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Geotechnical Investigation

Proposed Multi-Storey Building
989 Somerset Street West
Ottawa, Ontario

Prepared For

Taggart Realty Management

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Report PG5885-1 Revision 3

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by Taggart Realty Management to conduct a geotechnical investigation for the proposed multi-storey building to be located at 989 Somerset Street West in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2).

The objectives of the current investigation were to:

❑ determine the subsurface and groundwater conditions by means of new and existing test holes information.

❑ provide geotechnical recommendations for the design of the proposed development including construction considerations which may affect the design based on current and existing borehole information..

A preliminary geotechnical report was previously issued by Paterson under Report G8584-1 dated January 7, 2003. Additional boreholes were completed during a supplemental investigation as part of a Environmental Site Assessment - Phase II on January 16, 2004.

The following report has been prepared specifically and solely for the aforementioned project which is described herein. This report contains the findings and includes geotechnical recommendations pertaining to the design and construction of the mixed-use (commercial and residential) development as understood at the time of writing this report.

2.0 Proposed Project

It is understood that the proposed development is understood to consist of a 15 storey residential building with 3 levels of underground parking, which will be encompassing the majority of the subject site. The building is proposed to have an eight storey portion that would occupy the majority of the site while the western half will consist of a 15-storey structure. It is also understood 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 geotechnical investigation was undertaken on April 11, 2022. At that time, a total of 3 test pits (TP1-22 to TP3-22) were advanced to a maximum depth of 4.4 m below ground surface.

The field program for a previous supplemental investigation was conducted on April 11, 2014. One borehole was advanced to a maximum depth of 7.0 m. A supplemental geotechnical investigation was completed in 2019 by this firm which included advancing three (3) boreholes to a maximum depth of 10.2 m within the footprint of the proposed building and one (1) test pit excavated along the eastern bridge abutment to a depth of 4.1 m below existing grade. Relevant test holes completed as part of a previous investigation from 2002 and 2004 were included as part of the report. The test hole locations were distributed in a manner to provide general coverage of the subject site based on the previous test holes completed. The test hole locations are presented on Drawing PG5885-1 - Test Hole Location Plan included in Appendix 2.

The test holes were completed using a backhoe excavator and track-mounted drill rig for test pits and boreholes, respectively. All fieldwork was conducted under the full-time supervision of Paterson personnel with the direction of a senior engineer. The drilling procedure consisted of augering to the required depths at the selected locations to sample and test the overburden.

Sampling and In Situ Testing

Soil samples were recovered from a 50 mm diameter split-spoon, the auger flights or grab samples. The split-spoon, auger and grab 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, auger and grab samples were recovered from the boreholes are presented as SS, AU and G, respectively, on the Soil Profile and Test Data sheets.

The Standard Penetration Test (SPT) was conducted in conjunction with the recovery of the split-spoon samples and 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.

Diamond drilling was completed at one location to confirm the depth to bedrock and

quality of bedrock. A recovery value and a Rock Quality Designation (RQD) value were calculated for each drilled section of bedrock and are shown on the Soil Profile and Test Data sheets in Appendix 1.
The recovery value is the ratio of the recovered sample length over the drilled section length. The RQD value is the ratio of the total intact rock longer than 100 mm per drilled section over the drilled section length. The values obtained indicate bedrock quality.

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

Groundwater

One monitoring well was installed during the current investigation and four during the 2004 investigation. Flexible PVC standpipes were installed in a selected number of the boreholes from the original investigation to monitor the ground water levels subsequent to the completion of the sampling program.

3.2 Field Survey

The test hole locations were selected and determined in the field by Paterson personnel to provide general coverage of the subject site. The test hole locations and elevations were surveyed in the field by Paterson. A manhole located at the centerline of Spruce Street across the northwest corner of the subject site was surveyed as a temporary benchmark (TBM) with a geodetic elevation of 54.98 m provided by Annis, Sullivan, Vollebek Ltd. The original TBM elevation of 55.53 m used for the 2004 investigation is higher due to Spruce Street recently undergoing a road rehabilitation program that altered the road grades.

The location and ground surface elevation at each test hole location is presented on Drawing PG5885-1 - Test Hole Location Plan in Appendix 2.

3.3 Laboratory Testing

The soil samples recovered from the subject site were examined in the laboratory to review the field logs.

3.4 Analytical Testing

Two soil samples were submitted for analytical testing to determine the concentration of sulphate and chloride, resistivity and pH of the soil. The results are provided in Appendix 1, and are discussed further in Subsection 6.7.

4.0 Observations

4.1 Surface Conditions

The subject site is currently occupied by a commercial building within the southeast portion of the site. The majority of the subject site is asphalt covered with some landscaped areas and the ground surface is gently sloping towards the north (Spruce Street). It should be noted that an overpass is located directly to the south of the subject site. Also, the subject site is at grade with City Centre Avenue and approximately 3 to 4 m below Somerset Street West.

4.2 Subsurface Profile

Generally, the subsurface profile at the borehole locations consists of a pavement structure with varying granular fill material overlying a peat layer, silty sand and a deposit glacial till. The glacial till consists of dense silty sand with gravel, cobbles and boulders. The fill material varied between a silty sand to a clayey silt with organics, debris, gravel, cobbles and boulders. The glacial till deposit is underlain by an interbedded limestone and shale bedrock. Bedrock and/or practical refusal to augering/advancement of the shovel was encountered at the majority of the test hole locations at depths between 3.3 m to 4.4 m.

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and shale from the Verulam Formation. The overburden thickness is estimated to be between 3 to 10 m depth.

East Bridge Abutment

The current geotechnical investigation completed in April 2022 , included three (3) test pits which extended against the concrete abutment of the Somerset Street West bridge to confirm details of the foundation and evaluate the founding conditions of the structure. Our findings are presented below and photographs of each test pit are presented in Appendix 2.

Test Pit - TP 1-22

The concrete abutment of Somerset bridge observed at the test hole location extended to a depth of 1.15 m below existing ground surface. The concrete abutment was founded by a 0.68 m thick footing extending 0.31m beyond the face of the wall. It should be further noted that the upper portion of the footing was observed to tapered at an approximate 45 degree angle.

The concrete footing was observed at a depth of 1.15 m below existing ground surface and bearing on a well compacted, engineered granular fill pad over a native glacial till deposit. It should be further noted that test pit was relocated approximately 2 m from the concrete retaining wall and further extended to a depth of 4.35 m and terminated within a mixture of dense silty sand and gravel (glacial till).

Test Pit TP 2-22

The concrete abutment of Somerset bridge observed at the test hole location extended to a depth of 1.54 m below existing ground surface. The concrete abutment was founded by a 0.68 m thick footing extending 0.32 m beyond the face of the wall. It should be further noted that the upper portion of the footing was observed to tapered at an approximate 45 degree angle.

The concrete footing was observed at a depth of 1.54 m below existing ground surface and bearing on a well compacted, engineered granular fill pad over a native glacial till deposit. TP 2-22 was relocated approximately 2 m from the concrete retaining wall and further extended to a depth of 3.55 m and terminated within a mixture of dense silty sand and gravel (glacial till).

Test Pit - TP 3-22

The concrete abutment of Somerset bridge observed at the test hole location extended to a depth of 2.69 m below existing ground surface. The concrete abutment was founded by a 0.61 m thick footing extending 0.33 m beyond the face of the wall. It should be further noted that the upper portion of the footing was observed to tapered at an approximate 45 degree angle.

The concrete footing was observed at a depth of 2.69 m below existing ground surface and bearing on a well compacted, engineered granular fill pad over a native glacial till deposit. The TP3-22 was relocated approximate 2 m from the concrete retaining wall and further extended to a depth of 4.42 m and terminated on bedrock. The inferred bedrock surface was encountered at elevation 53.4 m.

Test Pit - TP 1-19

The concrete abutment of Somerset bridge observed at the test hole location extended to a depth of 2.15 m below existing ground surface. The concrete abutment was founded by a 0.65 m thick footing extending 0.45 m beyond the face of the wall. It should be further noted that the upper portion of the footing was observed to tapered at an approximate 45 degree angle.

The concrete footing was observed at a depth of 2.15 m below existing ground surface and bearing on a well compacted, engineered granular fill pad over a native glacial till deposit. It should be further noted that TP1-19 was relocated approximately 3 m from the concrete retaining wall and further extended to a depth of 4.1 m and terminated within a mixture of dense silty sand and gravel (glacial till).

The inferred bedrock surface was encountered at geodetic elevation at 53.2 and 52.9 m at the boreholes nearest the Somerset Bridge within the 989 Somerset Street West property. A very dense to dense, glacial till deposit was found overlying the bedrock. The east bridge abutment is expected to be founded upon a dense glacial till or bedrock bearing surface or dense fill placed over the above noted bearing medium.

4.3 Groundwater

The groundwater level (GWL) readings are presented in Table 1. It is important to note that groundwater level readings could be influenced by surface water infiltrating the backfilled boreholes. Groundwater conditions can also be estimated based on the observed colour, moisture levels and consistency of the recovered soil samples. Based on these observations, it is estimated that the long-term groundwater level can be expected between 2.5 to 3.5 m below existing ground surface. It should be noted that groundwater levels are subject to seasonal fluctuations and therefore levels could differ at the time of construction.

