



**March 2018**

## **REPORT ON**

# **Preliminary Geotechnical Assessment Proposed Development Hunt Club Road and Riverside Drive Ottawa, Ontario**

**Submitted to:**  
Taggart Realty Management  
225 Metcalfe Street, Suite 708  
Ottawa, Ontario  
K2P 1P9

**REPORT**



**A world of  
capabilities  
delivered locally**

**Report Number:** 1670692-1000

**Distribution:**

3 copies - Taggart Realty Management  
1 e-copy - Golder Associates Ltd.





## Table of Contents

<b>1.0</b>	<b>INTRODUCTION .....</b>	<b>1</b>
<b>2.0</b>	<b>DESCRIPTION OF PROJECT AND SITE .....</b>	<b>2</b>
<b>3.0</b>	<b>PROCEDURE .....</b>	<b>4</b>
3.1	Review of Previous Investigations .....	4
3.2	Site Reconnaissance and Slope Mapping .....	5
<b>4.0</b>	<b>SUBSURFACE CONDITIONS.....</b>	<b>6</b>
4.1	General.....	6
4.2	Subsurface Conditions.....	6
4.3	South Area.....	6
4.4	North Area .....	7
<b>5.0</b>	<b>RESULTS OF SITE RECONNAISSANCE AND SLOPE MAPPING .....</b>	<b>9</b>
5.1	South Area.....	9
5.2	North Area .....	10
<b>6.0</b>	<b>DISCUSSION.....</b>	<b>11</b>
6.1	General.....	11
6.2	Overview.....	11
6.3	Seismic Considerations .....	12
6.3.1	Liquefaction Assessment .....	12
6.3.2	Site Classification for Seismic Site Response.....	13
6.4	Slope Stability Assessment .....	13
6.4.1	General .....	13
6.4.2	Static Conditions .....	15
6.4.3	Seismic Conditions (Earthquake).....	15
6.4.4	Limit of Hazard Lands .....	15
6.4.5	Surface Drainage and Erosion Protection.....	16
6.4.6	Fill Slopes .....	17
6.5	Site Grading and Ground Improvement.....	17
6.6	Foundation Options .....	18
6.7	Floor Slab Construction .....	19
6.8	Excavation .....	20
6.9	Site Service, Roadways and Parking.....	20
<b>7.0</b>	<b>CLOSURE .....</b>	<b>22</b>



---

## PRELIMINARY GEOTECHNICAL ASSESSMENT HUNT CLUB ROAD AND RIVERSIDE DRIVE

---

### IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT

#### FIGURES

Figure 1 - Site Plan

Figures 2 to 8 - Slope Cross Sections

#### APPENDICES

##### APPENDIX A

MASW Testing - Previous Investigation



## **1.0 INTRODUCTION**

This report provides a summary of the available subsurface information for the proposed development located at the northwest corner of Hunt Club Road and Riverside Drive in Ottawa, Ontario. The objective of this study is to provide a preliminary assessment of the general geotechnical issues relating to the possible development of the site.

More specifically, the purpose of this geotechnical assessment is to:

- Collect and collate the existing available subsurface information for the site;
- Assess the stability of the existing slopes and to establish the Limit of Hazard Lands (i.e., setback) along the west side of the site, bordering the Rideau River; and,
- Provide preliminary geotechnical input relating to development of the property (e.g., site grading, building foundation options, seismic requirements, and ground improvement).

It is understood that this report is to be used in support of an application to the City of Ottawa for re-zoning of the property. The report therefore needs to confirm the overall feasibility of developing this site, from a geotechnical perspective.

The reader is referred to the “Important Information and Limitations of This Report” which follows the text but forms an integral part of this document.



## **2.0 DESCRIPTION OF PROJECT AND SITE**

The site is located immediately northwest of the intersection of Riverside Drive and Hunt Club Road, in the City of Ottawa, Ontario (see Key Plan Inset, Figure 1). The site is located between Riverside Drive and the Rideau River, extending north from Hunt Club Road and south from Kimberwick Crescent.

It is understood that plans are being prepared to develop the property. While only conceptual details of the proposed site development are available, it is understood that the plans include developing this site with a retirement residence, a hotel, car dealerships, a retail block, and a restaurant.

The site was previously used for granular material extraction (i.e., 'sand pit') activities that lasted at least until the 1970's. Over the subsequent years, the site has been sequentially filled to reclaim the land for development purposes. In the order of 10 to 15 metres thickness of fill material have been placed on the site in some locations.

The property is an irregular pentagon in shape. The average length of the site, between the north and south boundaries, is about 400 metres. The site averages about 200 metres in width. Vehicular access to the site is from the east boundary via Riverside Drive. There also exists an unpaved access road that runs in the east-west direction, dividing the site into two approximately equal trapezoidal portions (i.e., northern and southern portions).

The property area between Riverside Drive and Rideau River includes both an upland area and a lowland area. The upland area consists of higher elevation table land and is the area currently proposed for the development. The ground surface elevation varies across the upland area, ranging from about 92 to 98 metres and 88 to 98 metres in the southern and northern portions of the site, respectively. Previous filling of these areas has resulted in an uneven ground surface across these areas.

The lowland area consists of a relatively narrow strip of land separating the table land from the Rideau River. The upland area is separated from the lowland area by relatively moderate slopes. The lowland area is separated from Rideau River by variable slopes. The slopes along the Rideau River are relatively steep and about 8 to 12 metres in height within the southern portion of the site; however, within the northern portion of the site, the river bank slopes (beneath the 'lowlands') are only about 2 metres high.

The high river bank slope within the southern portion of the site is bisected by a major drainage gully, which drains the upland area runoff into the Rideau River. Several minor gullies (rills) also exist throughout the river bank slopes.

The upland area is primarily vegetated with tall grass and occasional trees. The lowland and slope areas are vegetated with dense vegetation including young and mature trees, shrubs and tall grass.

A privately-owned pump station is located within the lowlands on the north part of the site. It is understood that the pump station provides irrigation water for the Hunt Club golf course.

Based on the results of previous geotechnical investigations on this site and published geologic mapping, the subsurface conditions consist of variable thicknesses (up to 15 metres) of miscellaneous fill underlain by native granular soils consisting of sand as well as sand and gravel deposits, which are in turn underlain by glacial till. The underlying bedrock is mapped as sandstone of the March Formation or dolostone of the Oxford Formation.



---

## PRELIMINARY GEOTECHNICAL ASSESSMENT HUNT CLUB ROAD AND RIVERSIDE DRIVE

---

The site falls within the Western Quebec Seismic Zone (WQSZ), as defined by the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montreal to Témiscaming, and which encompasses the Ottawa area. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki axis. Historical seismicity within the WQSZ from 1900 to 2000 includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2 and the 1944 Cornwall Massena event which had a magnitude of 5.6. In comparison to other seismically active areas in the world (e.g., California, Japan, New Zealand), the frequency of earthquake activity within the WQSZ is significantly lower, but there still exists the potential for significant earthquake events to be generated.



## **3.0 PROCEDURE**

### **3.1 Review of Previous Investigations**

For the present assessment, subsurface information for the site was collected from several previous geotechnical investigations carried out by Golder Associates. No intrusive investigation works such as boreholes, test pits and the like were carried out for this study.

The results of the previous investigations are presented in the following Golder Associates reports:

- Report to the City of Ottawa titled "*Geotechnical Study, Uplands-River Road Study Area, Ottawa, Ontario*", dated October 1981 (report No. 811-2269);
- Report to the Regional Municipality of Ottawa-Carleton titled "*Soil Investigation, Drummond Pit, Ottawa, Ontario*", dated November 1983 (report No. 831-2386);
- Report to the Regional Municipality of Ottawa-Carleton titled "*Additional Soil Investigation, Drummond Pit, Ottawa, Ontario*", dated April 1984 (report No. 841-2088);
- Report to Delcan titled "*Geotechnical Considerations Proposed Widening and Realignment, Hunt Club Road and Riverside Drive, Ottawa, Ontario*", dated December 1984 (report No. 841-2470);
- Report to Perez Bramalea Ltd. titled "*Preliminary Subsurface Investigation, Proposed Commercial Development, St. Mary's Site, Ottawa, Ontario*", dated July 1991 (report No. 911-2151);
- Report to Cumming Cockburn Ltd. titled "*Phase I and Partial Phase II Environmental Site Assessment, Riverwalk Park and St. Mary's Sites, Riverside Drive, Ottawa, Ontario*", dated June 1994 (report No. 941-2735);
- Report to Perez Bramalea Ltd. titled "*Additional Geotechnical Investigation, Feasibility of Dynamic Compaction, St. Mary's Site, Riverside Drive, Ottawa, Ontario*", dated July 1994 (report No. 941-2135); and,
- Report to Taggart Realty Management titled "*Phase II Environmental Site Assessment, Riverside Drive and Hunt Club Road, Ottawa, Ontario*", dated September 2001 (reports No. 011-2898-5000 and 5500).
- Report to Taggart Corporation titled "*Preliminary Geotechnical Assessment, St. Mary's Site, Ottawa, Ontario*", dated September 2009 (report number 09-1121-0101)
- Technical Memorandum to The Taggart Group titled "*Site Conditions Report, Proposed PSAC Headquarters, Riverside Drive, Ottawa, Ontario*", dated May 2, 2011 (report No. 11-1121-0050)

The approximate locations of relevant boreholes from these previous subsurface investigations are shown on Figure 1.

Geotechnical information for the lowland area on the north part of the site is also available from the report prepared by McRostie Genest St-Louis and Associates (MGS) for the Ottawa Hunt and Golf Club titled "*Report on Geotechnical Investigation at Pumphouse Rebuilding Project, Ottawa Hunt and Golf Club*" dated September 2005 (report no. SF-4927).



In addition to reviewing the borehole information, the thickness of fill material placed across the site has been assessed using available site topographic maps from the previous investigation reports. In particular, the topographic data given in the 1983 and 1984 investigation reports show the approximate site conditions prior to the placement of significant fill (only relatively minor filling had been carried out by that time). The topographic data was then compared with collected topographic data in about 2007 and again in 2017 for the site, and the resulting assessment of the fill thicknesses across the site are shown on Figure 1.

The site has been divided into two areas based on topographical characteristics at the site. These two areas, hereafter called the North Area and South Area, are shown on Figure 1. The two areas have then been subdivided into a total of six sub-areas based on the estimated amount of filling present at the site, as shown on Figure 1. An overview of the subsurface conditions within each area, based on the previous boreholes data and available topographic elevation contours, is given in Sections 4.3 and 4.4, respectively.

### **3.2 Site Reconnaissance and Slope Mapping**

A site reconnaissance was previously carried out on July 9, 2009. At that time, seven slope cross sections were surveyed at relevant slope locations along the Rideau River bank.

At that time, the topography along each slope cross section was surveyed (both for horizontal and vertical positions) using a Trimble R8 GPS survey instrument, with a vertical and horizontal accuracy of less than 0.1 metres. A hand clinometer was also used to confirm the slope inclination at selected locations. The data was then used to develop approximate cross sections of the slope geometry at each location. The approximate locations of the slope cross sections are shown on the Site Plan, Figure 1. The slope cross sections were updated based on the topographic plans from 2017. The cross-sections of the surveyed slopes are shown on Figures 2 to 8.

Observations were also made on the state of erosion at the slope toe/river bank in July 2009. Locations of minor to moderate to severe erosion observed at that time are also shown on Figure 1.





## **4.0 SUBSURFACE CONDITIONS**

### **4.1 General**

The ground surface elevation across the upland (table land) area ranges from about elevations 92 to 98 metres over the South Area and from about elevations 88 to 98 metres over the North Area. The subsurface conditions consist of variable thicknesses of random fill material (generally very loose to dense silty sand, silty clay, or silty sand with variable amounts of miscellaneous material) overlying loose to very dense native granular soil (sand overlying sand and gravel), overlying glacial till and then bedrock. The fill thickness ranges between about 5 and 15 metres within the South Area (table land). Within the North Area (table land), the fill thickness ranges from about 3 to 8 metres. The groundwater level has been recorded to be at about elevation 76 to 77 metres within South Area and between about elevation 87 and 89 within North Area. The bedrock was encountered at elevations ranging from about 60 to 65 metres.

Since the time of borehole completion during the previous geotechnical investigations, the site ground surface was further raised using miscellaneous fill. The available borehole records do not therefore reflect the full thickness and composition of the fill material.

The following sections present a more detailed overview of the interpreted subsurface conditions on this property.

### **4.2 Subsurface Conditions**

#### **4.3 South Area**

The South Area includes an upland (table land) area and a significant slope down to the Rideau River. The table land ground surface elevation decreases from about 100 metres at Riverside Drive to about 92 to 94 metres at the north and west boundaries of the table land. The slope down to the Rideau River is about 16 to 20 metres high.

Boreholes 101, 102, 104, 105, 4, 01-5, 01-6, 11-3, and 11-4 and test pit 11-103 from previous investigations define subsurface conditions within the table land, while borehole 103 defines the subsurface conditions with the slope area.

Significant infilling of the former sand pits was carried out through this area. From the available borehole information and topographic mapping, it appears that essentially the whole area (except the slope) is underlain by a layer of fill of variable composition and thickness. The fill generally consists of sandy silt or silty sand with variable amounts of one or more of the following materials: gravel, clay, cobbles, boulders, topsoil, wood, concrete, bricks, plastic, metal, glass, and organic matter. The fill material in borehole 11-3 generally consists of layered silty sand and silty clay. Further fill materials may have been placed since the time that the previous boreholes were advanced, and thus the fill composition could vary.

The surface of the natural/original ground (beneath the fill) is indicated to vary between about elevations 77 and 92 metres. The existing ground elevations within the table land area, based on the recent topographic mapping, vary between about 92 and 98 metres. Based on this information, the fill thickness is expected to vary between about 5 and 15 metres within the table land area, with the fill being thickest in the central portion. The fill is indicated to range from very loose to dense in state of packing. Based on the borehole information and a review of topographic elevation contours from previous investigation reports, it appears that the deepest portion of the sand pit was essentially contained within this south part of the overall site. The fill thickness therefore tapers:

- To the east, adjacent to Riverside Drive;



- To the south, adjacent to Hunt Road and its approach to the bridge over the Rideau River; and,
- To the north, along the boundary with the North Area of the site.

These locations coincide with the slopes which formed the perimeter of the former pit. It also appears that a ridge of sand was left in-place (i.e., un-excavated) between the pit and the Rideau River, so that at least the lower part of the existing slope is the natural slope which pre-existed the sand pit. Small quantities of fill material appear to have been sporadically dumped over that slope, but otherwise there is no fill on the lower part of this slope. The overall site has however been filled up above the original ridge level, such that the upper part of the existing slope is composed of fill.

A thin layer of very stiff grey brown silty clay (about 0.8 metres thick) exists below the fill at boreholes 103, 104, and 105, located along the south and west edges of the site.

