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Geotechnical Engineering

Environmental Engineering

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Materials Testing

Building Science

Noise and Vibration Studies

Geotechnical Investigation
Proposed Multi-Storey Building
403 Richmond Road
Ottawa, Ontario

Prepared For

SoHo Westboro Inc.

April 12, 2022

Report: PG5101-1

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1.0 Introduction

Paterson Group (Paterson) was commissioned by SoHo Westboro Inc. to prepare a geotechnical report for the proposed multi-storey building to be located at 403 Richmond Road in the City of Ottawa, Ontario (refer to Figure 1 - Key Plan in Appendix 2 of this report).

The objectives of the geotechnical investigation were to:

☐ Review available subsurface soil and groundwater information prepared by others.

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

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

2.0 Proposed Development

Based on available drawings, it is understood that the proposed development will consist of a multi-storey building with up to 3 levels of underground parking. Asphalt-paved access lanes, walkways and landscaped areas are also anticipated at finished grades surrounding the proposed building.

Construction of the proposed development is expected to require demolition of the existing residential dwelling and commercial building presently located at the site.

3.0 Method of Investigation

3.1 Field Investigation

Field Program

A geotechnical investigation was previously completed by others at the subject site on February 7, 2017, and consisted of advancing a total of 3 boreholes to a maximum depth of 4.4 m. The locations of the boreholes are shown on Drawing PG5101-1 - Test Hole Location Plan in Appendix 2.

The boreholes were advanced using an auger drill rig. The drilling procedure consisted of augering and bedrock coring to the required depths at the selected locations and sampling the overburden.

Groundwater

Groundwater monitoring wells were installed in all boreholes to permit monitoring of the groundwater levels subsequent to the completion of the sampling program. The groundwater observations are discussed in Section 4.3 and are presented in the Monitoring Well Logs by Others in Appendix 1.

All monitoring wells should be decommissioned in accordance with Ontario Regulations O.Reg 903 by a qualified licensed well technician and prior to construction.

Sample Storage

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

3.2 Field Survey

The ground surface elevations at each borehole location were surveyed by Others and are understood to be referenced to a local benchmark with an assumed elevation of 100.0 m. The location of the boreholes and ground surface elevation at each borehole location are presented on Drawing PG5101-1 - Test Hole Location Plan in Appendix 2.

4.0 Observations

4.1 Surface Conditions

The subject site consists of 2 contiguous properties: 403 Richmond Road and 389 Roosevelt Avenue. The property at 403 Richmond is occupied by an existing commercial development and associated asphalt-paved access lanes and parking areas.

The majority of the property at 389 Roosevelt Avenue is occupied by a residential dwelling located within the northern portion of the property as well as landscaped areas. An asphalt-paved driveway is located within the southwest corner of the property, fronting onto Roosevelt Avenue.

The subject site is bordered to the north and northeast by residential dwellings, to the southeast by a commercial development, to the south by Richmond Road and to the west by Roosevelt Avenue. The ground surface across the subject site is relatively flat and at-grade with the surrounding roadways.

4.2 Subsurface Profile

Overburden

Generally, the subsurface profile encountered at the test hole locations consists of asphaltic concrete underlain by an approximate 0.9 to 1.0 m thickness of fill which is further underlain by bedrock. The fill material was generally observed to consist of brown silty sand with varying amounts of gravel and clay

Bedrock

Bedrock cored at all boreholes and was encountered at approximate depths ranging from 1.0 to 1.1 m below the existing ground surface. The bedrock was observed to consist of interbedded grey limestone and black shale and was generally of fair to good quality based on the RQDs of the bedrock core. A zone of weathered, highly fractured bedrock was observed between approximate depths of 1.4 and 2.3 m in borehole MW17-3. The bedrock was cored to approximate depths of 4.4 m below the existing ground surface.

Based on available geological mapping, bedrock in the area of the subject site consists of interbedded limestone and dolomite of the Gull River Formation with an overburden thickness ranging from approximately 1 to 2 m.