Table 1
Summary of Groundwater Levels

Test Hole Number	Ground Surface Elevation (m)	Measured Groundwater Level (m)	Groundwater Elevation (m)	Recording Date
BH 5	55.85	1.80	54.05	April 16, 2002
BH 7	55.90	2.80	53.10	Dec 24, 2002
BH 8	56.81	3.45	53.36	Dec 24, 2002
BH 9	56.50	2.60	53.90	Dec 24, 2002
BH 10	55.73	Dry	--	Dec 24, 2002
BH 11	56.06	Dry	--	Dec 24, 2002
BH 14	55.85	2.60	53.25	Jan 16, 2004
BH 15	56.86	Dry	--	Jan 16, 2004
BH1-14	55.90	3.31	52.59	April 9, 2014
BH 1-19	56.53	3.35	53.18	December 9, 2019
BH 2-19	56.33	3.30	53.03	December 9, 2019
BH 3-19	55.74	3.70	52.04	December 9, 2019
TP1-22*	56.35	3.10	53.25	April 11, 2022
TP2-22*	56.38	2.80	53.58	April 11, 2022
TP3-22*	57.82	3.30	54.52	April 11, 2022

Notes: "*"- Groundwater levels in test pits considered as depth which minor sidewall infiltration had been observed in the open pit at the time of excavation

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is adequate for the proposed multi-storey development. The proposed buildings are expected to be founded on conventional footings placed on clean, surface sounded bedrock.

Due to the absence of a sensitive silty clay deposit, no permissible grade raise restrictions are required for the subject site.

Bedrock removal will be required to complete the underground parking to the current proposed founding depth. Line drilling and controlled blasting where large quantities of bedrock need to be removed is recommended. The blasting operations should be planned and completed under the guidance of a professional engineer with experience in blasting operations.

It should be noted that an overpass is located directly to the south of the subject site.

5.2 Site Grading and Preparation

Stripping Depth

Due to the relatively shallow bedrock depth and the anticipated founding level for the proposed building, all existing overburden material should be excavated from within the proposed building(s) footprint. Bedrock removal should be required for the construction of the underground parking levels.

Bedrock Removal

Based on the bedrock depth, line-drilling in conjunction with hoe-ramming or controlled blasting is expected to remove the bedrock. In areas of weathered bedrock and where only a small quantity of bedrock could be excavated with only hoe-ramming.

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

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

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

Vibration Considerations

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

The following construction equipments could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles or sheet piling will require the above equipment. Vibrations, caused by blasting or construction operations could cause detrimental affects on the adjoining buildings and structures. Therefore, all vibrations are recommended to be limited.

Two parameters determine the recommended vibration limit, the maximum peak particle velocity and the frequency. For low frequency vibrations, the maximum allowable peak particle velocity (PPV) 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 there are several sensitive buildings in close proximity to the subject site, consideration to lowering the PPV is recommended. The guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Horizontal Rock Anchors

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

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

Fill Placement

Fill placed for grading beneath the building area should consist, unless otherwise specified, of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. The fill material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick lifts and compacted to 98% of the material's standard Proctor maximum dry density (SPMDD).

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

Fill used for grading beneath the base and subbase layers of paved areas should consist, unless otherwise specified, of clean imported granular fill, such as OPSS Granular A, Granular B Type II or select subgrade material. This material should be tested and approved prior to delivery to the site. The fill should be placed in lifts no greater than 300 mm thick and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the paved areas should be compacted to at least 95% of its SPMDD.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on a clean, surface sounded limestone bedrock surface could be designed for a factored bearing resistance value at ultimate limit states (ULS) of **3,000 kPa**, incorporating a geotechnical resistance factor of 0.5.

A clean, surface-sounded bedrock bearing surface should be free of loose materials, and should not contain surface seams, voids, fissures or open joints which can be detected from surface sounding with a rock hammer.

Settlement

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

5.4 Design for Earthquakes

The site class for seismic site response is **Class C** for the shallow foundations at the subject site. A higher seismic site class, such as Class A or B is available for the subject site. However, a site specific seismic shear wave test is required to provide the higher site classes according to the 2012 Ontario Building Code.

5.5 Basement Slab

The native subsurface, approved granular fill or lean concrete mudslab will be considered to be an acceptable subgrade on which to commence backfilling for floor slab construction.

The upper 200 mm of sub-slab fill is recommended to consist of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

Any soft areas should be removed and backfilled with appropriate backfill material prior to placing any additional backfill. 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(s) should be placed in maximum 300 mm thick loose layers and compacted to a minimum of 98% of the SPMDD.

5.6 Basement Wall

There are several combinations of backfill and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions could be well-represented by assuming the retained soil consist of a material with an angle of internal friction of 30 degrees and a dry unit weight of 20 kN/m^3. A portion of the basement walls are expected to be poured against a composite drainage blanket, which will be placed against the exposed bedrock face. A nominal coefficient of at-rest earth pressure of 0.05 is recommended in conjunction with a dry unit weight of 23.5 kN/m^3 (effective unit weight of 15.5 kN/m^3). A seismic earth pressure component will not be applicable for the foundation wall, which is to be poured against the bedrock face. The seismic earth pressure is expected to be transferred to the underground floor slabs, which should be designed to accommodate the pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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

Static Conditions

The static horizontal earth pressure (p_o) could be calculated with 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 with a magnitude equal to $K_o \cdot q$ and acting on the entire wall height should be added to the above diagram for any surcharge loading, q (kPa), that may be placed at ground surface adjacent to the wall. The surcharge pressure will only be applicable for static analyses and should not be calculated in conjunction with the seismic loading case.

Actual earth pressures could be higher than the "at-rest" case if care is not exercised during the backfill compaction. A minimum separation of 0.3 m from any wall with the compaction equipment should be provided.

Seismic Conditions

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 = $(1.45\text{-}a_{max}/g)a_{max}$
γ = unit weight 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 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 rock cone pullout with the cone apex near the middle of the anchor bonded length. Interaction may develop between the anchor failure cones that are relatively close to one another resulting in a total group capacity smaller than the sum of each individual anchor capacity.

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

Permanent anchors should be provided with corrosion protection. As a minimum, the entire drill hole should be filled with cementious grout. The free anchor length is provided by installing a plastic sleeve to act as a bond break, with the sleeve filled with grout or a corrosion inhibiting mastic. Double corrosion protection can be provided with factory assembled systems from Dywidag Systems International or Williams Form Engineering Corporation. Recognizing the importance of the anchors for the proposed building(s) long term foundation performance, 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 limetsone ranges between about 50 and 80 MPa, which is stronger than most routine grouts. A factored tensile grout to rock bond resistance value at ULS of **1.0 MPa**, incorporating a resistance factor of 0.4, can be calculated. A minimum grout strength of 40 MPa is recommended.

Rock Cone Uplift

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

Recommended Grouted Rock Anchor Lengths

The parameters calculated for grouted rock anchor lengths are provided in Table 2.

Table 2 - Parameters used in Rock Anchor Review	
Grout to Rock Bond Strength - Factored at ULS	1.0 MPa
Compressive Strength - Grout	40 MPa
Rock Mass Rating (RMR) - Good quality Limestone Hoek and Brown parameters	69 m=0.575 and s=0.00293
Unconfined compressive strength - Limestone	60 MPa
Effective unit weight - Bedrock	15 kN/m^3
Apex angle of failure cone	60^o
Apex of failure cone	mid-point of fixed anchor length

The fixed anchor length will depend on the drill hole diameter. The recommended anchor lengths are provided in Table 3. The factored tensile resistance values provided are based on a single anchor. The group influence effects has not been accounted for in the calculations below.

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	2.1	1	3.1	500
	3.2	1	4	750
	4	1	5	1000
	5.7	1	6.7	1500
125	1.4	1.3	2.7	500
	1.9	1.5	3.4	750
	2.6	1.5	4.1	1000
	3.9	1.5	5.4	1500

Other considerations

The anchor drill holes should be a maximum of 1.5 to 2 times the rock anchor 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 hole. 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 proof testing can be provided upon request. Compressive strength testing is recommended to be completed for the rock anchor grout. A set of grout cubes should be tested for each day grout is prepared.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

A perimeter foundation drainage system is recommended to be provided for the proposed structure. Insufficient room is expected to be available for exterior backfill. The system should consist of the following:

❏ A waterproofing membrane could be applied to the prepared vertical bedrock surface and shoring system above the bedrock face (from the founding level and up to 1 m above the estimated long-term groundwater table) to limit long-term groundwater infiltration to be handled by the building's sump pits. The membrane will serve as a water infiltration suppression system.

❏ Composite drainage layer will be placed from the surface to the proposed founding elevation.

The composite drainage system (such as Miradrain G100N or equivalent) is recommended to extend to the footing level. Sleeves, 150 mm diameter, at 3 m centres are recommended to be placed in the footing or at the foundation wall/footing interface to allow the water infiltration to the interior perimeter drainage pipe. The perimeter drainage pipe and underfloor drainage system should direct water to sump pit(s) within the lower basement area.

Underfloor Drainage

Underfloor drainage is recommend to control water infiltration due to groundwater infiltration at the proposed founding elevation. For design purposes, Paterson recommends a 150 mm in diameter perforated pipes be placed at 6 m centres. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be more accurately assessed.

Adverse Effects of Dewatering on Adjacent Properties

Any minor dewatering will be within the bedrock layer which is considered relatively shallow at the subject site. Therefore, adverse effects to the surrounding buildings or properties are not expected with the lowering of the groundwater in this area.

Foundation Backfill

Above the bedrock surface, backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. The greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for placement as backfill against the foundation walls, unless placed in conjunction with a drainage geocomposite, such as Miradrain G100N or Delta Drain 6000, connected to the perimeter foundation drainage system. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should otherwise be used for this purpose.

6.2 Protection of Footings Against Frost Action

The parking garage is expected to not require protection against frost action due to the founding depth. Unheated structures such as the access ramp may required to be insulated against the deleterious effect of frost action.

Perimeter footings of heated structures are required to be insulated against the deleterious effects of frost action. A minimum of 1.5 m of soil cover alone, or a minimum of 0.6 m of soil cover, in conjunction with adequate foundation insulation, should be provided. More details regarding foundation insulation can be provided, if requested.

