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Noise and Vibration Studies

Geotechnical Investigation

Proposed Multi-Storey Building
1400 Bank Street
Ottawa, Ontario

Prepared For

SerCo Reality Group

September 10, 2021

Report: PG5922-1

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

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Appendices

1.0 Introduction

Paterson Group was commissioned by SerCo Reality Group to conduct a geotechnical investigation for the proposed multi-storey building to be located at 1400 Bank Street in the City of Ottawa (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

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

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

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

Investigating the presence or potential presence of contamination on the subject property was not part of the scope of work of the present investigation. Therefore, the present report does not address environmental issues.

2.0 Proposed Development

Based on the available drawings, it is understood that the proposed development will consist of a multi-storey building with 3 levels of underground parking which will occupy the majority of the site footprint. Associated walkways, landscaped areas, and asphalt-paved access lanes are also anticipated surrounding the proposed building. It is expected that the proposed development will be municipally serviced.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

The field program for the current geotechnical investigation was carried out on July 22, 2021, and consisted of advancing a total of 5 boreholes to a maximum depth of 8.6 m below existing ground surface. The test hole locations were distributed in a manner to provide general coverage of the subject site and taking into consideration underground utilities and site features.

Paterson also conducted a previous investigation as part of a Phase II Environmental Site Assessment between June 24 and 26, 2019. At that time, 5 boreholes were advanced to a maximum depth of 6.1 m below the existing ground surface. The borehole locations are shown on Drawing PG5922-1 - Test Hole Location Plan included in Appendix 2.

The boreholes were completed using a low-clearance, track-mounted 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 advancing each test hole to the required depths at the selected locations and sampling the overburden.

Sampling and In Situ Testing

Soil samples were recovered using a 50 mm diameter split-spoon sampler or from the auger flights. The split-spoon and auger samples were classified on site and placed in sealed plastic bags. All samples were transported to our laboratory. The depths at which the split-spoon 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. 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.

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

The thickness of the overburden was evaluated during the investigation by a dynamic cone penetration test (DCPT) at borehole BH 4-21. The DCPT consists of driving a steel drill rod, equipped with a 50 mm diameter cone at its 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 were installed in boreholes BH 3-21 and BH 4-21, and flexible polyethylene standpipes were installed in the remaining 3 boreholes to permit monitoring of the groundwater levels subsequent to the completion of the current sampling program.

Monitoring wells were also installed in boreholes BH 1, BH 2 and BH 4 as part of the previous environmental investigation.

Sample Storage

All samples from the current investigation will be stored in the laboratory for a period of one (1) month after issuance of this report. They will then be discarded unless otherwise directed.

3.2 Field Survey

The test hole locations from the current investigation were selected by Paterson to provide general coverage of the proposed development, taking into consideration the existing site features and underground utilities. The test hole locations and ground surface elevations for the current geotechnical investigation were surveyed by Paterson using a handheld GPS and referenced to a geodetic datum. The test hole ground surface elevations from the previous investigation were surveyed with reference to a temporary benchmark (TBM), consisting of the finished floor level of the existing building at the site, and assigned an assumed elevation of 100.00 m. The location of the boreholes and ground surface elevation at each test hole location are presented on Drawing PG5922-1 - 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.

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, which was collected from borehole BH 2-21. The sample was submitted to determine the concentration of sulphate and chloride, the resistivity, and the pH of the samples. The results are presented in Appendix 1 and are discussed further in Section 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a low-rise, multi-unit commercial building surrounded by asphalt-paved access lanes and parking areas with landscaped margins.

However, as observed from available aerial photographs, the subject was occupied by a sheet metal fabrication business until the 1970s and the property was re-developed in the 1980s for the present-day commercial building. Reference should be made to aerial photographs in Figure 2 through Figure 6 presented in Appendix 2, which illustrate the former and the present site conditions.

The site is bordered by Belanger Street to the north, a multi-storey residential building to the west, automobile garage to the south and by Bank Street to the east. The existing ground surface across the site is relatively level at approximate geodetic elevations of 77.5 to 78.5 m.

4.2 Subsurface Profile

Generally, the soil profile at the test hole locations consists of asphaltic concrete underlain by fill extending to approximate depths ranging from 0.3 to 3.1 m. The fill was generally observed to consist of brown silty clay and/or brown silty sand with traces of topsoil and brick.

A hard to stiff, brown silty clay was encountered underlying the fill, transitioning into a stiff to firm grey silty clay at an approximate depth of 3.7 m below the existing ground surface.

A glacial till deposit was encountered underlying the silty clay at borehole BH 2-21 at a depth of about 7 m below the existing ground surface. The glacial till deposit was observed to consist of grey silty clay with sand, gravel, cobbles and boulders.

Practical refusal to the auger/DCPT was encountered at depths of 8.6 and 7.8 m in boreholes BH 2-21 and BH 4-21, respectively.

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

Bedrock

Based on available geological mapping, the bedrock in the subject area consists of Paleozoic shale of the Upper Ordovician formation, with an overburden drift thickness of 5 to 10 m.

4.3 Groundwater

Groundwater levels were measured on August 10th and 24th, 2021 within the installed groundwater monitoring wells and standpipe piezometers. 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 1-21	78.17	1.90	76.27	August 24, 2021
BH 2-21	77.97	1.75	76.22	
BH 3-21	78.47	2.25	76.22	
BH 4-21	78.26	2.30	75.96	
BH 5-21	78.10	2.41	75.69	
BH 1	78.10	1.75	76.35	August 10, 2021
BH 2	78.10	1.60	76.50	
BH 4	78.20	1.94	76.26	

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

It should be noted that groundwater measurements can be influenced by surface water infiltrating the backfilled boreholes. The long-term groundwater levels can also be estimated based on the observed colour and consistency of the recovered soil samples. Based on these observations, the long-term groundwater table can be expected at approximately 3.5 to 4.5 m below ground surface.

The recorded groundwater levels are noted on the applicable Soil Profile and Test Data sheet presented in Appendix 1.

It should also 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 development. It is recommended that foundation support for the proposed building consist of conventional spread footings bearing directly on clean, surface sounded bedrock.

It is anticipated that bedrock removal will be required for the proposed building excavation. Further, where glacial till is excavated, it is anticipated that cobbles and boulders will be encountered frequently. All contractors should be prepared for boulder removal at the subject site.

Due to the presence of a silty clay deposit, a permissible grade raise restriction is required for the subject site.

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

5.2 Site Grading and Preparation

Stripping Depth

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

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.

Bedrock Removal

Where the bedrock is weathered and/or where only small quantities of bedrock need to be removed, hoe ramming is an option for bedrock removal. Where large quantities of bedrock need to be removed, line drilling and controlled blasting may be required.The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

Prior to considering blasting operations, the blasting effects on the existing services, buildings and other structures should be addressed. A pre-blast or pre-construction survey of the existing structures located in proximity of the blasting operations shouldbe conducted prior to commencing construction.

The extent of the survey should be determined by the blasting consultant and sufficient to respond to any inquiries/claims related to the blasting operations.

Vibration Considerations

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

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

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

Fill Placement

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

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

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance at ULS.

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

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

The bearing medium under footing-supported structures is required to be provided withadequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a planeextending horizontally and vertically from the footing perimeter at a minimum of 1H:6V(or shallower) passes through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

Permissible Grade Raise

Due to the presence of the silty clay deposit, a permissible grade raise restriction of **1 m** is recommended for grading at 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 settlements.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. A higher seismic site class, such as Class A or B, may be applicable to the proposed development at the subject site. However, a site-specific shear wave velocity test would be required to accurately determine the higher 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.

Soils underlying the subject site are not susceptible to liquefaction. Reference shouldbe made to the latest revision of the Ontario Building Code 2012 for a full discussionof the earthquake design requirements.

5.5 Basement Slab Construction

For the proposed development, it is anticipated that all overburden soil will be removed from the building footprint, leaving the bedrock as the founding medium for the basement floor slab. It is anticipated that the basement area for the proposed building will be mostly parking and the recommended pavement structures noted in Section 5.8 will be applicable. However, if storage or other uses of the lower level will involve the construction of a concrete floor slab, the upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone.

Any soft areas in the basement slab subgrade should be removed and backfilled with appropriate backfill material prior to placing fill. OPSS Granular A or Granular B Type II, with a maximum particle size of 50 mm, are recommended for backfilling below the floor slab. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

In consideration of the groundwater conditions at the site, a sub-slab drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the subfloor fill under the lower basement floor. This is discussed further in Subsection 6.1.

5.6 Basement Wall

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

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

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

Lateral Earth Pressure

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

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

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

Actual earth pressures could be higher than the "at-rest" case if care is not exercisedduring 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}) can be calculated using $0.375 \cdot a \cdot \gamma \cdot H^2/g$ where:

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

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

The earthforce component (P_o) under seismic conditions can be calculated using:

$P = 0.5\ K \cdot \gamma \cdot H^2$, where K = 0.5 for the soil conditions noted above.

