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July 15, 2020
Report PG5101-LET.02

Starwood Group Inc.
188 Eglinton Avenue East, Suite 800
Toronto, ON
M4P 2X7

Attention: **Mr. Bruce Greenberg**

Subject: **Preliminary Geotechnical Investigation**
Proposed Multi-Storey Building
403 Richmond Road - Ottawa

Dear Sir,

Paterson Group (Paterson) was commissioned by Starwood Group to prepare a preliminary geotechnical investigation report based on the available subsoil and groundwater information for the proposed multi-storey building at 403 Richmond Road in the City of Ottawa.

1.0 Background Information

It's understood that the proposed development will consist of a 9 storey building with 1 to 2 levels of underground parking which is expected to occupy the majority of the site. It's expected that the proposed multi-storey building will be serviced with municipal water and sewer. The subject site is currently occupied by an existing two storey funeral home with associated asphalt covered car parking and access lanes. The site is generally flat and at grade with the adjacent roadways and neighbouring properties. It is our understanding that the existing structure will be demolished prior to construction to accommodate the proposed multi-storey building.

A field program for the geotechnical investigaiton was completed by others on February 9, 2017. At the time, a total of 3 boreholes were advanced to a maximum depth of 4.4 m below existing ground surface at selected locations across the site to provided general coverage of the proposed development taking into consideration of existing site features. All three boreholes were instrumented with a monitoring well installation to evaluate the groundwater conditions at the subject site. The approximate location of the test holes are presented on Drawing PG5101-1 - Test Hole Location Plan appended to this report.

2.0 Available Geotechnical Information

Subsoil Conditions

Based on our review of the available subsoil and groundwater information recovered by others, the subsurface profile at the test hole locations consists of asphaltic concrete underlain by a brown silty sand fill with varying amounts of gravel extending to a maximum depth of 1.1 m below existing ground surface at the test hole locations. A grey limestone interbedded with black shale was encountered directly below the overlying fill material at all three borehole locations.

Bedrock

As part of the geotechnical field investigation, the bedrock was cored at all three test hole locations to confirm bedrock and assess its quality. The recovery value of 100% and RQD values varied between 20 and 80% which are indicative of a fair to good quality bedrock.

Based on available geological mapping, the bedrock in the area consists of limestone and dolomite interbedded of the Gulf River formation with an overburden drift thickness of 1 to 2 m.

Groundwater

Upon reviewing the existing groundwater data collected by others, the groundwater level is estimated to be between 4 to 5 m below the existing grade. It is important to note that groundwater levels are subject to seasonal fluctuations. Therefore groundwater levels could differ at the time of construction.

3.0 Geotechnical Assessment

From a geotechnical perspective, the subject site is satisfactory for the proposed development. It’s expected that the proposed building will be founded on conventional spread footings placed on a clean, surface sounded bedrock bearing surface.

Bedrock removal will be required to complete the underground parking levels. Hoe ramming is an option where only small quantities of bedrock need to be removed. 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.

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

Site Grading and Preparation

Due to the shallow bedrock depth at the subject site and the anticipated founding level for the proposed building, all existing overburden material will be excavated from within the proposed building footprint. Bedrock removal will be required for the construction of the parking garage level.

Bedrock Removal

Based on the bedrock encountered in the area, it is expected that line-drilling in conjunction with hoe-ramming or controlled blasting will be required to remove the bedrock. 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 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 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.

Excavation side slopes in sound bedrock could be completed with almost vertical side walls. Where bedrock is of lower quality, the excavation face should be free of any loose rock. An area specific review should be completed by the geotechnical consultant at the time of construction to determine if rock bolting or other remedial measures are required to provide a safe excavation face for areas where low quality bedrock is encountered.

Vibration Considerations

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

The following construction equipment could cause vibrations: piling equipment, hoe ram, compactor, dozer, crane, truck traffic, etc. The construction of the shoring system with soldier piles, should it be utilized, would require these pieces of equipment. Vibrations, caused by blasting or construction operations could cause detrimental vibrations on the adjoining buildings and structures. Therefore, it is recommended that all vibrations 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 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). These guidelines are for current construction standards. These guidelines are above perceptible human level and, in some cases, could be very disturbing to some people. Therefore, a pre-construction survey is recommended to minimize the risks of claims during or following the construction of the proposed building.

Fill Placement

Fill used for grading beneath the 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. 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 proposed building areas should be compacted to at least 98% of its standard Proctor maximum dry density (SPMDD).