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

6.3 Excavation

Temporary Side Slopes

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

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. A shallower slope is required for excavation below groundwater level. The subsurface soil is considered to be mainly 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 be installed at all times 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 exposed for extended periods of time.

In bedrock, almost vertical side slopes can be used provided that all loose rock and blocks with unfavourable weak planes are removed or stabilized with rock anchors.

Temporary Shoring

Temporary shoring may be required for the overburden soil to complete the required excavations 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 implementation of these temporary systems will be the responsibility of the excavation contractor and their design team. Inspections and approval of the temporary system will also be the responsibility of the designer. Geotechnical information provided below is to assist the designer in completing a suitable and safe shoring system. The designer should take into account the 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. These systems could 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. The shoring system is recommended to be adequately supported to resist toe failure and inspected to ensure that the sheet piles extend well below the excavation base. It should be noted if consideration is being given to utilizing a raker style support for the shoring system that lateral movements can occur and the structural engineer should ensure that the design selected minimizes these movements to tolerable levels.

The earth pressures acting on the 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
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 no movement is permissible. The dry unit weight should be calculated above the groundwater level while the effective unit weight should be calculated below the groundwater level.

The hydrostatic groundwater pressure should be included to the earth pressure distribution wherever the effective unit weight are calculated for earth pressures. If the groundwater level is lowered, the dry unit weight for the soil/bedrock should be calculated 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. If the bedding is placed on bedrock, the thickness of the bedding should be increased to 300 mm for sewer pipes. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to a minimum of 300 mm above the obvert of the pipe should consist of OPSS Granular A (concrete or PSM PVC pipes) or sand (concrete pipe). The bedding and cover materials should be placed in maximum 225 mm thick lifts and compacted to 95% of the SPMDD.

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

6.5 Groundwater Control

Groundwater Control for Building Construction

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.

Long-term Groundwater Control

The recommendations for the proposed building(s) long-term groundwater control are presented in Subsection 6.1. Any groundwater encountered along the building(s) perimeter or sub-slab drainage system will be directed to the proposed building(s) cistern/sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, the groundwater flow should be low (i.e.- less than 20,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided at the time of construction, once groundwater infiltration levels are observed. The groundwater flow should be controllable using conventional open sumps.

Impacts on Neighbouring Structures

Based on observations, the groundwater level is anticipated within the bedrock. A local groundwater lowering is anticipated under short-term conditions due to construction of the proposed building. The extent of any significant groundwater lowering should occur within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded within native glacial till and/or directly over a bedrock bearing surface. No issues are expected, with respect to groundwater lowering, that would cause long term damage to adjacent structures surrounding the proposed building.

6.6 Winter Construction

Precautions should be provided if winter construction is considered for this project.

The subsurface conditions mainly consist of frost susceptible materials. In presence of water and freezing conditions, ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the installation of straw, propane heaters, tarpaulins or other suitable means. Any excavation base should be insulated from sub-zero temperatures immediately upon exposure and until such time as heat is adequately supplied to the building and the footings are protected with sufficient soil cover to prevent freezing at founding level.

The trench excavations should be constructed in a manner to avoid the introduction of frozen materials, snow or ice into the trenches.

6.7 Corrosion Potential and Sulphate

The analytical test results indicate the sulphate content is less than 0.1%. The result is indicative that Type 10 Portland cement. The chloride content and the pH of the sample are indicative of non significant factors in creating a corrosive environment for exposed ferrous metals, whereas the resistivity is indicative of an aggressive to highly aggressive corrosive environment.

6.8 Impacts on Somerset Bridge and Monitoring Action Plan

Based on our review of available information, the existing bridge abutment is located in close proximity to the south property boundary of the subject site. To ensure that we are aware of the current bridge construction, Paterson reviewed the following design drawings for the adjacent bridge as part of the geotechnical assessment:

- Drawing No. B-053408 - Sheet 1 of 5 - Site Plan and Profile I - Somerset Street Bridges Renovations - prepared by The Regional Municipality of Ottawa - Carleton - Transportation Department, dated February 1976.
- Drawing No. B-053408-4 - Sheet 4 of 5 - Site Plan and Profile I - Somerset Street Bridges Renovations - prepared by The Regional Municipality of Ottawa - Carleton - Transportation Department, Revision 1 dated January 26, 1977.
- Contract No. 76-532 - Drawing No. B-053411-1- Site Plan and Profile I - Somerset Street Bridges Renovations - prepared by The Regional Municipality of Ottawa - Carleton - Transportation Department, dated February 1976.

Assessment Summary

Due to the close proximity of east bridge abutment footing to the subject site's property line and the soil and bedrock conditions observed, it is recommended that an underpinning program be completed to extend the existing bridge abutment footing to bedrock by means of a series of lean concrete in-filled, narrow excavations (ie.- less than 1.5 m) extended to bedrock. The narrow sections would be excavated in a sequential order and in-filled with concrete to ensure that no significant movement of the bridge structure will occur. Once details of the proposed building are finalized, it is recommended that details of the underpinning program be prepared by a structural engineer specialized in underpinning design. Based on our observations, the long-term groundwater level is located within the bedrock, so any groundwater lowering (if any) will not negatively impact the existing bridge structure and the proposed underpinning. Alternatively, a temporary shoring system could be designed to support the existing bridge abutment structure.

Construction Monitoring: A monitoring program should be implemented during excavation, bedrock removal and installation of the shoring system and/or underpinning panels. This will allow the construction crew to have a live feed of the vibrations and immediate alert system to stop any construction activities the vibrations exceed the recommended threshold.

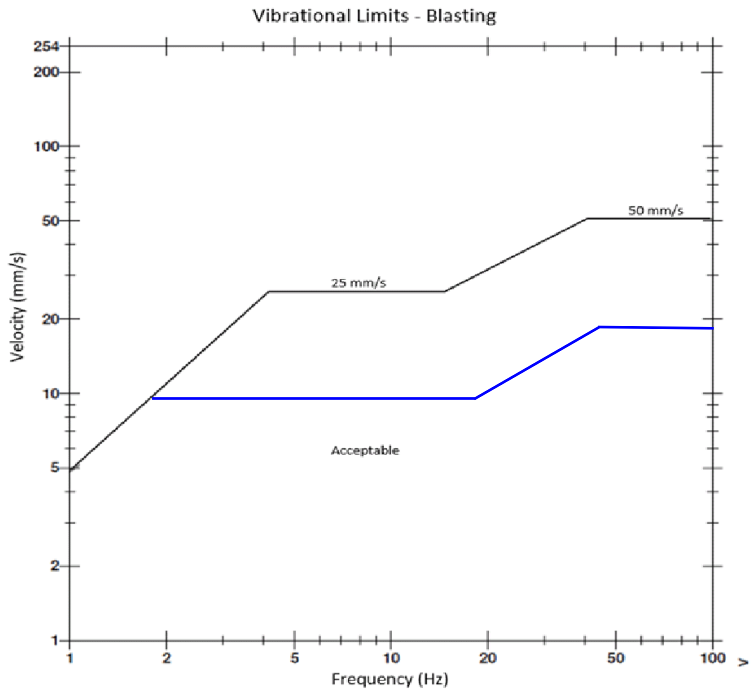
- *Deflection Monitoring*: It is recommended that 3 deflection monitoring points be installed on top of the east abutment, adjacent to the guard rail, to monitor horizontal and vertical deflection of the abutment. The deflection monitoring points will be monitored weekly until the foundation extends above the exterior finished grade.

A review level of 10 to 14 mm of deflection will require an assessment. An alert level of greater than 15 mm will required immediate attention and possible mitigation measures as noted in Table 5.

Table 5 - Settlement/Displacement Criteria & Associated Actions

Displacement Value	Description of Event	Contractor Required Action
Up to 10 mm	*Allowable Level*	- Work may continue, no action required.
10 to 14 mm	*Review Limit*	- Immediately notify all relevant emergency contact parties within 24 hours of the survey. - Complete an additional survey of all monitoring points for confirmation of the readings. Give verbal notification of the results to the Contract Administrator within 1 hour of the additional survey and a written report within 24 hours. - Review the potential cause of the displacement and adjust the monitoring program as required. - Work may continue, however the contractor should give consideration to adjusting construction activities accordingly to minimize potential further movement.
Over 15 mm	*Alert Limit*	- Stop excavation work immediately. - Immediately notify all relevant emergency contact parties within 2 hours of the survey. - Complete an additional survey of all monitoring points for confirmation of the readings. Give verbal notification of the results to the Contract Administrator within 1 hour of the additional survey and a written report within 24 hours. - Complete a geotechnical review of the site within 12 hours to identify any obvious visual indications of ground subsidence, movement, sink holes, etc. - Complete a structural review of the affected structure(s) within 12 hours. - Notify all relevant emergency contact parties of the additional survey, geotechnical, and structural reviews within 24 hours. - Coordinate a meeting with the owner, construction manager, and all relevant emergency contact parties to discuss the results, mitigative actions, and plan for moving forward. - Construction work shall not begin until the meeting group has reached a conclusion and appropriate actions are implemented.

- ❑ ***Vibration Monitoring:*** Vibration levels at the south boundary of the site will also be continuously monitored during the excavation and blasting programs. The vibration levels would be monitored using 2 vibration monitors installed at the site boundary. Refer to Figure 1 below for proposed vibration limits.



Blue line designates vibration levels, which require notification of all parties, if exceeded.

Figure 1 - Proposed Vibration Limits at the East Abutment fo the Somerset Street Bridge.

If the vibrations are observed to exceed the warning level event (Blue Line in the above chart), the contractor will be notified and a field assessment will be completed to prevent any exceedances from occurring.