The total earth force (P_{AE}) is considered to act at a height, h (m), from the base of thewall, 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 shouldbe factored as live loads, as per OBC 2012.

5.7 Rock Anchor Design

Overview of Anchor Features

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

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

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

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

Grout to Rock Bond

The Canadian Foundation Engineering Manual recommends a maximum allowable grout to rock bond stress (for sound rock) of 1/30 of the unconfined compressive strength (UCS) of either the grout or rock (but less than 1.3 MPa) for an anchor of minimum length (depth) of 3 m. Generally, the UCS of shales ranges between about 30 and 40 MPa, which is equivalent to most routine grouts. A factored tensile grout to rock bond resistance value at ULS of 1.0 MPa, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

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

Recommended Rock Anchor Lengths

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

Table 2 - Parameters used in Rock Anchor Review

Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Shale Hoek and Brown parameters	65 m=0.575 and s=0.00293
Unconfined compressive strength – Shale bedrock	30 MPa
Unit weight - Submerged Bedrock	15.5 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

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

Table 3 - Recommended Rock Anchor Lengths - Grouted Rock Anchor

Diameter of Drill Hole (mm)	Anchor Lengths (m)			Factored Tensile Resistance (kN)
	Bonded Length	Unbonded Length	Total Length	
75	2.0	0.8	2.8	450
	2.6	1.0	3.6	600
	3.2	1.3	4.5	750
	4.5	2.0	6.5	1000
125	1.6	1.0	2.6	600
	2.0	1.2	3.2	750
	2.6	1.4	4.0	1000
	3.2	1.8	5.0	1250

Other considerations

The anchor drill holes should be within 1.5 to 2 times the rock anchor tendon diameter, inspected by geotechnical personnel and should be flushed clean prior to grouting. A tremie tube is recommended to place grout from the bottom of the anchor holes. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day that grout is prepared.

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

5.8 Pavement Design

For design purposes, it is recommended that the rigid pavement structure for the underground parking level 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 4 on the next page. The flexible pavement structure presented in Table 5 should be used for at grade access lanes and heavy loading parking areas.

Table 4 - 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 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 hour after the concrete has been poured during warm temperatures and up to 12 hours during cooler temperatures.

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

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

If soft spots develop in the subgrade during compaction or due to construction traffic,the affected areas should be excavated and replaced with OPSS Granular B Type I orType II material. Weak subgrade conditions may be experienced over service trenchfill materials. This may require the use of a geotextile, thicker subbase or other measures that can be recommended at the time of construction. Minimum Performance Graded (PG) 58-34 asphalt cement should be used.

The pavement granular base and subbase should be paced in maximum 300 mm thick liftsand compacted to a minimum of 100% of the material's SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage & Waterproofing

To manage and control groundwater infiltration over long term conditions, the following water suppression system is recommended to be installed for the exterior foundation walls. Due to the proposed depth of excavation and the proximity of the proposed building to the property lines, it is anticipated that the waterproofing membrane will be installed directly against the temporary shoring system and the underlying vertical bedrock face.

☐ A waterproofing membrane will be required to lessen the effect of water infiltration for the underground parking levels starting at a 3 m depth, down to the footing level. The waterproofing membrane should consist of bentonite panels such as Paraseal LG (20 mil HDPE Bentomat), or approved equivalent, fastened to the shoring system or vertical bedrock face. Where the vertical bedrock face is present, it should be grinded and any cavities should be in-filled with concrete or shotcrete prior to the installation of the waterproofing membrane.

☐ A composite drainage board will be placed between the waterproofing membrane and the foundation wall, from finished grade to the bottom of the foundation wall. It is recommended that the composite drainage system (such as Delta Drain 6000 or equivalent) extend down to the bottom of the foundation wall. It is expected that 150 mm diameter sleeves placed at 3 m centres be cast in the foundation wall at the footing interface to allow for the infiltration of water to flow to an interior perimeter drainage pipe.

☐ The perimeter drainage pipe should direct water to the sump pit(s) within the lower basement area. Water infiltration will result from two sources. The first will be water infiltration from the upper 3 m which is above the vertical waterproofed area. The second source will be water from minor breaching of the waterproofing membrane.

Underfloor Drainage

Underfloor drainage will be required to control water infiltration below the lowest underground parking level slab. For design purposes, it is recommended that 150 mm diameter perforated pipes be placed along the interior perimeter and at approximate 6 m spacing underlying the lowest level floor slab. The final spacing of the underfloor drainage system 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 insulation equivalent) should be provided in this regard.

Exterior unheated footings, such as those for isolated exterior piers, are more prone to deleterious movement associated with frost action than the exterior walls of the heated structure and require additional protection, such as soil cover of 2.1 m or an equivalent combination of soil cover and foundation insulation.

However, foundations which are founded directly on clean, surface-sounded bedrock,and which is approved by Paterson at the time of construction, is not considered frostsusceptible and does not require soil cover.

6.3 Excavation Side Slopes

The side slopes of shallow excavations anticipated at this site should either be cut back at acceptable slopes or be retained by shoring systems from the start of the excavation until the structure is backfilled. Based on the proposed depth of excavation and the proximity of the proposed building to the property lines, it is anticipated that a temporary shoring system will be required to support the overburden, and any weathered bedrock which is encountered.

Unsupported Side Slopes

The excavation side slopes in the overburden and, where present, weathered bedrock, 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 subsoil at this site 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 be kept away from the excavation sides. Slopes in excess of 3 m in height should be periodically inspected by the geotechnical consultant in order to detect if the slopes are exhibiting signs of distress.

It is recommended that a trench box be used at all times to protect personnel working in trenches with steep or vertical sides. It is expected that services will be installed by “cut and cover” methods and excavations will not be left open for extended periods of time.

Rock stabilization

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

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

The requirements for horizontal rock anchors and bedrock stabilization measures will be evaluated during the excavation program and determined by Paterson at the time of construction.

Temporary Shoring

Temporary shoring may be required for the overburden soil, and if encountered, weathered bedrock, to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements should be designed by a structural engineer, specializing in shoring design. The shoring will depend on the depth of the excavation, the proximity of the adjacent structures and the elevation of the adjacent building foundations, roadways and underground services.

The design and implementation of the temporary systems will be the responsibility of the excavation contractor. The geotechnical information provided below is to assist the Contractor in preparing a safe design.

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 design prior to implementation.

The temporary system could consist of soldier pile and lagging system or interlocking steel sheet piling. Any additional loading due to street traffic, construction equipment,adjacent structures and facilities, etc., should be included to the earth pressures described below.

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

Table 6 - Soil Parameters	
Paramet ers	**Values**
Active Earth Pressure Coefficient (K_a)	0.33
Passive Earth Pressure Coefficient (K_p)	3
At-Rest Earth Pressure Coefficient (K_o)	0.5
Dry Unit Weight (γ), kN/m^3	20
Effective Unit Weight (γ), kN/m^3	13

The active earth pressure should be calculated where wall movements are permissible, while the at-rest pressure should be calculated if movement is not permissible. The dryunit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

A hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If thegroundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated full weight, with no hydrostatic groundwater pressure component. For designpurposes, 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 and 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 orwater pipes when placed on soil subgrade. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The beddingshould extend to the spring line of the pipe.

The cover material, which should consist of OPSS Granular A, should extend from thespring line of the pipe to at least 300 mm above the obvert of the pipe. The material should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98%of its SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfillmaterial within the frost zone (about 1.8 m below finished grade) should match the soilsexposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 300 mm thick loose lifts and compacted to a minimum of95% of the material’s SPMDD.

6.5 Groundwater Control

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

Groundwater Control for Building Construction

A temporary Category 3 Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to5 months should be allowed for completion of the application and issuance of the permitby the MECP.

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

Impacts on Neighbouring Structures

It is understood that 3 underground parking levels are proposed for the development, with the lower portions of the foundation wall having a groundwater infiltration control system in place.

Based on field observations and assessment, the groundwater level is anticipated at an approximate 3.5 to 4.5 m depth below existing grade. Localized groundwater lowering is expected under short-term conditions due to construction of the proposed building. However, it should be noted that no significant groundwater lowering will take place once the foundation wall construction has been completed, as the waterproofing membrane will mitigate any significant groundwater drainage.

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 by the use of 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 an aggressive to very aggressive corrosive environment.

7.0 Recommendations

It is a requirement for the foundation design data provided herein to be applicable that the following material testing and observation program be performed by the geotechnical consultant.

- Grading plan review from a geotechnical perspective, once the final grading plan is available.
- Observation of all bearing surfaces prior to the placement of concrete.
- Sampling and testing of the concrete and fill materials.
- Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.
- 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.
- Review bedrock excavation activities and exposed vertical bedrock faces.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection 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 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 SerCo Realty Group 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.

O. Canton

Owen Canton, E.I.T.