Non-specified existing fill along with site-excavated soil can be used as general landscaping fill and beneath parking areas where settlement of the ground surface is of minor concern. In landscaped areas, these materials should be spread in thin lifts and at least compacted by the tracks of the spreading equipment to minimize voids. If these materials are to be used to build up the subgrade level for areas to be paved, they should be compacted in thin lifts to a minimum density of 95% of their respective SPMDD. Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls unless a composite drainage blanket connected to a perimeter drainage system is provided.

Foundation Design

Based on the subsurface profile encountered, limestone and shale bedrock is expected to be encountered at the founding levels.

For auxiliary structures (canopy and shallow foundations), footings placed on a clean surface sounded limestone bedrock surface can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **2,500 kPa**, incorporating a geotechnical resistance factor of 0.5.

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.

For the building foundations, a factored bearing resistance value at ULS of **6,000 kPa**, incorporating a geotechnical resistance factor of 0.5, is available for footings founded on the sound bedrock at depth.

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.

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 down and out from the bottom edge of the footing at a minimum of 1H:6V (or flatter) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete. A weathered bedrock bearing medium will require a lateral support zone of 1H:1V (or flatter).

Design for Earthquakes

For preliminary design purposes, the site class for seismic site response can be taken as **Class A**. With this higher seismic site class, a site specific shear wave velocity test will be required for the detail geotechnical report to accurately determine the applicable seismic site classification for foundation design of the proposed building, as presented in Table 4.1.8.4.A of the Ontario Building Code (OBC) 2012.

Soil underlying the subject site are not susceptible to liquefaction. Reference should be made to the latest revision of the Ontario Building Code 2012 for a full discussion of the earthquake design requirements.

Basement Slab

All overburden soil will be removed from the subject site leaving the bedrock as the founding medium for the lower basement floor slab. It is expected that the basement area will be mostly parking and the recommended rigid pavement structure noted in Subsection 5.8 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 300 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 the fieldwork, a subfloor 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 (discussed in Section 4.0).

Basement Wall

It’s expected that a portion of the basement walls are 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 0.01 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. It is expected that the seismic earth pressure will be transferred to the underground floor slabs, which should be designed to accommodate these pressures. A hydrostatic groundwater pressure should be added for the portion below the groundwater level.

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 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 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 $\Delta P_{AE} = 0.375 \cdot a_c \cdot \gamma \cdot H^2/g$, where:

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

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

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

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

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

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

Pavement Structure

For design purpose, the pavement structure in the following tables are recommended to be used for the design of car parking areas and access lanes.

Table 1 - 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 imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

Table 2 - Recommended Flexible Pavement Structure - Surface Parking (if any)

Thickness (mm)	Material Description
50	**Wear Course -** Superpave 12.5 Asphaltic Concrete
150	**BASE** - OPSS Granular A Crushed Stone
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over bedrock.	

Table 3 - Recommended Rigid Pavement Structure - Lower Parking Level

Thickness (mm)	Material Description
125	**32 MPa Concrete** - Exposure Class C2
300	**BASE** - OPSS Granular A Crushed Stone
SUBGRADE - Existing imported fill, or OPSS Granular B Type I or II material placed over 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.

Pavement Structure Drainage

Satisfactory performance of the pavement structure is largely dependent on keeping the contact zone between the subgrade material and the base stone in a dry condition. Failure to provide adequate drainage under conditions of heavy wheel loading can result in the fine subgrade soil being pumped into the voids in the stone subbase, thereby reducing its load carrying capacity.

4.0 Design and Construction Precautions

Foundation Drainage

It is recommended that a perimeter foundation drainage system be provided for the proposed structure. It is expected that insufficient room is available for exterior backfill. It is suggested that this system could be as follows:

- Bedrock vertical surface should be prepared to receive the proposed membrane for the underground parking levels. The surface will be prepared by grinding or using shotcrete to smooth out angular sections for the placement of the composite foundation drainage board.

- The composite foundation (such as Miradrain G100N, Delta Drain 6000 or equivalent) will be applied to the prepared vertical bedrock surface from the ground surface to the proposed founding elevation.

- It'is 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 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

It is anticipated that underfloor drainage will be required to control water infiltration. For preliminary design purposes, it’s recommended that 150 mm perforated pipes be placed within each bay. The spacing of the underfloor drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

Foundation Backfill

If applicable, 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 re-use as backfill against the foundation walls, unless used 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.

Protection of Footings Against Frost Action

Perimeter footings of heated structures are required to be insulted against the deleterious effect of frost action. A minimum 1.5 m thick soil cover (or equivalent) should be provided in this regard. A minimum of 2.1 m thick soil cover (or equivalent) should be provided for other unheated footings.