- ❑ If the recommended vibration limit is exceeded (Black Line in the above chart), Paterson will notify the site superintendent and operation will be stopped, and the deflection monitoring points will be surveyed. Weekly reporting of the monitoring program and recommendations will be provided to the owner and the City of Ottawa. For warning/exceedance level events, please refer to the Vibration Response Action Plan included on the following page.
- ❑ The contractor should implement mitigation measures for future excavation of any construction activities as necessary and provide updates on the effectiveness of the improvement. Response actions should be pre-determined prior to excavation, depending on the approach provided to protect elements. Processes and procedures should be in-place prior to completing any activities, which cause vibrations to identify issues and react in a quick manner in the event of an exceedance.

Vibration Response Action Plan

No Exceedances:

1 – An automatic emailer, set up by Paterson, will provide a report of vibration events on a daily basis to all stakeholders. The report(s) will include the following:

- Measure Vibration Levels
- Distance from the construction activity to the monitoring location
- Vibration Type

2 - Any vibrations reaching a warning level or exceeding the specified threshold will be assessed by Paterson as follows:

Warning Level Event

3 – Paterson will review all vibrations over the established warning level which is bound by the blue line in Figure 1 below.

4 – Paterson will notify the contractor if any vibrations occur due to the construction activities and are close to the exceedance level.

5 – A site visit may be completed to review the field vibrations and provide an action plan to mitigate the vibration levels to prevent any foreseen damage.

Exceedance Level Event:

6 – Paterson will notify all the relevant stakeholders via email if any vibrations surpass the exceedance level, illustrated by the black line in Figure 1. Site activities will be requested to be stopped until an assessment is completed.

7 – Complete a site visit to ensure the monitors are functioning well and review the condition of the bridge abutment.

8 – Issue the vibration exceedance result(s) along with an assessment report completed by a senior project manager.

Exceedance Reports will Include:

9 – Identify the location of vibration exceedance.

10 – The date, time and nature of the exceedance and/or incident.

11 – Purpose of the exceeded monitor and current vibration criteria.

12 – Identify the cause of the exceedance/incident

13 – Describe the initial response action that has been completed to date.

14 – Describe the proposed measures to address the exceedance/incident and provide an immediate action plan and prevention measures to eliminate the cause of the exceeded vibration(s) during future work.

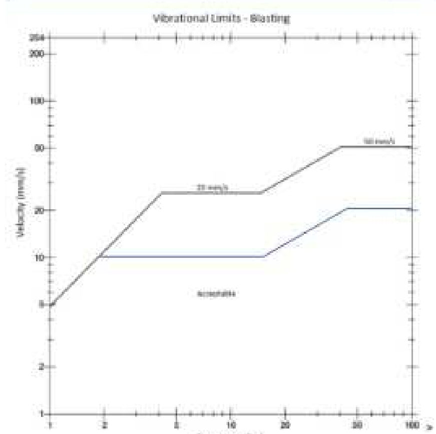


Figure 1- Proposed Vibration Limits at East Bridge Abutment

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7.0 Recommendations

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

- ❏ Review the bedrock stabilization and excavation requirements.
- ❏ Observation of all bearing surfaces prior to the placement of concrete.
- ❏ Sampling and testing of the concrete and fill materials used.
- ❏ 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 in the report are in accordance with Paterson's present understanding of the project. Paterson request 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 the test locations, Paterson requests immediate notification to permit reassessment of the recommendations.

The recommendations provided should only be used by the design professionals associated with this project. The recommendations are not intended for contractors bidding on or constructing the project. The latter should evaluate the factual information provided in the report. The contractor should also determine the suitability and completeness for the intended construction schedule and methods. Additional testing may be required for the contractors purpose.

The present report applies only to the project described in the report. The use of the report for purposes other than those described above or by person(s) other than Taggart Realty Management or their agents is not authorized without review by Paterson.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
May 26, 2022
D. J. GILBERT
100116130
PROVINCE OF ONTARIO

Faisal I. Abou-Seido, P.Eng

David J. Gilbert, P.Eng.

Report Distribution:

- ❑ Taggart Realty Management (3 copies)
- ❑ Paterson Group (1 copy)

APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS

SYMBOLS AND TERMS

ANALYTICAL RESULTS

VIBRATION RESPONSE ACTION PLAN

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

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 989 Somerset St. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE April 11, 2022

FILE NO. PG5885

HOLE NO. TP 1-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	56.35		
FILL: Topsoil with brown silty sand, crushed stone and gravel (0.30)		G	1						
FILL: Crushed stone - 0.25m thick concrete slab under crushed stone layer (0.95)		G	2			1	55.35		
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders (3.10)		G	3			2	54.35		
GLACIAL TILL: Grey silty clay to clayey silt with gravel and cobbles (4.35)		G	4			3 4	53.35 52.35		∇
End of Test Pit (Groundwater infiltration at 3.1m depth)									

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
Prop. Multi-Storey Building - 989 Somerset St. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE April 11, 2022

FILE NO. PG5885

HOLE NO. TP 2-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20 40 60 80)	Piezometer Construction
GROUND SURFACE						0	56.38		
FILL: Topsoil with brown silty sand		G	1						
0.90									
GLACIAL TILL: Dense, brown silty sand with gravel, cobbles and boulders		G	2			1	55.38		
						2	54.38		
		G	3						▽
						3	53.38		
3.40									
GLACIAL TILL: Grey silty clay to clayey silt with gravel and cobbles		G	4						
3.55									
End of Test Pit Practical refusal to excavation on glacial till at 3.55m depth. (Groundwater infiltration at 2.8m depth)									

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

Geotechnical Investigation
Prop. Multi-Storey Building - 989 Somerset St. West
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY Backhoe

DATE April 11, 2022

FILE NO. PG5885

HOLE NO. TP 3-22

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE						0	57.82		
FILL: Topsoil with crushed stone		G	1						
1.10						1	56.82		
GLACIAL TILL: Dense brown silty sand with gravel, cobbles and boulders		G	2			2	55.82		
		G	3			3	54.82		▽
		G	4			4	53.82		
4.42									

End of Test Pit

TP terminated on bedrock surface at 4.42m depth.

(Groundwater infiltration at 3.3m depth)

Pen. Resist. scale: 20 40 60 80

Shear Strength (kPa): 20 40 60 80 100

▲ Undisturbed △ Remoulded

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 & 160 Spruce Street W. and 989 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 December 5

FILE NO. PG5214

HOLE NO. BH 1-19

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.05	AU	1		
FILL: Brown sand with gravel, some silt, occasional asphalt pieces	2.29	SS	2	17	6
		SS	3	12	50+
Very dense, brown **SILTY SAND-GRAVEL**	3.45	SS	4	58	84
		SS	5	86	50+
Possible weathered **BEDROCK**	4.10				
BEDROCK: Good to excellent quality, black shale interbedded with limestone	5.74	RC	1	91	78
BEDROCK: Grey llimestone with shale partings	10.21	RC	2	98	98
		RC	3	100	75
		RC	4	100	86

End of Borehole

(GWL @ 3.35m - Dec. 9, 2019)

DEPTH (m)	ELEV. (m)
0	56.53
1	55.53
2	54.53
3	53.53
4	52.53
5	51.53
6	50.53
7	49.53
8	48.53
9	47.53
10	46.53

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

○ Water Content %

20 40 60 80

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

Monitoring Well Construction

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 & 160 Spruce Street W. and 989 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 December 5

FILE NO. PG5214

HOLE NO. BH 2-19

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
FILL: Brown sand, some gravel	0.30	AU	1		
FILL: Dark brown sand with gravel	1.52	SS	2	25	11
Dense to very dense, brown **SILTY SAND-GRAVEL**	3.05	SS	3	67	46
		SS	4	41	50+
Brown **SILTY CLAY** with sand and gravel	3.20	SS	5	62	76
Very dense, brown **SILTY SAND** with gravel, occasional cobbles	3.79				
BEDROCK: Poor to excellent quality, black shale interbedded with limestone		RC	1	100	38
		RC	2	93	45
- limestone content increasing with depth	6.61	RC	3	94	94
End of Borehole					
(GWL @ 3.30m - Dec. 9, 2019)					

DEPTH (m)	ELEV. (m)
0	56.33
1	55.33
2	54.33
3	53.33
4	52.33
5	51.33
6	50.33

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 & 160 Spruce Street W. and 989 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

REMARKS

BORINGS BY CME 55 Power Auger

DATE 2019 December 5

FILE NO. PG5214

HOLE NO. BH 3-19

SOIL DESCRIPTION	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE					0	55.74
FILL: Brown sand with gravel, occasional concrete pieces	AU	1				
	SS	2	33	10	1	54.74
1.52						
Very dense, brown **SILTY SAND-GRAVEL** with cobbles, possible boulders	SS	3	60	50+		
					2	53.74
	SS	4	50	50+		
	SS	5	89	50+	3	52.74
	SS	6	100	50+	4	51.74
4.65	SS	7	0	50+		
Possible weathered **BEDROCK**					5	50.74
5.13						

End of Borehole

Practical refusal to augering at 5.13m depth.

(GWL @ 3.70m - Dec. 9, 2019)

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

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154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigation
158 & 160 Spruce Street W. and 989 Somerset Street W.
Ottawa, Ontario

DATUM Geodetic

FILE NO. PG5214

REMARKS

HOLE NO. TP 1-19

BORINGS BY Backhoe

DATE 2019 December 5

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Monitoring Well Construction
GROUND SURFACE						0	57.17		
TOPSOIL (0.25)		G	1						
FILL: Brown sand with gravel and cobbles (2.20)		G	2			1	56.17		
						2	55.17		
Brown **SILTY SAND-GRAVEL** (4.10)		G	3			3	54.17		
		G	4			4	53.17		
End of Test Pit									

End of Test Pit

Concrete grade beam extended to a depth of 1.5m which in turn was overlying a 0.65m thick footing extending 0.45m beyond the face of the concrete grade beam.