LICENSED PROFESSIONAL ENGINEER
Sept. 10-2021
S. S. DENNIS
100519516
Scott Dennis
PROVINCE OF ONTARIO

Scott Dennis

Scott S. Dennis, P.Eng.

Report Distribution:

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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
1400 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

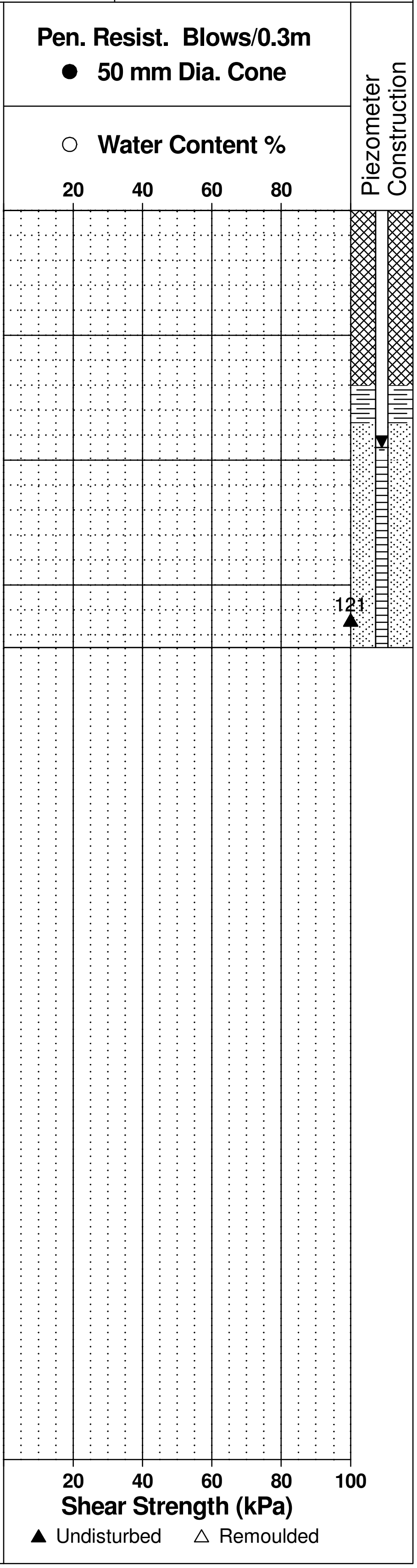
BORINGS BY CME-55 Low Clearance Drill

DATE July 22, 2021

FILE NO. PG5922

HOLE NO. BH 1-21

SOIL DESCRIPTION	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	78.17
Asphaltic concrete 0.08	AU	1				
FILL: Crushed stone with brown silty sand 0.41						
FILL: Brown silty clay with topsoil, trace sand and gravel 1.07	SS	2	42	16	1	77.17
Hard to very stiff, brown **SILTY CLAY**	SS	3	58	6	2	76.17
	SS	4	83	6	3	75.17
3.50						
End of Borehole (GWL @ 1.90m - August 10, 2021)						



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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1400 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 22, 2021

FILE NO. PG5922

HOLE NO. BH 2-21

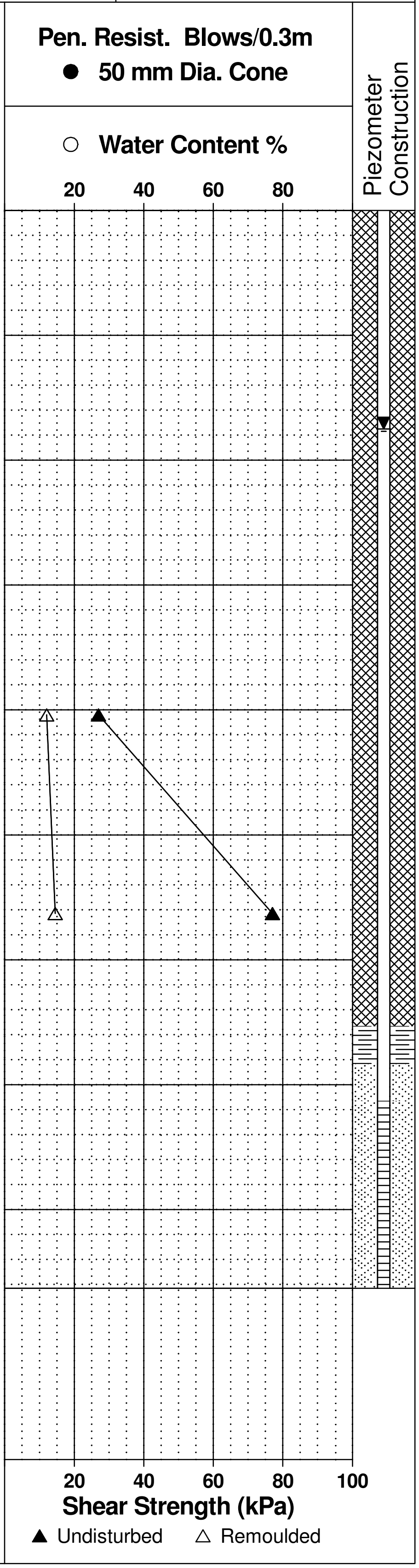
SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE				
Asphaltic concrete 0.08	AU	1		
FILL: Crushed stone with sand 0.46				
FILL: Brown silty clay, some sand and gravel, trace brick 1.55	SS	2	58	6
Very stiff, brown **SILTY CLAY** - firm to stiff by 3.7m depth - silt content increasing with depth	SS	3	67	7
	SS	4	83	5
	SS	5	75	2
	SS	6	83	P
	SS	7	100	P
7.01	SS	8	83	2
GLACIAL TILL: Grey silty clay with sand, gravel, cobbles and boulders - clay content decreasing with depth	SS	9	58	35
	SS	10	67	64
8.63	SS	11	67	50+

End of Borehole

Practical refusal to augering at 8.63m depth.

(GWL @ 1.75m - August 10, 2021)

DEPTH (m)	ELEV. (m)
0	77.98
1	76.98
2	75.98
3	74.98
4	73.98
5	72.98
6	71.98
7	70.98
8	69.98



SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1400 Bank Street
Ottawa, Ontario

DATUM Geodetic

FILE NO. **PG5922**

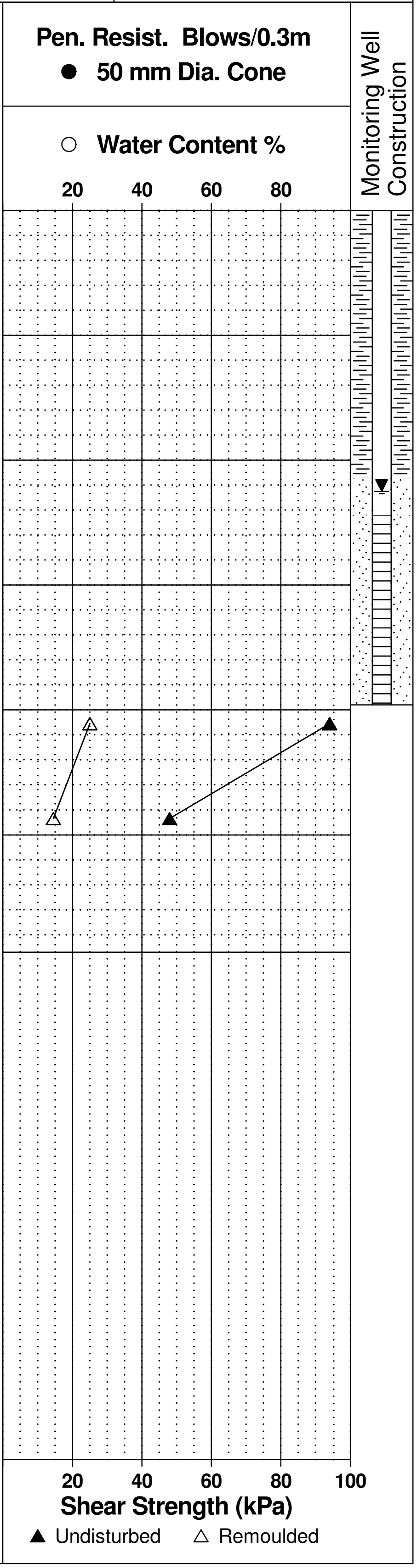
REMARKS

HOLE NO. **BH 3-21**

BORINGS BY CME-55 Low Clearance Drill

DATE July 22, 2021

SOIL DESCRIPTION	STRATA PLOT	SAMPLE				DEPTH (m)	ELEV. (m)
GROUND SURFACE		TYPE	NUMBER	% RECOVERY	N VALUE or RQD		
TOPSOIL 0.25						0	78.47
FILL: Brown silty sand, some crushed stone and gravel 0.76		AU	1				
FILL: Brown silty clay with sand and gravel 1.58		SS	2	33	12	1	77.47
		SS	3	33	12	2	76.47
		SS	4	75	10	3	75.47
Very stiff to stiff, brown **SILTY CLAY**		SS	5	83	5	4	74.47
		SS	6	83	P		
- firm and grey by 4.6m depth		SS	7	75	P	5	73.47
5.94		SS	8		P		
End of Borehole (GWL @ 2.25m - August 10, 2021)							