Unsupported Excavations

The side slopes of excavations in the soil and fill overburden materials should be either cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled. It is assumed that sufficient room will be available for the greater part of the excavation to be undertaken by open-cut methods (i.e. unsupported excavations).

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 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.

Temporary Shoring

With shallow bedrock at 1 m below the existing grade, a temporary shoring system may not be required. Further recommendations will be provided in the detailed geotechnical investigation report.

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 through the excavation face. The groundwater infiltration will be controllable with open sumps and pumps.

A temporary Ministry of the Environment, Conservation and Parks (MECP) permit to take water (PTTW) Category 3 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 MECP.

For typical ground or surface water volumes being pumped during the construction phase, typically between 25,000 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.

Long-term Groundwater Control

Any groundwater encountered along the building’s perimeter or underfloor drainage system will be directed to the proposed building’s sump pit. Provided the proposed groundwater infiltration control system is properly implemented and approved by the geotechnical consultant at the time of construction, it is expected that groundwater flow will be low (i.e.- less than 25,000 L/day) with peak periods noted after rain events. A more accurate estimate can be provided during the final geotechnical investigation program.

Impacts on Neighbouring Structures

A local groundwater lowering is anticipated under short term conditions due to construction of the proposed building. It should be noted that the extent of any significant groundwater lowering will take place within a limited range of the subject site due to the minimal temporary groundwater lowering.

The neighbouring structures are expected to be founded on bedrock. No adverse effects are expected due to groundwater lowering on neighbouring structures.

Winter Construction

Precautions must be taken if winter construction is considered for this project. Where excavations are completed in proximity of existing structures which may be adversely affected due to the freezing conditions. In particular, where a shoring system is constructed, 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.

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

Trench excavations and pavement construction are difficult activities to complete during freezing conditions without introducing frost in the subgrade or in the excavation walls and bottoms. Precautions should be considered if such activities are to be completed during freezing conditions. Additional information could be provided, if required.

5.0 Recommendations

Further testing and observation services program are required to confirm geotechnical recommendations. The following aspects of the program should be performed by Paterson:

- Undertake a detailed geotechnical investigation report based on the proposed final design and issue a final geotechnical investigation report.
- Review the bedrock stabilization and excavation requirements.
- Review proposed elevator waterproofing and foundation drainage design and 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.

A report confirming that the construction has been conducted in general accordance with Paterson's recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

6.0 Statement of Limitations

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

Further, as noted in Section 5.0, a site specific investigation consisting of boreholes should be completed prior to construction to confirm our design recommendations which are based on nearby investigations and an investigations done by others.

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 Starwood Group Inc., or their agents, is not authorized without review by Paterson Group for the applicability of our recommendations to the altered use of the report.

Best Regards,

Paterson Group Inc.

Carlos P. Da Silva, P.Eng., ing., QP_{ESA}

LICENSED PROFESSIONAL ENGINEER
July 15, 2020
C. P. DA SILVA
PROVINCE OF ONTARIO

Attachments

☐ Boreholes by Others
☐ Drawing PG5101-1 - Test Hole Location Plan

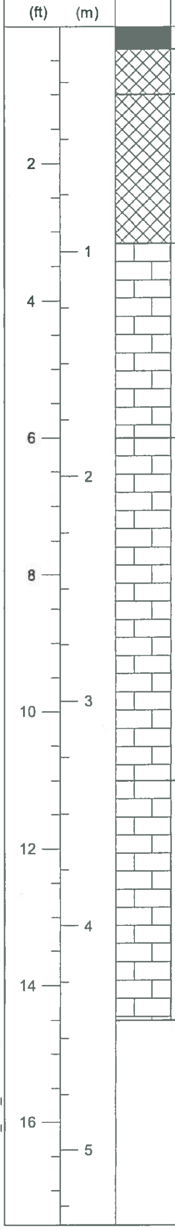
Report Distribution

☐ Starwood Group Inc. (Ontario) Ltd. (3 copies)
☐ Paterson Group (1 copy)

Monitoring Well: MW17-1

Project:	Phase II ESA and Geotechnical Assessment	**Drilling method:**	Solid Stem Auger / HQ Core
Client:	A. L. Tubman Ltd.	**Date started/completed:**	09-Feb-2017
Location:	403 Richmond Road, Ottawa, ON	**Ground surface elevation:**	99.53 m RTD
Number:	122170005.200	**Top of casing elevation:**	99.39 m RTD
Field investigator:	R. Lee	**Easting:**	440745
Contractor:	George Downing Estate Drilling Ltd.	**Northing:**	5026659