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

Geotechnical Investigation
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 54.98m, as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE 4 April 2014

FILE NO. PG3158

HOLE NO. BH 1-14

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	55.90
Asphaltic concrete 0.08							
FILL: Crushed stone 0.15							
FILL: Brown silty sand, trace clay, gravel and wood 2.13		SS	1		6	1	54.90
FILL: Boulders with sand 3.96		SS	2		50+	2	53.90
						3	52.90
BEDROCK: Grey limestone 6.96		RC	1	80	0	4	51.90
		RC	2	93	75	5	50.90
		RC	3	98	88	6	49.90
End of Borehole							
(GWL @ 3.31m-April 9, 2014)							

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, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigaton
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m.

REMARKS

BORINGS BY CME 75 Power Auger

DATE Apr 10, 02

FILE NO. G8584

HOLE NO. BH 5

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	55.85
Asphaltic concrete	0.04						
FILL: Silty sand with gravel	0.76						
PEAT - grey marl by 1.2m depth	1.37	SS	27	58	8	1	54.85
Dense, brown-grey fine to coarse **SAND** with gravel		SS	28	75	48	2	53.85
		SS	29	62	42	3	52.85
- brown by 3.0m depth		SS	30	62	45		
- brown-grey by 3.8m depth		SS	31	100	50+	4	51.85
- black by 4.6m depth	5.18	SS	32	92	50+	5	50.85
End of Borehole							
(GWL @ 1.88m-April 16/02)							

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

patersongroup Consulting Engineers

154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigaton
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m.

REMARKS

BORINGS BY CME 45 Power Auger

DATE Dec 18, 02

FILE NO. G8584

HOLE NO. BH 7

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	55.90
Asphaltic concrete	0.05						
FILL: Dark brown silty sand with crushed gravel	0.28	AU	1				
FILL: Brown clayey silt with fine sand and gravel	1.22	SS	2	46	10	1	54.90
PEAT: Dark brown organic matter	1.58						
White **MARL**	2.21	SS	3	100	1	2	53.90
GLACIAL TILL: Very dense, brown silty fine to coarse sand with gravel, cobbles and boulders	3.05	SS	4	75	82	3	52.90

End of Borehole

Practical refusal to augering @ 3.05m depth

(GWL @ 2.80m-Dec. 24/02)

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, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigaton
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m.

FILE NO. **G8584**

REMARKS

HOLE NO. **BH 8**

BORINGS BY CME 45 Power Auger **DATE** Dec 18, 02

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content % (20, 40, 60, 80)	Piezometer Construction
GROUND SURFACE						0	56.81		
Asphaltic concrete — 0.08									
FILL: Grey crushed stone — 0.18									
FILL: Brown silty fine sand with gravel and concrete pieces		SS	5	17	50+				
						1	55.81		
— 1.88		SS	6	67	22				
Compact to very dense, brown **SANDY SILT** with gravel						2	54.81		
— 2.48		SS	7	62	50+				
GLACIAL TILL: Very dense, brown silty fine to coarse sand with gravel, cobbles and boulders						3	53.81		
		SS	8	76	85				
— 3.63									
End of Borehole									
Practical refusal to augering @ 3.63m depth									
(GWL @ 3.45m-Dec. 24/02)									

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

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SOIL PROFILE AND TEST DATA

Geotechnical Investigaton
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m.

REMARKS

BORINGS BY CME 45 Power Auger

DATE Dec 18, 02

FILE NO. G8584

HOLE NO. BH 9

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	56.50
25mm Topsoil							
FILL: Brown silty sand with gravel, cobbles, concrete and extruded polystyrene pieces	0.91						
GLACIAL TILL: Very dense, brown silty fine to coarse sand with gravel, cobbles and boulders		SS	9	72	80+	1	55.50
		SS	10	71	66	2	54.50
		SS	11	71	89+		
- grey silty fine sand by 2.9m depth	3.45	SS	12	100	50+	3	53.50
End of Borehole							
Practical refusal to augering @ 3.45m depth							
(GWL @ 2.60m-Dec. 24/02)							

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, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigaton
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m.

FILE NO. G8584

REMARKS

HOLE NO. BH10

BORINGS BY CME 45 Power Auger

DATE Dec 18, 02

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	55.73
Asphaltic concrete	0.05	AU	13				
FILL: Dark brown silty fine sand with crushed gravel and brick fragments	1.07	SS	14	17	15	1	54.73
FILL: Dark brown organic matter over grey silty fine sand with gravel	1.98	SS	15	33	6	2	53.73
PEAT: Dark brown organic matter	2.59	SS	16	54	22		
GLACIAL TILL: Compact, grey fine to coarse sand with gravel, cobbles and boulders	3.00					3	52.73

End of Borehole

Practical refusal to augering @ 3.00m depth

(Dry - Dec. 24/02)

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, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigaton
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m.

FILE NO. G8584

REMARKS

HOLE NO. BH11

BORINGS BY CME 45 Power Auger **DATE** Dec 18, 02

SOIL DESCRIPTION		SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	56.61
Asphaltic concrete	0.05						
FILL: Brown silty fine sand with crushed gravel	0.25	AU	17				
FILL: Brown silty fine sand with gravel, plastic, concrete and asphalt pieces	1.83	SS	18	54	13	1	55.61
PEAT and **MARL**	1.90	SS	19	54	30	2	54.61
GLACIAL TILL: Dense, brown silty fine to coarse sand with gravel, cobbles and boulders	2.64	SS	20	40	50+		

End of Borehole

Practical refusal to augering @ 2.64m depth

(BH dry-Dec. 24/02)

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 II - Environmental Site Assessment
160 Spruce Street / 989 Somerset St.
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m, as provided by Farley, Smith & Murray Surveying Ltd.

REMARKS

BORINGS BY Portable Drill

DATE January 16, 2003

FILE NO. E2387

HOLE NO. BH12

SOIL DESCRIPTION		STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE							0	
Concrete slab	0.08							
FILL: Crushed stone	0.23							
FILL: Dark brown sand	0.33							
			SS	1	25	16		
							1	
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders			SS	2	62	58		
			SS	3	100	152		
	2.03						2	
BEDROCK: Weathered grey limestone	2.06							
End of Borehole								

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
160 Spruce Street / 989 Somerset St.
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m, as provided by Farley, Smith & Murray Surveying Ltd.

REMARKS

BORINGS BY Portable Drill

DATE January 16, 2003

FILE NO. E2387

HOLE NO. BH13

SOIL DESCRIPTION	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Photo Ionization Detector ● Volatile Organic Rdg. (ppm) / ○ Lower Explosive Limit %	Monitoring Well Construction
GROUND SURFACE						0			
Concrete slab 0.11									
Concrete slab 0.25		G	4						
GLACIAL TILL: Very dense, brown silty sand with gravel, cobbles and boulders		G	5						
		SS	6	58	50+				
						1			
		SS	7	88	50+				
1.62		SS	8	25	50+				
End of Borehole									
Practical refusal to augering at 1.62m depth									

LEL scale: 20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
160 Spruce Street / 989 Somerset St.
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m, as provided by Farley, Smith & Murray Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger **DATE** January 16, 2004

FILE NO. E2387

HOLE NO. BH14

SOIL DESCRIPTION	Depth to base (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD
GROUND SURFACE					
Asphaltic concrete	0.08				
FILL: Crushed stone	0.30				
FILL: Dark brown sandy silt with gravel	0.84	AU	1		
Light brown MARL, some sand and shells	2.57	SS	2	30	50+
		SS	3	0	6
		SS	4	46	14
Light brown SANDY SILT, some gravel and cobbles	3.38	SS	5		50+
End of Borehole					

Practical refusal to augering at 3.38m depth

(GWL @ 2.60m-Jan. 19, 2004)

DEPTH (m)	ELEV. (m)
0	55.85
1	54.85
2	53.85
3	52.85

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500

RKI Eagle Rdg. (ppm)

▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road South, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Phase II - Environmental Site Assessment
160 Spruce Street / 989 Somerset St.
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 55.53m, as provided by Farley, Smith & Murray Surveying Ltd.

REMARKS

BORINGS BY CME 55 Power Auger

DATE January 16, 2004

FILE NO. E2387

HOLE NO. BH15

SOIL DESCRIPTION	Strata depth (m)	SAMPLE TYPE	NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)
GROUND SURFACE						0	56.86
Asphaltic concrete	0.08						
FILL: Crushed stone	0.30						
FILL: Brown to dark brown sandy silt, some gravel	0.81	AU	6				
Light brown **SANDY SILT** with gravel		SS	7	8	16	1	55.86
		SS	8	40	50+		
						2	54.86
	2.90	SS	9	50	50+		
End of Borehole							

Practical refusal to augering at 2.90m depth

(BH dry - Jan. 19, 2004)

Photo Ionization Detector
● Volatile Organic Rdg. (ppm)

○ **Lower Explosive Limit %**

20 40 60 80

100 200 300 400 500
RKI Eagle Rdg. (ppm)
▲ Full Gas Resp. △ Methane Elim.

Monitoring Well Construction

patersongroup Consulting Engineers

154 Colonnade Road, Ottawa, Ontario K2E 7J5

SOIL PROFILE AND TEST DATA

Geotechnical Investigaton
160 Spruce Street/989 Somerset Street West
Ottawa, Ontario

DATUM TBM - Top of grate of manhole (see plan). Elevation = 54.98m, as provided by Annis, O'Sullivan, Vollebekk Ltd.

