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Geotechnical Investigation

Proposed Multi-Storey Mixed-Use Building
289 Carling Avenue
Ottawa, Ontario

Prepared For

City of Ottawa
c/o PBC Group

February 21, 2019

Report: PG4801-1

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Appendices

1.0 Introduction

Paterson Group (Paterson) was commissioned by PBC Group on behalf of the City of Ottawa to conduct a geotechnical investigation for a proposed multi-storey mixed-use building to be located at 289 Carling Avenue, in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the current investigation were to:

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

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

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

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

2.0 Proposed Project

It is understood that the proposed project will consist of a six (6) storey, mixed-use building with a partial basement level daylighting to the south. It is anticipated that the proposed building will occupy the majority of the site.

It is expected that the proposed structure will be municipally service with water and sewer.

3.0 Available Soils Information

Two (2) separate subsoil and groundwater investigations were previously conducted by others at the aforementioned site during September 1997 and February 2017. During that time, a combined total of eleven (11) boreholes were advanced to a maximum depth of 4 m below existing ground surface. The locations of the boreholes are shown on Drawing PG4801-1 - Test Hole Location Plan included in Appendix 2.

The majority of the site is currently vacant and covered with a thin pavement structure which is bordered by an approximately 300 to 900 mm high stone and mortar retaining wall followed by landscaping consisting of grass with several mature trees. The site is approximately at grade with the intersection of Carling Avenue and Bell Street South at approximate elevation 72 to 73 m, but lower than the neighbouring properties which border the north and east boundaries of the site. The existing at-grade, asphalt parking area located immediately to the north of the northern property line is at approximate elevation 75 m, which is above the majority of the subject site. The east boundary of the subject site is bordered by the exposed exterior concrete foundation wall of the underground parking structure for the multi-storey apartment building occupying the neighbouring property to the east.

At the time of the previous subsoil investigation completed in September 1997 and February 2017, the subsurface profile at the borehole locations consists of a pavement structure underlain by imported fill consisting of a mixture of silty sand with gravel, crushed stone and ash fragments extending to a maximum depth of 0.8 m below existing ground surface. Based on the bedrock core samples recovered during the subsoil investigations and available geological mapping, the subject site is located in an area where the bedrock consists of interbedded limestone and shale of the Lindsay Formation at depths varying between 0.2 and 0.8 m below existing ground surface.

Based on the five (5) monitoring wells that were installed within the boreholes upon completion of the sampling program, the stabilized long term groundwater levels were recorded within the underlying bedrock at depths ranging between 1.5 to 2 m below exiting ground surface.

The subsoil and groundwater conditions at the test hole locations are presented in the Soil Profile and Test Data sheets in Appendix 1.

4.0 Discussion

4.1 Geotechnical Assessment

The subject site is considered adequate, from a geotechnical perspective, for the proposed building. It is anticipated that the proposed multi-storey building will be founded on conventional shallow footings placed on clean, surface sounded bedrock.

Considering the shallow depth to bedrock, it is expected that the neighbouring building located along the east property boundary is founded on bedrock. Therefore, underpinning is not expected to be required at this site. However, an assessment should be completed by the geotechnical engineer at the time of excavation to confirm founding conditions of the neighbouring building to the east, and to evaluate rock bolt locations and specific rock bolt details, should they be required, for the bedrock underlying the neighbouring building's foundations.

Due to the existing topography and subsoil conditions in conjunction with the depth and proximity of the proposed structure to the property boundaries, it is suspected that a temporary shoring system will be required to adequately support the at grade asphalt covered parking area to the north and the existing sidewalk and underground utilities located along the west property boundary.

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

4.2 Site Grading and Preparation

Stripping Depth

Due to the depth of the bedrock at the subject site and the anticipated founding level for the proposed multi-storey building, it is anticipated that all existing overburden material will be excavated from within the footprint of the proposed multi-storey building.

Bedrock Removal

It is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock for the basement level of the proposed structure. In areas of weathered bedrock and where only a small quantity of bedrock is to be removed, bedrock removal may be possible by hoe-ramming.