The fill is otherwise underlain by a sand deposit that grades into sand and gravel with depth in some of the boreholes. The sand ranges from loose to very dense while the sand and gravel ranges from compact to very dense, however both materials would more typically be characterized as compact to dense.

A deposit of clayey silt exists below the sand deposit in borehole 102 at a depth of about 23.5 metres below the existing ground surface (at about elevation 75.3 metres).

The underlying bedrock surface appears to dip down to the north or northwest. Borehole 101, as well as previous boreholes (not shown on Figure 1) advanced by Golder Associates at the east abutment of the existing Hunt Club Road bridge (for its design) indicate that the bedrock surface beneath the south part of the site is at about elevation 60 to 65 metres, which is about 30 metres below the general table land level.

The groundwater level in the sand to sand and gravel deposit was previously recorded between about elevations 76 and 78 metres, but up to about elevation 87 metres near Riverside Drive, reflecting a downward gradient from east to west across the site, towards the river. An artesian water level was also recorded for the bedrock, at about elevation 82 metres, in borehole 101 on November of 1983 (i.e., artesian relative to the ground level at that time).

The general groundwater level of about elevation 76 to 78 metres approximately corresponds to the bottom of the fill material and likely controlled the lowest level to which the pit was apparently excavated.

## **4.4 North Area**

The North Area includes of two relatively flat areas, discussed as 'upland' and 'lowland' areas, which are separated by a slope. The lowland area abuts the Rideau River on its western boundary. The upland area, which is the area proposed for development, slopes from about elevation 99 metres at Riverside Drive to 87 metres at the northern site boundary. The upland area is higher than the lowland area by about 8 metres (due to the placement of fill material within the upland area).

Boreholes 01-1, 01-2, 01-3, 91-1, 91-3, 91-4, 11-1, and 11-2 along with test pits 94-8, 94-9, 94-15, 94-17, 94-18, 01-1, 01-2, 01-5, 01-6, 01-7, 01-8, 11-101, 11-102 define the subsurface conditions within the table land, while borehole 81-6 and test pit 01-9 defines the conditions within the lowland area (along with the MGS geotechnical data for the pump station adjacent to the Rideau River).



---

## PRELIMINARY GEOTECHNICAL ASSESSMENT HUNT CLUB ROAD AND RIVERSIDE DRIVE

---

From the available boreholes and topographic maps, it appears that eastern part of the North Area has also been filled though not as extensively as the South Area. The fill is of variable composition and thickness, consisting of silty sand, sand or silty clay with variable amounts of one or more than one of the following materials: organic matter, gravel, cobbles, bricks, wood fragments, asphalt, metal etc.

The original/native ground surface level, beneath the fill, is indicated to vary between elevations 86 and 90 metres. The existing ground elevation within the upland area, based on the recent topographic mapping, varies between about 90 and 95 metres, except within the extreme east part where the ground level rises up to Riverside Drive. Based on this information, the fill thickness is expected to generally vary between about 3 and 8 metres within the table land, but could be potentially thicker near Riverside Drive where the ground surface level rises. The fill generally ranges from very loose to compact.

The fill is underlain by a sand deposit with trace to some silt and gravel. There is very little information available on the overall thickness of this deposit. Beneath the upland area, borehole 91-1 encountered auger refusal at about elevation 61 metres, which *could* indicate the bedrock surface (at a depth of about 30 metres beneath the current ground level). In the lowland area, borehole 81-6 indicates that the sand may be very thin and overlies glacial till at about elevation 79 metres (about 3 metres depth). The sand ranges from loose to very dense, but would more typically be described as compact to dense.

A discontinuous layer of stiff to very stiff silty clay, about 4 to 5 metres thick, exists within the sand deposit at the north end of the site in borehole 91-1.

The groundwater level was generally recorded between elevations 85 and 89 metres, but potentially as low as about elevation 78 metres in the area closer to the river, likely reflecting a downward gradient in that direction.



## **5.0 RESULTS OF SITE RECONNAISSANCE AND SLOPE MAPPING**

### **5.1 South Area**

The slopes within this portion of the site are composed of an 'upper' slope formed by the filling and a 'lower' slope composed of the native sand which extends down to the bank of the Rideau River. The approximate height and slope angle of the upper (between upland and lowland areas) and lower (Rideau River bank) slopes are as follows:

<b>Slope Section</b>	<b>Slope Locations</b>			
	<b>Upper Slope</b>		<b>Rideau River Slope</b>	
	<b>Slope Height (m)</b>	<b>Slope Angle (degree)</b>	<b>Slope Height (m)</b>	<b>Slope Angle (degree)</b>
AA'	7	19	12	36
BB'	7	28	11	49
CC'	9	14	8	47
DD'	9	18	9	41

From the 2009 slope reconnaissance, the Rideau River slopes are generally covered with mature and dense vegetation (tall grass, shrubs and trees), while the upper slopes are grass covered. The vegetation along the Rideau River bank appears to be responsible for maintaining the surficial stability of these slopes. A major drainage gully (about 2 metres wide by 2 metres deep) has been cut through the river bank slope by surface erosion.

No erosion protection is present along the Rideau River bank bordering the site. Areas of active erosion were noted at several locations along the Rideau River bank, which have resulted in over-steepened slope toes along the River bank. The results of the erosion mapping (from the 2009 slope reconnaissance) along the Rideau River bank are provided on the Site Plan, Figure 1.

Above the zone of active erosion at the river bank toe, the remaining portion of the slope appeared to be quite dry and stable (surficially) at the time of the survey in 2009, with the exception of the slope at section AA'. At a height of about 6 to 7 metres above the river bank (i.e., slope toe), the slope at section AA' exhibits some evidence of soil softening and minor seepage. The soil within this area was observed to be bare of vegetation, indicating active erosion due to surface and seepage water runoff. However, this localized zone does not appear to be experiencing any deep-seated instability.



## 5.2 North Area

The slopes within this portion of the site are divided into table land slopes and Rideau River bank slopes. The approximate height and slope angle of the table land and Rideau River slopes are as follows:

Slope Section	Slope Locations			
	Table Land		Rideau River	
	Slope Height (m)	Slope Angle	Slope Height (m)	Slope Angle
EE'	8	14	2	54
FF'	8	15	1.2	60
GG'	6	7	2	45

Both the Rideau River and table land slopes are generally covered with thick vegetation (tall grass, shrubs and trees). A broken drainage pipe was encountered at some distance (about 50 metres) to the east of the river at the location of slope section EE'. A relatively deep gully has been formed between the pipe outlet and the Rideau River. Some sporadic rip rap erosion protection is present along the Rideau River bank at the locations of slope sections EE' and FF'.

Some moderate to severe active erosion of the Rideau River bank (over its 1 to 2 metres height) was observed at the locations of cross sections EE' and FF' during the 2009 slope reconnaissance. Several small drainage gullies also exist which discharge into the Rideau River (i.e., cut into the bank). It appears that large trees and shrubs present along the Rideau River bank are responsible for maintaining the stability of the bank.



## **6.0 DISCUSSION**

### **6.1 General**

This section of the report provides preliminary engineering guidelines on the geotechnical design aspects of developing this site based on our interpretation of the available borehole records from previous investigations and from a previous site slope reconnaissance in 2009. These guidelines are appropriate for project planning, but not detailed design. Additional investigation will need to be carried out at the design stage and additional geotechnical engineering input provided.

The guidelines in this section of the report are also subject to the 'Important Information and Limitations of this Report' which follows the text but forms an integral part of this document.

### **6.2 Overview**

The subsurface conditions on the site, based on the previous investigations, consist of variable thicknesses of fill material (generally silty sand, sandy silt, or silty clay with variable amount of miscellaneous material) overlying generally compact to dense native granular soils (sand overlying sand and gravel) extending to about 30 metres or more below the current site ground level. Discontinuous deposits of silty clay (up to 5 metres thick) within the native granular soils exists at the site. The compactness of the fill ranges from very loose to dense. Based on the current topographic mapping, additional filling has taken place since the previous investigations. The composition of the new fill is unknown at this time.

The fill thickness is greatest on the south part of the site (South Area), where the deepest part of the former sand pit was located. The fill material in this area ranges between about 10 and 15 metres thick. Over the north part of the site (table land area), the fill thickness appears to generally range from about 3 to 8 metres, but may be thicker adjacent to Riverside Drive.

The groundwater level was reported to be at elevations 76 and 78 metres within the South Area and between elevations 85 and 89 within the North Area.

The ground surface elevation across the upland area ranges from about 92 to 97 metres and 88 to 98 metres in the South Area and North Area, respectively, except where the ground level rises up to Riverside Drive, at about elevation 100 metres, along the east side of the site.

The soil conditions encountered in the previous boreholes coupled with the slope conditions along the west side of the site present the following key issues associated with development of this property. More detailed geotechnical guidelines on each issue are provided in the following sections of the report.

- The slopes along the west side of the South Area are only marginally stable under static conditions and are unstable under seismic loading conditions. Furthermore, the river bank is being actively eroded. The lands adjacent to the slope are therefore considered to be 'Hazard Lands' and the development will need to be set-back from the slope. Based on the current development plan, it appears that the proposed development plans will not be impacted by the slope hazard.
- The surficial fill material is unsuitable for the support of foundations, floor slabs, or pavement in its current condition. The proposed structures in the South Area (car dealerships, hotel, and the retail block) would need to be supported on deep foundations, which derive their support from below the fill layer. Buildings in the North Area (retirement residence and restaurant) can potentially be founded on the native ground beneath the fill material.



- The fill materials and the native coarse grained soils below the water table are potentially liquefiable under seismic events.
- After discussions with a ground improvement consultant, a ground improvement program, using Geopier Rammed Aggregate Pier impact system for low rise buildings and Geopier Geoconcrete Columns for high rise buildings should be considered for this site to transfer the building loads below the surficial fill materials and into the competent native soils.
- A ground improvement program (such as rapid impact compaction) should also be considered to improve the subgrade for the support of services and pavements. Otherwise subexcavation of the fill materials beneath service pipes could be required to avoid settlements that would otherwise be damaging to the operation and integrity of sewers and watermains. Pavements could also experience unacceptable settlement and distortion if a ground improvement program is not carried out.
- The feasibility of a ground improvement program will need to be evaluated based on the results of further investigation. If/where the fill contains undesirable material such as compressible organic matter, wood, peat etc., complete subexcavation of the fills beneath the services and roadway would likely be required.

### 6.3 Seismic Considerations

The site is located in an area where there exists a history of earthquake activity and saturated granular soils. The potential for seismic liquefaction of the overburden therefore needs to be assessed.

A seismic Site Class also needs to be assigned, in accordance with Section 4.1.8.4 of the Ontario Building Code, to be used by the structural designer in determining the seismic forces to be considered in the design of the structures.

#### 6.3.1 Liquefaction Assessment

Seismic liquefaction occurs when earthquake vibrations cause an increase in pore water pressures within the soil. The presence of excess pore water pressures reduces the effective stress between the soil particles, and therefore reduces the soil's frictional resistance to shearing. This phenomenon, which leads to a temporary reduction in the shear strength of the soil, may cause:

- Instability of slopes, and even gently sloping ground can experience large lateral movements, which is referred to as "lateral spreading";
- Reduced shear resistance (i.e., bearing capacity) of soils which support foundations, as well as reduced resistance to sliding; and,
- Reduced shaft resistance for deep foundations as well as reduced resistance to lateral loading.

In addition, 'seismic settlements' may occur once the vibrations and shear stresses have ceased. Seismic settlement is the process whereby the soils stabilize into a denser arrangement after an earthquake, causing potentially large surface settlements (which can be highly differential).

The following conditions are more prone to experiencing seismic liquefaction:

- Coarse grained soils (i.e., more probable for sands than for silts);



- Soils having a loose state of packing; and,
- Soils located below the groundwater level.

A preliminary assessment of the liquefaction potential of the existing fill materials and natural granular soil deposits (i.e., the sand plus the deeper sand and gravel deposits) was carried out using the Idriss and Boulanger (2008) simplified procedure based on SPT  $N_{60}$ -values from the boreholes. The SPT N-values reported on the borehole records were corrected for overburden stress, rod length during sampling, and hammer energy efficiencies. The results of this assessment suggest that the existing fill and native submerged sands at the site would generally be classified as potentially liquefiable under an earthquake with a magnitude of 6.5 (Ottawa specified design value) and a peak ground acceleration of 0.31 g. Ground surface settlements of up to 100 millimetres could be generated following a seismic event.

Note that the liquefaction assessment carried out for this study is preliminary in nature and a detailed analysis will be required at the project design stage. At that time the potential for lateral slope movements should be assessed. Much more extensive investigation will be required, to confirm the compactness of the granular soil deposits. These liquefiable soils could also be improved (i.e., densified) to reduce or eliminate their liquefaction potential.

### **6.3.2 Site Classification for Seismic Site Response**

The results of the previous MASW work are presented in Appendix A. For sites where potential liquefaction is a concern, as identified above for this site, the 2012 OBC requires a Site Class of F (i.e., special soils) designation. The 2012 OBC allows the use of a “non-liquefied” seismic site class where the fundamental period of the proposed structure is less than 0.5 seconds (i.e., typically 3 stories or less). Thus, for preliminary planning purposes, a seismic site class designation of Site Class D, based on the MASW results, appears appropriate for buildings with a fundamental period of less than 5 seconds. For Structures with a fundamental period greater than 0.5 seconds, the development of a site-specific response spectra will be required unless a ground improvement program to mitigate potential liquefaction is undertaken.

## **6.4 Slope Stability Assessment**

### **6.4.1 General**

The stability of three critical slope cross sections was assessed using the measured slope geometry and available information on the subsurface and overburden thickness conditions.

The evaluation of the stability of a slope depends on several parameters, including:

- 1) The geometry of the slope
- 2) The ground conditions which form the slope (i.e., the thickness and orientation of the soil/bedrock strata)
- 3) The shear strength parameters of the soils which form the slope
- 4) The unit weight (i.e., density) of the soils which form the slope
- 5) The groundwater levels and flow gradients within the slope.

The slope geometry used in the analyses was established from the topographical plans from June 24, 2009 provided by Annis O’Sullivan, Vollebakk Ltd. (see Section 5.0 of this report).





## PRELIMINARY GEOTECHNICAL ASSESSMENT HUNT CLUB ROAD AND RIVERSIDE DRIVE

The ground conditions within the slope were based on the available borehole records as well as observations of the exposed soils made during the slope reconnaissance in 2009. For the slopes within the South Area, the lower portion of the slope was modelled as being composed of the native sand while the upper slope was modelled as being composed of fill material. The geometry of the former sand 'ridge' which separated the pit from the Rideau River was inferred from previous topographic records.

The slopes within the North Area were modelled as being composed of the native sand soils, but with a layer of fill material existing across the table land.