Reference should be made to the Monitoring Well Logs by Others in Appendix 1 for specific details of the soil profiles encountered at each test hole location.

4.3 Groundwater

Groundwater level readings were measured in the monitoring wells by others on February 17, 2017. The measured groundwater level (GWL) readings are presented in Table 1 below.

Table 1 - Summary of Groundwater Level Readings

Borehole Number	Ground Surface Elevation (m)	Groundwater Levels (m)	Groundwater Elevation (m)	Recording Date
MW1-17	99.53	3.48	96.05	February 17, 2017
MW2-17	99.26	3.70	95.56	
MW3-17	99.27	3.16	96.11	

Note: Ground surface elevations at test hole locations were surveyed by others and are referenced to a local benchmark with an assumed elevation of 100.00 m.

Based on the measured groundwater levels, the long-term groundwater level is anticipated at a depth of approximately 3 to 4 m below the existing ground surface.

5.0 Discussion

5.1 Geotechnical Assessment

From a geotechnical perspective, the subject site is considered suitable for the proposed multi-storey building. The proposed multi-storey building is expected to be founded on conventional spread footings placed on clean surface sounded bedrock.

Bedrock removal will be required to complete the underground parking levels. Hoe ramming is an option where the bedrock is weathered and/or where only small quantities of bedrock need to be removed. Line drilling and controlled blasting will be required where large quantities of bedrock need to be removed. 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 discussed in the following sections.

5.2 Site Grading and Preparation

Stripping Depth

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

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

Due to the relatively shallow depth of the bedrock surface and the anticipated founding level for the proposed building, all existing overburden material should be excavated from within the proposed building footprint.

Bedrock Removal

Bedrock removal can be accomplished by hoe ramming where the bedrock is weathered and/or where only small quantities of the bedrock need to be removed. Sound bedrock may be removed by line drilling in conjunction with controlled blasting and/or 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 completed 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 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 are the cause of vibrations, and possibly, sources of nuisance to the community. Therefore, means to reduce the vibration levels 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 the source of vibrations: hoe ram, compactor, dozer, crane, truck traffic, etc. Vibrations, whether caused by blasting operations or by construction operations, could be the source of detrimental vibrations on the nearby buildings and structures. Therefore, all vibrations are recommended to be limited.

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

Fill Placement

Fill used for grading beneath the proposed building should consist of clean imported granular fill, such as Ontario Provincial Standard Specifications (OPSS) Granular A or Granular B Type II. This material should be tested and approved prior to delivery to the site. The fill should be placed in maximum 300 mm thick loose lifts and compacted by suitable compaction equipment. Fill placed beneath the building and paved areas should be compacted to a minimum of 98% of the standard Proctor maximum dry density (SPMDD).

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

Non-specified existing fill and site-excavated soils are not suitable for use as backfill against foundation walls, unless used in conjunction with a geocomposite drainage membrane.

5.3 Foundation Design

Bearing Resistance Values

Footings placed on clean, surface sounded limestone bedrock can be designed using a factored bearing resistance value at ultimate limit states (ULS) of **5,000 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.

Footings placed on clean, surface-sounded bedrock will be subjected to negligible 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 shallower) passes only through sound bedrock or a material of the same or higher capacity as the bedrock, such as concrete.

5.4 Design for Earthquakes

The site class for seismic site response can be taken as **Class C**. If a higher seismic site class is required (Class A or B), a site specific shear wave velocity test may be completed 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.

Soils 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.

5.5 Basement Floor Slab

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

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

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

5.6 Basement Wall

There are several combinations of backfill materials and retained soils that could be applicable for the basement walls of the subject structure. However, the conditions can be well-represented by assuming the retained soil consists of a material with an angle of internal friction of 30 degrees and a bulk (drained) unit weight of 20 kN/m^3.