REMARKS

BORINGS BY Backhoe

DATE May 24, 02

FILE NO. G8584

HOLE NO. TP 4

SOIL DESCRIPTION	Depth to base (m)	STRATA PLOT	SAMPLE TYPE	SAMPLE NUMBER	% RECOVERY	N VALUE or RQD	DEPTH (m)	ELEV. (m)	Pen. Resist. Blows/0.3m ● 50 mm Dia. Cone / ○ Water Content %	Piezometer Construction
GROUND SURFACE							0			
Asphaltic concrete	0.05									
FILL: Brown silty sand with gravel	0.80									
Brown **PEAT**	2.60						1			
							2			
Grey **SILTY SAND**, some shells, trace clay	2.80									∇
End of Test Pit (Open hole GWL @ base of TP)										

Water Content % axis: 20 40 60 80

Shear Strength (kPa) axis: 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 strength of cohesionless soils is the relative density, 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.

Relative Density	'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 vane tests, penetrometer tests, unconfined compression tests, or occasionally by Standard Penetration Tests.

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

SOIL DESCRIPTION (continued)

Cohesive soils can also be classified according to their "sensitivity". The sensitivity is the ratio between the undisturbed undrained shear strength and the remoulded undrained shear strength of the soil.

Terminology used for describing soil strata based upon texture, or the proportion of individual particle sizes present is provided on the Textural Soil Classification Chart at the end of this information package.

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 NXL size core. However, it can be used on smaller core sizes, such as BX, 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
PS	-	Piston sample
AU	-	Auger sample or bulk sample
WS	-	Wash sample
RC	-	Rock core sample (Core bit size AXT, BXL, etc.). Rock core samples are obtained with the use of standard diamond drilling bits.

SYMBOLS AND TERMS (continued)

GRAIN SIZE DISTRIBUTION

MC%	-	Natural moisture 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 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 = $(D30)^2 / (D10 \times D60)$
Cu	-	Uniformity coefficient = D60 / D10

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

CONSOLIDATION TEST

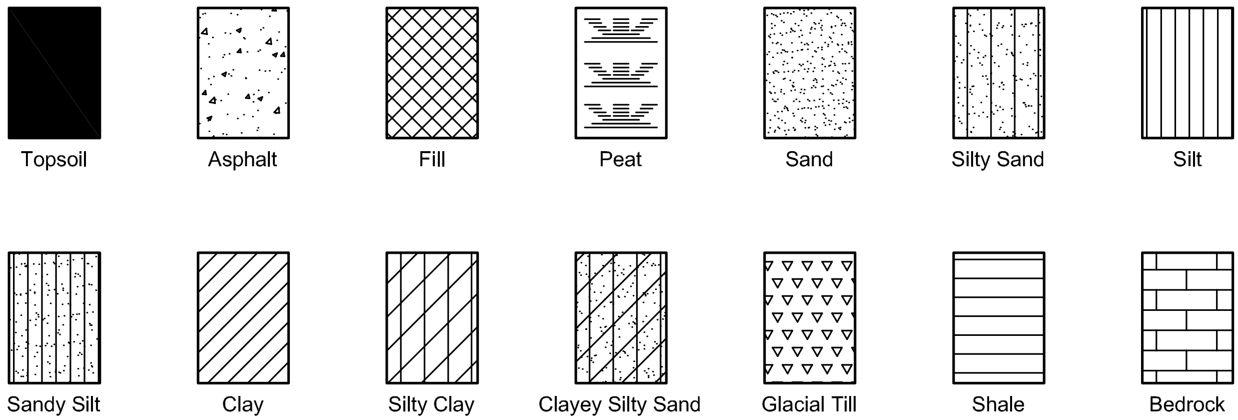
p'_o	-	Present effective overburden pressure at sample depth
p'_c	-	Preconsolidation pressure of (maximum past pressure on) sample
Ccr	-	Recompression index (in effect at pressures below p'_c)
Cc	-	Compression index (in effect at pressures above p'_c)
OC Ratio		Overconsolidaton ratio = p'_c / p'_o
Void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

PERMEABILITY TEST

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

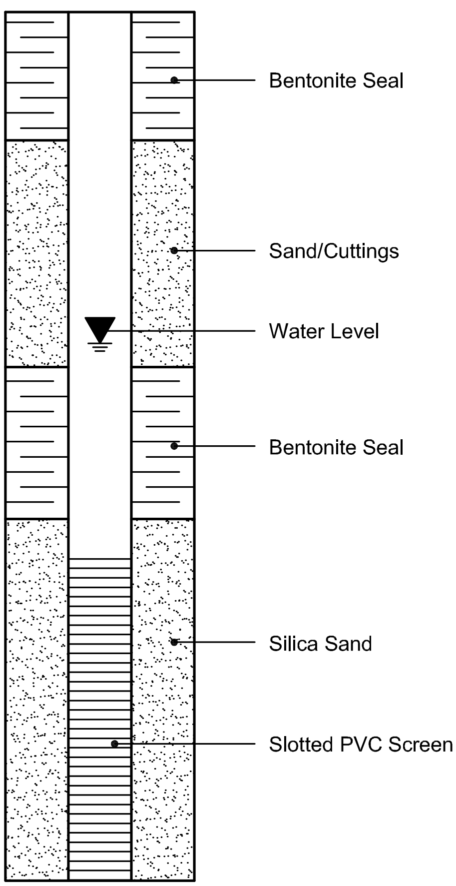
SYMBOLS AND TERMS (continued)

STRATA PLOT

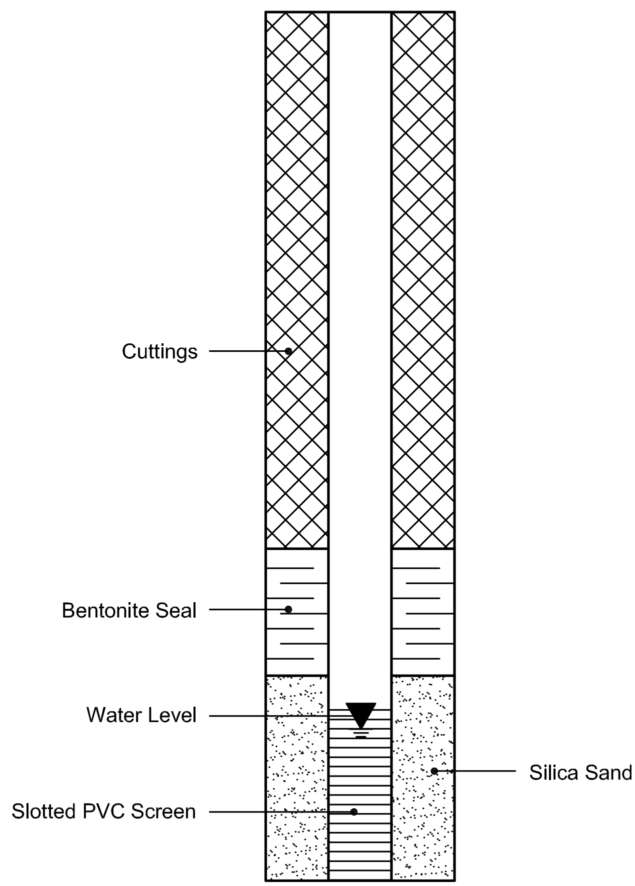


MONITORING WELL AND PIEZOMETER CONSTRUCTION

MONITORING WELL CONSTRUCTION



PIEZOMETER CONSTRUCTION



Paracel Laboratories Ltd.

Certificate of Analysis

Order #: H871?

Report Date: 04/29/0?

Order Date: 04/22/0?

Sample Date: 04/11/0?

Client: **J.D. Paterson and Associates**

Client Ref: 6717

Project: **G8584**

Matrix: Soil

Parameter	MDL	BH4 SS21 H8719.1
Chloride	5.0 ug/g	55
Sulphate	5.0 ug/g	250
pH	0.050 pH units	8.1
Resistivity	0.10 ohm.m	22

Order #: H2830

Report Date: 01/06/03
Order Date: 12/20/02
Sample Date: 12/18/02

Client: **J.D. Paterson and Associates**
Client Ref: 7397

Project: **G8584**

Matrix: Soil

Parameter	MDL	BH2 SS6 H2830.1
Chloride	5 ug/g	45
Sulphate	5 ug/g	40
pH	0.05 pH units	8.20
Resistivity	0.1 ohm.m	29