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

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

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

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

Vibration Considerations

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

The following construction equipment could be a source of vibrations: piling rig, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system using soldier piles would require the use of such equipment. Vibrations, whether it is caused by blasting operations or by construction operations, could be the cause of the source of detrimental vibrations on the adjacent 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). It should be noted that these guidelines are for today's construction standards. Considering that several old or sensitive buildings are encountered in the vicinity of the subject site, considerations should be given to lowering these guidelines. Considering that these guidelines are above perceptible human level and, in some cases, could be very disturbing to some people, it is recommended that a pre-construction survey 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 areas 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 should be placed in lifts of 300 mm thick or less and compacted using suitable compaction equipment for the lift thickness. Fill placed beneath the building areas should be compacted to at least 98% of the Standard Proctor Maximum Dry Density (SPMDD).

4.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface-sounded bedrock at the proposed founding elevation can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **4,000 kPa** and a bearing resistance value at serviceability limit states (SLS) of **2,000 kPa**. A geotechnical resistance factor of 0.5 was applied to the bearing resistance value 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 detected from surface sounding with a rock hammer.

Lateral Support

The bearing medium under footing-supported structures is required to be provided with adequate lateral support with respect to excavations and different foundation levels. Adequate lateral support is provided to a sound bedrock bearing medium when a plane extending 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.

Settlement

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

4.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C** for the foundations anticipated at the subject site. The soils underlying the subject site are not susceptible to liquefaction. A higher seismic site class, such as Class A or B, is available for design provided footings are extended to the bedrock surface and a site specific seismic shear wave velocity test is conducted by the geotechnical consultant. Reference should be made to the latest revision of the 2012 Ontario Building Code for a full discussion of the earthquake design requirements. The soils underlying the subject site are not considered to be susceptible to liquefaction

4.5 Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the basement floor slab. It is expected that the basement area will be mostly parking and the recommended pavement structure noted in Subsection 4.7 will be applicable. However, if storage or other uses of the lower level where a concrete floor slab will be used it is recommended that the upper 200 mm of sub-slab fill consists of 19 mm clear crushed stone. All backfill material within the footprint of the proposed building should be placed in maximum 300 mm thick loose layers and compacted to at least 98% of its SPMDD.

In consideration of the groundwater conditions encountered at the time of construction, an underfloor drainage system, consisting of lines of perforated drainage pipe subdrains connected to a positive outlet, should be provided in the clear stone under the lower basement floor.

4.6 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. 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.

Lateral Earth Pressures

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 wall (m)

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

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

Seismic 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 $0.375 \cdot a_c \cdot \gamma \cdot H^2/g$ where:

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

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

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

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

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

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

4.7 Pavement Structure

It is anticipated that the proposed basement slab will be considered a rigid pavement structure. The following rigid pavement structure is suggested to support car parking only.

Table 1 - Recommended Rigid Pavement Structure - Car Only Parking Areas

Thickness (mm)	Material Description
125	**Wear Course -** Concrete slab
150	**BASE** - 20 mm clear stone
	SUBGRADE - Bedrock

For design purposes, the pavement structure presented in the following table could be used for the design of access lanes.

Table 2 - Recommended Pavement Structure - Access Lanes	
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 - Existing fill, or OPSS Granular B Type I or II material placed over weathered bedrock.	

Minimum Performance Graded (PG) 58-34 asphalt cement should be used for this project.

If soft spots develop in the subgrade during compaction or due to construction traffic, the affected areas should be excavated and replaced with OPSS Granular B Type II material.

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 98% of the material's SPMDD using suitable vibratory equipment.

5.0 Design and Construction Precautions

5.1 Foundation Drainage and Backfill

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. The system should consist of a 150 mm diameter perforated corrugated plastic pipe, surrounded on all sides by 150 mm of 19 mm clear crushed stone, placed at the footing level around the exterior perimeter of the structure.

Backfill against the exterior sides of the foundation walls should consist of free-draining non frost susceptible granular materials. Imported granular materials, such as clean sand or OPSS Granular B Type I granular material, should be used for this purpose. However, the greater part of the site excavated materials will be frost susceptible and, as such, are not recommended for re-use as backfill against the foundation walls, unless a composite drainage blanket (such as Delta Drain 6000) connected to a drainage system is provided.

In areas where the building foundation walls will be placed in close proximity to the site boundaries, it is expected that insufficient room will be available for exterior backfill along those walls and, therefore, it is expected that the foundation wall will be poured against a drainage system placed against the bedrock face and/or temporary shoring system. It is further recommended that 150 mm diameter sleeves at 3 m centres be cast in the footing or at the foundation wall/footing interface to allow the infiltration of water to flow to an interior perimeter drainage pipe.

The foundation drainage pipe should have a positive outlet, such as a gravity connection to the storm sewer.