Where undrained conditions are anticipated (i.e below the groundwater level), the applicable effective (undrained) unit weight of the retained soil can be taken as 13 kN/m^3 where applicable. A hydrostatic pressure should be added to the total static earth pressure when calculating 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 0.32 g 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.

5.7 Pavement Design

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

Table 2 - Recommended Rigid Pavement Structure - Lower Parking Level

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

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

Table 3 - Recommended Asphalt Pavement Structure - Access Lanes and Heavy Loading Parking Areas

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
300	**SUBBASE** - OPSS Granular B Type II
SUBGRADE - OPSS Granular B Type II overlying the Concrete Podium Deck.	

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

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

The pavement granular base and subbase should be placed in maximum 300 mm thick lifts and compacted to a minimum of 99% of the SPMDD using suitable vibratory equipment.

6.0 Design and Construction Precautions

6.1 Foundation Drainage and Backfill

Foundation Drainage

It is anticipated that the proposed building foundation walls will be blind poured and placed against a composite drainage system. For the perimeter drainage system for the foundation walls, the following is recommended:

- ☐ Line drill the excavation perimeter (usually at 150 to 200 mm spacing).
- ☐ Grind the bedrock surface up to the outer face of the line drill holes to ensure a satisfactory surface for the below grade foundation drainage system.
- ☐ If bedrock overbreaks occur, shotcrete these areas to fill in cavities and to smooth out angular features at the bedrock surface, as required based on site inspection by Paterson.
- ☐ Place a composite drainage layer, such as Delta Drain 6000 or equivalent, over the approved bedrock surface. The composite drainage layer should extend from finished grade to underside of footing level.
- ☐ Pour foundation wall against the composite drainage system.

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 any water to flow to an interior perimeter drainage pipe. The perimeter drainage pipe should direct water to sump pit(s) within the lower basement area.

A waterproofing system should also be provided for the elevator pits (pit bottom and walls).

Sub-slab Drainage System

Sub-slab drainage will be required to control water infiltration for the underground parking levels. For preliminary design purposes, we recommend that 150 mm perforated pipes be placed at approximate 6 m centres underlying the lowest level floor slab. The spacing of the sub-slab drainage system should be confirmed at the time of completing the excavation when water infiltration can be better assessed.

6.2 Protection of Footings Against Frost Action

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

Exterior unheated footings, such as isolated 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 an equivalent combination of soil cover and foundation insulation.

However, the footings are generally not expected to require protection against frost action due to the founding depth. Unheated structures such as the access ramp may require insulation for protection against the deleterious effects of frost action.

6.3 Excavation Side Slopes

The side slopes of excavations in the soil and fill overburden materials should either be cut back at acceptable slopes or should be retained by shoring systems from the start of the excavation until the structure is backfilled.

Unsupported Excavations

The excavation side slopes in the overburden and above the groundwater level extending to a maximum depth of 3 m should be excavated at 1H:1V or shallower. The shallower slope is required for excavation below groundwater level. The subsurface soils are considered to be 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.

Bedrock Stabilization

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

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

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

Temporary Shoring

Dependent on the foundation elevation of the proposed building and its proximity to the site boundaries, temporary shoring may be required to support the overburden soils. The design and approval of the shoring system will be the responsibility of the shoring contractor and the shoring designer who is a licensed professional engineer and is hired by the shoring contractor. It is the responsibility of the shoring contractor to ensure that the temporary shoring is in compliance with safety requirements, designed to avoid any damage to adjacent structures and include dewatering control measures.

In the event that subsurface conditions differ from the approved design during the actual installation, it is the responsibility of the shoring contractor to commission the required experts to re-assess the design and implement the required changes.

The designer should also take into account the impact of a significant precipitation event and designate design measures to ensure that a precipitation will not negatively impact the shoring system or soils supported by the system. Any changes to the approved shoring design system should be reported immediately to the owner's representative prior to implementation.