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1400 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

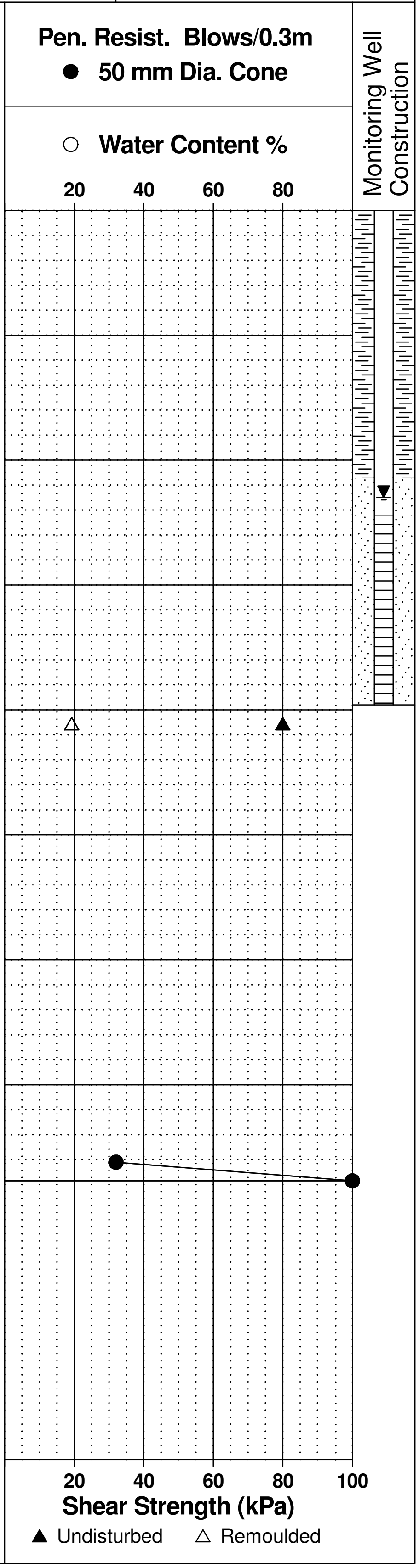
DATE July 22, 2021

FILE NO. PG5922

HOLE NO. BH 4-21

SOIL DESCRIPTION	Depth of base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Patio stone	0.05				
FILL: Crushed stone with sand	0.76	AU	1		
FILL: Brown silty clay, trace sand, gravel and topsoil	1.22	SS	2	33	9
Hard to very stiff, brown **SILTY CLAY**		SS	3	50	9
		SS	4	83	7
		SS	5	83	3
- stiff to firm and grey by 3.7m depth		SS	6		P
	5.18	SS	7		1
Dynamic Cone Penetration Test commenced at 5.18m depth. Cone pushed to 7.3m depth.	7.77				
End of Borehole Practical DCPT refusal at 7.77m depth. (GWL @ 2.30m - August 10, 2021)					

DEPTH (m)	ELEV. (m)
0	78.27
1	77.27
2	76.27
3	75.27
4	74.27
5	73.27
6	72.27
7	71.27



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SOIL PROFILE AND TEST DATA

Geotechnical Investigation
1400 Bank Street
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME-55 Low Clearance Drill

DATE July 22, 2021

FILE NO. PG5922

HOLE NO. BH 5-21

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	78.11
Asphaltic concrete 0.08							
FILL: Crushed stone with sand 0.30		AU	1				
FILL: Brown silty clay, some sand and gravel, trace topsoil 0.97							
Hard to very stiff, brown **SILTY CLAY,** trace sand		SS	2	42	13	1	77.11
		SS	3	33	14	2	76.11
		SS	4	58	5		
3.66		SS	5	75	3	3	75.11
End of Borehole							
(GWL @ 2.41m - August 10, 2021)							

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

○ Water Content %

20 40 60 80

Piezometer Construction

20 40 60 80 100

Shear Strength (kPa)

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
1400 Bank Street
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 24, 2019

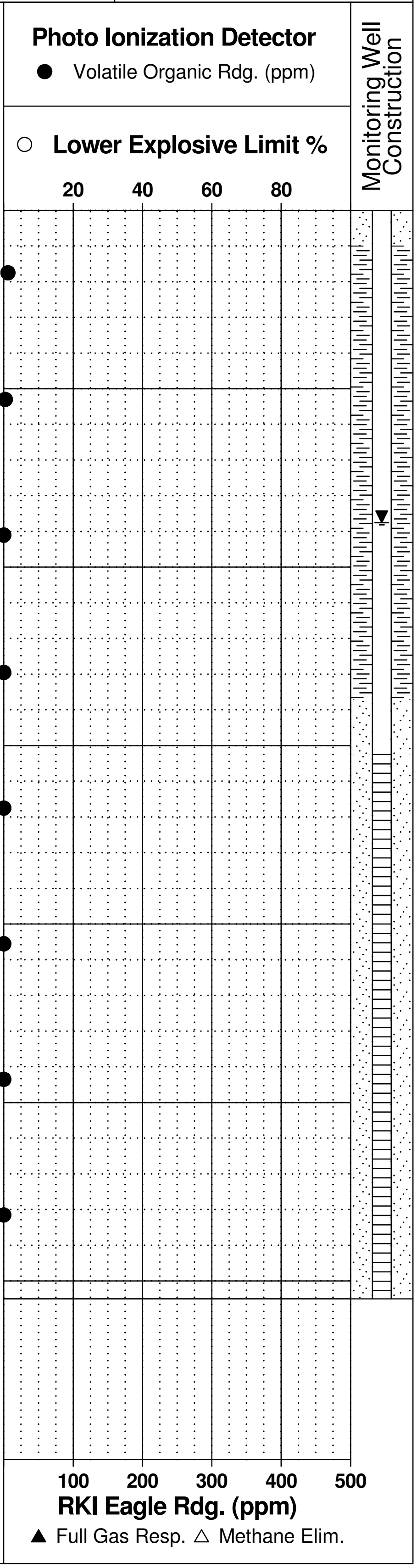
FILE NO. PE4632

HOLE NO. BH 1

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with gravel	0.76
FILL: Brown silty clay, trace sand	1.52
Brown **SILTY CLAY** - grey by 3.8m depth	6.10
End of Borehole (GWL @ 1.75m - August 10, 2021)	

SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	18	10
SS	3	67	10
SS	4	96	6
SS	5	100	4
SS	6	100	2
SS	7	100	2
SS	8	100	W

DEPTH (m)	ELEV. (m)
0	78.10
1	77.10
2	76.10
3	75.10
4	74.10
5	73.10
6	72.10



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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
1400 Bank Street
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

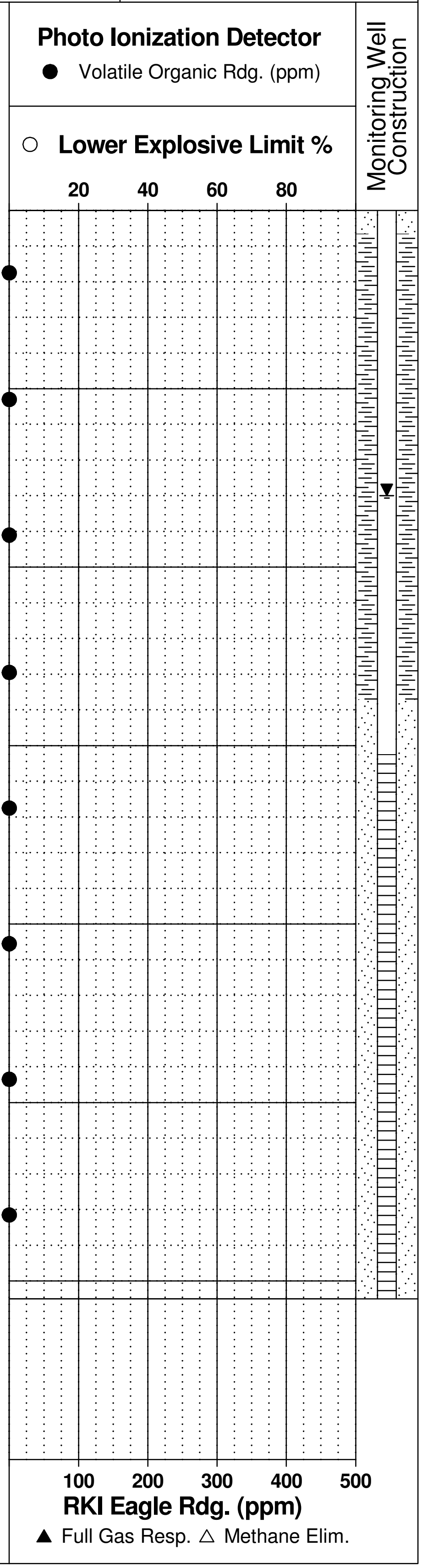
DATE June 24, 2019

FILE NO. PE4632

HOLE NO. BH 2

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.10				
FILL: Brown silty sand with gravel	0.76	AU	1		
Brown **SILTY CLAY,** trace sand - grey by 3.0m depth	6.10	SS	2	54	11
		SS	3	50	8
		SS	4	96	5
		SS	5	100	3
		SS	6	100	2
		SS	7	100	2
		SS	8	100	2
End of Borehole (GWL @ 1.60m - August 10, 2021)					