Underfloor Drainage

It is anticipated that underfloor drainage will be required to control water infiltration in the basement area. For preliminary design purposes, we recommend that 100 or 150 mm 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 better assessed.

5.2 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 equivalent) should be provided in this regard. If perimeter footings of heated structures are founded directly on clean, surface-sounded bedrock with no cracks or fissures and approved by the geotechnical consultant at the time of excavation, a minimum of 0.6 m of soil cover should be provided for frost protection.

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 structure proper and require additional protection, such as soil cover of 2.1 m or a combination of soil cover and foundation insulation.

5.3 Excavation Side Slopes

The side slopes of the 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.

The excavation side slopes above the groundwater level extending to a maximum depth of 3 m should be cut back at 1H:1V or flatter. The flatter slope is required for excavation below groundwater level. The 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.

As noted above, 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.

Rock Stabilization

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

Temporary Shoring

Temporary shoring may be required on the overburden soil to complete the required excavations where insufficient room is available for open cut methods. The shoring requirements will depend on the depth of the excavation, the proximity of the adjacent buildings and underground structures and the elevation of the adjacent building foundations and underground services. Additional information can be provided when the above details are known.

For design purposes, the temporary system may consist of a soldier pile and lagging system. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below. These systems can be cantilevered, anchored or braced. Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. It is further recommended that the toe of the shoring be adequately supported to resist toe failure by means of rock bolts or extending the piles into the bedrock through pre-augered holes if a soldier pile and lagging system is used.

Generally, it is expected that the shoring systems will be provided with tie-back rock anchors to ensure their stability. The geotechnical design of grouted rock anchors in sedimentary bedrock is based upon two possible failure modes. The anchor can fail either by shear failure along the grout/rock interface or by pullout of a 60 to 90 degree cone of rock with the apex of the cone near the middle of the bonded length of the anchor.

The anchor derives its capacity from the bonded portion, or fixed anchor length, at the base of the anchor. An unbonded portion, or free anchor length, is also usually provided between the rock surface and the start of the bonded length. Because the depth at which the apex shear failure cone develops is midway along the bonded length, a fully bonded anchor would tend to have a much shallower cone, and therefore less capacity, than one where the bonded length was just the bottom part of the overall anchor.

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

Table 3 - Soil Parameters for Shoring System Design

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

Soldier Pile and Lagging System

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

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

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

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

5.4 Pipe Bedding and Backfill

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

At least 150 mm of OPSS Granular A should be used for bedding for sewer and water pipes when placed on soil subgrade. The bedding should extend to the spring line of the pipe. Cover material, from the spring line to at least 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 compacted to a minimum of 95% of the material's SPMDD.

Where hard surface areas are considered above the trench backfill, the trench backfill material within the frost zone (about 1.8 m below finished grade) should match the soils exposed at the trench walls to reduce the potential 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 material's SPMDD.

5.5 Groundwater Control

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.

Infiltration levels are anticipated to be low to moderate through the excavation face, depending on the local groundwater table. The groundwater infiltration is anticipated to be controllable with open sumps and pumps.

If the anticipated pumping volumes exceed 400,000 L/day of ground and/or surface water, a temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) will be required for this project during the construction phase. A minimum of 4 to 5 months should be allowed for completion of the PTTW application package and issuance of the permit by the MECP.

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 MECP review of the PTTW application.

5.6 Winter Construction

Precautions must be taken if winter construction is considered for this project. The subsoil conditions at this site mostly consist of frost susceptible materials. In presence of water and freezing conditions ice could form within the soil mass. Heaving and settlement upon thawing could occur.

In the event of construction during below zero temperatures, the founding stratum should be protected from freezing temperatures by the 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.

The trench excavations should be carried out in a manner to avoid the introduction of frozen materials, snow or ice into the trenches. Precaution must be taken where excavations are carried in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, it should be recognized that where a shoring system is used, the soil behind the shoring system will be subjected to freezing conditions and could result in heaving of the structure(s) placed within or above frozen soil. Provisions should be made in the contract document to protect the walls of the excavations from freezing, if applicable.

6.0 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:

❏ Review of the geotechnical aspects of the excavating contractor's shoring design, prior to construction.

❏ Confirm founding conditions of the neighbouring structure during the excavation program.

❏ Periodic inspections of the vertical bedrock excavation to determine rock bolt locations and specific rock bolt details (if required).