SUBSURFACE PROFILE



Lithologic Description	Elevation (m RTD) Depth (m BGS)	Sample Number	Sample Type	Recovery	RQD	Fractures per 1.52m	N Value	Lab Analyses	%LEL Comb▲ 20 40 60 80 / ppm OTOV Comb ● 200 400 600 800
Ground Surface	99.53								
100 mm ASPHALT	0.00 99.43	1	GS	n/a	n/a	n/a	n/a	Metals, PAH, pH, PHC F1-F4, VOC	<5 ●
FILL: light to dark brown, silty sand, some gravel	0.10 99.23								
Light to dark brown, SILTY SAND (possible fill)	0.30 98.56	2	SS	50%	n/a	n/a	60		<5 ●
Fair quality, light grey LIMESTONE - Iron oxidation in fractures	0.97 97.70	B1	CORE	100%	34" 59%	2.5	n/a		
Fair quality, interlayered grey LIMESTONE and dark grey SHALE	1.83 96.18								
Fair quality, light grey LIMESTONE to SHALEY LIMESTONE	3.35 95.11	B2	CORE	100%	40.5" 52%	2.5	n/a		
End of Borehole	4.42								

INSTALLATION DETAILS

Description

Flushmount protective cover with concrete seal

Backfilled with bentonite

50 mm ID schedule 40 PVC pipe

Backfilled with silica sand

50 mm ID slotted schedule 40 PVC pipe

Groundwater Level: 3.48 m BGS 17-Feb-17

Screen Interval: 1.37 - 4.42 m BGS
Sand Pack Interval: 0.61 - 4.42 m BGS
Well Seal Interval: 0.23 - 0.61 m BGS

Notes:
m BGS - metres below ground surface
SS - split-spoon sample
GS - grab sample
ppm - parts per million by volume
n/a - not available

CORE - HQ core size
PAH - polycyclic aromatic hydrocarbons
PHC F1-F4 - petroleum hydrocarbon fractions 1 to 4
VOC - volatile organic compounds

Easting and Northing coordinates based on UTM 17T projection

Stantec

Drawn By/Checked By: M. Ford / R. Lee

Monitoring Well: MW17-2

Project:	Phase II ESA and Geotechnical Assessment	**Drilling method:**	Solid Stem Auger / HQ Core
Client:	A. L. Tubman Ltd.	**Date started/completed:**	09-Feb-2017
Location:	403 Richmond Road, Ottawa, ON	**Ground surface elevation:**	99.26 m RTD
Number:	122170005.200	**Top of casing elevation:**	99.16 m RTD
Field investigator:	R. Lee	**Easting:**	440744
Contractor:	George Downing Estate Drilling Ltd.	**Northing:**	5026703

SUBSURFACE PROFILE

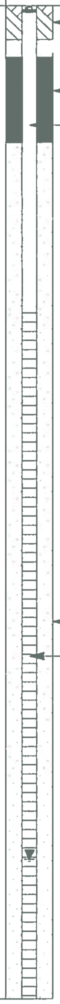
Lithologic Description	Elevation (m RTD) Depth (m BGS)
Ground Surface	99.26
ASPHALT	0.00
FILL: dark brown, clayey sand to sandy clay, with gravel	99.16 0.10
Dark brown, SILTY SAND, with some clay (possible fill)	98.95 0.30
Good quality, light grey LIMESTONE with SHALE laminations - Iron oxidation in fractures	98.19 1.07
Poor quality, dark grey interlayered SHALE and LIMESTONE	96.51 2.74
Fair quality, grey LIMESTONE to SHALEY LIMESTONE	95.55 3.70
End of Borehole	94.84 4.42

SAMPLE DETAILS

Sample Number	Sample Type	Recovery	RQD	Fractures per 1.52m	N Value	Lab Analyses	%LEL Comb
1	GS	n/a	n/a	n/a	n/a	Metals, PAH, PHC F1-F4, VOC	1%
2	SS	50%	n/a	n/a	60		1%
B1	CORE	100%	46.5" 78%	2.5	n/a		
B2	CORE	100%	43.5" 63%	2.5	n/a		

%LEL: Comb▲ 20 40 60 80; ppm: OTOV, Comb ● 200 400 600 800

INSTALLATION DETAILS



- Flushmount protective cover with concrete seal
- Backfilled with bentonite
- 50 mm ID schedule 40 PVC pipe
- Backfilled with silica sand
- 50 mm ID slotted schedule 40 PVC pipe
- Groundwater Level: 3.70 m BGS 17-Feb-17