APPENDIX 2

FIGURE 1 - KEY PLAN

SITE PHOTOGRAPHS

DRAWING PG5885-1 - TEST HOLE LOCATION PLAN

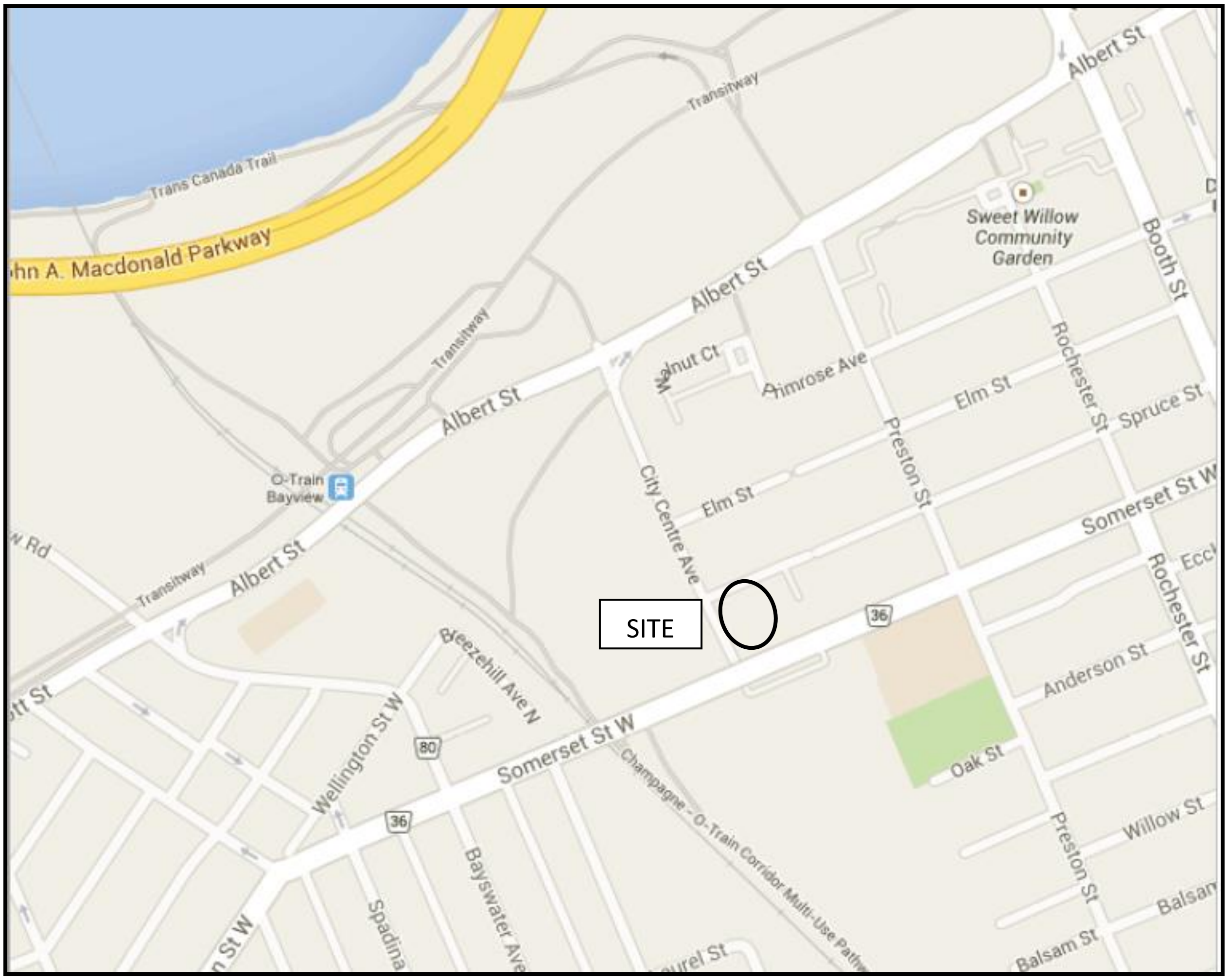


FIGURE 1

KEY PLAN

patersongroup

Photo 1 – Test pit at adjacent retaining wall along Somerset Street.

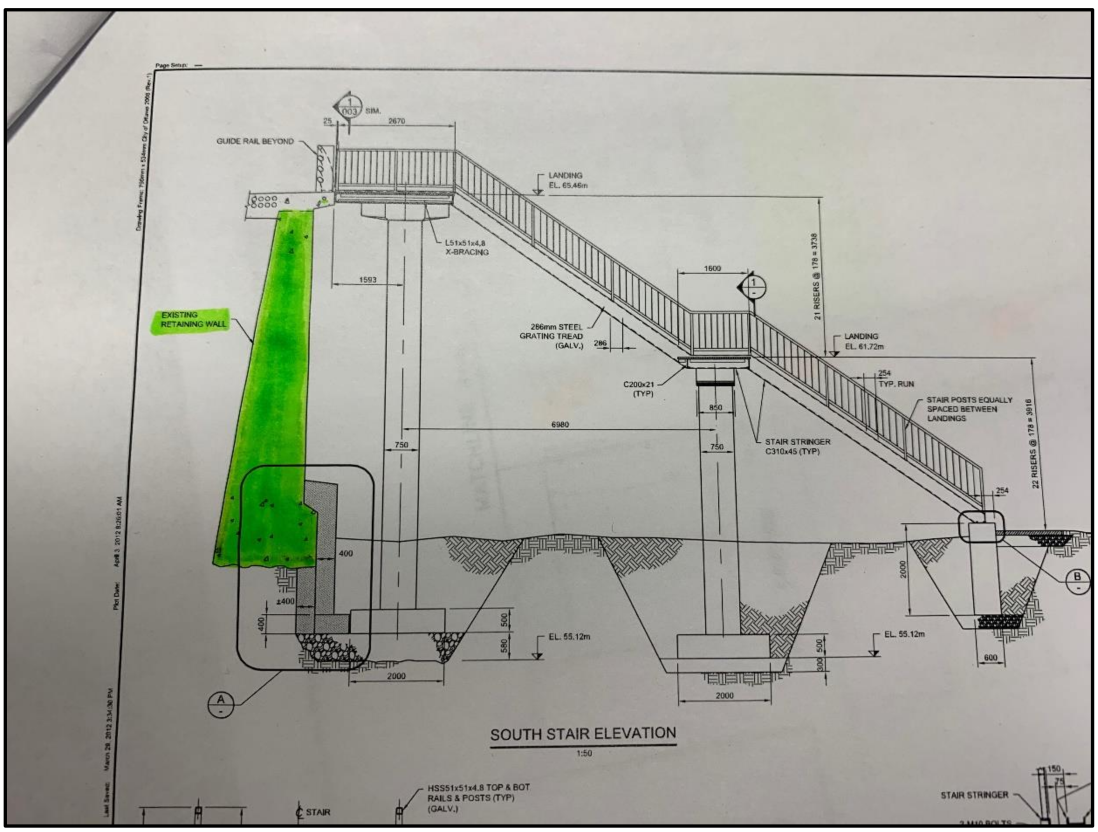


Photo 2 – Cross-section detail from Somerset Street retaining wall abutment at nearby stairway for Multi-Use Path.

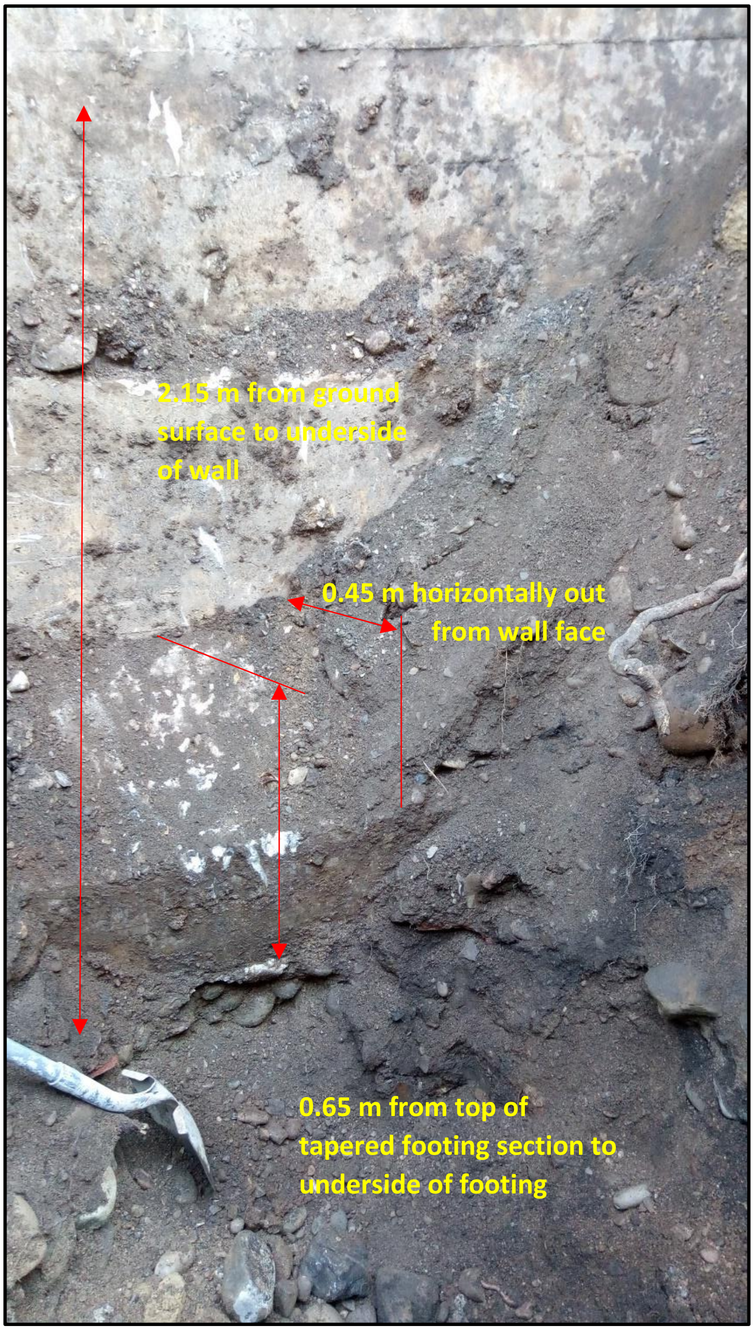


Photo 3 – Details of existing retaining wall abutment system for adjacent section of Somerset Street from field program dated December 2019. Wall abutment placed over engineered granular fill.

Photo 4 – Details of TP1-22, of the existing retaining wall abutment system for adjacent section of Somerset Street from field program dated April 2022. Wall abutment placed over engineered granular fill.

Photo 5 – Details of TP2-22, of the existing retaining wall abutment system for adjacent section of Somerset Street from field program dated April 2022. Wall abutment placed over engineered granular fill.