The temporary shoring system may consist of a soldier pile and lagging system or steel sheet piles which could be cantilevered, anchored or braced. The shoring system is recommended to be adequately supported to resist toe failure. Any additional loading due to street traffic, construction equipment, adjacent structures and facilities, etc., should be added to the earth pressures described below.

The earth pressures acting on the temporary shoring system may be calculated using the parameters outlined in Table 4 below.

Table 4 - Soil Parameters for Calculating Earth Pressures Acting on Shoring System

Parameter	Value
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	21
Submerged Unit Weight(γ'), kN/m^3	13

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

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

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

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

6.4 Pipe Bedding and Backfill

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

A minimum of 150 mm of OPSS Granular A should be placed for bedding for sewer 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) and above the cover material should match the soils exposed at the trench walls to minimize differential frost heaving. The trench backfill should be placed in maximum 225 mm thick loose lifts and compacted to a minimum of 95% of the material’s SPMDD.

6.5 Groundwater Control

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

Groundwater Control for Building Construction

A temporary Ministry of Environment, Conservation and Parks (MECP) permit to take water (PTTW) may be required if more than 400,000 L/day of ground and/or surface water are to be pumped during the construction phase. At least 4 to 5 months should be allowed for completion of the application and issuance of the permit by the MECP.

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

Impacts on Neighbouring Properties

Based on the geotechnical investigation by others, it is anticipated that the existing buildings in proximity to the subject site are founded on bedrock. Therefore, dewatering impacting neighbouring properties is not a concern for the proposed development.

6.6 Winter Construction

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

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

Trench excavations and pavement construction are also difficult activities to complete during freezing conditions without introducing frost into the subgrade or in the excavation walls and bottoms. Precautions should be taken if such activities are to be carried out during freezing conditions.

7.0 Recommendations

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

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

☐ Sampling and testing of the concrete and fill materials.

☐ Periodic observation of the condition of unsupported excavation side slopes in excess of 3 m in height, if applicable.

☐ Observation of all subgrades prior to backfilling.

☐ Field density tests to determine the level of compaction achieved.

☐ Sampling and testing of the bituminous concrete including mix design reviews.

A report confirming that these works have been conducted in general accordance with our recommendations could be issued upon the completion of a satisfactory inspection program by the geotechnical consultant.

8.0 Statement of Limitations

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

A soils investigation is a limited sampling of a site. Should any conditions at the site be encountered which differ from those at the test locations, Paterson requests immediate notification to permit reassessment of our recommendations.

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

The present report applies only to the project described in this document. Use of this report for purposes other than those described herein or by person(s) other than SoHo Westboro Inc., or their agents, is not authorized without review by Paterson for the applicability of our recommendations to the alternative use of the report.

Paterson Group Inc.

LICENSED PROFESSIONAL ENGINEER
Apr. 12, 2022
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PROVINCE OF ONTARIO

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Report Distribution:

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APPENDIX 1

MONITORING WELL LOGS BY OTHERS

SYMBOLS AND TERMS

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)
Ground Surface	99.53
100 mm ASPHALT	0.00
FILL: light to dark brown, silty sand, some gravel	99.43 0.10
Light to dark brown, SILTY SAND (possible fill)	99.23 0.30
Fair quality, light grey LIMESTONE - Iron oxidation in fractures	98.56 0.97
Fair quality, interlayered grey LIMESTONE and dark grey SHALE	97.70 1.83
Fair quality, light grey LIMESTONE to SHALEY LIMESTONE	96.18 3.35
End of Borehole	95.11 4.42

SAMPLE DETAILS

Sample Number	Sample Type	Recovery	RQD	Fractures per 1.52m	N Value	Lab Analyses	%LEL Comb / ppm OTOV, Comb
1	GS	n/a	n/a	n/a	n/a	Metals, PAH, pH, PHC F1-F4, VOC	<5 ●
2	SS	50%	n/a	n/a	60		<5 ●
B1	CORE	100%	34" 59%	2.5	n/a		
B2	CORE	100%	40.5" 52%	2.5	n/a		