The soil parameters used in the analyses were based on experience with similar soils in the Ottawa area as well as published correlations with the results of the in-situ and laboratory testing. The soil parameters used in the analyses are:

Material	Material Thickness (m)	Drained Parameters		Unit Weight (kN/m <sup>3</sup> )
		Effective Angle of Internal Friction (degrees)	Effective Cohesion (kPa)	
Fill	2.1 – 11.8	28	0	19.0
Sand/ Silty Sand	4.0 – 12.5	31	0	19.0
Sand and Gravel	13.0 – 21.0	34	0	20.5

For the South Area, the groundwater level was modelled as being at the level of the bottom of the fill material within the former sand pit (as indicated by the boreholes), with a slight gradient towards the river. The 'ridge' of sand between the former pit and the river was therefore modelled as being unsaturated. For the North Area, the groundwater level was modelled as being about 2 to 3 metres below the slope surface, with flow generally parallel to the slope.

The stability of each slope cross section was evaluated for under both 'static' and seismic loading conditions. Effective stress soil parameters (as given above) were used under both the static and seismic loading conditions, since the fill material and native soils are generally granular in nature.

The stability of the slopes was evaluated using the SLOPE/W software. The Morgenstern-Price method was used to compute a factor of safety. The factor of safety is defined as the ratio of the magnitude of the forces/moments tending to resist failure to the magnitude of the forces/moments tending to cause failure. Theoretically, a slope with a factor of safety of less than 1.0 will fail and one with a factor of safety of 1.0 or greater will stand. However, because the modelling is not exact and natural variations exist for all of the parameters affecting slope stability, a factor of safety of 1.5 is used to define a stable slope (for static loading conditions), or alternatively to define the acceptable set-back distance for permanent structures or valuable infrastructure from an unstable slope (i.e., the Limit of Hazard Lands). Under seismic loading conditions, a minimum factor of safety of 1.1 is used in a pseudo-static analysis along with a 10 percent increase in mobilized shear strength to account for "strain-rate" effects. During detailed design, the need to assess a "post-earthquake" case with liquefied soil strengths should be undertaken in consideration of any ground improvement works (Section 6.5).



### **6.4.2 Static Conditions**

The results of the stability analyses carried out under static conditions for the sandy slopes indicate that the factor of safety against global instability of the existing Rideau River bank slopes (cross sections AA' to DD') within the South Area is generally less than 1.0 (i.e., potentially unstable).

For the shallower and flatter sand slopes with in the North Area, which includes cross sections EE', FF', and GG', the calculated factors of safety were greater than 1.5 (stable).

Based on these analyses, it is considered that the tall and steep existing Rideau River slopes within the South Area are not stable and could fail given appropriately high groundwater conditions, such as those that could be experienced during the spring thaw, or due to continuing erosion.

For the North Area, although the overall slopes are considered to be stable, continuing erosion at the creek bank could result in localized sloughing.

### **6.4.3 Seismic Conditions (Earthquake)**

The potential instability under seismic (earthquake) loading was also evaluated at each of the selected cross section locations. These analyses were carried out using a simple “pseudo-static” model where a horizontal force is applied to the failure mass. This horizontal force is proportional to the weight of the failure mass and is determined using a “seismic coefficient”.

As discussed in Section 6.3.1, these analyses were carried out using soil parameters consistent with the soil not being vulnerable to liquefaction during an earthquake.

For the South Area, the factors of safety against instability under seismic loading are less than 1.1. The slopes could therefore fail under the design seismic loading event.

For the North Area, the slopes are considered to be stable under seismic loading conditions but should be re-assessed during final design to address any potentially liquefiable areas.

### **6.4.4 Limit of Hazard Lands**

In view of the low factors of safety against slope instability obtained for the slopes in the South Area, a setback from the slope crest for development was assessed at the cross-section locations. This setback was developed by carrying out further stability analyses to assess the limit beyond which there is an acceptable factor of safety (i.e., greater than about 1.5 static or 1.1 seismic) against slope failure. This setback is shown on Figure 1 as the “Limit of Hazard Lands.”

The land between the slope and the Limits of Hazard Lands, plus the slope area itself, would be defined as Hazard Lands in accordance with Ministry of Natural Resources (MNR) guidelines and provincial planning policies, as well as City of Ottawa guidelines. Hazard Lands are unsuitable for development with either publicly owned infrastructure or private development. No permanent structures or infrastructure (i.e. buildings, walkways, bridges, roadways, parking, etc.) should be constructed within the Hazard Lands.

In accordance with the MNR guidelines, the setback distance from the crest of an unstable slope to the Limit of Hazard Lands includes three components, as appropriate, namely:

- 1) A “Stable Slope Allowance”, which is determined as the limit beyond which there is an acceptable factor of safety (i.e., greater than about 1.5 static or 1.1 seismic) against slope instability.
- 2) An “Erosion Allowance”, to account for future movement of the slope toe, in the table land direction, as a result of erosion along the slope toe/creek bank



- 3) An “Access Allowance” of 6 metres, to allow a corridor by which equipment could travel to access and repair a future slope failure. This Access Allowance has not been included in the determination of the Limit of Hazard Lands for this site since the development will not restrict the future slope access.

The magnitude of the *Erosion Allowance* is described in the MNR guidelines and is a function of the soil type, state of erosion, and water course characteristics. The reconnaissance survey assessment carried out on July 9, 2009 identified active erosion along the Rideau River bank, adjoining to the site, and therefore an *Erosion Allowance* of 15 metres has been included in the determination of the Limit of Hazard Lands. Note: The *Erosion Allowance* need not be considered if erosion protection were installed along the Rideau River bank.

The resulting Limit of Hazard Lands based on the stable Slope Allowance and Erosion Allowance is shown on Figure 1. Based on the current development plans and this preliminary assessment, the proposed development plans do not appear to conflict with the Limit of Development.

The location of the Limit of Hazard Lands is based on the current slope geometry and site grading. It is assumed that the ground level within the South Area (i.e., within that area adjacent to the highest and least stable slopes) is unlikely to be raised significantly. However, the location of the Limit of Hazard Lands will need to be re-assessed once the final site grading has been confirmed. Increases in the site grade could shift the Limit of Hazard Lands further from the slope and reduce the amount of developable land.

Conversely, the completion of a ground improvement program (see Section 6.5 of this report) could have a beneficial impact on the stability of the slope (by increasing the shear strength of the fill materials), which could shift the Limit of Hazard Lands closer to the slope, and allow for more developable land.

For the North Area, although the overall slope is considered to be stable, the approximately 2 metre high river bank could be subject to erosion and sloughing. A modest set-back from the bank is therefore proposed, however there is no planned development for this part of the site.

#### **6.4.5 Surface Drainage and Erosion Protection**

Although the Limit of Hazard Lands indicated on Figure 1 does not apparently impact on (i.e., restrict) the current development plans, the line could be shifted towards the slope, and more table land defined as useable/developable land, if erosion protection were installed at the slope toe. With the installation of erosion protection, the ‘Erosion Allowance’ need not be considered in the evaluation of the Limit of hazard Lands.

Ongoing erosion of the slope toe is also one of the most likely potential triggers for a slope movement which, even if those movements did not impact on the development (since the development would be located outside of the Limit of Hazard Lands), might have negative impacts on river navigation and aquatic habitat, and also be a cause of concern to the public.

The installation of erosion protection along the Rideau River bank could therefore have the following possible benefits:

- More developable land might be identified for the table land, by defining a Limit of Hazard Lands closer to the Rideau River bank slope;
- The risk of a future slope failure occurring and having to be repaired may be reduced; and,
- Fish habitat and riparian habitat might be improved.



The erosion protection measures could conceivably be of several forms, including: rip-rap, gabion basket walls, or biotechnical measures such as live crib walls.

The decision as to whether to implement such measures (and which measures to implement) would however require consultation with the Rideau Valley Conservation Authority (RVCA) which regulates this waterway. An assessment of the regulatory or biological/ecological impacts would also be required and might preclude such measures being implemented. The RVCA has previously expressed a preference to not have erosion protection installed along the slope toe adjacent to this site.

As a more general guideline, grading of the site should direct surface runoff away from the slopes into drainage channels designed specifically for this purpose. Uncontrolled surface water runoff over the existing slopes can reduce the factor of safety against instability and should not be allowed.

#### **6.4.6 Fill Slopes**

The assessment provided in this report focuses on the 'global' stability of the slopes adjacent to the Rideau River, and on determination of the Limit of Hazard Lands associated with deep-seated failure of those slopes. There are however localized fill slopes on this site that, having been created by end-dumping, are overly steep. Surficial instability of these slopes could be expected. Therefore, where these slopes exist within the development area, it should be planned to re-grade them to a flatter geometry. The required slope angle depends on the height of each filled slope but, as a preliminary guideline, it should be planned to flatten all slopes within the development area to no steeper than 3H: 1V (horizontal: vertical).

### **6.5 Site Grading and Ground Improvement**

As described previously, the fill materials on this site were apparently placed under uncontrolled conditions and are therefore highly variable in composition and compactness. These fill materials cannot be relied upon to support foundations, floor slabs, or grade-sensitive services. The fill materials are likely still consolidating under their own self weight and could settle significantly if stressed by additional load. The magnitude of the potential settlements cannot be predicted with any accuracy but would be significant. Even without the addition of further load, it could be expected that the fill materials would continue to settle over many years.

It is therefore proposed that consideration be given to carrying out a ground improvement program for this site. Since the fill materials are highly variable in nature (consisting of a mixture of clay, silt, sand, and gravel with organic matter), it might be feasible to densify these materials in-place using ground improvement methods, such as Geopier Rammed Aggregate Pier Impact System (RAP) or Geopier GeoConcrete Columns (GCC). Rammed Aggregate Piers are a ground improvement method whereby the soil is densified by installing closely spaced columns of compacted granular material (clear stone). Geopier GeoConcrete Columns is a ground improvement method that might be more feasible for buildings with higher loads, which involves the installation of concrete columns within the soil by pumping ready-mix concrete into the soil under pressure.

The ground improvement programs above would likely allow for the densified fill to have adequate capacity to support the building loads. These ground improvement programs would also permit slab on grade floor slabs, sewers, and watermains to be supported within the fill material.

In regards to the site grading, although the placement of additional fill materials could add further load and increase the magnitude of potential long-term settlements, it is expected that this effect could be mitigated by the ground improvement program. From that perspective, there is not considered to be a restrictive limit on the permissible grade raise for this site (although significant grade raises could negatively impact on the stability of the slopes and on the location of the Limit of Hazard Lands).



## **6.6 Foundation Options**

Preliminary development plans indicate a multi-story retirement residence, a multi-story hotel, a retail block, a restaurant, and car dealerships. These buildings would be constructed within the table land on both southern and northern portions of the site.

As discussed in Section 6.5 of this report, the random fill materials that cover most of this site are not suitable for the support of foundations. These materials are variable in composition and state of packing, and were placed under unknown and likely uncontrolled conditions. Foundations supported on these materials could be expected to undergo unpredictable, highly differential, and potentially large settlements. In general, it should be planned to:

- 1) Provide ground densification to the fill materials as described in Section 6.5;
- 2) Remove these materials from beneath structures and replace them with compacted engineered fills; or,
- 3) Extend the foundations through these materials to the more competent native soils.

The first option of ground improvement is likely the most feasible in the South Area where the fill material is the thickest. This will allow for the structures to be founded on conventional spread footing at typical depths within the densified fill.

The second option may be more feasible/applicable to the North Area where the fill materials are thinner. Depending on the design site grading and the design founding level for structures with underground parking, the founding levels may already be below the fill materials.

The third option is likely not a practical option for the South Area. In view of greater thickness of fill material in the South Area (ranging up to 16 metres), it would be unfeasible to subexcavate these materials to reach the competent native material.

Golder had discussions with a ground improvement subcontractor to assess the feasibility of undertaking Geopier Rammed Aggregate Pier Impact System or Geopier GeoConcrete Columns systems for the fill material at the site. After the densification of the existing fill, the bearing resistance for high capacity spread footings on the fill may be increased up to about 250 to 400 kilopascals at Serviceability Limit States, however this value is dependent on the success of the ground improvement methods.

Furthermore, where the fill thickness is greater than about 10 metres (i.e., southern portion of the site), it is expected that densification of the full thickness of the fill by either Dynamic Compaction or Rapid Impact Compaction may not be feasible.

The native sand underlying the fill deposits in the North Area is inferred to be located between elevations 86 and 89 metres and is typically compact to dense. This stratum would potentially be suitable for the support of the retirement residence foundation loads on spread footings on these native soils. An Ultimate Limit States (ULS) factored bearing resistance for spread footings in the order of 300 kilopascals and a Serviceability Limit States (SLS) resistance of 200 kilopascals could be assumed for preliminary design, however these values are highly dependent on the founding level and foundation geometry and are difficult to confirm/evaluate at this preliminary stage.



For the hotel, retail block, restaurant, and car dealerships, where the surface of the native sand deposit is located well below the likely founding level, and/or if the above mentioned bearing pressure values are not sufficient for the retirement residence, then consideration can be given to supporting the buildings on the following deep foundation options:

- Driven steel piles (either pipe piles or H-piles) end-bearing on the bedrock surface at unknown depth (but likely 20 to 25 metres or more below basement level). The piles may however have difficulty penetrating the sandy deposits to that depth.
- Driven steel 'friction' piles, supported within the sand deposit, but with lower capacity than piles end-bearing on the bedrock.
- Expanded base concrete piles (such as Franki piles), advanced to below the existing fill materials and bearing within the native compact to very dense sand deposits. There are however very few contractors with the equipment to construct this pile system. It is relatively costly, slow to construct, and no longer commonly used.
- Cast-in-place concrete caissons, socketed into the bedrock at depth. However, this system is unlikely to be economical considering the significant depth to bedrock at this site.
- Continuous Flight Auger (CFA) piles, founded in the sand deposit. CFA piles consist of cast-in-place concrete piles installed using a hollow-auger system.

The choice of foundation type will likely depend on the particular subsurface conditions at each building location and the required capacities.

## **6.7 Floor Slab Construction**

Floor slabs should not be constructed on the unimproved fill materials. Excessive settlement could occur for floor slabs constructed on the fill materials. The fill materials could alternatively be densified (per the ground improvement program described in Section 6.5 of this report) or, where feasible, subexcavated and replaced with compacted engineered fill.

Within the North Area, if the buildings are provided with basement levels, it may be feasible to construct the slabs as slabs-on-grade on the native competent sand. However, in the South Area, where there exists up to about 16 metres of fill, construction of slabs-on-grade would require densification of under-slab fill, or structural slabs could be used.

Deeper basement levels would potentially extend below the groundwater level. This is considered to be a particular issue only for the retirement residence and hotel (which it is assumed to have one to two basement levels). Given the expected high permeability of the sandy deposits at this site, the rate of groundwater inflow to the foundation drainage system could be high and may require a dewatering program.

Under the new regulations, a Permit-To-Take-Water (PTTW) is required from the Ministry of the Environment and Climate Change (MOECC) if a volume of water greater than 400,000 litres per day is pumped from the excavations. If the volume of water to be pumped will be less than 400,000 litres per day, but more than 50,000 litres per day, the water taking will not require a PTTW, but will need to be registered in the Environmental Activity and Sector Registry (EASR) as a prescribed activity. The groundwater level and hydrogeologic conditions in this area should be confirmed before the grading design and basement levels are finalized to assess the need for a Permit-To-Take-Water.