DEPTH (m)	ELEV. (m)
0	78.10
1	77.10
2	76.10
3	75.10
4	74.10
5	73.10
6	72.10



SOIL PROFILE AND TEST DATA

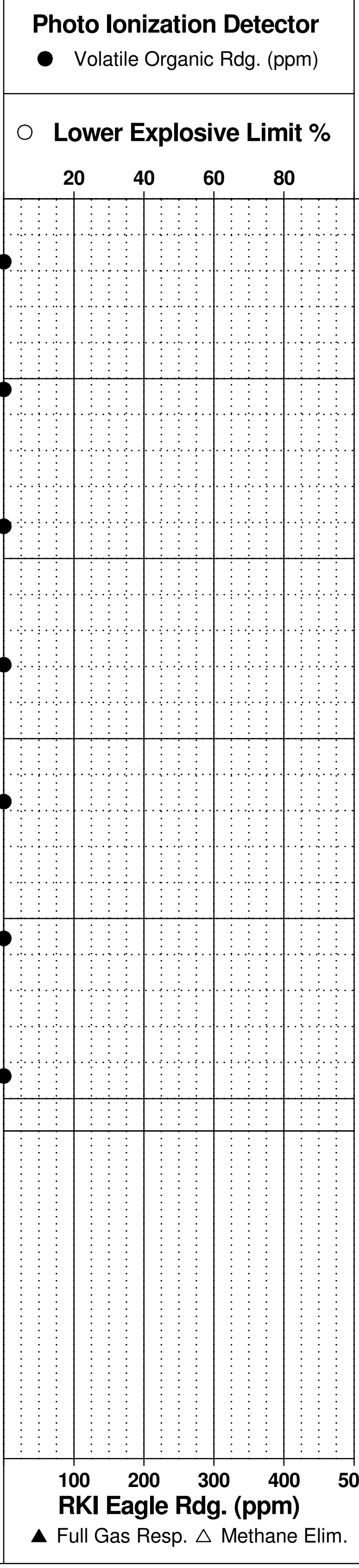
Phase I - II Environmental Site Assessment
1400 Bank Street
Ottawa, Ontario

DATUM Approximate geodetic
REMARKS
BORINGS BY CME 55 Power Auger
DATE June 26, 2019
FILE NO. PE4632
HOLE NO. BH 3

SOIL DESCRIPTION	Depth to base (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with gravel, trace brick	1.52
Grey **SILTY CLAY**	5.18
End of Borehole	

TYPE	NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	21	8
SS	3	58	5
SS	4	96	4
SS	5	100	2
SS	6	100	2
SS	7	100	W

DEPTH (m)	ELEV. (m)
0	78.00
1	77.00
2	76.00
3	75.00
4	74.00
5	73.00



Monitoring Well Construction

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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
1400 Bank Street
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE June 26, 2019

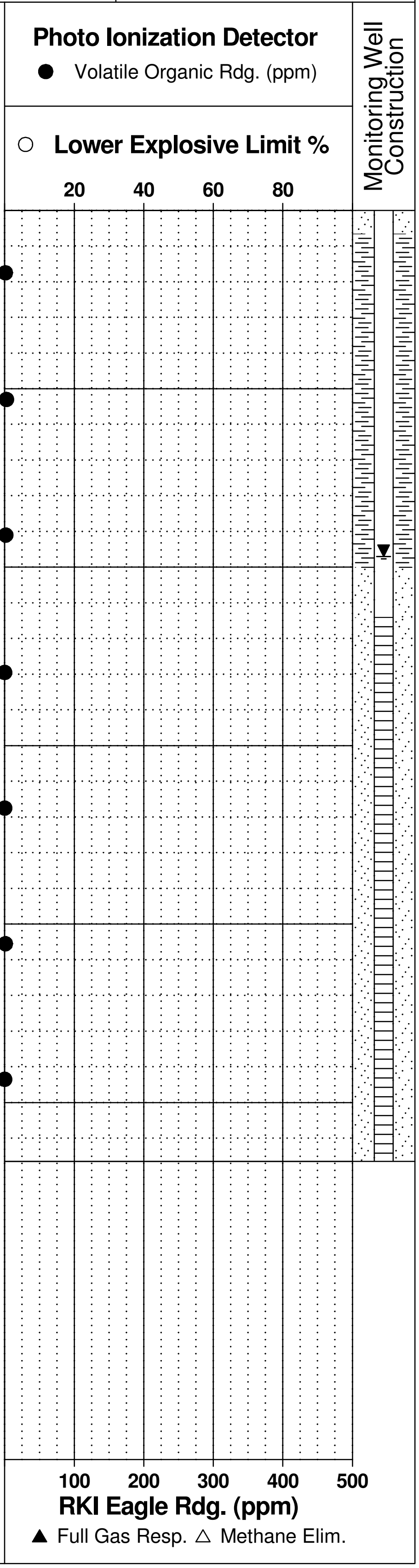
FILE NO. PE4632

HOLE NO. BH 4

SOIL DESCRIPTION	Depth (m)
GROUND SURFACE	
Asphaltic concrete	0.10
FILL: Brown silty sand with gravel	3.05
Brown **SILTY CLAY** - grey by 3.7m depth	5.33
End of Borehole (GWL @ 1.94m - August 10, 2021)	

SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD
AU	1		
SS	2	58	15
SS	3	21	18
SS	4	54	7
SS	5	29	3
SS	6	54	2
SS	7	100	W

DEPTH (m)	ELEV. (m)
0	78.20
1	77.20
2	76.20
3	75.20
4	74.20
5	73.20



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SOIL PROFILE AND TEST DATA

Phase I - II Environmental Site Assessment
1400 Bank Street
Ottawa, Ontario

DATUM Approximate geodetic

REMARKS

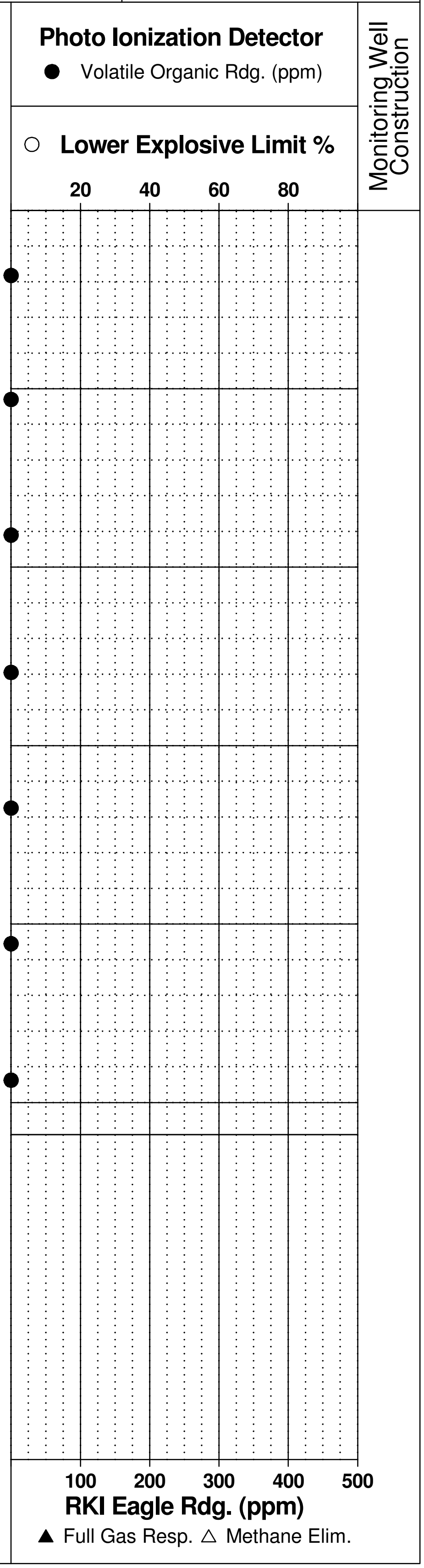
BORINGS BY CME 55 Power Auger

DATE June 26, 2019

FILE NO. PE4632

HOLE NO. BH 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	SAMPLE % RECOVERY	SAMPLE N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	78.20
Asphaltic concrete 0.13							
FILL: Brown silty sand with gravel		AU	1				
		SS	2	46	15	1	77.20
1.52							
Brown **SILTY CLAY**		SS	3	54	12		
						2	76.20
		SS	4	100	4		
						3	75.20
		SS	5	100	3		
- grey by 3.8m depth							
						4	74.20
		SS	6	100	2		
		SS	7	100	1		
						5	73.20
5.18							
End of Borehole							