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: 3.48 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

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

SAMPLE DETAILS

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)
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		

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

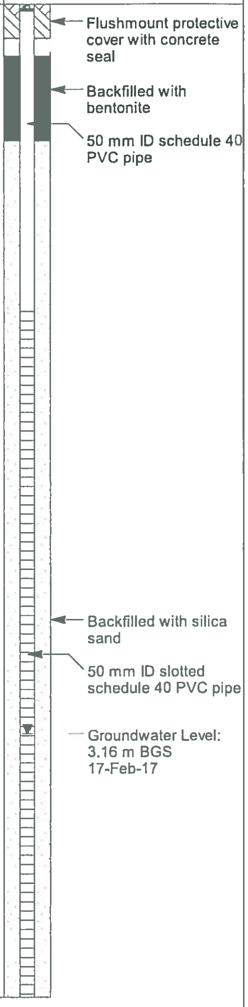
SUBSURFACE PROFILE

Elevation (m RTD) Depth (m BGS)	Lithologic Description
99.27	Ground Surface
0.00	100 mm ASPHALT
99.17 0.10	FILL: dark brown, clayey sand, with gravel
98.97 0.30	Dark brown, SILTY SAND, with some clay (possible fill)
98.21 1.07	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
96.99 2.29	Good quality, dark grey interlayered LIMESTONE and SHALE
95.92 3.35	Good quality, light grey SHALEY LIMETONE to LIMESTONE
94.85 4.42	End of Borehole

SAMPLE DETAILS

Sample Number	Sample Type	Recovery	RQD	Fractures per 1.52m	N Value	Lab Analyses	%LEL Comb▲ / ppm OTOV Comb ●
1	GS	n/a	n/a	n/a	n/a	Metals, PAH, PHC F1-F4, VOC	<5 ●
2	SS	50%	n/a	n/a	50 / 0.1 m		<5 ●
B1	CORE	100%	10" 20%	10	n/a		
B2	CORE	100%	18" 75%	5	n/a		
B3	CORE	100%	46" 80%	5	n/a		

INSTALLATION DETAILS



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:

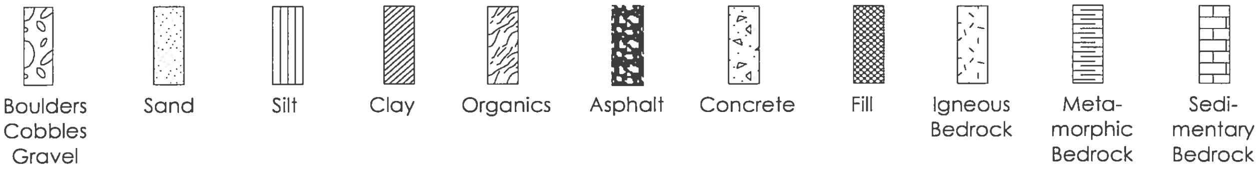
Strength Classification	Grade	Unconfined Compressive Strength (MPa)
Extremely Weak	R0	<1
Very Weak	R1	1 – 5
Weak	R2	5 – 25
Medium Strong	R3	25 – 50
Strong	R4	50 – 100
Very Strong	R5	100 – 250
Extremely Strong	R6	>250