## **6.8 Excavation**

It is assumed that basement levels will be required for the proposed retirement residence and hotel, at depths of up to 10 metres below the existing ground surface. The groundwater level was reported to be between elevations 76 and 78 metres within the South Area and between elevations 85 and 89 within the North Area.

Excavations for the construction of the buildings or services would likely be carried out within the fill materials above the groundwater level; however, the excavation for the retirement residence and hotel may potentially be slightly below the groundwater level.

No unusual problems are anticipated in excavating in the overburden using conventional hydraulic excavating equipment although some significant boulder or rubble excavation could be required. Based on the groundwater level data, most of the excavations would be carried out above the groundwater level, and hence no significant issue with respect to groundwater control is generally anticipated. In accordance with the Occupational Health and Safety Act (OHSA) of Ontario, the existing fill above the groundwater table would generally be classified as Type 3 soils. Accordingly, side slopes in these materials may be temporarily sloped at no steeper than 1 horizontal to 1 vertical from the bottom of the excavation (i.e., Type 3 soil).

The exception could be the retirement residence and hotel where the excavation level may approach, or extend slightly below, the groundwater level; the groundwater level in the immediate area of these structures will need to be confirmed. In these areas, the soils below the groundwater level would be classified as Type 4 soils, and the excavation side slopes would need to be sloped no steeper than 3 horizontal to 1 vertical.

If excavation needs to be carried out below groundwater level, then an active groundwater management program, such as pumping from wells or well points around the excavation, would be required. The rate of pumping could be very high. As discussed above, a Permit-To-Take-Water would need to be obtained from the Ministry of the Environment and Climate Change. An evaluation of the impacts of the groundwater level lowering on the settlement of surrounding structures would be required as part of that permit application. The disposal options for the pumped groundwater would also need to be evaluated. Given the permeable ground conditions and related issues, it is recommended that excavations below the groundwater level on this site, for both foundations and services, be avoided.

## **6.9 Site Service, Roadways and Parking**

The subgrade for the site services, roadways, and parking areas will consist of an up to 16 metre thick layer of the random fill that covers the site, and which was placed under un-controlled compaction conditions.

It will not be generally feasible to re-compact the full thickness of that fill since ground improvement methods, such as dynamic compaction and rapid impact compaction, would be ineffective in the potentially clayey fill material. Subgrade treatment should therefore consist of surface compaction by means of proof rolling with a heavy smooth drum roller. There will be some potential for post-construction settlement due to long term consolidation of the deeper fill. But those settlements should not be excessive and, given the slope of the site, should probably not be noticeable or impact on surface drainage.

Where main services for the building will be constructed within/through the fill, some subexcavation of the fill beneath the pipes and replacement with compacted engineered fill could be required (if the services will be shallow enough to be within the fill, rather than below). Consideration could also be given to increasing the normal sanitary and storm sewer pipe gradients in the fill and to providing flexible joints at the connections between the pipe



---

## PRELIMINARY GEOTECHNICAL ASSESSMENT HUNT CLUB ROAD AND RIVERSIDE DRIVE

---

sections. Flexible connections or sub-excavation of the fills would be required where the service pipes enter buildings. Watermains could be provided with joints that are both flexible yet restrained, if loss of pipe integrity is to be avoided. Welded HDPE pipe or similar systems could be considered. Service pipes could also potentially be supported on piles.

Shallow piping for surface water collection (e.g., catchbasin leads) can probably be constructed directly within the fill since any pipe settlement would probably conform to the pavement settlements.

The effects of fill settlement of the service pipes and pavements could be mitigated by means of a general ground improvement program to densify the fill materials (see Section 6.4 of this report).





## 7.0 CLOSURE

The guidelines provided in this report are based on a limited amount of factual information and are intended solely for the planning of the proposed redevelopment. The detailed design of this development will require site-specific geotechnical investigations and design guidance.

Golder Associates should be retained to review the final drawings and specifications for this project prior to tendering to ensure that the guidelines in this report have been adequately interpreted.

### GOLDER ASSOCIATES LTD.

*W. Alex Meacoe*  
Alex Meacoe, P.Eng.  
Geotechnical Engineer



*Michael Snow*  
Michael Snow, P.Eng.  
Principal, Senior Geotechnical Engineer

WAM/MSS/mvrd

\\golder.gds\gal\ottawa\active\2016\3 proj\1670692 taggart riverside & hunt club ottawa\08\_reports\1670692 prt-001 geotechnical study march 2018.docx

## **IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT**

**Standard of Care:** Golder Associates Ltd. (Golder) has prepared this report in a manner consistent with that level of care and skill ordinarily exercised by members of the engineering and science professions currently practicing under similar conditions in the jurisdiction in which the services are provided, subject to the time limits and physical constraints applicable to this report. No other warranty, expressed or implied is made.

**Basis and Use of the Report:** This report has been prepared for the specific site, design objective, development and purpose described to Golder by the Client, Taggart Realty Management. The factual data, interpretations and recommendations pertain to a specific project as described in this report and are not applicable to any other project or site location. Any change of site conditions, purpose, development plans or if the project is not initiated within eighteen months of the date of the report may alter the validity of the report. Golder cannot be responsible for use of this report, or portions thereof, unless Golder is requested to review and, if necessary, revise the report.

The information, recommendations and opinions expressed in this report are for the sole benefit of the Client. No other party may use or rely on this report or any portion thereof without Golder's express written consent. If the report was prepared to be included for a specific permit application process, then the client may authorize the use of this report for such purpose by the regulatory agency as an Approved User for the specific and identified purpose of the applicable permit review process, provided this report is not noted to be a draft or preliminary report, and is specifically relevant to the project for which the application is being made. Any other use of this report by others is prohibited and is without responsibility to Golder. The report, all plans, data, drawings and other documents as well as all electronic media prepared by Golder are considered its professional work product and shall remain the copyright property of Golder, who authorizes only the Client and Approved Users to make copies of the report, but only in such quantities as are reasonably necessary for the use of the report by those parties. The Client and Approved Users may not give, lend, sell, or otherwise make available the report or any portion thereof to any other party without the express written permission of Golder. The Client acknowledges that electronic media is susceptible to unauthorized modification, deterioration and incompatibility and therefore the Client cannot rely upon the electronic media versions of Golder's report or other work products.

The report is of a summary nature and is not intended to stand alone without reference to the instructions given to Golder by the Client, communications between Golder and the Client, and to any other reports prepared by Golder for the Client relative to the specific site described in the report. In order to properly understand the suggestions, recommendations and opinions expressed in this report, reference must be made to the whole of the report. Golder cannot be responsible for use of portions of the report without reference to the entire report.

Unless otherwise stated, the suggestions, recommendations and opinions given in this report are intended only for the guidance of the Client in the design of the specific project. The extent and detail of investigations, including the number of test holes, necessary to determine all of the relevant conditions which may affect construction costs would normally be greater than has been carried out for design purposes. Contractors bidding on, or undertaking the work, should rely on their own investigations, as well as their own interpretations of the factual data presented in the report, as to how subsurface conditions may affect their work, including but not limited to proposed construction techniques, schedule, safety and equipment capabilities.

**Soil, Rock and Groundwater Conditions:** Classification and identification of soils, rocks, and geologic units have been based on commonly accepted methods employed in the practice of geotechnical engineering and related disciplines. Classification and identification of the type and condition of these materials or units involves judgment, and boundaries between different soil, rock or geologic types or units may be transitional rather than abrupt. Accordingly, Golder does not warrant or guarantee the exactness of the descriptions.

## **IMPORTANT INFORMATION AND LIMITATIONS OF THIS REPORT (cont'd)**

Special risks occur whenever engineering or related disciplines are applied to identify subsurface conditions and even a comprehensive investigation, sampling and testing program may fail to detect all or certain subsurface conditions. The environmental, geologic, geotechnical, geochemical and hydrogeologic conditions that Golder interprets to exist between and beyond sampling points may differ from those that actually exist. In addition to soil variability, fill of variable physical and chemical composition can be present over portions of the site or on adjacent properties. **The professional services retained for this project include only the geotechnical aspects of the subsurface conditions at the site, unless otherwise specifically stated and identified in the report.** The presence or implication(s) of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this project and have not been investigated or addressed.

Soil and groundwater conditions shown in the factual data and described in the report are the observed conditions at the time of their determination or measurement. Unless otherwise noted, those conditions form the basis of the recommendations in the report. Groundwater conditions may vary between and beyond reported locations and can be affected by annual, seasonal and meteorological conditions. The condition of the soil, rock and groundwater may be significantly altered by construction activities (traffic, excavation, groundwater level lowering, pile driving, blasting, etc.) on the site or on adjacent sites. Excavation may expose the soils to changes due to wetting, drying or frost. Unless otherwise indicated the soil must be protected from these changes during construction.

**Sample Disposal:** Golder will dispose of all uncontaminated soil and/or rock samples 90 days following issue of this report or, upon written request of the Client, will store uncontaminated samples and materials at the Client's expense. In the event that actual contaminated soils, fills or groundwater are encountered or are inferred to be present, all contaminated samples shall remain the property and responsibility of the Client for proper disposal.

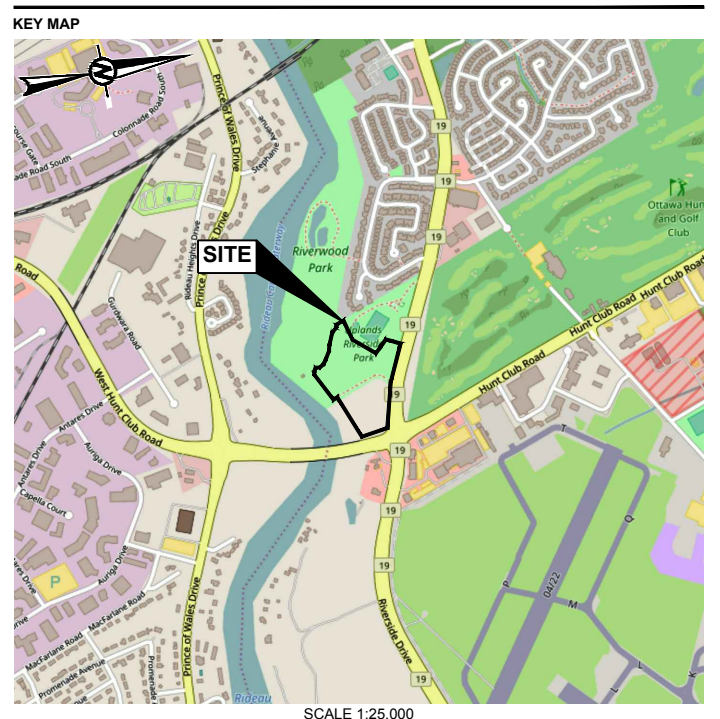
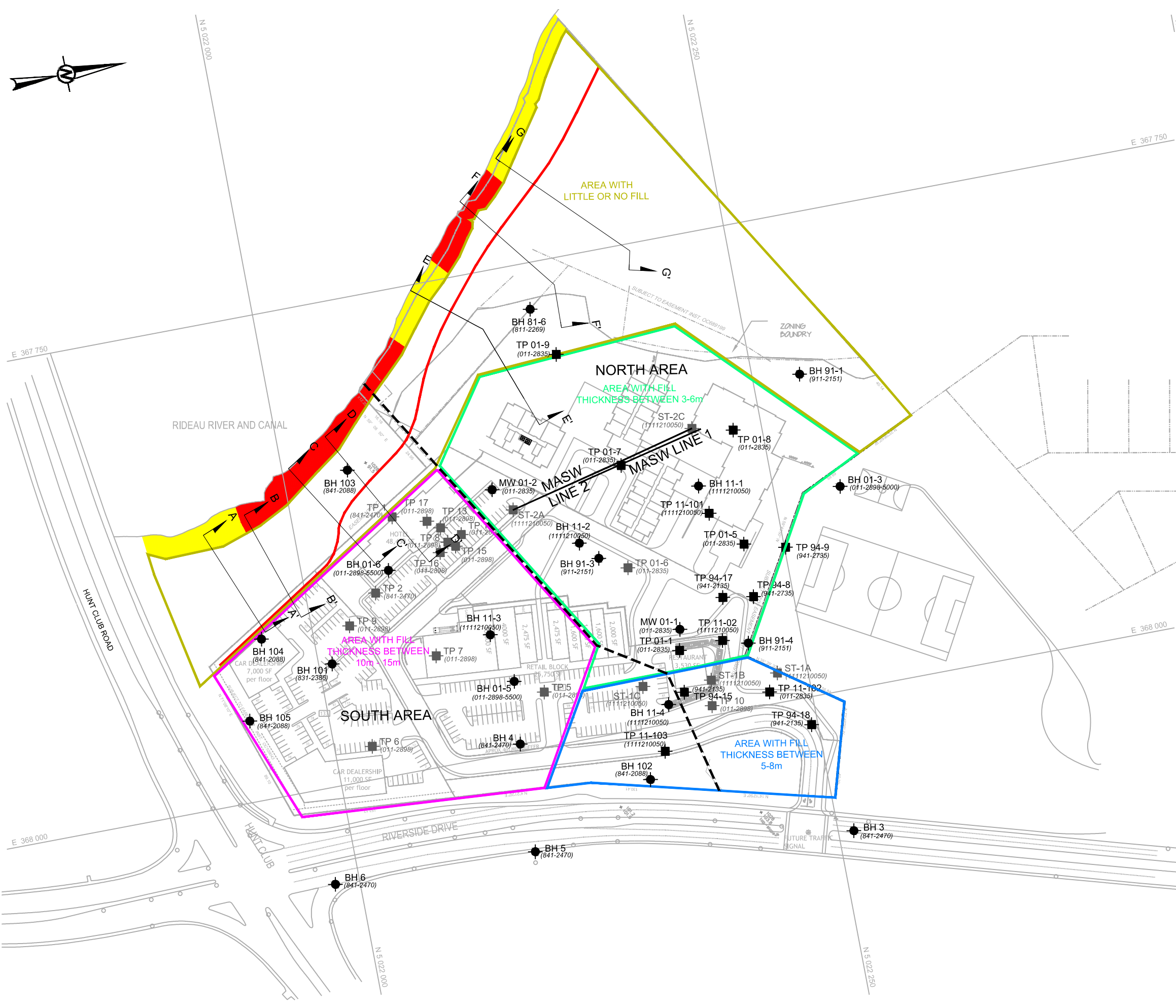
**Follow-Up and Construction Services:** All details of the design were not known at the time of submission of Golder's report. Golder should be retained to review the final design, project plans and documents prior to construction, to confirm that they are consistent with the intent of Golder's report.