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)

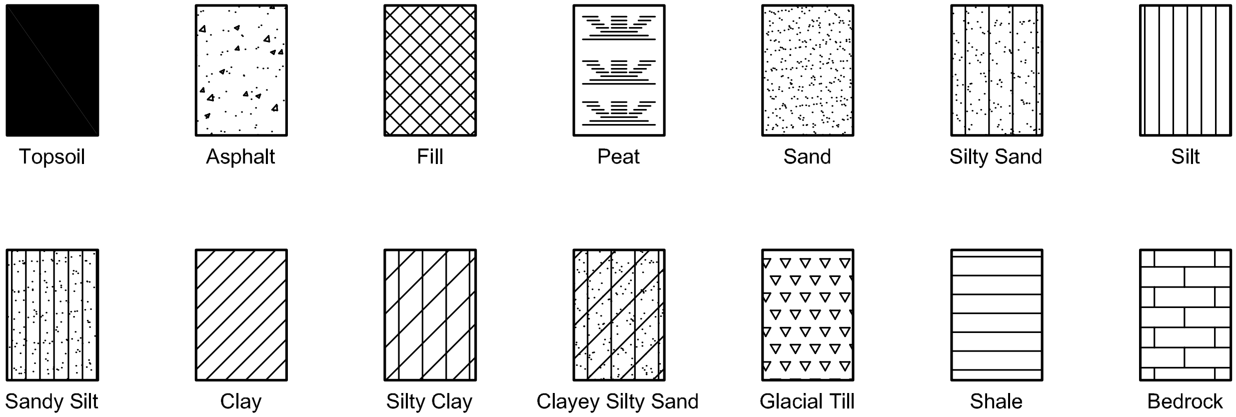
OC Ratio Overconsolidaton ratio = p'_c / p'_o
Void Ratio Initial sample void ratio = volume of voids / volume of solids
Wo - Initial water content (at start of consolidation test)

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.

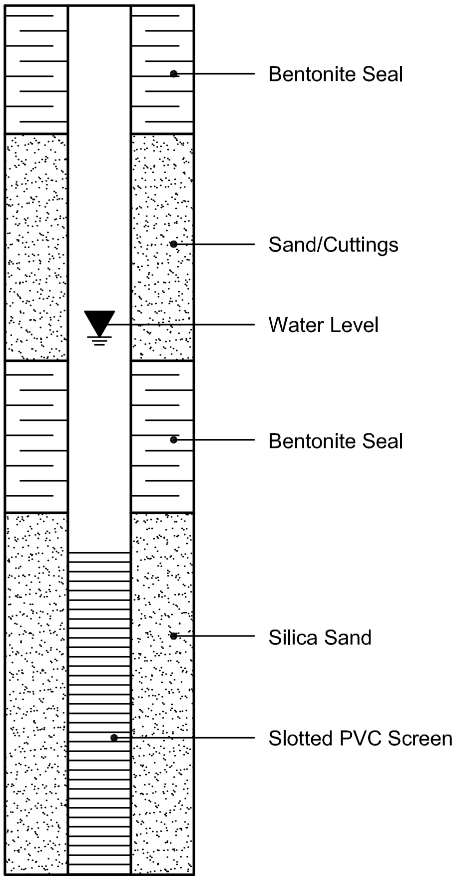
SYMBOLS AND TERMS (continued)

STRATA PLOT

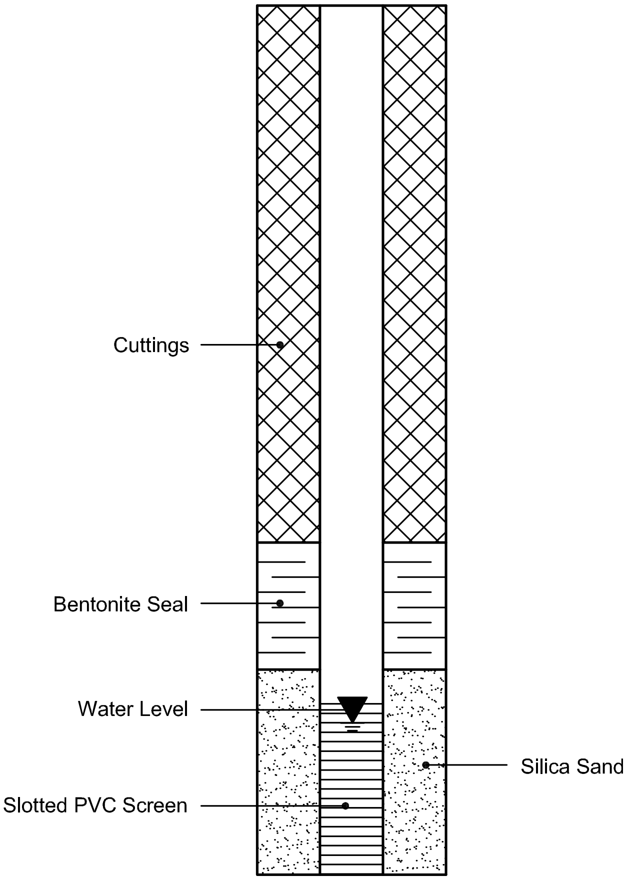


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Certificate of Analysis

Client: Paterson Group Consulting Engineers

Client PO: 32524

Report Date: 30-Jul-2021

Order Date: 26-Jul-2021

Project Description: PG5922

	Client ID:	BH2-21 SS5	-	-	-
	Sample Date:	22-Jul-21 00:00	-	-	-
	Sample ID:	2131185-01	-	-	-
	MDL/Units	Soil	-	-	-
Physical Characteristics					
% Solids	0.1 % by Wt.	68.1	-	-	-
General Inorganics					
pH	0.05 pH Units	6.90	-	-	-
Resistivity	0.10 Ohm.m	14.0	-	-	-
Anions					
Chloride	5 ug/g dry	223	-	-	-
Sulphate	5 ug/g dry	312	-	-	-