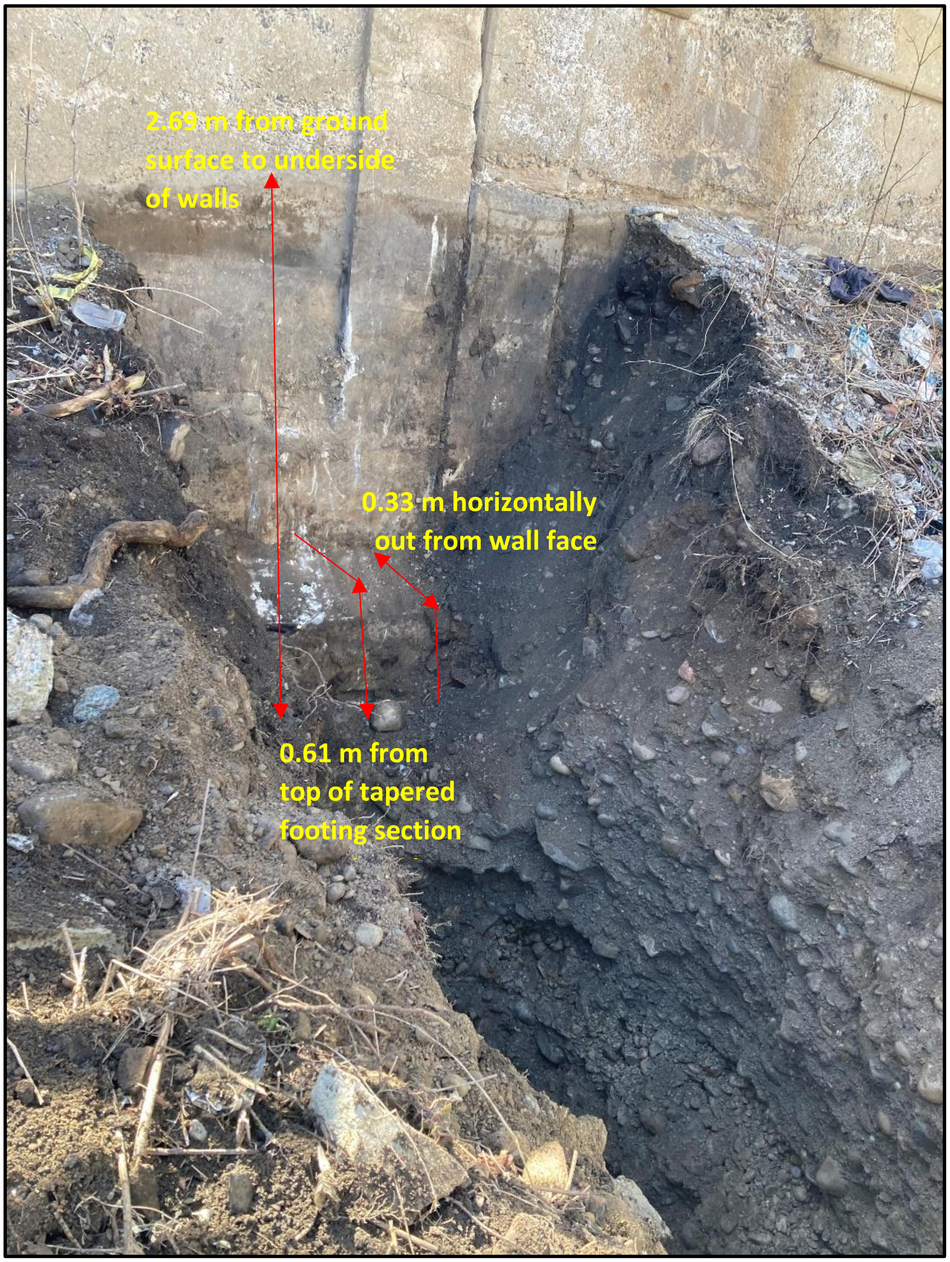
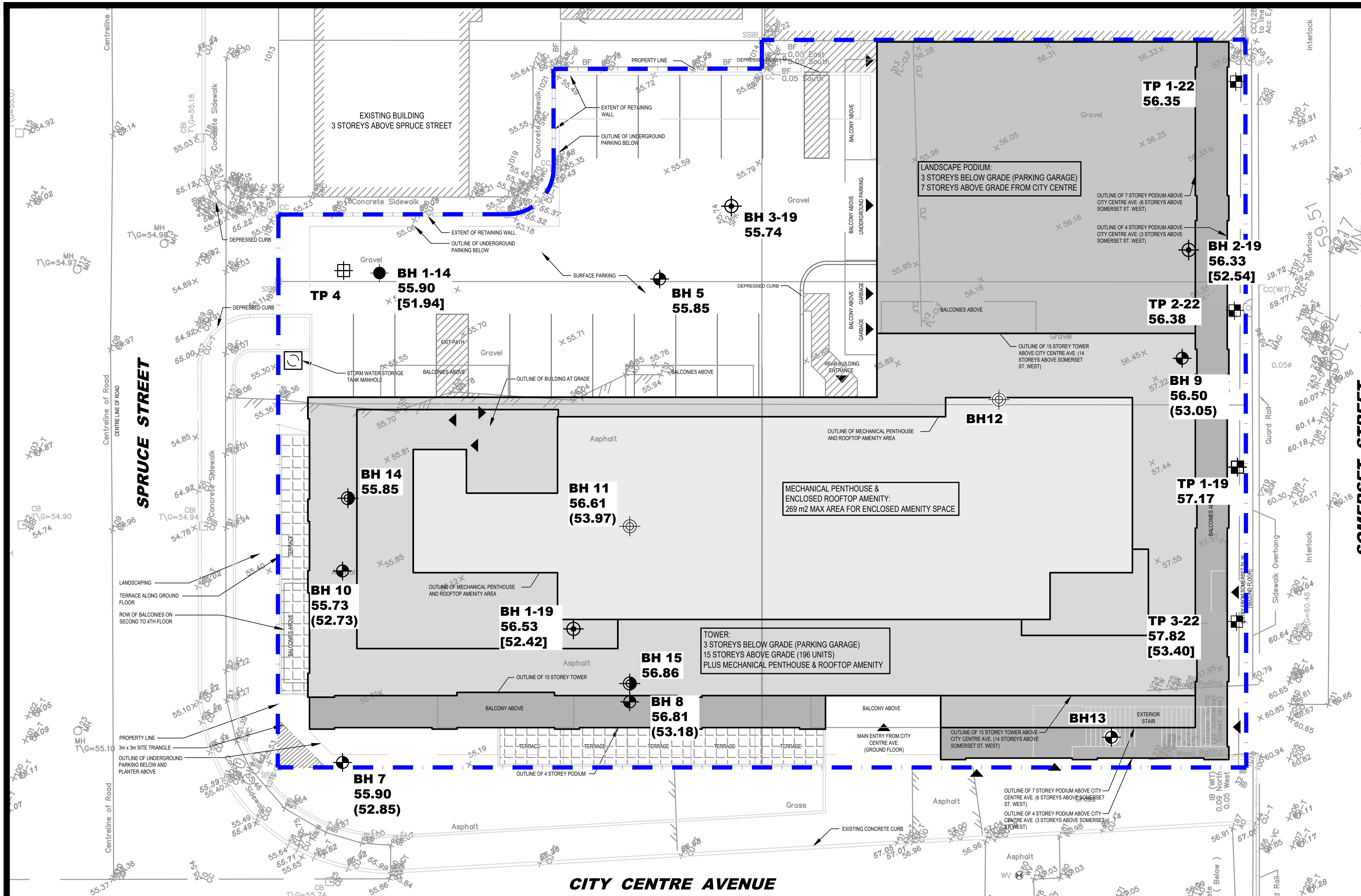


Photo 6 – Details of TP3-22, of the existing retaining wall abutment system for adjacent section of Somerset Street from field program dated April 2022. Wall abutment placed over engineered granular fill.



LEGEND:

⌖	BOREHOLE WITH MONITORING WELL LOCATION (PATERSON GROUP REPORT PG5214-1, DECEMBER 2019)
◩	TEST PIT LOCATION PATERSON GROUP REPORT PG5214-1, DECEMBER 2019)
◈	BOREHOLE LOCATION, JDPA REPORT G8584, 2002
⊞	TEST PIT LOCATION, JDPA REPORT G8584, 2002
◈	BOREHOLE WITH MONITORING WELL LOCATION, JDPA REPORT E2387, 2004
⊕	BOREHOLE LOCATION, JDPA REPORT E2387, 2004
●	BOREHOLE WTH MONITORING WELL, PATERSON GROUP PE3185, 2013
55.18	GROUND SURFACE LEVATION (m)
(52.18)	PRACTICAL REFUSAL TO AUGERING ELEVATION (m)
[51.94]	BEDROCK SURFACE ELEVATION (m)

BASEPLAN PROVIDED BY ALCAIDE WEBSTER ARCHITECTS INC.

TOPOGRAPHIC MAPPING PROVIDED BY ANNIS O'SULLIVAN VOLLEBEKK , JULY 2, 2013

TBM - TOP OF MANHOLE COVER, GEODETIC ELEVATION = 54.98m, PROVIDED BY ANNIS, O'SULLIVAN, VOLLEBEKK LTD.

SCALE: 1:250

0 1 2 3 4 5 10 15m

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consulting engineers

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NO.	REVISIONS	DATE	INITIAL
1	UPDATED TO INCLUDE LATEST 2022 TEST PITS	04/29/2022	PB

TAGGART REALTY MANAGEMENT
GEOTECHNICAL INVESTIGATION
PROPOSED MULTI-STOREY BUILDING
989 SOMERSET STREET WEST

OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:250	Date: 01/2022
Drawn by: GK	Report No.: PG5885-1
Checked by: FA	Dwg. No.: PG5885-1
Approved by: DJG	Revision No.: 1

c:\users\robertg\downloads\pg5885-1-test hole location plan (1).dwg