During construction, Golder should be retained to perform sufficient and timely observations of encountered conditions to confirm and document that the subsurface conditions do not materially differ from those interpreted conditions considered in the preparation of Golder's report and to confirm and document that construction activities do not adversely affect the suggestions, recommendations and opinions contained in Golder's report. Adequate field review, observation and testing during construction are necessary for Golder to be able to provide letters of assurance, in accordance with the requirements of many regulatory authorities. In cases where this recommendation is not followed, Golder's responsibility is limited to interpreting accurately the information encountered at the borehole locations, at the time of their initial determination or measurement during the preparation of the Report.

**Changed Conditions and Drainage:** Where conditions encountered at the site differ significantly from those anticipated in this report, either due to natural variability of subsurface conditions or construction activities, it is a condition of this report that Golder be notified of any changes and be provided with an opportunity to review or revise the recommendations within this report. Recognition of changed soil and rock conditions requires experience and it is recommended that Golder be employed to visit the site with sufficient frequency to detect if conditions have changed significantly.

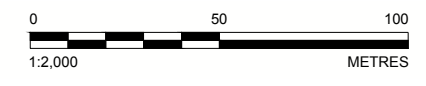
Drainage of subsurface water is commonly required either for temporary or permanent installations for the project. Improper design or construction of drainage or dewatering can have serious consequences. Golder takes no responsibility for the effects of drainage unless specifically involved in the detailed design and construction monitoring of the system.

Path: \\golder\gpc\gpc\external\active\spatial\mkt\taggart\Riverside-HuntClub\09\_PROJ\1670692\_Taggart\_CenDev\env\env\40\_PROJ\0002\_Geotech | File Name: 1670692-0009-BG-0001.dwg | Last Edited By: jproccornell Date: 2018-03-19 Time: 3:49:20 PM | Printed By: jproccornell Date: 2018-03-19 Time: 3:50:16 PM



- LEGEND**
- APPROXIMATE BOREHOLE LOCATION, PREVIOUS INVESTIGATION BY GOLDER
  - APPROXIMATE TEST PIT LOCATION, PREVIOUS INVESTIGATION BY GOLDER
  - A' APPROXIMATE SLOPE CROSS-SECTION LOCATION IN PLAN
  - LIMIT OF HAZARD LANDS
  - - - APPROXIMATE BOUNDARY BETWEEN ASSESSMENT AREAS
  - AREA WITH MODERATE TO SEVERE EROSION
  - AREA WITH MINOR TO MODERATE EROSION

**REFERENCE(S)**  
 1. PROJECTION: TRANSVERSE MERCATOR DATUM: NAD 83,  
 COORDINATE SYSTEM: MTM ZONE 9, VERTICAL DATUM: CGVD28



CLIENT  
**ST. MARY'S LANDS CORPORATION**

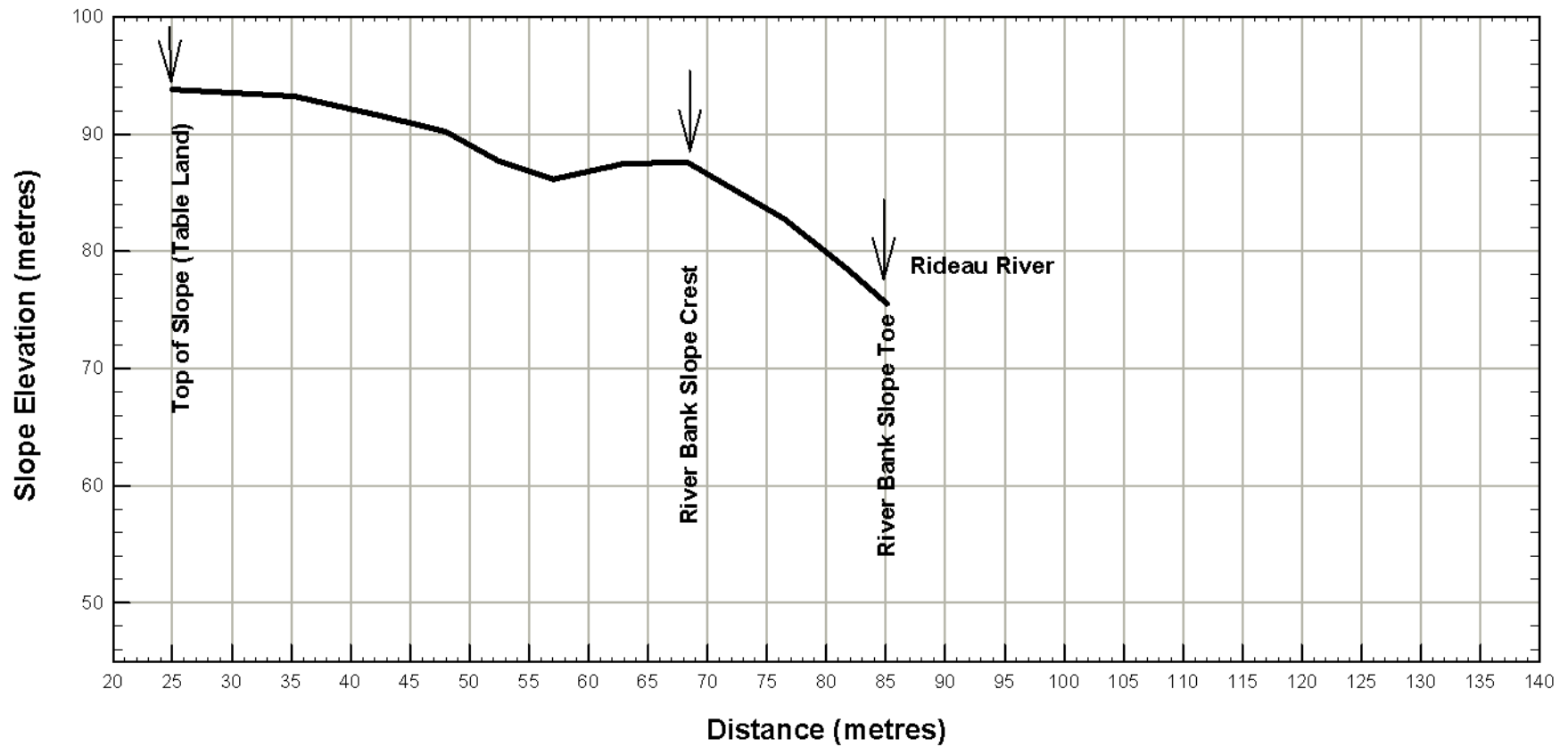
PROJECT  
**GEOTECHNICAL INVESTIGATION  
 PROPOSED DEVELOPMENT  
 HUNT CLUB ROAD AND RIVERSIDE DRIVE, OTTAWA, ONTARIO**

TITLE  
**SITE PLAN**

CONSULTANT	DATE	REVISION
	YYYY-MM-DD	2018-03-19
	DESIGNED	---
	PREPARED	JM
	REVIEWED	WAM
	APPROVED	MSS

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM A3/B3

# Cross Section A-A'



Date: May 8, 2017

Drawn: WAM

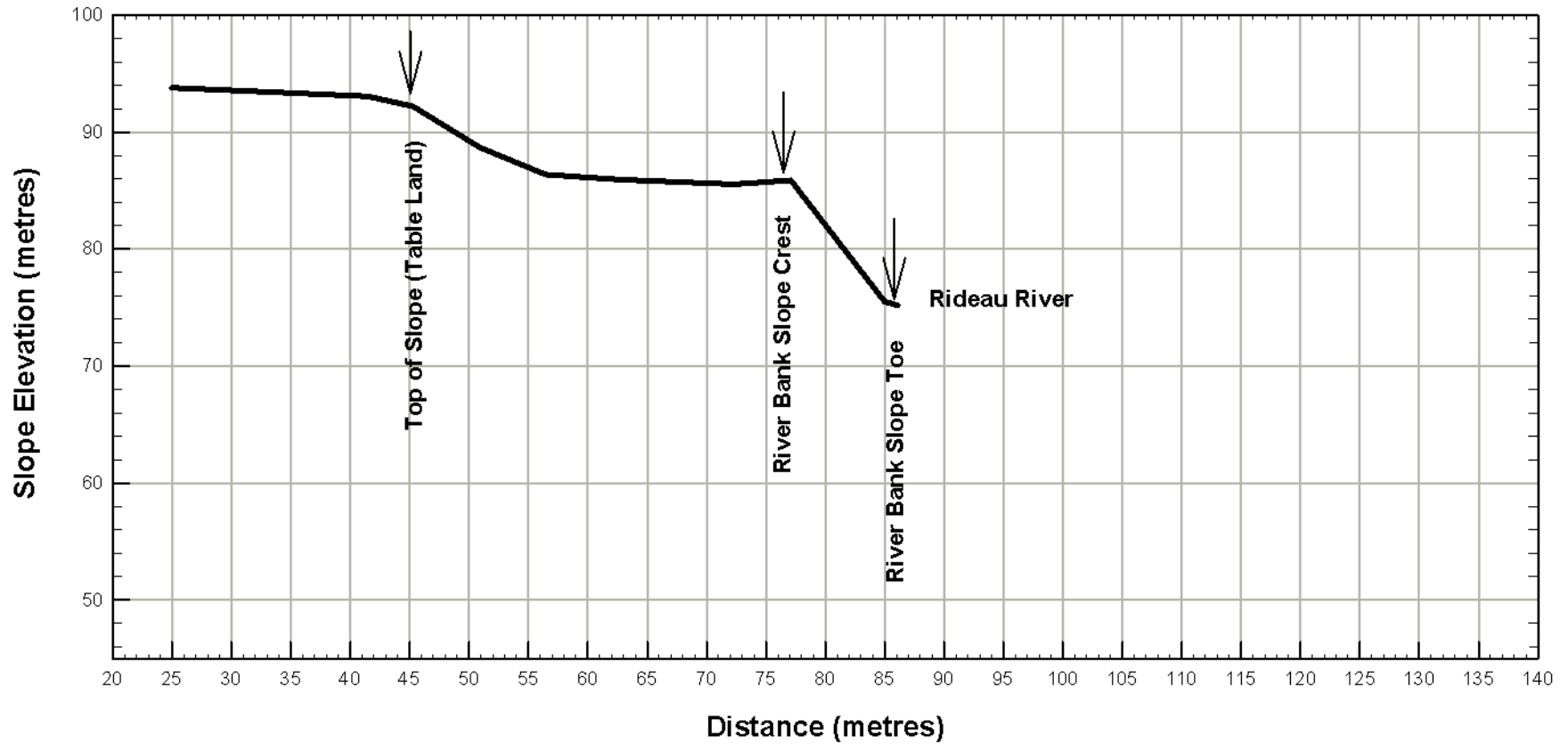
Project: 1670692

Chkd:

**SLOPE CROSS SECTION**

**FIGURE 2**

# Cross Section B-B'



Date: May 8, 2017

Drawn: WAM

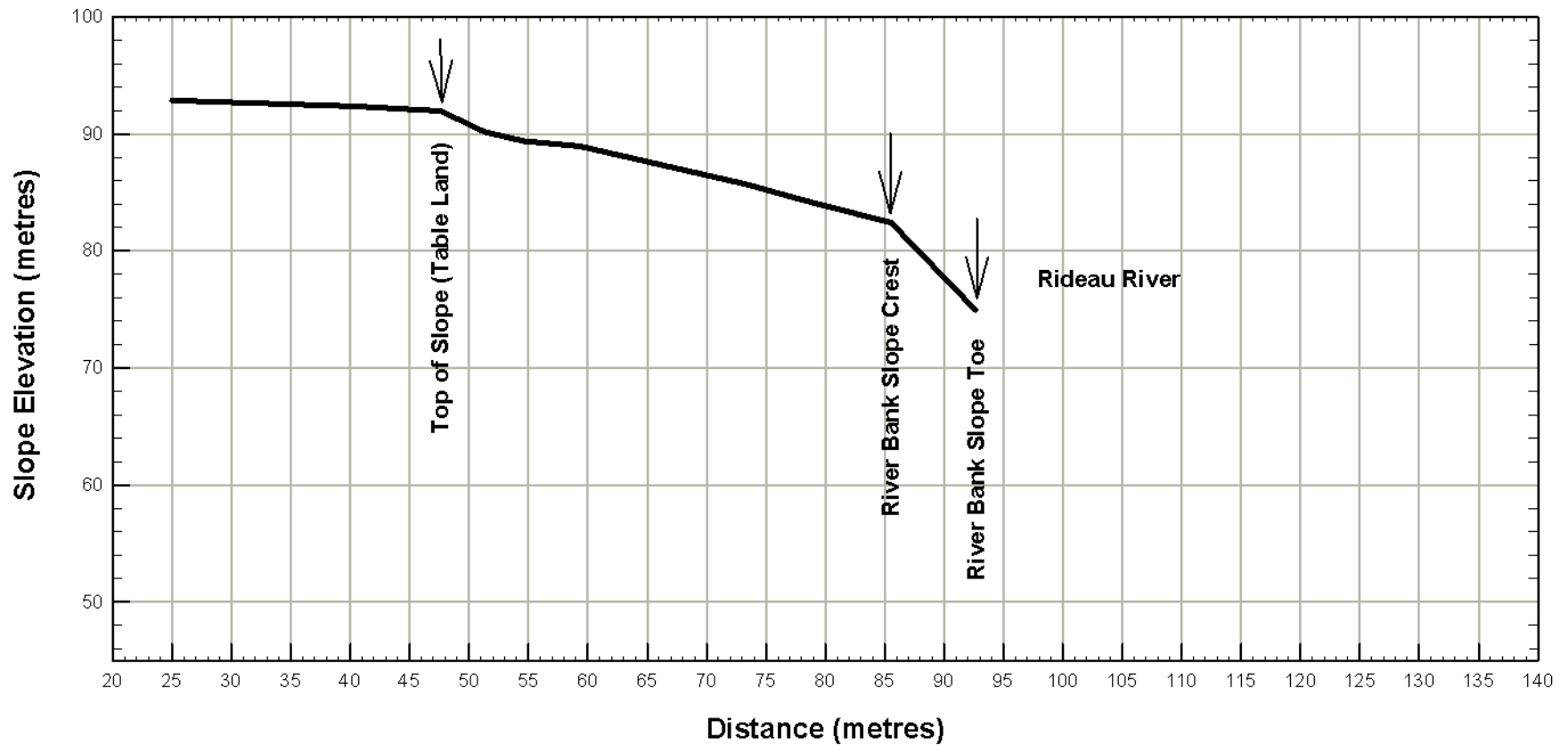
Project: 1670692

Chkd:

**SLOPE CROSS SECTION**

**FIGURE 3**

# Cross Section C-C'



Date: May 8, 2017

Drawn: WAM

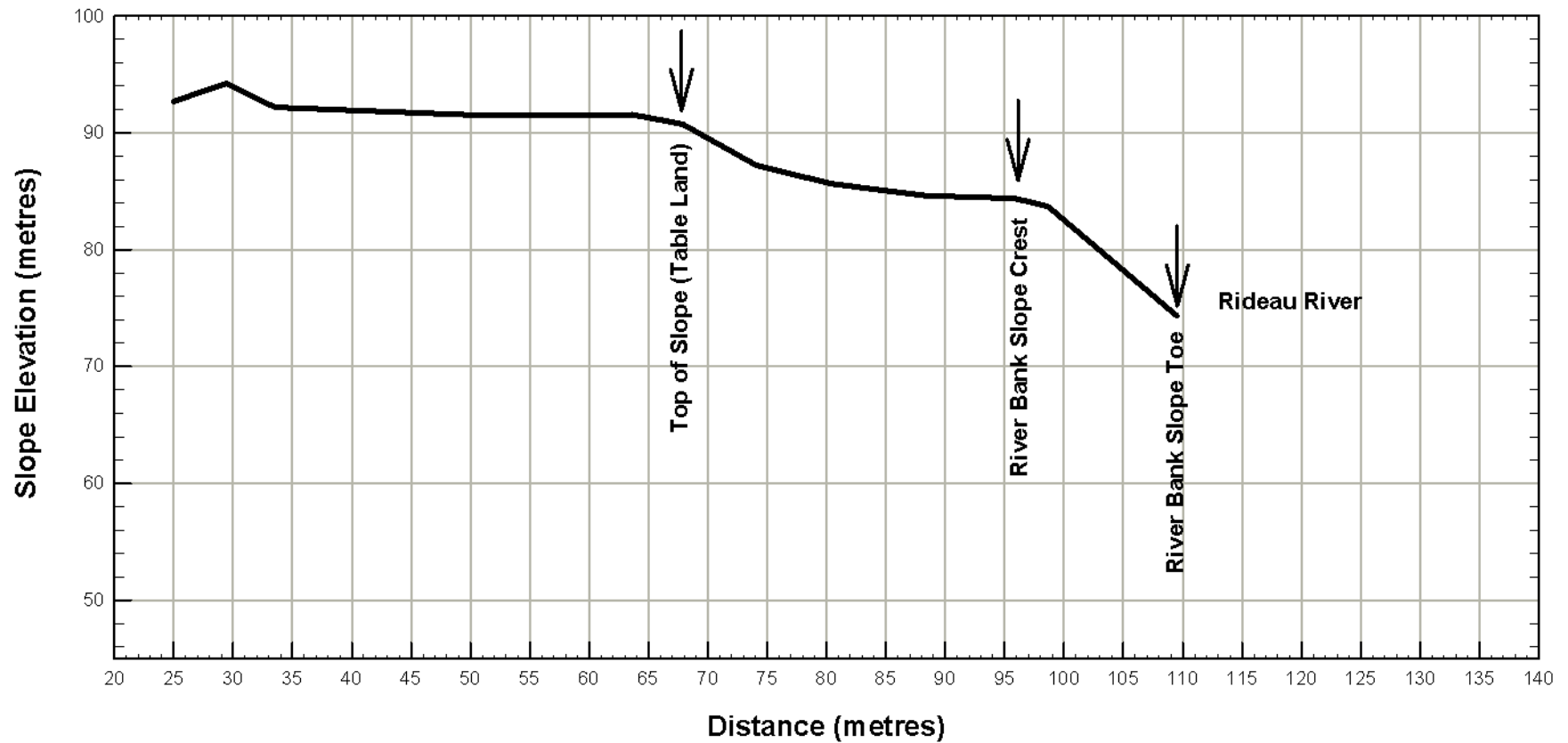
Project: 1670692

Chkd:

**SLOPE CROSS SECTION**

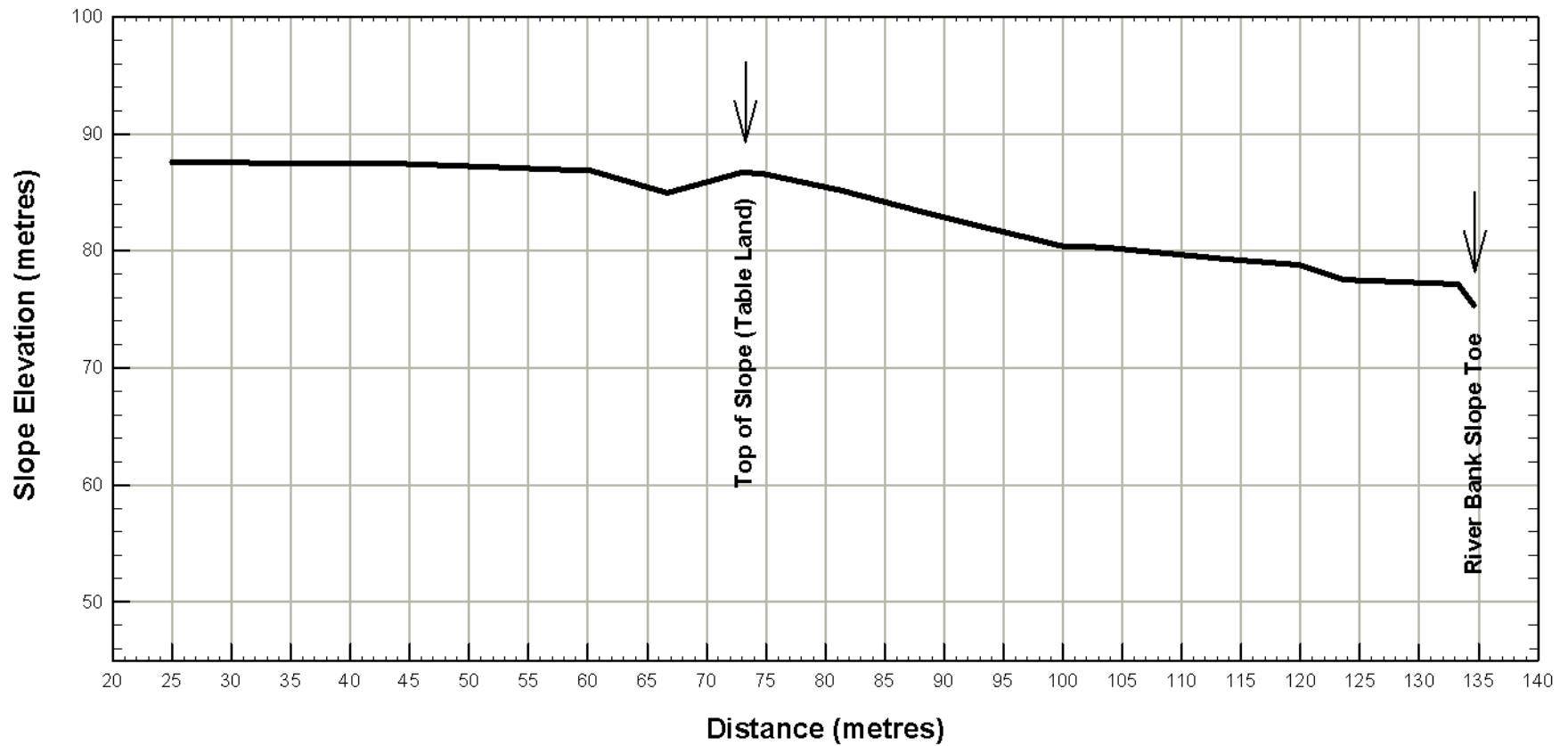
**FIGURE 4**

## Cross Section D-D'





# Cross Section E-E'



Date: May 8, 2017

Drawn: WAM

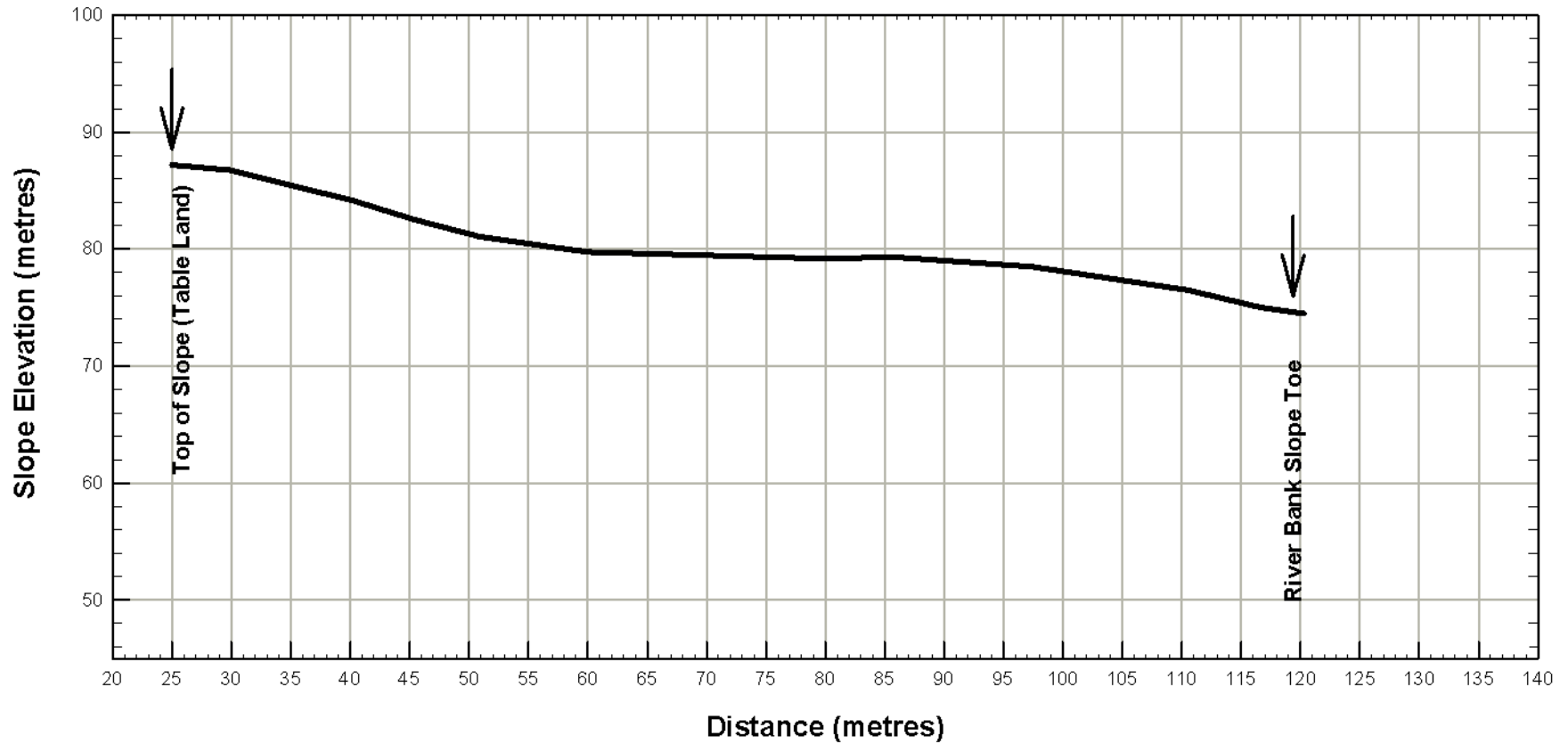
Project: 1670692

Chkd:

**SLOPE CROSS SECTION**

**FIGURE 6**

# Cross Section F-F'



Date: May 8, 2017

Drawn: WAM

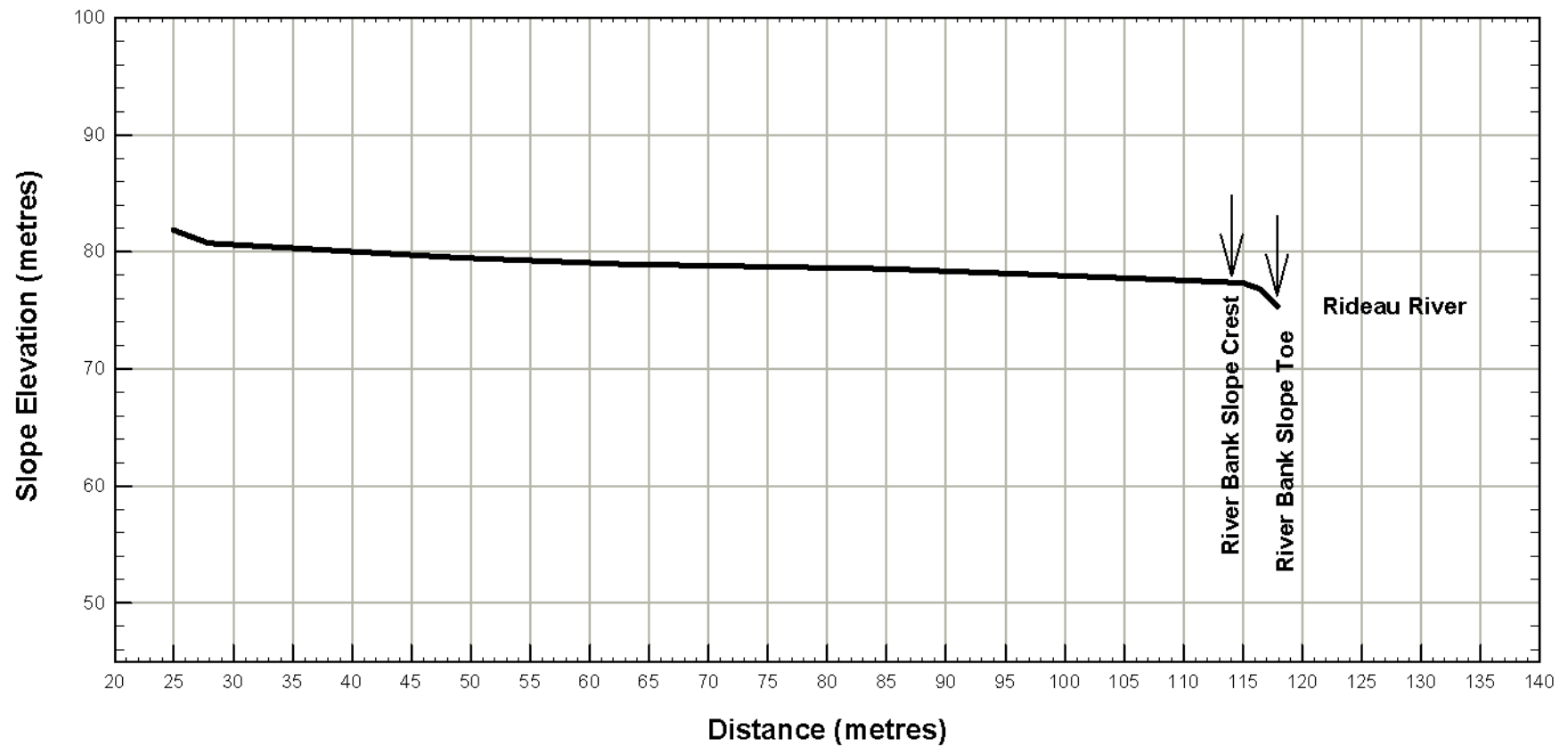
Project: 1670692

Chkd:

**SLOPE CROSS SECTION**

**FIGURE 7**

# Cross Section G-G'



Date: May 8, 2017

Drawn: WAM

Project: 1670692

Chkd:

**SLOPE CROSS SECTION**

**FIGURE 8**



# **APPENDIX A**

## **MASW Testing Report – Previous Investigation**

**DATE** April 5, 2011**PROJECT No.** 11-1121-0050**TO** Mike Cunningham  
Golder Associates Ltd.**CC****FROM** Stephane Sol, Christopher Phillips**EMAIL** ssol@golder.com, cphillips@golder.com**NBCC SEISMIC SITE CLASS TESTING RESULTS – ST. MARY’S SITE, OTTAWA, ONTARIO**

---

This technical memorandum presents the processing and results of two Multichannel Analysis of Surface Waves (MASW) tests performed for the purpose of National Building Code of Canada Seismic Site Classification for a site located Northwest of the intersection of Hunt Club Road and Riverside Drive in Ottawa, Ontario. The geophysical testing was performed by Golder personnel on April 1, 2011.