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

APPENDIX 2

FIGURE 1 - KEY PLAN

DRAWING PG5101-1 - TEST HOLE LOCATION PLAN

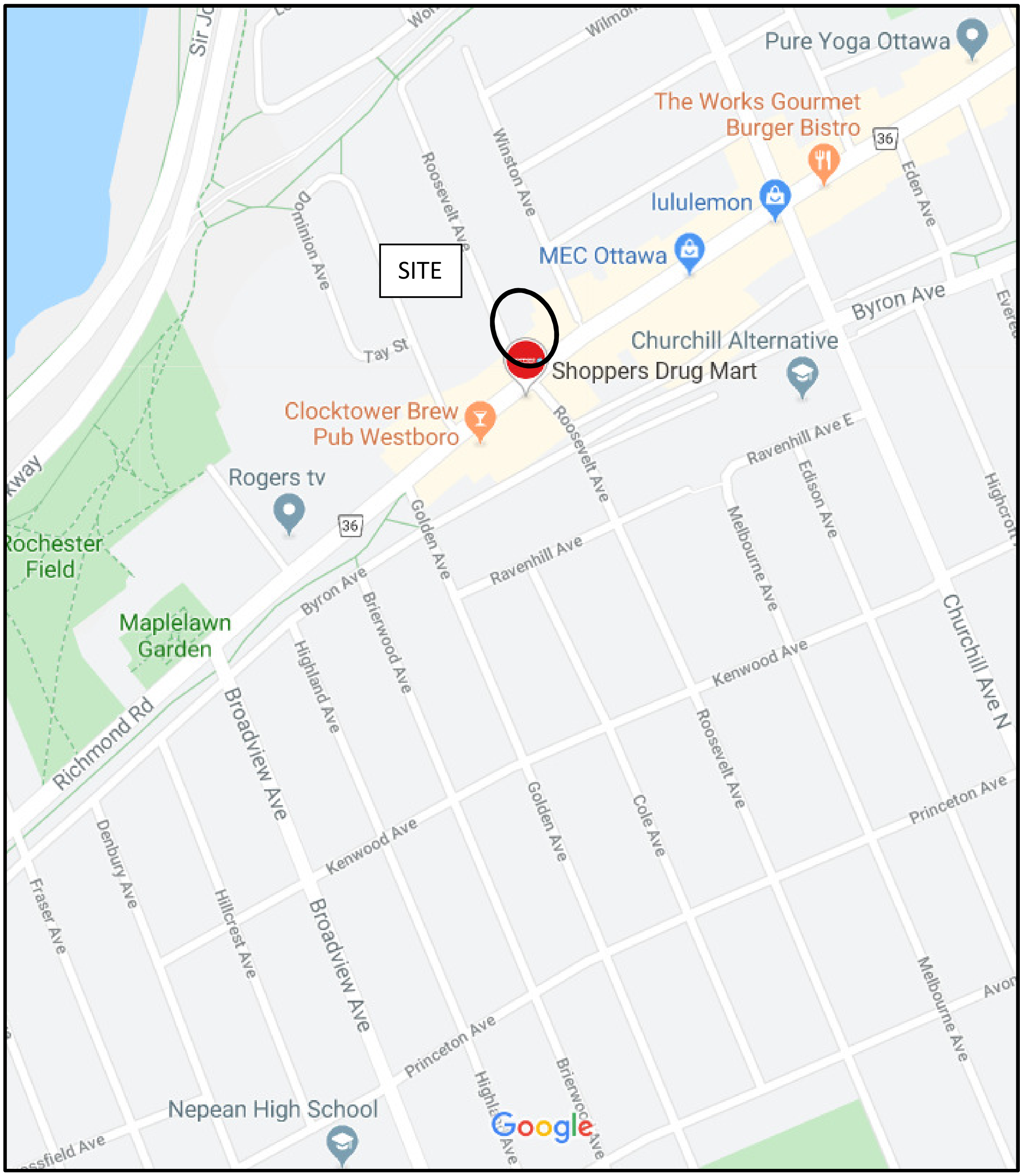
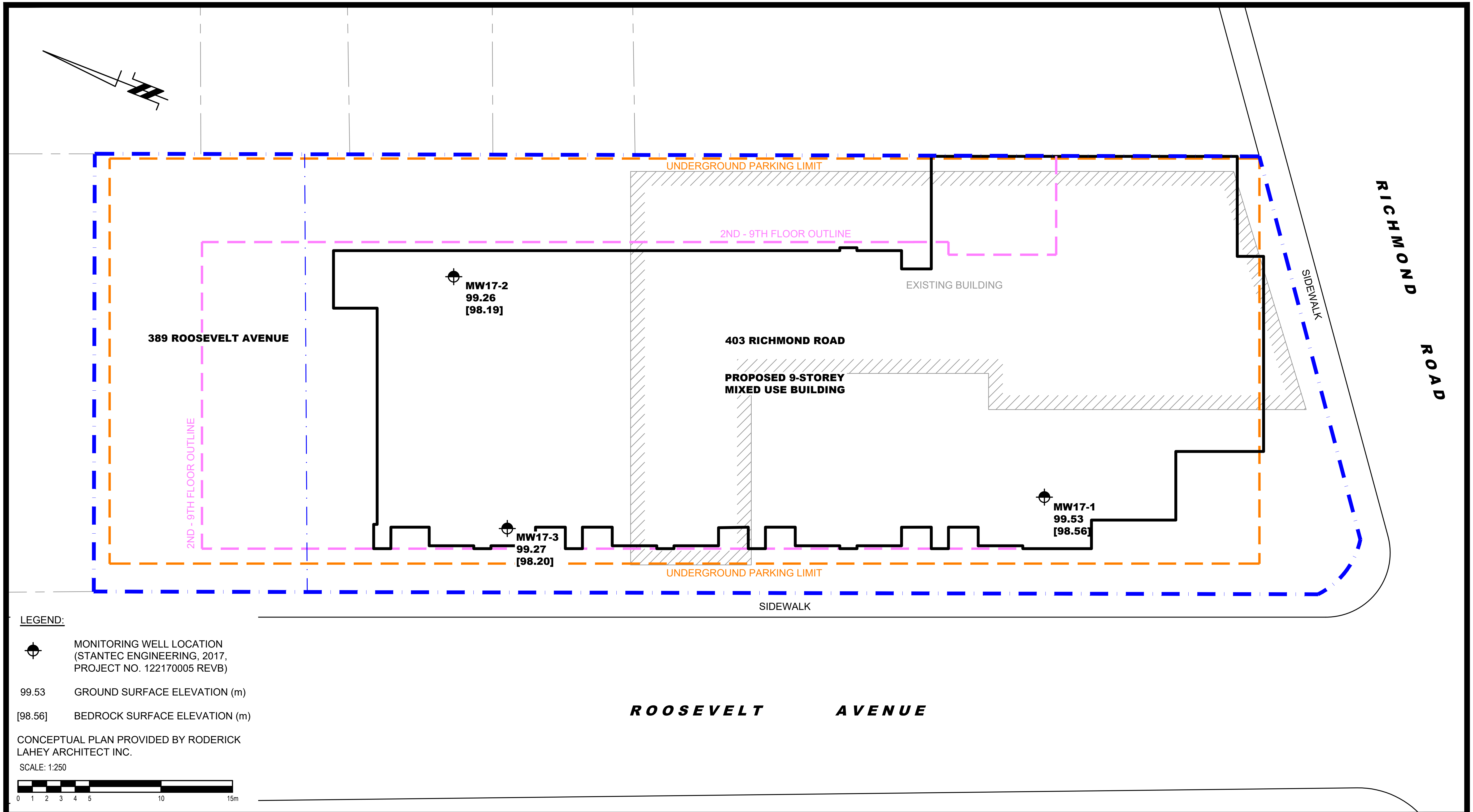


FIGURE 1

KEY PLAN



LEGEND:

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

CONCEPTUAL PLAN PROVIDED BY RODERICK LAHEY ARCHITECT INC.

SCALE: 1:250

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consulting engineers

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Ottawa, Ontario K2E 7J5
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NO.	REVISIONS	DATE	INITIAL
1	UPDATED CONCEPTUAL PLAN TO DRAWING	07/04/2022	KP

STARWOOD GROUP INC.
GEOTECHNICAL INVESTIGATION
PROPOSED MIXED USE BUILDING
403 RICHMOND ROAD & 389 ROOSEVELT AVENUE
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.: 1

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