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

❏ Sampling and testing of the concrete and fill materials used.

❏ 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 that these works have been conducted in general accordance with our recommendations could be issued, upon request, following the completion of a satisfactory materials testing and observation program by the geotechnical consultant.

7.0 Statement of Limitations

The recommendations provided in this report are in accordance with our present understanding of the project. We request permission to review our 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, we request immediate notification to permit reassessment of our recommendations.

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than the City of Ottawa, PBC 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.

Richard Groniger, C. Tech.

Scott S. Dennis, P.Eng.

LICENSED PROFESSIONAL ENGINEER
Feb. 21, 2019
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PROVINCE OF ONTARIO

Report Distribution:

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APPENDIX 1

SOIL PROFILE AND TEST DATA SHEETS (BY OTHERS)

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Project No.:	97-237	Borehole No.:	BH-22
Client:	PWGSC	Date Completed:	September 25, 1997
Location:	401 LeBreton Street	Drilling Method:	Hollow Stem Auger
Ground Surface Elevation:		Drill Supervisor:	K.B.T.

DEPTH (mBGS)	SAMPLE	BLOWS	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
0		5 11 16 50+	14	0.0		FILL - brown clayey sand with stones overlying black and orange sand - no odour	
						Borehole terminated on bedrock @ 0.61 mBGS	
1							
2							
3							
4							
5							

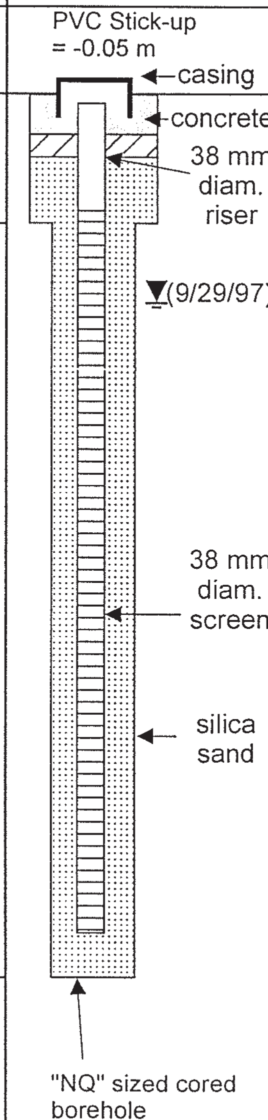
INTERA

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Project No.:	97-237	Borehole No.:	BH-23
Client:	PWGSC	Date Completed:	September 25, 1997
Location:	Bell Street Parking Lot	Drilling Method:	Hollow-Stem Auger/NQ Coring
Ground Surface Elevation:	99.52 mASD	Drill Supervisor:	K.B.T.

DEPTH (mBGS)	SAMPLE	BLOWS	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION
0		10 8	20	0.0		FILL - black and orange sandy gravel - no odour
1						LIMESTONE - black and grey shaley limestone with frequent mud seams - frequent fractures - RQD = 31%
2						- black and grey shaley limestone with frequent mud seams - frequent fractures - RQD = 66%
3						- black and grey shaley limestone with mud seams - fractured - RQD = 73%
4						**Borehole terminated in limestone @ 3.89 mBGS**
5						

INSTALLATION



INTERA

BOREHOLE STRATIGRAPHIC AND INSTRUMENTATION LOG

Project No.:	97-237	Borehole No.:	BH-24
Client:	PWGSC	Date Completed:	September 25, 1997
Location:	401 LeBreton Street	Drilling Method:	Hollow Stem Auger
Ground Surface Elevation:		Drill Supervisor:	K.B.T.

DEPTH (mBGS)	SAMPLE	BLOWS	CGI (ppm)	PID (ppm)	LOG	STRATIGRAPHIC DESCRIPTION	INSTALLATION
0		8 9 10 9	0	0.0		FILL - black and brown sand and stones - no odour - brown clayey sand overlying grey silty clay with stones	
		6	50	0.0		- no odour, no staining	
1						**Borehole terminated on bedrock @ 0.84 mBGS**	
2							
3							
4							
5							

INTERA

LOG OF BOREHOLE MW17-01

REF. No.: GV-SO-027667	DST CONSULTING ENGINEERS INC.
CLIENT: City of Ottawa	SURFACE ELEVATION: 100.83 m
PROJECT: Phase 2 Environmental Site Assessment	METHOD: Hollow Stem Auger
LOCATION: 289 Carling Avenue	DIAMETER: 75 mm (Inner diameter)
COORDINATES: **5027694.36** m N, **445057.66** m E	DATE: February 11, 2017