**Methodology**

The Multichannel Analysis of Surface Waves (MASW) method measures variations in surface wave velocity with increasing distance and wavelength and can be used to infer the rock/soil types, stratigraphy and soil conditions.

A typical MASW survey requires a seismic source, to generate surface-waves, and a minimum of two geophone receivers, to measure the ground response at some distance from the source. Surface waves are a special type of seismic wave whose propagation is confined to the near surface medium.

The depth of penetration of a surface-wave into a medium is directly proportional to its wavelength. In a non-homogeneous medium surface-waves are dispersive, i.e., each wavelength has a characteristic velocity owing to the subsurface heterogeneities within the depth interval that particular wavelength of surface-wave propagates through. The relationship between surface-wave velocity and wavelength is used to obtain the shear-wave velocity and attenuation profile of the medium with increasing depth.

The seismic source used can be either active or passive, depending on the application and location of the survey. Examples of active sources include explosives, weight-drops, sledge hammer and vibrating pads. Examples of passive sources are road traffic, micro-tremors and water-wave action (in near-shore environments).

The geophone receivers measure the wave-train associated with the surface wave travelling from a seismic source at different distances from the source.

The participation of surface-waves with different wavelengths can be determined from the wave-train by transforming the wave-train results into the frequency domain. The surface-wave velocity profile with respect to



wavelength (called the 'dispersion curve') is determined by the delay in wave propagation measured between the geophone receivers. The dispersion curve is then matched to a theoretical dispersion curve using an iterative forward-modelling procedure. The result is a shear-wave velocity profile of the tested medium with depth, which can be used to estimate the dynamic shear modulus of the medium as a function of depth.

## Field Work

The MASW field work was conducted on April 1, 2011, by personnel from the Golder Mississauga and Ottawa offices. The two MASW lines were oriented nearly parallel to Riverside Road. The location of the lines is provided in Table 1. At each line, a shallow trench was dug to remove the frozen layer, which would affect testing results. For both MASW lines, a series of 24 low frequency (4.5 Hz) geophones were laid out at 3 m intervals. A seismic weight drop of 45 kg and a 5.5 kg sledge hammer were used as seismic sources for this investigation. Seismic records were collected with seismic sources located 5, 10 and 20 m from and collinear to the geophone array. An example of an active seismic record collected at MASW Lines 1 and 2 is shown in Figures 1 and 2, respectively (below).

**Table 1: Surveyed MASW Lines**

<b>MASW LINES</b>	<b>Easting (m)</b>	<b>Northing (m)</b>
Line 1 - Start	445,630E	5,020,577N
Line 1 - End	445,638E	5,020,649N
Line 2 - Start	445,520E	5,020,565N
Line 2 - End	445,506E	5,020,631N

Datum: UTM NAD 83, Zone 18

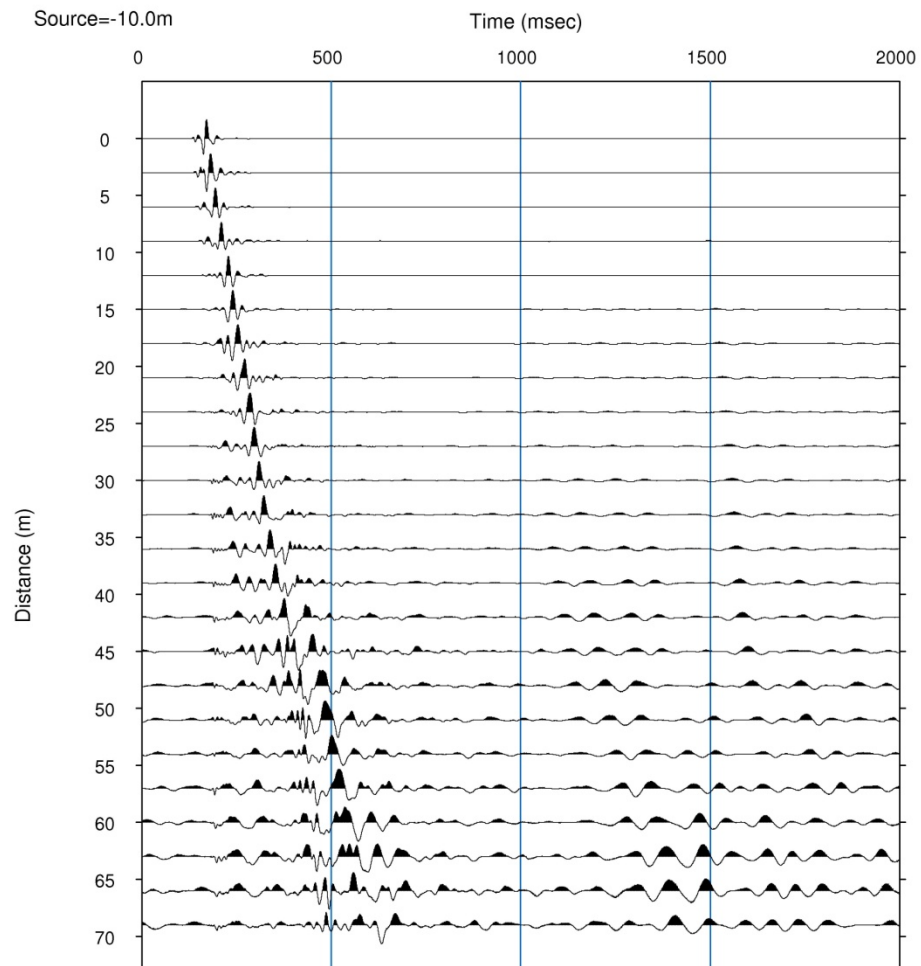


Figure 1: Typical seismic record collected along MASW Line1.

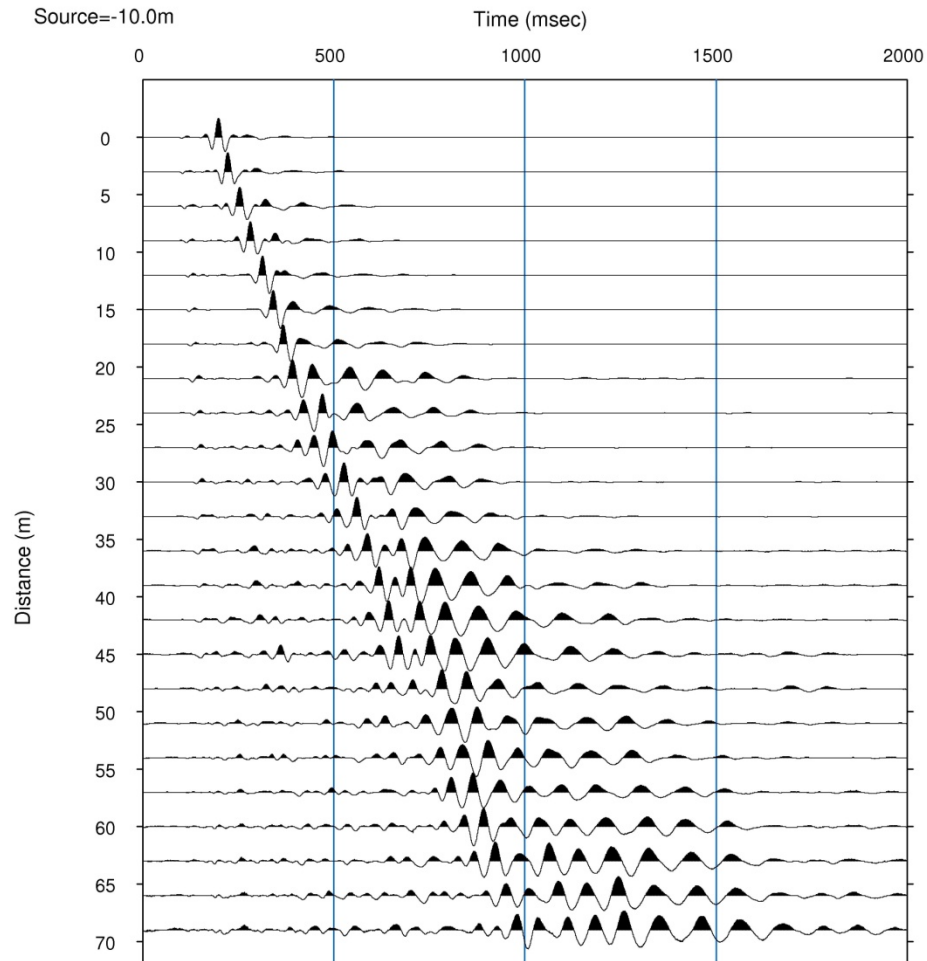


Figure 2: Typical seismic record collected along MASW Line 2.

## Data Processing

Processing of the MASW test results consisted of the following main steps:

- 1) Transformation of the time domain data into the frequency domain using a Fast-Fourier Transform (FFT) for each source location;
- 2) Calculation of the phase for each frequency component;
- 3) Linear regression to calculate phase velocity for each frequency component;
- 4) Filtering of the calculated phase velocities based on the Pearson correlation coefficient ( $r^2$ ) between the data and the linear regression best fit line used to calculate phase velocity;
- 5) Generation of the dispersion curve by combining calculated phase velocities for each shot location of a single MASW test; and
- 6) Generation of the stiffness profile, through forward iterative modelling and matching of model data to the field collected dispersion curve.



Processing of the MASW data was completed using the SeisImager/SW software package (Geometrics Inc.). The calculated phase velocities for a seismic shot point were combined and the dispersion curve generated by choosing the minimum phase velocity calculated for each frequency component as shown on Figures 3 and 4. Shear wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves.

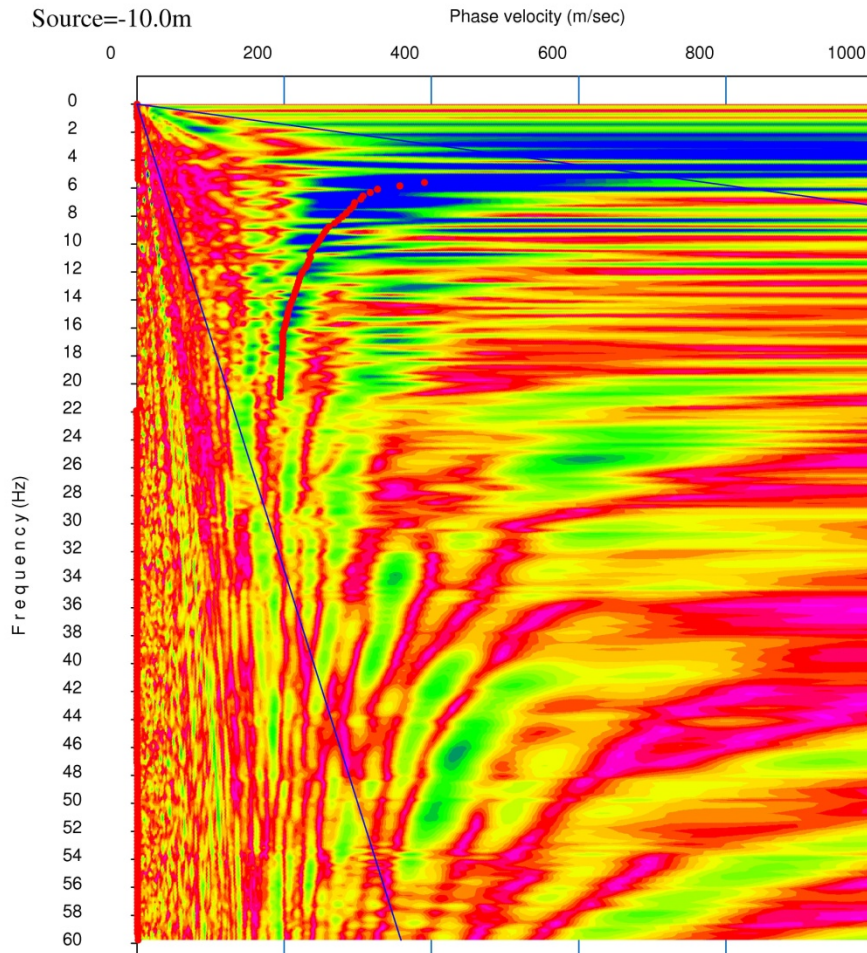


Figure 3: MASW Dispersion Curve Picks for Line 1 (red dots).

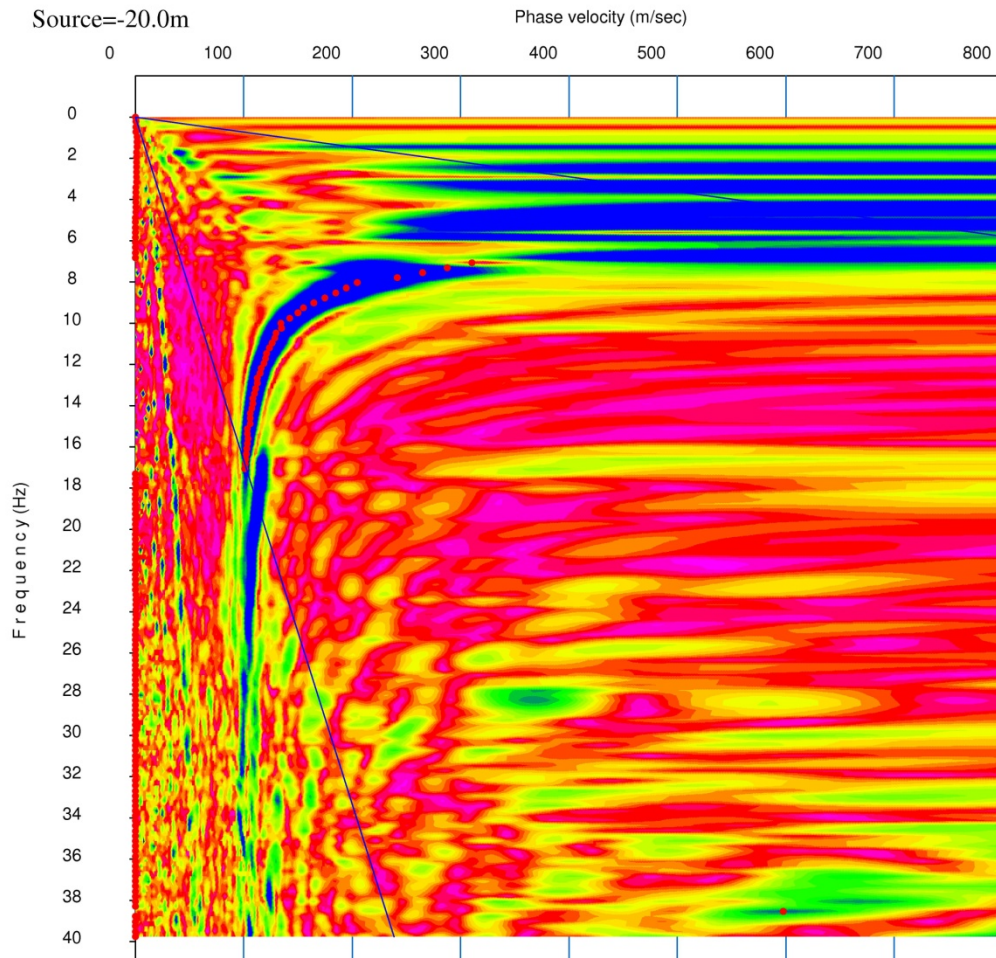


Figure 4: MASW Dispersion Curve Picks for Line 2 (red dots).