APPENDIX 2

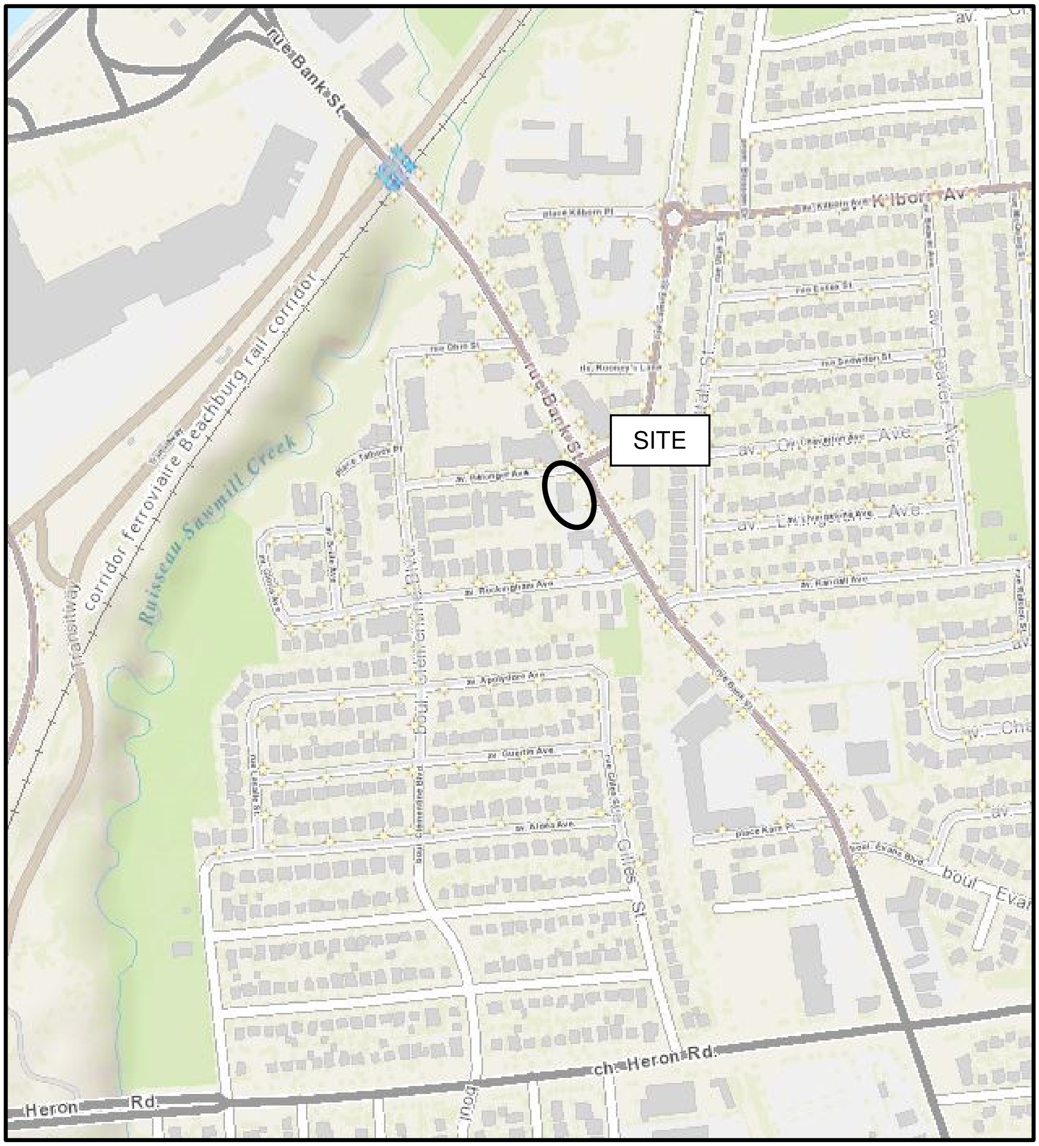
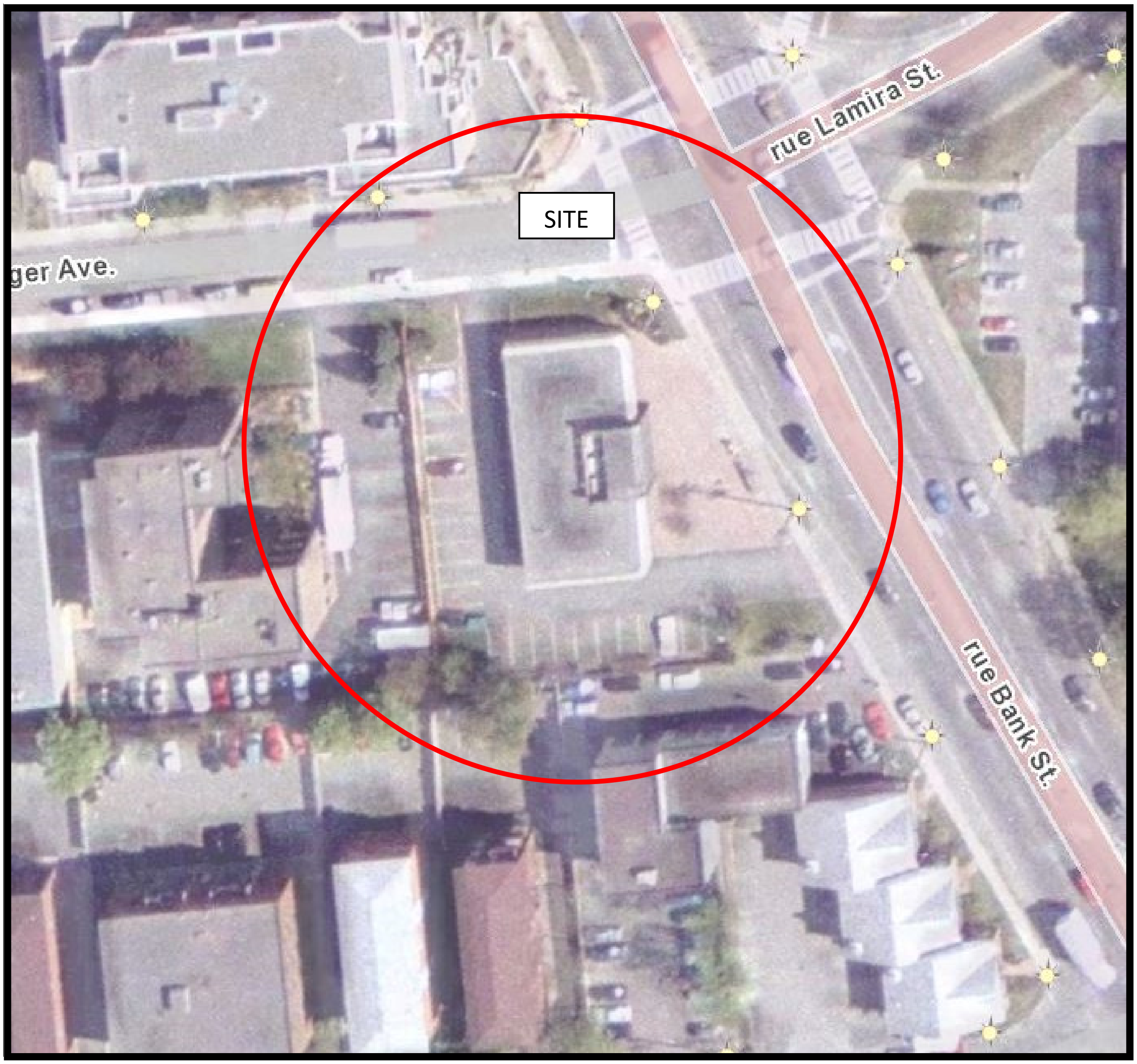