Screen Interval:	1.37 - 4.42 m BGS
Sand Pack Interval:	0.61 - 4.42 m BGS
Well Seal Interval:	0.23 - 0.61 m BGS

Notes:
m BGS - metres below ground surface
SS - split-spoon sample
GS - grab sample
ppm - parts per million by volume
n/a - not available

CORE - HQ core size
PAH - polycyclic aromatic hydrocarbons
PHC F1-F4 - petroleum hydrocarbon fractions 1 to 4
VOC - volatile organic compounds

Easting and Northing coordinates based on UTM 17T projection

Drawn By/Checked By: M. Ford / R. Lee

Monitoring Well: MW17-3

Project:	Phase II ESA and Geotechnical Assessment	**Drilling method:**	Solid Stem Auger / HQ Core
Client:	A. L. Tubman Ltd.	**Date started/completed:**	09-Feb-2017
Location:	403 Richmond Road, Ottawa, ON	**Ground surface elevation:**	99.27 m RTD
Number:	122170005.200	**Top of casing elevation:**	99.19 m RTD
Field investigator:	R. Lee	**Easting:**	440729
Contractor:	George Downing Estate Drilling Ltd.	**Northing:**	5026693

SUBSURFACE PROFILE | SAMPLE DETAILS

Lithologic Description	Elevation (m RTD) Depth (m BGS)	Sample Number	Sample Type	Recovery	RQD	Fractures per 1.52m	N Value	Lab Analyses	%LEL Comb▲ (20 40 60 80) / ppm OTOV Comb● (200 400 600 800)
Ground Surface	99.27								
100 mm ASPHALT	0.00								
FILL: dark brown, clayey sand, with gravel	99.17 0.10	1	GS	n/a	n/a	n/a	n/a	Metals, PAH, PHC F1-F4, VOC	<5
Dark brown, SILTY SAND, with some clay (possible fill)	98.97 0.30								
		2	SS	50%	n/a	n/a	50 / 0.1 m		<5
Poor quality, light grey LIMESTONE - Highly fractured and highly weathered zone between 1.4 to 1.6 mBGS - Highly fractured and highly weathered zone between 2.1 to 2.3 m BGS	98.21 1.07	B1	CORE	100%	10" 20%	10	n/a		
Good quality, dark grey interlayered LIMESTONE and SHALE	96.99 2.29	B2	CORE	100%	18" 75%	5	n/a		
Good quality, light grey SHALEY LIMETONE to LIMESTONE	95.92 3.35	B3	CORE	100%	46" 80%	5	n/a		
End of Borehole	94.85 4.42								

Depth (ft): 2, 4, 6, 8, 10, 12, 14, 16 — (m): 1, 2, 3, 4, 5

INSTALLATION DETAILS

Flushmount protective cover with concrete seal

Backfilled with bentonite

50 mm ID schedule 40 PVC pipe

Backfilled with silica sand

50 mm ID slotted schedule 40 PVC pipe

Groundwater Level: 3.16 m BGS 17-Feb-17

Screen Interval: 1.37 - 4.42 m BGS
Sand Pack Interval: 0.61 - 4.42 m BGS
Well Seal Interval: 0.23 - 0.61 m BGS

Notes:
m BGS - metres below ground surface
SS - split-spoon sample
GS - grab sample
ppm - parts per million by volume
n/a - not available

CORE - HQ core size
PAH - polycyclic aromatic hydrocarbons
PHC F1-F4 - petroleum hydrocarbon fractions 1 to 4
VOC - volatile organic compounds

Easting and Northing coordinates based on UTM 17T projection

Drawn By/Checked By: M. Ford / R. Lee

SYMBOLS AND TERMS USED ON BOREHOLE/MONITORING WELL RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

Rootmat	- vegetation, roots and moss with organic matter and topsoil typically forming a mattress at the ground surface
Topsoil	- mixture of soil and humus capable of supporting vegetative growth
Peat	- mixture of visible and invisible fragments of decayed organic matter
Till	- unstratified glacial deposit which may range from clay to boulders
Fill	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

Desiccated	- having visible signs of weathering by oxidization 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 successions of different soil types, e.g. silt and sand
Layer	- > 75 mm in thickness
Seam	- 2 mm to 75 mm in thickness
Parting	- < 2 mm in thickness