The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 6 Hz and 7 Hz for MASW Lines 1 and 2, respectively.

## Results

The MASW test results are presented in Figures 5 and 6, which present the calculated shear wave velocity profiles measured from the field testing at the two locations. The results at each line have been inferred using a weight drop located at 10 m from the first geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figures 7 and 8. At MASW Line 1 there is a good correlation between the field collected and model calculated dispersion curves, with a root mean squared error of 3.5%. At MASW Line 2 there is an excellent correlation between the field collected and model calculated dispersion curves, with a root mean squared error of 0.8%.

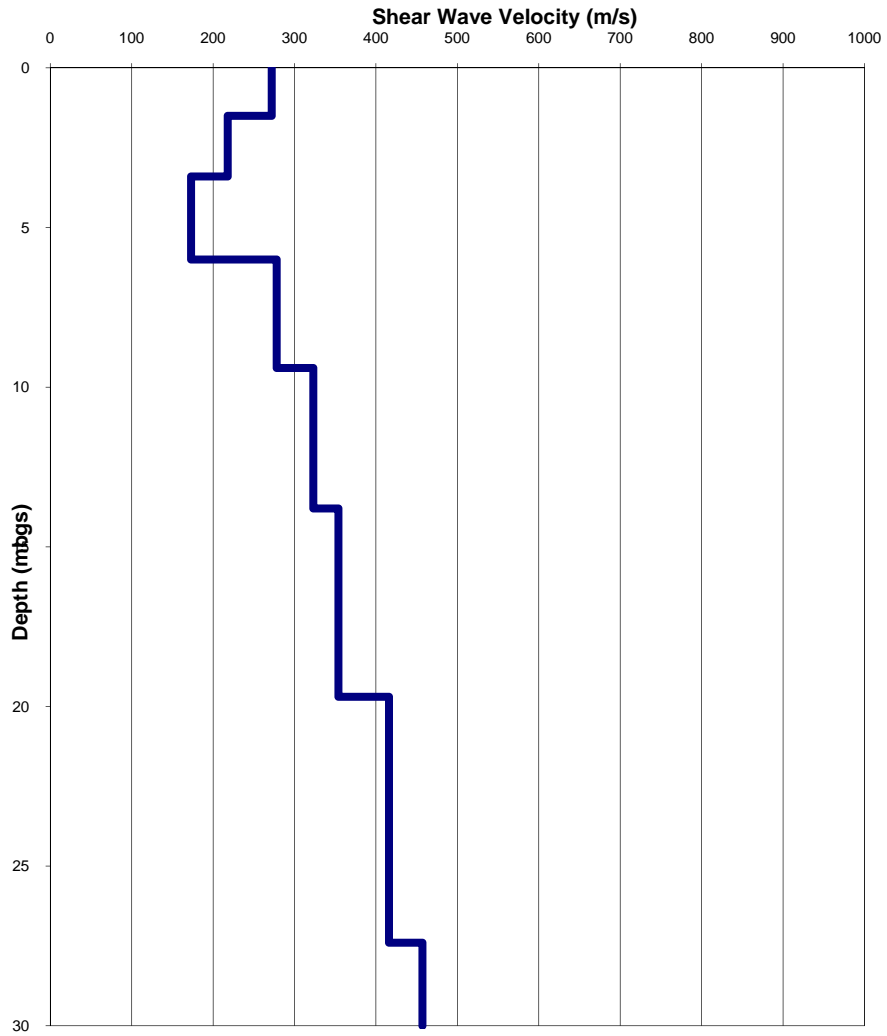


Figure 5: MASW Modelled Shear Wave Velocity Depth profile for MASW Line 1.

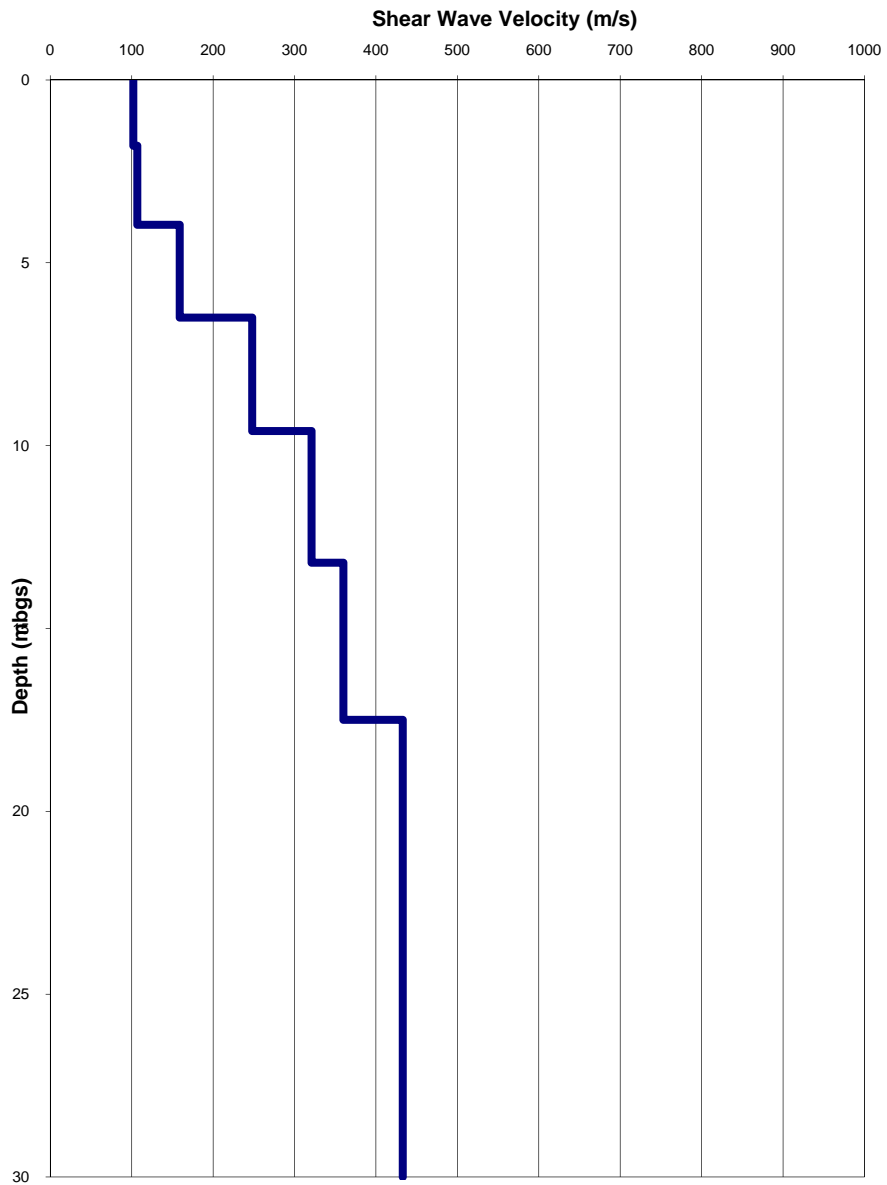


Figure 6: MASW Modelled Shear Wave Velocity Depth profile for MASW Line 2.

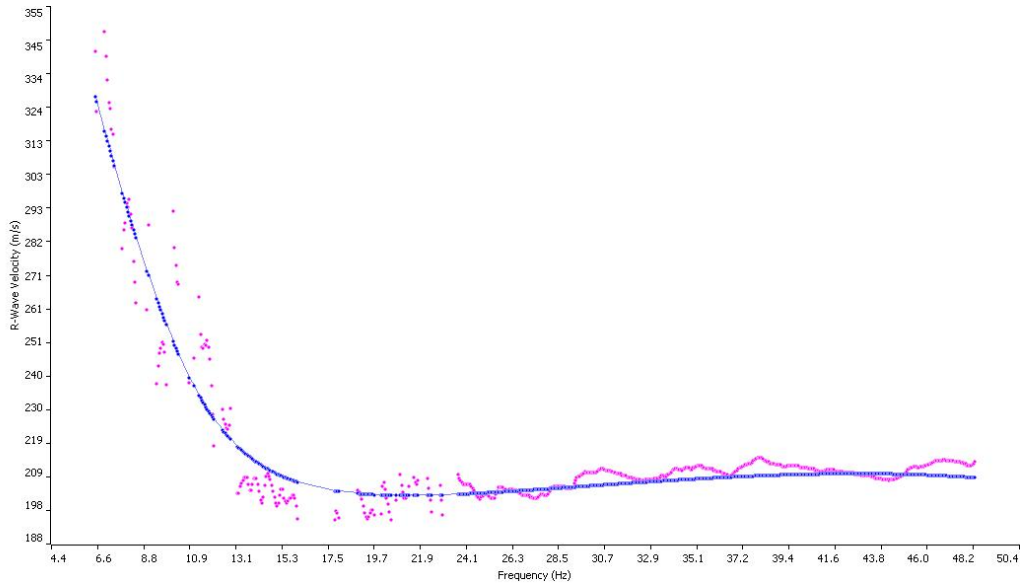


Figure 7: Comparison of Field (pink dots) vs. Modelled Data (blue dots) for the MASW Line 1.

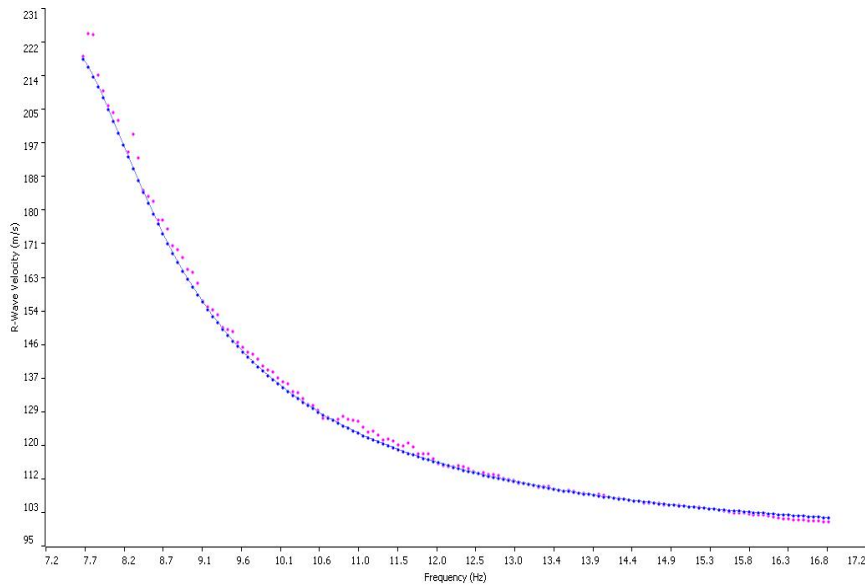


Figure 8: Comparison of Field (pink dots) vs. Modelled Data (blue dots) for the MASW Line 2.

To calculate the average shear wave velocity as required by the National Building Code of Canada, 2005 (NBCC2005), the results were modelled to 30 metres below ground surface.

At MASW Line 1, the limited low frequency content of the dispersion curve did not allow us to sufficiently resolve shear-wave velocities at depth below 27 m. Therefore the average velocity was calculated assuming that the velocity from the maximum resolved depth to a depth of 30 m was constant and equal to the velocity of the maximum resolved depth layer. The average shear-wave velocity was found to be 313 m/s (Table 2).

At MASW Line 2, the limited low frequency content of the dispersion curve did not allow us to sufficiently resolve shear-wave velocities at depth below 17.5 m. Therefore the average velocity was calculated assuming that the velocity from the maximum resolved depth to a depth of 30 m was constant and equal to the velocity of the maximum resolved depth layer. The average shear-wave velocity was found to be 254 m/s (Table 3).

**Table 2: Shear Wave Velocity Profile MASW Line 1**

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.50	1.50	272	0.005515
1.50	3.40	1.90	218	0.008716
3.40	6.00	2.60	173	0.015029
6.00	9.40	3.40	278	0.012230
9.40	13.80	4.40	323	0.013622
13.80	19.70	5.90	354	0.016667
19.70	27.40	7.70	416	0.018510
27.40	30.00	2.60	457	0.005689
<b>Vs Average to 30 mbgs (m/s)</b>				<b>313</b>

**Table 3: Shear Wave Velocity Profile MASW Line 2**

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.80	1.80	102	0.017647
1.80	3.96	2.16	107	0.020187
3.96	6.50	2.54	159	0.015975
6.50	9.60	3.10	248	0.012500
9.60	13.20	3.60	321	0.011215
13.20	17.50	4.30	360	0.011944
17.50	30.00	12.50	433	0.028868
<b>Vs Average to 30 mbgs (m/s)</b>				<b>254</b>

## Closure

We trust that this letter report meets your needs at the present time. If you have any questions or require clarification, please contact the undersigned at your convenience



Stephane Sol, Ph.D.  
Geophysics Group

SS/CRP/wlm



Christopher Phillips, M.Sc.  
Senior Geophysicist, Associate

n:\active\2011\other offices\11-1121-0050 st marys masw\reporting\11-1121-0050 tm st marys masw 05apr11.docx

At Golder Associates we strive to be the most respected global group of companies specializing in ground engineering and environmental services. Employee owned since our formation in 1960, we have created a unique culture with pride in ownership, resulting in long-term organizational stability. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees now operating from offices located throughout Africa, Asia, Australasia, Europe, North America and South America.

Africa	+ 27 11 254 4800
Asia	+ 852 2562 3658
Australasia	+ 61 3 8862 3500
Europe	+ 356 21 42 30 20
North America	+ 1 800 275 3281
South America	+ 55 21 3095 9500

[solutions@golder.com](mailto:solutions@golder.com)  
[www.golder.com](http://www.golder.com)

**Golder Associates Ltd.**  
**1931 Robertson Road**  
**Ottawa, Ontario, K2H 5B7**  
**Canada**  
**T: +1 (613) 592 9600**