FIGURE 1

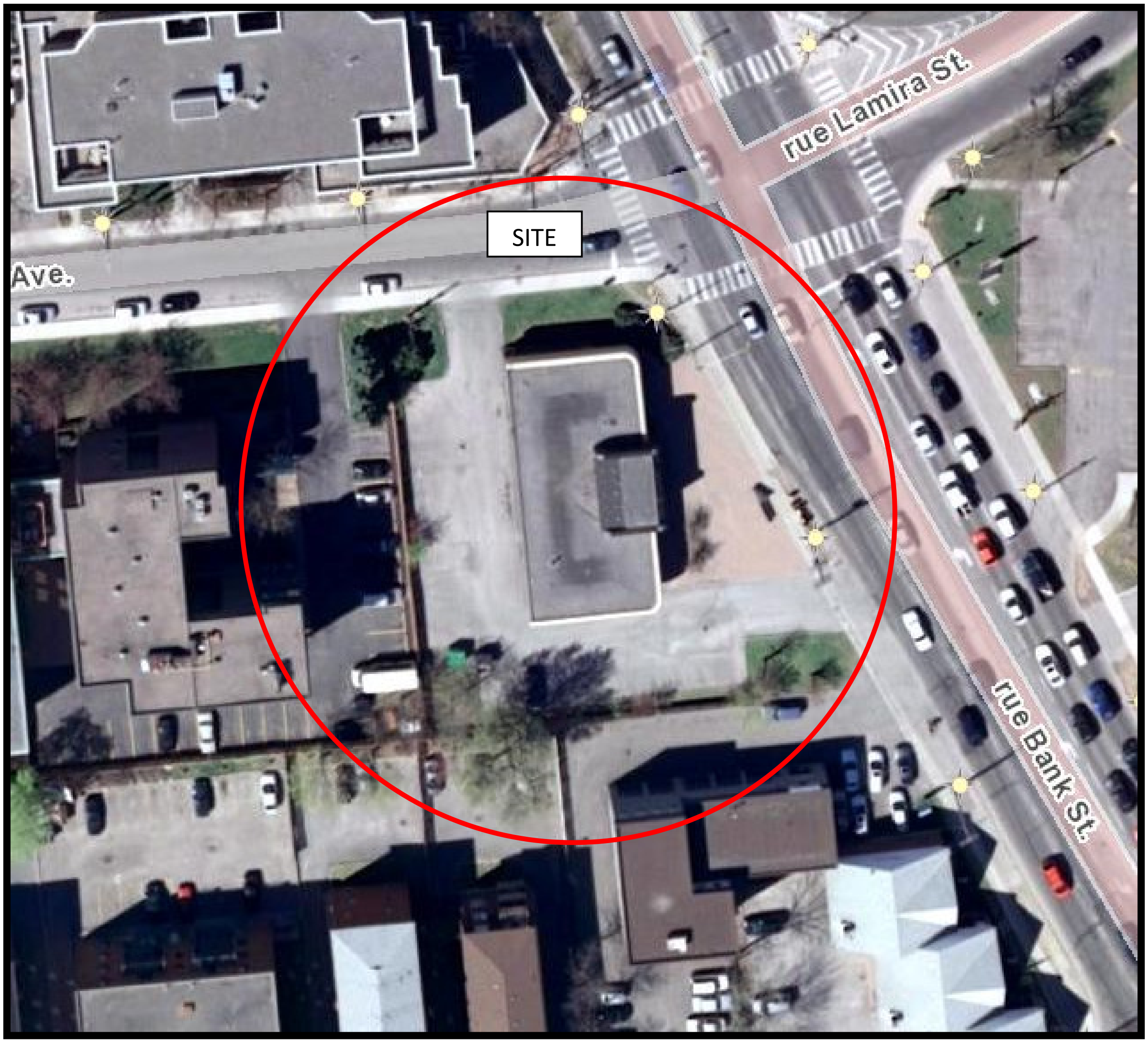
KEY PLAN

AERIAL PHOTOGRAPH
1965

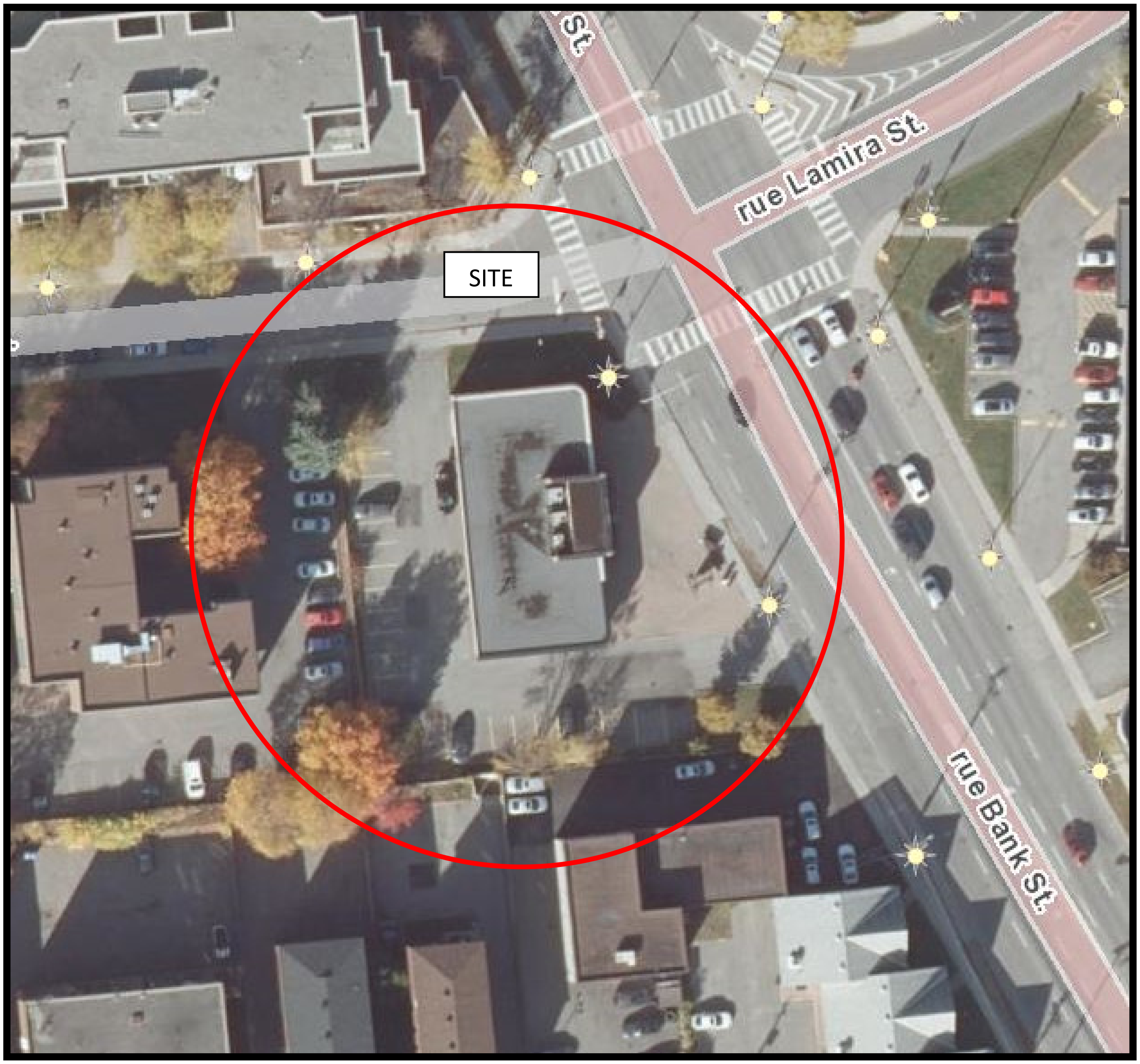
AERIAL PHOTOGRAPH
1976



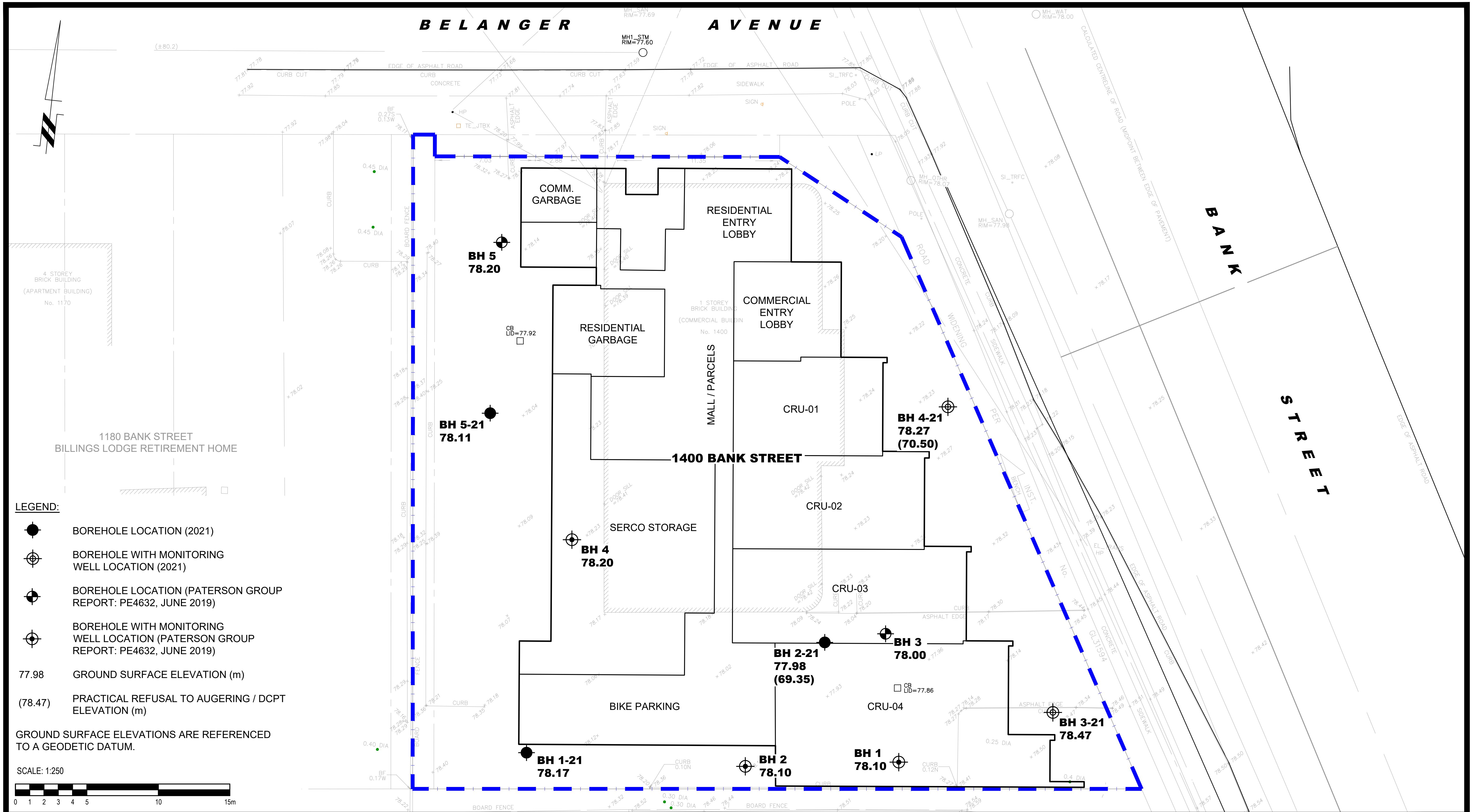
AERIAL PHOTOGRAPH
2002



AERIAL PHOTOGRAPH
2007



AERIAL PHOTOGRAPH
2019



LEGEND:

●	BOREHOLE LOCATION (2021)
⊕	BOREHOLE WITH MONITORING WELL LOCATION (2021)
◆	BOREHOLE LOCATION (PATERSON GROUP REPORT: PE4632, JUNE 2019)
⌖	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT: PE4632, JUNE 2019)
77.98	GROUND SURFACE ELEVATION (m)
(78.47)	PRACTICAL REFUSAL TO AUGERING / DCPT ELEVATION (m)

GROUND SURFACE ELEVATIONS ARE REFERENCED TO A GEODETIC DATUM.

SCALE: 1:250

0 1 2 3 4 5 10 15m

patersongroup
consulting engineers

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

NO.	REVISIONS	DATE	INITIAL

SerCo REALTY GROUP

GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING - 1400 BANK STREET

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 07/2021
Drawn by: YA	Report No.: PG5922-1
Checked by: OC	Dwg. No.: PG5922-1
Approved by: SD	Revision No.:

c:\users\robertg\documents\pg5922\pg5922-1-test hole location plan.dwg