CVC * — ■ % LEL 20 40 60 80; ○ PPM 20 40 60 80

PPM	Samples No.	Type	SPT N-Value	MATERIAL DESCRIPTION	DPTH m	ELEV m	REMARKS
				SURFACE			
				ASPHALT			
	SS1	Soil Sample	21	FILL - Sand, some silt, light brown, frozen			
				FILL - Sand, some silt, black ash fragments, black, frozen			
	CS1	Rock Core		LIMESTONE - Interbedded with black shale, dark grey - Fractured from 0.6 m to 3.0 m, fractures are rust stained	1.0 2.0		Groundwater depth 1.87 m on February 12, 2017
	CS2	Rock Core		- Heavily fractured from 3.0 m to 4.0 m	3.0 4.0		
				End of borehole at 4.05 m			

Soil Sample
Rock Core

* - Combustible Vapour Concentration
ND - Non-Detectable

Bentonite & Riser
Sand Pack & Screen

GASTECBH PROJECT FILE GV-SO-027667 - 2017-03-22 - PHASE 2 ESA V1.1.GPJ DST_MIN.GDT 4-13-17

LOG OF BOREHOLE MW17-02

REF. No.: GV-SO-027667	**DST CONSULTING ENGINEERS INC.**
CLIENT: City of Ottawa	SURFACE ELEVATION: 101.18 m
PROJECT: Phase 2 Environmental Site Assessment	METHOD: Hollow Stem Auger
LOCATION: 289 Carling Avenue	DIAMETER: 75 mm (Inner diameter)
COORDINATES: **5027677.27** m N, **445055.81** m E	DATE: February 11, 2017

CVC * (■ % LEL 20 40 60 80; ○ PPM 20 40 60 80)	PPM	SAMPLES No.	SAMPLES Type	SPT N-Value	SUBSURFACE PROFILE MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	REMARKS
					SURFACE				
		SS1	Soil Sample	50	ASPHALT - Black			Bentonite & Riser	N/A = ppm not enough sample to screen CVC's
					FILL - Sand, some gravel, light brown, frozen				
					FILL - Sand, some silt, some gravel, black ash fragments, black stained, frozen				
		CS1	Rock Core		LIMESTONE - interbedded with black shale, dark grey	1.0		Sand Pack & Screen	
						2.0			Groundwater depth 1.72 m on February 12, 2017
					- Fractured from 0.6 m to 3.1 m. Fractures are rust stained				
		CS2	Rock Core		- Heavily fractured from 3.1 m to 4.0 m	3.0			
					End of Borehole 4.05 m	4.0			

GASTECBH PROJECT FILE GV-SO-027667 - 2017-03-22 - PHASE 2 ESA V1.1.GPJ DST_MIN.GDT 4-13-17

DST consulting engineers

Soil Sample
Rock Core

* - Combustible Vapour Concentration
ND - Non-Detectable

Bentonite & Riser
Sand Pack & Screen

LOG OF BOREHOLE MW17-03

REF. No.: GV-SO-027667	DST CONSULTING ENGINEERS INC.
CLIENT: City of Ottawa	SURFACE ELEVATION: 100.88 m
PROJECT: Phase 2 Environmental Site Assessment	METHOD: Hollow Stem Auger
LOCATION: 289 Carling Avenue	DIAMETER: 75 mm (Inner diameter)
COORDINATES: **5027687.56** m N, **445074.51** m E	DATE: February 11, 2017

CVC * (■ % LEL 20 40 60 80; ○ PPM 20 40 60 80)	PPM	SAMPLES No.	Type	SPT N-Value	SYMBL	SUBSURFACE PROFILE MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	REMARKS
						SURFACE				
						ASPHALT - Black				
						FILL - Sand some silt, light brown, frozen				
						FILL - Sand, some silt, black ash fragments, black stained				
		SS1		47		FILL - Sand, debris and ash fragments, black stained, frozen				
						LIMESTONE - Interbedded with black shale, dark grey				
							1.0			
		CS1								Groundwater depth 1.8 m on February 12, 2017
							2.0			
		CS2				- Fractured from 2.0 m to 3.0 m, fractures are rust stained				
							3.0			
		CS3								
							4.0			
						End of Borehole at 4.05 m				