Terminology describing soil types:
The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488) which excludes particles larger than 75 mm. For particles larger than 75 mm, and for defining percent clay fraction in hydrometer results, definitions proposed by Canadian Foundation Engineering Manual, 4th Edition are used. The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):
Terminology describing materials outside the USCS, (e.g. particles larger than 75 mm, visible organic matter, and construction debris) is based upon the proportion of these materials present:

Trace, or occasional	Less than 10%
Some	10-20%
Frequent	> 20%

Terminology describing compactness of cohesionless soils:
The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test (SPT) N-Value - also known as N-Index. The SPT N-Value is described further on page 3. A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
Very Loose	<4
Loose	4-10
Compact	10-30
Dense	30-50
Very Dense	>50

Terminology describing consistency of cohesive soils:
The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests. Consistency may be crudely estimated from SPT N-Value based on the correlation shown in the following table (Terzaghi and Peck, 1967). The correlation to SPT N-Value is used with caution as it is only very approximate.

Consistency	Undrained Shear Strength		Approximate SPT N-Value
	kips/sq.ft.	kPa	
Very Soft	<0.25	<12.5	<2
Soft	0.25 - 0.5	12.5 - 25	2-4
Firm	0.5 - 1.0	25 - 50	4-8
Stiff	1.0 - 2.0	50 – 100	8-15
Very Stiff	2.0 - 4.0	100 - 200	15-30
Hard	>4.0	>200	>30

ROCK DESCRIPTION

Except where specified below, terminology for describing rock is as defined by the International Society for Rock Mechanics (ISRM) 2007 publication "The Complete ISRM Suggested Methods for Rock Characterization, Testing and Monitoring: 1974-2006"

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	*Very Poor Quality*
25-50	*Poor Quality*
50-75	*Fair Quality*
75-90	*Good Quality*
90-100	*Excellent Quality*

Alternate (Colloquial) Rock Mass Quality	
Very Severely Fractured	*Crushed*
Severely Fractured	*Shattered or Very Blocky*
Fractured	*Blocky*
Moderately Jointed	*Sound*
Intact	*Very Sound*

RQD (Rock Quality Designation) denotes the percentage of intact and sound rock retrieved from a borehole of any orientation. All pieces of intact and sound rock core equal to or greater than 100 mm (4 in.) long are summed and divided by the total length of the core run. RQD is determined in accordance with ASTM D6032.

SCR (Solid Core Recovery) denotes the percentage of solid core (cylindrical) retrieved from a borehole of any orientation. All pieces of solid (cylindrical) core are summed and divided by the total length of the core run (It excludes all portions of core pieces that are not fully cylindrical as well as crushed or rubble zones).

Fracture Index (FI) is defined as the number of naturally occurring fractures within a given length of core. The Fracture Index is reported as a simple count of natural occurring fractures.

Terminology describing rock with respect to discontinuity and bedding spacing:

Spacing (mm)	Discontinuities	Bedding
>6000	*Extremely Wide*	-
2000-6000	*Very Wide*	*Very Thick*
600-2000	*Wide*	*Thick*
200-600	*Moderate*	*Medium*
60-200	*Close*	*Thin*
20-60	*Very Close*	*Very Thin*
<20	*Extremely Close*	*Laminated*
<6	-	*Thinly Laminated*

Terminology describing rock strength:

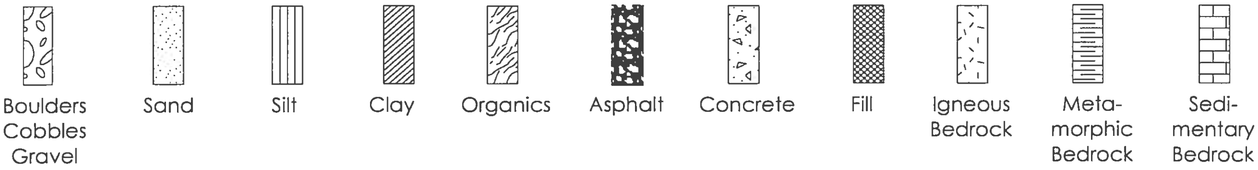
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

Terminology describing rock weathering:

Term	Symbol	Description
Fresh	W1	No visible signs of rock weathering. Slight discoloration along major discontinuities
Slightly	W2	Discoloration indicates weathering of rock on discontinuity surfaces. All the rock material may be discolored.
Moderately	W3	Less than half the rock is decomposed and/or disintegrated into soil.
Highly	W4	More than half the rock is decomposed and/or disintegrated into soil.
Completely	W5	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.
Residual Soil	W6	All the rock converted to soil. Structure and fabric destroyed.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT

▼ measured in standpipe, piezometer, or well

▽ inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (63.5 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (300 mm) into the soil. In accordance with ASTM D1586, the N-Value equals the sum of the number of blows (N) required to drive the sampler over the interval of 6 to 18 in. (150 to 450 mm). However, when a 24 in. (610 mm) sampler is used, the number of blows (N) required to drive the sampler over the interval of 12 to 24 in. (300 to 610 mm) may be reported if this value is lower. For split spoon samples where insufficient penetration was achieved and N-Values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N-values corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to 'A' size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (300 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer

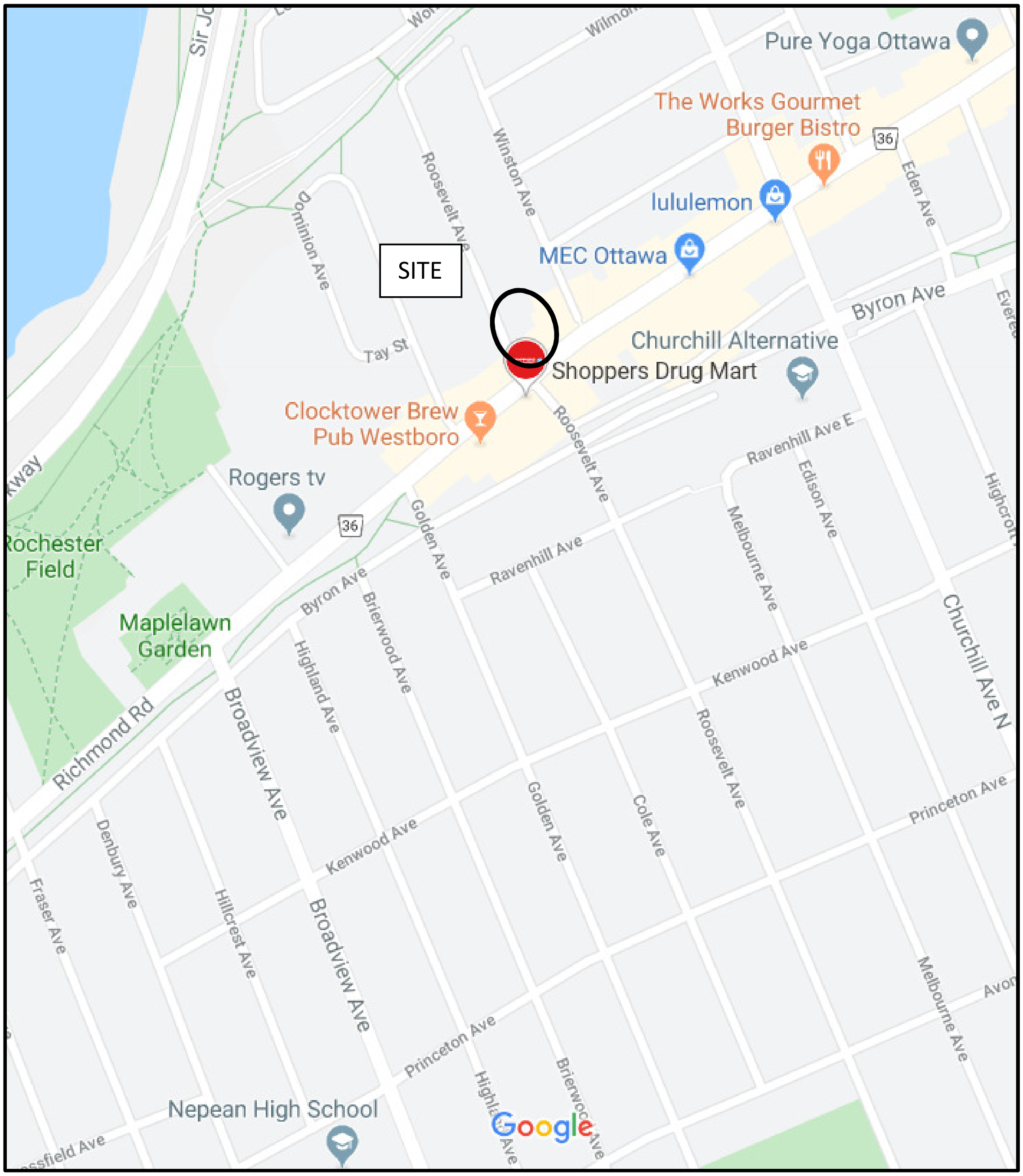
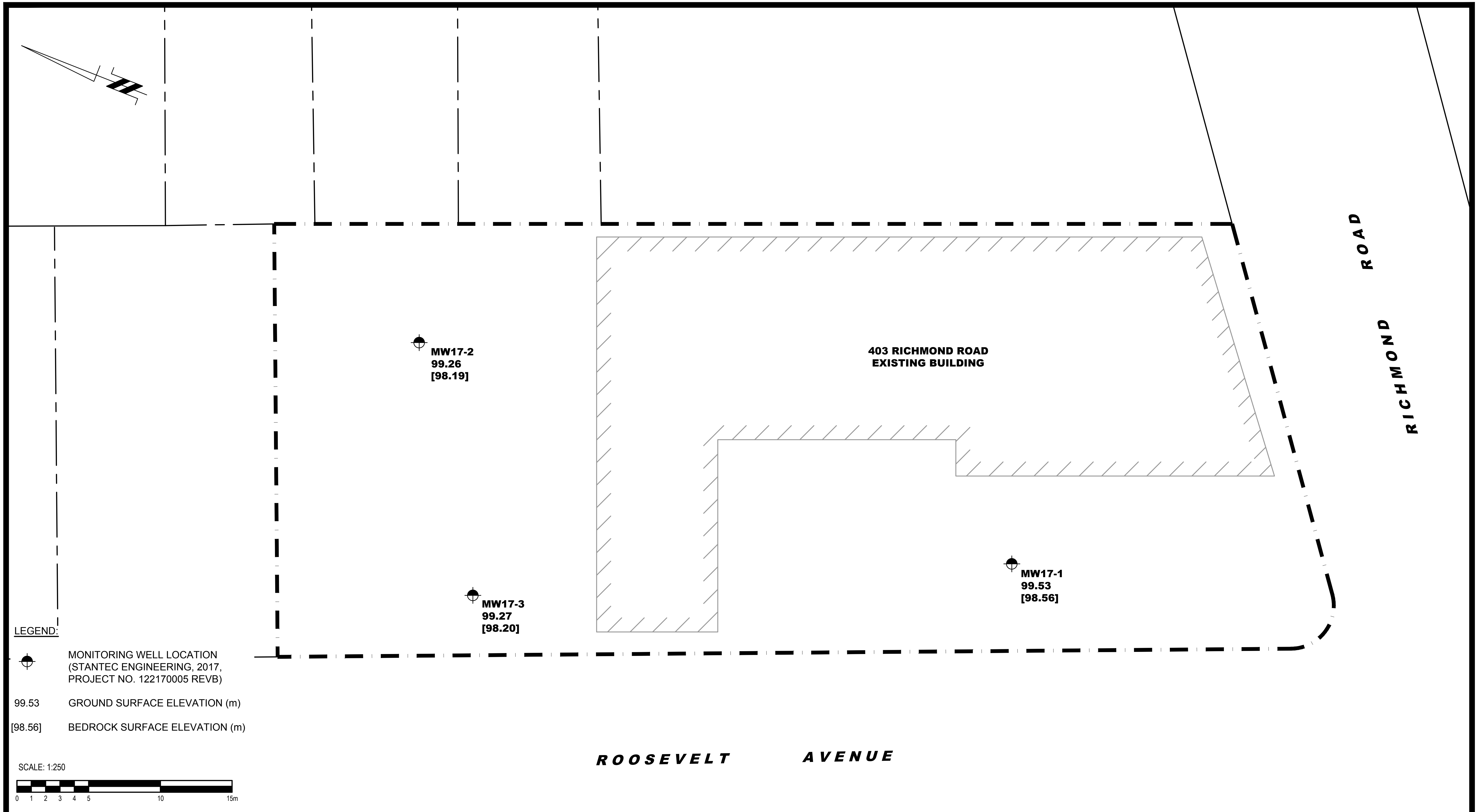


FIGURE 1

KEY PLAN

patersongroup



LEGEND:

⌖	MONITORING WELL LOCATION (STANTEC ENGINEERING, 2017, PROJECT NO. 122170005 REVB)
99.53	GROUND SURFACE ELEVATION (m)
[98.56]	BEDROCK SURFACE ELEVATION (m)

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NO.	REVISIONS	DATE	INITIAL

STARWOOD GROUP INC.
GEOTECHNICAL INVESTIGATION
403 RICHMOND STREET
OTTAWA, ONTARIO

Title:
TEST HOLE LOCATION PLAN

Scale: 1:250
Date: 10/2019
Drawn by: YA
Report No.: PG5101-1
Checked by: NG
Dwg. No.: PG5101-1
Approved by: DJG
Revision No.:

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