DST consulting engineers

Soil Sample
Rock Core

* - Combustible Vapour Concentration
ND - Non-Detectable

Bentonite & Riser
Sand Pack & Screen

LOG OF BOREHOLE MW17-04

REF. No.: GV-SO-027667	**DST CONSULTING ENGINEERS INC.**
CLIENT: City of Ottawa	SURFACE ELEVATION: 101.29 m
PROJECT: Phase 2 Environmental Site Assessment	METHOD: Hollow Stem Auger
LOCATION: 289 Carling Avenue	DIAMETER: 75 mm (Inner diameter)
COORDINATES: **5027671.43** m N, **445068.02** m E	DATE: February 12, 2017

CVC * (■ % LEL 20 40 60 80; ○ PPM 20 40 60 80)	PPM	SAMPLES No.	Type	SPT N-Value	SYMBL	MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	REMARKS
						SURFACE				
						ASPHALT				40 ppm - Hexane 1 ppm - Isob. Groundwater depth 0.3 m on February 12, 2017
		SS1	Soil Sample	50		FILL - Sand with silt, some gravel, light brown, frozen				
		CS1	Rock Core			LIMESTONE - Interbedded with black shale, dark grey - Heavily fractured from 0.85 m to 1.2 m, rust staining in fractures	1.0			
		CS2	Rock Core				2.0 3.0 4.0			
						End of Borehole at 4.0 m				

Soil Sample
Rock Core

* - Combustible Vapour Concentration
ND - Non-Detectable

Bentonite & Riser
Sand Pack & Screen

DST consulting engineers

LOG OF BOREHOLE BH17-05

REF. No.: GV-SO-027667	DST CONSULTING ENGINEERS INC.
CLIENT: City of Ottawa	SURFACE ELEVATION: 101.43 m
PROJECT: Phase 2 Environmental Site Assessment	METHOD: Hollow Stem Auger
LOCATION: 289 Carling Avenue	DIAMETER: 75 mm (Inner diameter)
COORDINATES: **5027665.21** m N, **445071.93** m E	DATE: February 12, 2017

CVC * (■ % LEL 20 40 60 80; ○ PPM 20 40 60 80)	PPM	SAMPLES No.	SAMPLES Type	SAMPLES SPT N-Value	SYMBL	MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	REMARKS
						SURFACE				
						ASPHALT				0 ppm - Hexane
						FILL - Sand, some silt, light brown, frozen				0 ppm - Isob.
		SS1	Soil Sample	69		FILL - Sand, ash fragments, black stained, frozen				Duplicate sample (DUP #1) collected at depth of 0.03 - 0.60 mbgs
						End of Borehole at 0.7 m Auger refusal				No monitoring well installed
							1.0			
							2.0			
							3.0			
							4.0			

DST consulting engineers

Soil Sample

Rock Core

* - Combustible Vapour Concentration
ND - Non-Detectable

Bentonite & Riser

Sand Pack & Screen

LOG OF BOREHOLE BH17-06

REF. No.: GV-SO-027667	DST CONSULTING ENGINEERS INC.
CLIENT: City of Ottawa	SURFACE ELEVATION: 101.22 m
PROJECT: Phase 2 Environmental Site Assessment	METHOD: Hollow Stem Auger
LOCATION: 289 Carling Avenue	DIAMETER: 75 mm (Inner diameter)
COORDINATES: **5027684.04** m N, **445064.82** m E	DATE: February 12, 2017

CVC * ■ % LEL 20 40 60 80 ○ PPM 20 40 60 80	PPM	SAMPLES No.	Type	SPT N-Value	SYMBL	SUBSURFACE PROFILE MATERIAL DESCRIPTION	DPTH m	ELEV m	WATER DATA	REMARKS
						SURFACE				
						ASPHALT				0 ppm - Hexane
		SS1	Soil Sample	44		FILL - Sand, ash fragments, black staining, frozen				0 ppm - Isob.
						End of Borehole at 0.65 m Auger refusal				No monitoring well installed
							1.0			
							2.0			
							3.0			
							4.0			

Soil Sample
Rock Core

* - Combustible Vapour Concentration
ND - Non-Detectable

Bentonite & Riser
Sand Pack & Screen

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG4801-1 - TEST HOLE LOCATION PLAN

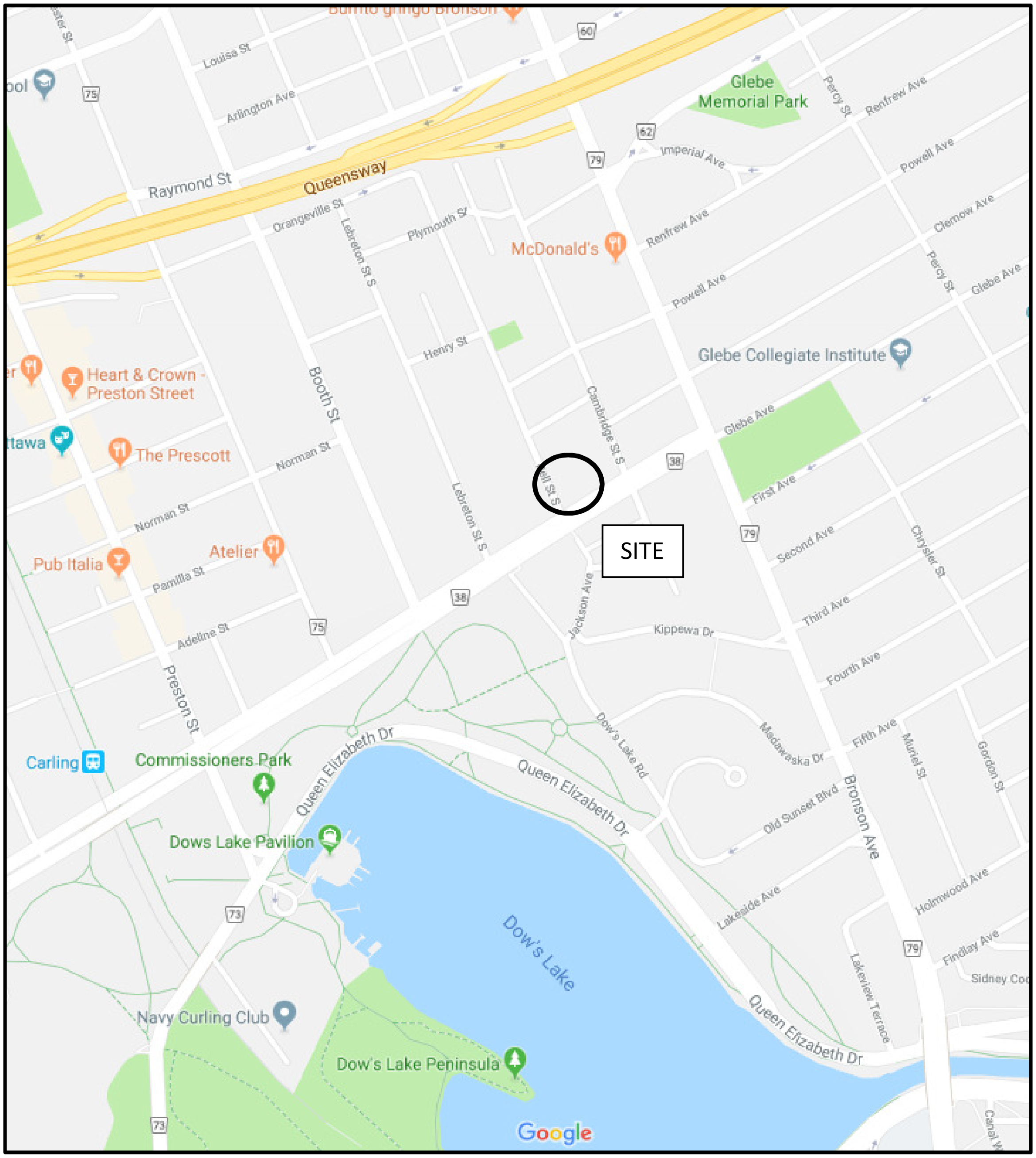


FIGURE 1

KEY PLAN

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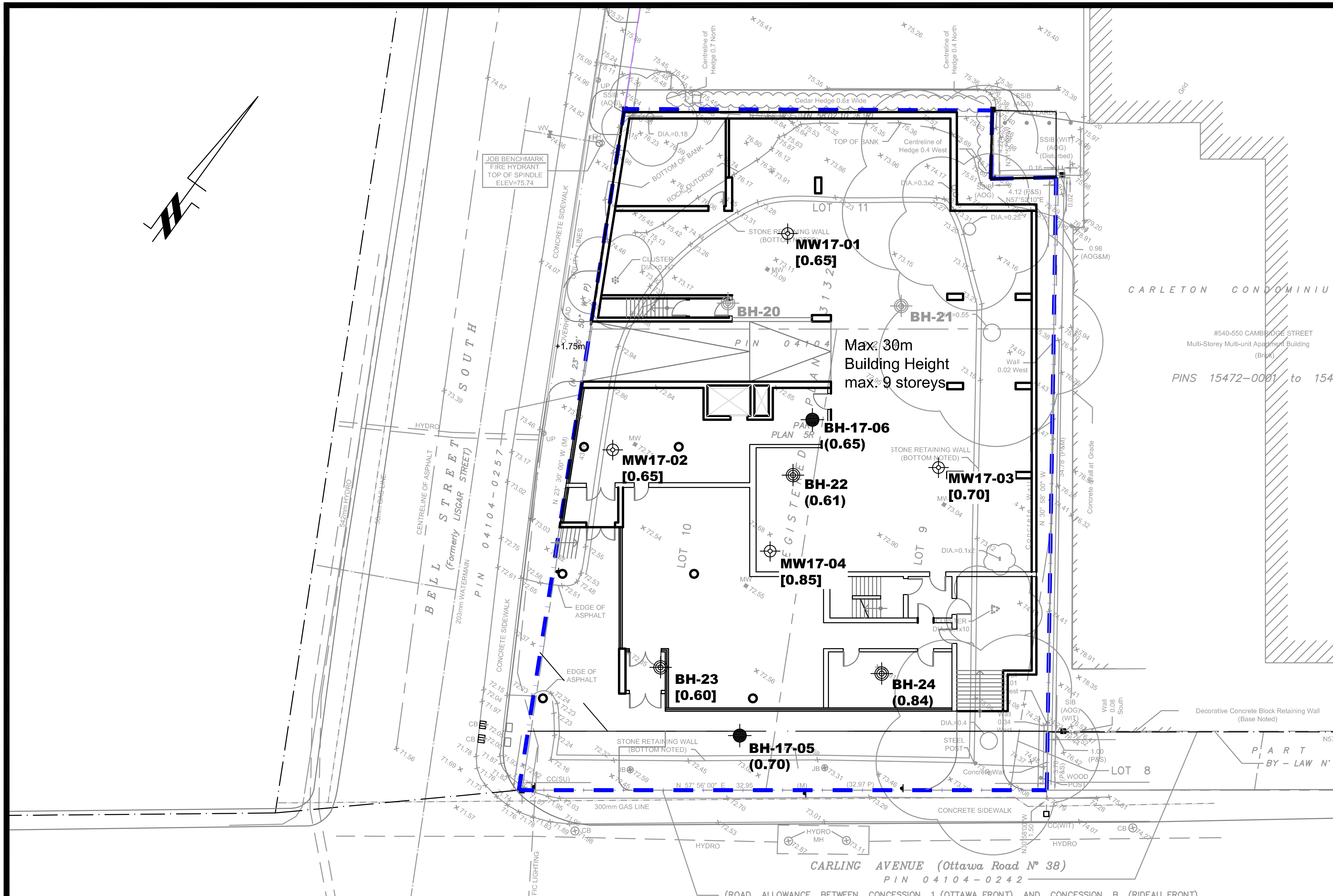

LEGEND:

BOREHOLE BY OTHERS LOCATION (DST, 2017)

BOREHOLE BY OTHERS LOCATION (INTERA, 1997)

MONITORING WELL BY OTHERS LOCATION (DST, 2017)

(3.89) DEPTH TO AUGER REFUSAL ON INFERRED BEDROCK (m)

[0.65] DEPTH TO BEDROCK SURFACE(m)

BASEPLAN AND TOPOGRAPHIC MAPPING PROVIDED BY FAIRHALL, MOFFAT AND WOODLAND, DATED SEPTEMBER 16, 2018

SCALE: 1:500

0 5 10 15 20 25 30m

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NO.	REVISIONS	DATE	INITIAL

CITY OF OTTAWA c/o PBC GROUP

PROPOSED MULTI-STOREY MIXED USE BUILDING
289 CARLING AVENUE

OTTAWA, ONTARIO

Title:

TEST HOLE LOCATION PLAN

Scale: 1:500	Date: 02/2019
Drawn by: RCG	Report No.: PG4801
Checked by: RG	PG4801-1
Approved by: SD	Revision No.: