

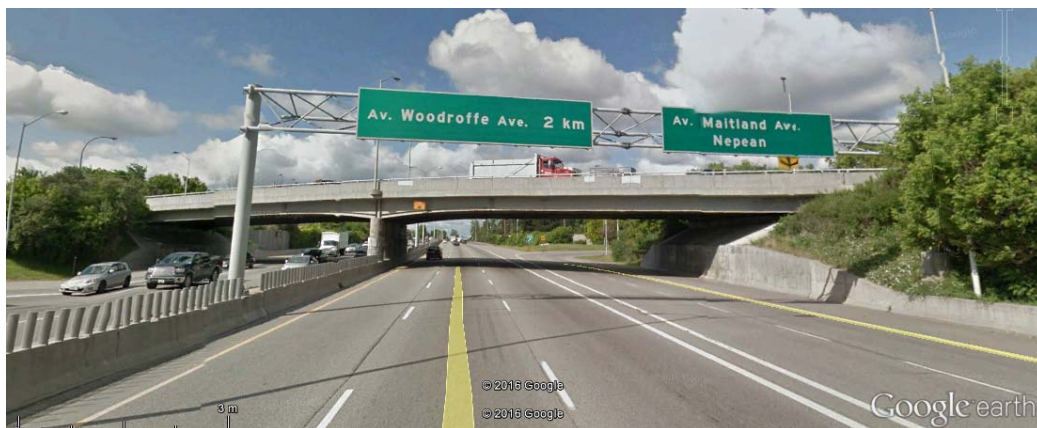


March 2017

## REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN  
MAITLAND AVENUE UNDERPASS  
SITE NO. 3-042  
HIGHWAY 417 WIDENING AND REHABILITATION  
FROM WEST OF HIGHWAY 416 TO EAST OF MAITLAND AVENUE  
G.W.P. 4124-14-00**

**Submitted to:**  
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REPORT



**GEOCRES NO:** 31G5-275

**Report Number:** 1546542-1040

**Distribution:**

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**FOUNDATION REPORT  
HIGHWAY 417 MAITLAND AVENUE UNDERPASS**

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# **PART A**

**FOUNDATION INVESTIGATION REPORT  
MAITLAND AVENUE UNDERPASS  
SITE NO. 3-042  
HIGHWAY 417 WIDENING AND REHABILITATION  
FROM WEST OF HIGHWAY 416 TO EAST OF MAITLAND AVENUE  
G.W.P. 4124-14-00**



### 1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by MMM Group Limited (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the design for the widening of the Maitland Avenue Underpass at Highway 417 in the City of Ottawa. The proposed work is part of the design associated with the Highway 417 widening from west of Highway 416 to east of Maitland Avenue (Assignment Number 4015-E-0017) in Ottawa, Ontario.

This report addresses the proposed widening of the Maitland Avenue Underpass (MTO Structure Site No. 3-042) and the associated retaining walls and underpass approach embankments only. The proposed widening of the existing bridge to the east by 4.4 metres is required to accommodate a lengthened northbound left hand turn lane on Maitland Avenue to westbound Highway 417, due to projected increased traffic demands. The highway platform beneath the Maitland Avenue structure is of sufficient width to accommodate the addition of new travel lanes.

The terms of reference and scope of work for the foundation investigation are outlined in the MTO's Request for Proposal, dated May 2015, and subsequent addenda. Golder's scope of work for foundation engineering services associated with the Maitland Avenue Underpass widening is contained in Table 17.8.3 of MMM's Technical Proposal for this assignment. The work has been carried out in accordance with Golder's Quality Control Plan for foundation engineering services for this project, dated May 2, 2016.



## 2.0 SITE DESCRIPTION AND GEOLOGY

### 2.1 Site Description

The Maitland Avenue Underpass is located within a mixed use (commercial-residential) area of the City of Ottawa, and is located approximately 6 kilometers east of the junction of Highway 416 and Highway 417. At this location, Highway 417 is a divided highway with three travel lanes in each direction separated by a concrete median. In the westbound direction, there are two off-ramp lanes exiting to Maitland Avenue. In the eastbound direction, there is a single off-ramp lane with a wide shoulder.

The existing Maitland Avenue Underpass is a two span continuous steel girder bridge with composite concrete deck. The spans are each approximately 35 m in length. The central piers are founded on spread footings on rock. The bridge abutments are supported on “perched” foundations on piles end bearing on bedrock. The front row of piles are battered towards Highway 417. The existing approach embankments are about 6 metres high relative to the highway profile. The foreslopes of both the north and south abutments were originally constructed at 2 Horizontal to 1 Vertical grade extending down to the roadway shoulders. In 1999, as part of intersection improvement works (GWP 203-86-02), the foreslope paving and approach fills in front of both the north and south bridge abutments were cut back to form a “truncated toe” with a retaining wall.

Based on the underground service locates completed, it is understood that hydro conduits are present within the existing underpass structure and that buried hydro conduits are located along the west and south fence lines for the St-Basil’s Church, in the northwest quadrant of the site.

Previous investigations were conducted for the design of the existing bridge by McRostie & Associates (McRostie) in 1958. The results of that investigation are contained in the report titled “Report on Foundation Investigation at Ottawa Queensway and Maitland Avenue, Bridge No. 3, to Deleuw, Cather and Company of Canada Limited” (Geocres 31G5-022). A subsequent investigation was carried out by Jacques Whitford Limited (JWL) for McCormick Rankin Corporation in 1998. The results of that investigation are contained in the report titled “Foundation Investigation Report to McCormick Rankin Corporation on GWP 203-86-02, Highway 417/Maitland Avenue Bridge Rehabilitation, District 42, Ottawa, Ministry of Transportation Ontario” (Geocres No. 31G5-190).

### 2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*<sup>1</sup>, this section of Highway 417 lies within the minor physiographic region known as the Ottawa Valley Clay Plain, which lies within the major physiographic region of the Ottawa-St. Lawrence Lowland.

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock<sup>2</sup>. This region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain at depth by igneous and metamorphic bedrock of the Precambrian Shield.

<sup>1</sup> Chapman, L. J. and Putnam, D. F., 1984. *The Physiography of Southern Ontario*, Ontario Geological Survey. Special Volume 2, Third Edition. Accompanied by Map P.2715, Scale 1:600,000. Ontario Ministry of Natural Resources.

<sup>2</sup> Belanger, J.R. “Urban Geology of Canada’s National Capital Area”, in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.



### **3.0 INVESTIGATION PROCEDURES**

The field work for this subsurface investigation was carried out between June 14 and July 27, 2016. During that time, a total of 8 boreholes were advanced at the locations shown on Drawing 1. Borehole 16-401 was advanced at the pier location, Boreholes 16-402 and 16-403 were advanced adjacent to the abutments at the east toe of each of the existing embankments, Boreholes 16-404 and 16-405 were advanced through each of the existing approach embankments, and Boreholes 16-406 to 16-408 were advanced within the currently proposed 'Construction Staging Area' within the northwest quadrant of the Maitland Avenue interchange. The boreholes were advanced using a combination of truck and track mounted drill rigs supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. The boreholes were advanced to depths of between 2.5 m and 9.2 m below present ground surface.

Samples of the overburden were obtained at 0.6 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In the bedrock, rotary diamond drilling techniques were used to retrieve HQ sized core. The bedrock was cored for depths of 3.2 to 3.7 m, after practical refusal to augering had been reached. Three monitoring wells were installed (in Boreholes 16-402, 16-403, and 16-406) to monitor the groundwater levels at the site. The monitoring wells consist of 50 mm outside diameter PVC tubing with a 1.5 to 3.0 m long slotted tip. The boreholes were backfilled with bentonite mixed with soil cuttings. The site conditions were restored following completion of the field work.

The field work was supervised on a full time basis by members of Golder's staff who located the boreholes in the field, directed the drilling, sampling, and in situ testing operations, and logged the boreholes. The soil and bedrock samples were identified in the field, placed in labelled containers, and transported to Golder's laboratory in Ottawa for further examination and to Golder's laboratories in London and Mississauga for testing. Index and classification tests consisting of water content determinations, Atterberg Limit testing, and grain size distribution analyses were carried out on selected soil samples at the London laboratory. Unconfined compressive strength testing was carried out on one sample of the bedrock core at the Mississauga laboratory.

The groundwater levels were measured in the monitoring wells in Boreholes 16-402, 16-403 and 16-406 on August 2 and September 30, 2016.

In addition to the borehole investigation, shear wave velocity profiling at the site was completed using the Multichannel Analysis of Surface Waves (MASW) technique and was conducted between May 26 and 27, 2016, by personnel from the Golder Associates' Mississauga and Ottawa offices. A series of 24 low frequency (4.5 Hz) geophones were laid out at 3 m intervals. A 9.9 kg sledge hammer and 45 kg weight drop were used as the seismic source. The source locations were offset at distances of 5, 10, 15, and 20 m off the end and collinear with the geophone array. A relatively high noise level was recorded at this site due to large amounts of road traffic.

The borehole locations were determined by Golder relative to existing site features. The borehole elevations were surveyed by Golder using a Trimble R8 GPS unit. The borehole locations in MTM NAD83 northing and easting coordinates, ground surface elevations referenced to geodetic datum and drilled depths are summarized in the following table and are shown on Drawing 1.





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Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
BH16-401	Pier	5025894.5	363195.9	82.1	7.9
BH16-402	South Embankment Toe	5025866.3	3623230.0	83.0	9.2
BH16-403	North Embankment Toe	5025947.4	363199.0	82.3	8.9
BH16-404	South Approach Embankment	5025829.0	363211.3	87.4	3.6
BH16-405	North Approach Embankment	5025938.7	363166.4	87.2	9.0
BH16-406	Staging Area	5025923.6	363102.7	83.5	3.3
BH16-407	Staging Area	5025898.4	363142.9	82.9	3.4
BH16-408	Staging Area	5025880.4	363097.4	82.9	2.5



## **4.0 DESCRIPTION OF SUBSURFACE CONDITIONS**

### **4.1 General**

The Record of Borehole sheets from the current investigation are presented in Appendix A. The results of the laboratory testing carried out during the current investigation are presented on the Record of Boreholes sheets and on Figures 1 to 7 in Appendix B. The Record of Borehole sheets from the previous investigations at the site (Geocres No. 31G5-022 and 31G5-190) are provided for reference in Appendix C.

The MASW test results and report are presented in Appendix D and include the calculated shear wave velocity profile measured from the field testing and a graphical representation of the shear wave velocity profile with depth.

As part of the current subsurface investigation at this site, five boreholes were advanced within or near the limits of the foundation elements for the proposed widening of the Maitland Avenue Underpass. The borehole locations from the current and previous investigations are shown on Drawing 1. The interpreted stratigraphic profile projected along the Maitland Avenue centreline is also shown on Drawing 1. An interpreted stratigraphic profile projected through the staging area is shown on Drawing 2.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile are inferred from observations of drilling progress and non-continuous sampling and, therefore, represent transitions between soil types rather than exact planes of geological change. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions at the site consist of a layer of fill and/or topsoil underlain by silty clay to clayey silt and glacial till overlying limestone bedrock.

A detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections. In the following discussion, emphasis is placed on the subsurface conditions indicated in the boreholes from the present investigation. The boreholes from the original 1958 investigation were advanced prior to the highway construction and therefore the ground conditions shown on those logs are different than what currently exist, particularly with respect to the composition and thickness of overburden and fill. The Geocres information is referenced herein only in regard to the bedrock surface elevation, which is in general agreement with the results from the present investigation. The bedrock depths from the subsequent 1998 investigation were inferred from auger refusal and could therefore indicate the bedrock surface or refusal on cobbles/boulders in the glacial till.

### **4.2 Topsoil and Fill**

Topsoil exists at ground surface at Boreholes 16-402, 16-403, and 16-406 to 16-408, with thicknesses of about 100 to 200 mm.

Asphaltic concrete exists at ground surface at Boreholes 16-401, 16-404 and 16-405, with thicknesses of about 200 to 300 mm.

Fill was encountered beneath the topsoil/asphaltic concrete at all boreholes with the exception of Borehole 16-408. Fill materials are quite variable between locations, consisting of sand, gravel, silty sand, silty clay and clayey silt. Cobbles and boulders were encountered in the fill at Boreholes 16-404 and 16-405 within the approach embankments. The layer of fill was fully penetrated in all of the boreholes, with the exception of Borehole 16-404, which was terminated due to refusal of auger advancement. At this location, the fill was proven to extend to a depth of about 3.6 m below the existing ground surface. Where fully penetrated, the fill varies from about 0.5 to 5.9 m in thickness, and is thickest at the approach embankments. SPT 'N' values obtained within this material generally range from about 7 to 44 blows per 0.3 m of penetration indicating a compact to dense state of packing.



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Two higher blow counts within the fill (i.e., in Boreholes 16-404 and 16-405) likely reflect the presence of cobbles and boulders, rather than the state of packing of the soil matrix.

Grain size distribution testing was carried out on five samples of the fill, the results of which are provided on Figure 1. The results of Atterberg limit testing carried out on two samples of the cohesive fill are summarized on Figure 2 and indicate plasticity index values of 19 and 22 percent and liquid limit values of 37 and 40 percent, reflecting a silty clay of intermediate plasticity. The measured water content of the fill ranges from approximately 3 to 20 percent.

### 4.3 Silty Clay to Clayey Silt to Clay

The fill is underlain by a deposit of sensitive silty clay to clayey silt to clay at all of the borehole locations, with the exception of borehole 16-404 (where the fill was not penetrated) and Borehole 16-408. Where encountered, the clayey deposit was fully penetrated and varies from about 1.4 to 3.9 m in thickness.

The upper portion of the silty clay to clayey silt at Boreholes 16-401 to 16-403, and the full thickness of the deposit at Boreholes 16-405 to 16-407, has been weathered to form a grey brown crust. Standard penetration tests carried out within the weathered crust gave 'N' values ranging from 3 to 18 blows per 0.3 m of penetration, indicating a generally stiff to very stiff consistency.

The results of Atterberg limit testing carried out on five samples of the weathered silty clay to clayey silt are summarized on Figure 3 and indicate plasticity index values generally ranging from 9 to 17 percent and liquid limit values ranging from 33 to 40 percent, reflecting a soil of low to intermediate plasticity. The measured water content of the weathered silty clay to clayey silt ranges from approximately 29 to 52 percent. Grain size distribution testing was carried out on two samples of the weathered silty clay to clayey silt, the results of which are provided on Figure 4.

A layer of unweathered grey silty clay to clayey silt to clay was encountered at Boreholes 16-401 to 16-403, below the upper weathered crust. This unweathered silty clay to clayey silt to clay is about 1.0 to 1.5 m thick at the borehole locations. Standard penetration tests carried out within the unweathered portion of the deposit gave 'N' values 1 and 2 blows per 0.3 m of penetration. In situ shear vane testing carried out where possible within this deposit measured undrained shear strengths of 63 and 65 kPa, indicating a stiff consistency. The calculated sensitivity ratio based on remoulded shear strengths of 11 kPa in this deposit is about 6, indicating a sensitive material in accordance with the CFEM.

The results of Atterberg limit testing carried out on three samples of the unweathered silty clay to clayey silt to clay are shown on Figure 5 and gave plasticity index values of 13, 18, and 51 percent and liquid limit values of 36, 38, and 71 percent, respectively, indicating intermediate and high plasticity soil. The measured water contents of three samples of unweathered portion of the deposit were between about 52 and 56 percent. Grain size distribution testing was carried out on one sample of the unweathered silty clay to clayey silt, the results of which are provided on Figure 6.

### 4.4 Till

A deposit of glacial till was encountered directly beneath the topsoil at Borehole 16-408, and below the silty clay at the other borehole locations (except for Borehole 16-404) at depths of about 2.9 to 7.6 m below the existing ground surface. The till consists of a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with some clay.

The till was fully penetrated at Boreholes 16-401 to 16-403 and about 0.7 to 1.4 m in thickness, extending to about 4.6 to 6.0 m depth below the existing ground surface (i.e., Elevations 77.0 to 77.6 m). The till was not fully penetrated at Boreholes 16-405 to 16-408 but was proven to extend to depths of about 2.5 to 9.0 m below the ground surface (i.e., Elevations 78.2 to 80.4 m). Standard penetration test 'N' values of 1 to in excess of 50 blows



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per 0.3 m of penetration were measured in the glacial till, indicating a very loose to very dense state of packing, although the higher 'N' values could reflect the presence of cobbles and boulders, rather than the state of packing of the soil matrix.

The measured water contents of three samples of till were between about 8 and 14 percent. Grain size distribution testing was carried out on two samples of the till, the results of which are provided on Figure 7. The results indicate that the till matrix consists of a silty sand with some clay and trace amounts of gravel. These samples were, however, retrieved using a 50 mm diameter sampler and therefore the results do not reflect the larger gravel, cobble and boulder content of the deposit.

### 4.5 Bedrock

The bedrock encountered at the bridge foundation elements consists of limestone with thin shale interbeds.

Fresh, thinly to medium bedded, limestone bedrock with thin shale interbeds was encountered at Boreholes 16-401, 16-402, and 16-403 at depths ranging from about 4.6 to 6.0 m below the existing ground surface (i.e., Elevations 77.0 to 77.6 m). At Borehole 16-401, the upper 0.3 m of limestone bedrock is weathered. These boreholes were advanced 3.2 to 3.7 m into the bedrock.

Photos of the bedrock core obtained during the current investigation are provided in Appendix A on Figures A1 to A6, inclusive.

The following table summarizes the bedrock surface (and refusal) depths and elevations as encountered at Boreholes 16-401 through 16-408 as part of the current investigations, and as encountered at the previous Boreholes 1 to 6 (Geocres 31G5-022) and 98-1 through 98-4 (Geocres 31G5-190). The bedrock was cored in Boreholes 16-401 to 16-403, and 1 to 6, inclusive. Boreholes 98-1 to 98-4, inclusive, were advanced to auger refusal on the inferred bedrock surface.

Borehole Location	Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
North Abutment (east side)	16-403	82.3	5.2	77.1
	98-2	81.8	2.8 <sup>R</sup>	79.0 <sup>R</sup>
	6	82.3	5.0	77.4
North Abutment (west side)	98-1	82.3	5.2 <sup>R</sup>	77.1 <sup>R</sup>
	1	82.6	4.7	77.9
North Approach Embankment	16-405	87.2	> 9.0	< 78.2
Pier (west side)	2	83.0	3.7	79.3
Pier (east side)	16-401	82.1	4.6	77.6
	5	82.7	4.7	78.0
South Approach Embankment	16-404	87.4	> 3.6	< 83.8
South Abutment (east side)	16-402	83.0	6.0	77.0
	98-4	82.0	4.2	77.8 <sup>R</sup>
	4	82.9	4.8	78.1
	98-3	84.4	5.9 <sup>R</sup>	78.5 <sup>R</sup>



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Borehole Location	Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
South Abutment (west side)	3	82.9	3.7	79.3
Staging Area (NW)	16-406	83.5	3.3 <sup>R</sup>	80.3 <sup>R</sup>
	16-407	82.9	3.4 <sup>R</sup>	79.5 <sup>R</sup>
	16-408	82.9	2.5 <sup>R</sup>	80.4 <sup>R</sup>

Note: <sup>R</sup> = Auger refusal on inferred bedrock surface.

The limestone bedrock at the site is a member of the Gull River Formation and medium strong to strong. Thin shale interbeds were also present in the rock core. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 47 to 100 percent. The lowest rock quality was recorded for the upper 0.3 m of the bedrock in Borehole 16-401. The result of one unconfined compressive strength test on a sample of the bedrock from Borehole 16-401 was 76.5 MPa, as shown on Figure 8. A description of some of the terms used in the description of the bedrock samples from this site is provided on the Lithological and Geotechnical Rock Description Terminology sheet which precedes the Record of Borehole sheets included with this report.

### 4.6 Groundwater Conditions

Monitoring wells were installed in Boreholes 16-402, 16-403, and 16-406. The water levels measured in the wells are summarized in the following table:

Borehole Number	Borehole Location	Screened Interval	Date	Depth (m)	Elevation (m)
16-402	South Abutment (east side)	Silty Clay/Glacial Till	August 2, 2016	3.0	79.9
			September 30, 2016	3.0	79.9
16-403	North Abutment (east side)	Limestone Bedrock	August 2, 2016	2.7	79.6
			September 30, 2016	3.2	79.1
16-406	Staging Area	Silty Clay/Glacial Till	August 2, 2016	Dry	< 80.3
			September 30, 2016	Dry	< 80.3

The water level at the site is expected to fluctuate seasonally in response to changes in precipitation and snow melt, and is expected to be higher during the spring and periods of precipitation.

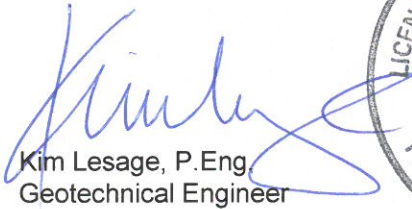


## FOUNDATION REPORT HIGHWAY 417 MAITLAND AVENUE UNDERPASS


### 5.0 CLOSURE


The field operations were supervised by Mr. Doug Grylls and Mr. Jason Derouin. This report was prepared by Ms. Kim Lesage, P.Eng., and was reviewed by Ms. Erin O'Neill, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Fintan Heffernan, P.Eng., a Senior Consultant with Golder and the Designated MTO Foundations Contact, conducted an independent quality control review of this report.

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KSL/ESO/FJH/ob

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**FOUNDATION REPORT  
HIGHWAY 417 MAITLAND AVENUE UNDERPASS**

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# **PART B**

**FOUNDATION DESIGN REPORT  
MAITLAND AVENUE UNDERPASS  
SITE NO. 3-042  
HIGHWAY 417 WIDENING AND REHABILITATION  
FROM WEST OF HIGHWAY 416 TO EAST OF MAITLAND AVENUE  
G.W.P. 4124-14-00**





### 6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the proposed widening of the existing Maitland Avenue Underpass structure (MTO Structure Site No. 3-042) above Highway 417 in Ottawa. These recommendations are based on an interpretation of the factual data obtained from the boreholes advanced during subsurface investigations at this site. The discussion and recommendations presented are intended only to provide the designer with sufficient information to assess the feasible foundation alternatives and to carry out the design of the proposed structure foundations, as may be required. The foundation investigation report, discussion and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

#### 6.1 General

Golder Associates Ltd. (Golder) has been retained by MMM Group Limited (MMM) on behalf of the MTO to provide recommendations on foundation aspect for the design of the proposed widening of the Maitland Avenue Underpass in Ottawa. The proposed widening of the existing bridge to the east by 4.4 m is required to accommodate a lengthened northbound left hand turn lane on Maitland Avenue to westbound Highway 417, due to projected increased traffic demands.

Based on the General Arrangement (GA) Drawing provided by MMM, the grade of Highway 417 beneath the Maitland Avenue Underpass is about Elevation 82 m. In comparison, the grade of Maitland Avenue where it crosses Highway 417 is between about Elevation 88 and 89 m.

The existing Maitland Avenue Underpass is a two span continuous steel girder bridge with composite concrete deck. The spans are each approximately 35 m in length. The central piers are founded on spread footings on rock. The existing bridge abutments are supported on “perched” foundations on piles end bearing on bedrock. The front row of piles are battered towards Highway 417. The existing approach embankments are about 6 to 7 m high relative to the highway profile. In 1999, the foreslope paving and approach fills in front of both the north and south bridge abutments were truncated to accommodate additional travel and off-ramp lanes on Highway 417. General Arrangement Drawings from that project indicate that the truncated toe retaining walls consist of a concrete-faced soldier pile retaining wall with piles augered and pinned to the bedrock for foundation support, and tie-back anchors extending into the bedrock provide lateral support to the retaining wall.

It is understood that widening of the existing Maitland Avenue Underpass to the east will require additional foundation elements to be added at the north and south abutments. It is further understood that, as a part of this project, the existing pier foundations may need to be replaced below both the existing and widened portions of the bridge structure. It is also understood that the highway platform beneath Maitland Avenue is of sufficient width to accommodate the addition of new travel lanes and that no additional modifications to the Maitland Avenue Underpass structure or approaches are required to accommodate the Highway 417 widening at this location.

This report addresses the proposed widening of the Maitland Avenue Underpass (MTO Structure Site No. 3-042) and the associated retaining walls and underpass approach embankments only.

It is understood that the bridge widening is to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC).





## 6.2 Seismic Design

### 6.2.1 Seismic Zone and Importance Category

The site falls within the Western Quebec Seismic Zone (WQSZ) according to the Geological Survey of Canada. The WQSZ constitutes a large area that extends from Montréal to Témiscaming. Within the WQSZ, recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montréal-Maniwaki axis. Historical seismicity within the WQSZ 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.

The CHBDC states that the seismic hazard values associated with the design earthquakes should be those established for the National Building Code of Canada (NBCC) by the Geological Survey of Canada (GSC). The GSC has developed a new set of seismic hazard maps (referred to as the 5th generation seismic hazard maps) that were made available for public use in December 2015.

In accordance with Section 4.4.2 of the CHBDC, and as specified by the MTO, the proposed bridge structure has been given an importance category of 'Major Route' bridge.

### 6.2.2 Seismic Site Classification

Multichannel Analysis of Surface Waves (MASW) geophysical testing was carried out at the proposed staging area location to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. The shear wave velocities measured are presented in a technical memorandum (see results in Appendix D) and indicate that the average shear wave velocity in the upper 30 m of the subsurface stratigraphy is 818 m/s. Despite relatively high average shear wave velocities in the overburden and bedrock, the underside of the footings for the bridge are perched more than 3 m above the rock surface and, as such, a Site Class B cannot be defined for this structure. A Site Class C designation should be used and would best represent the site response effects of the site onto the Maitland Avenue Underpass structure.

### 6.2.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the bridge (latitude 45.37 and longitude – 75.75), the following are the reference Site Class C (reference) peak seismic hazard values (based on the 5<sup>th</sup> generation seismic hazard maps published by the GSC) that can be used for the design of the proposed bridge structure.

Seismic Hazard Values	10% Exceedance in 50 years (475-year return period)	5% Exceedance in 50 years (975-year return period)	2% Exceedance in 50 years (2,475 return period)
PGA (g)	0.099	0.159	0.275
PGV (m/s)	0.067	0.109	0.193
Sa (0.2) (g)	0.157	0.249	0.431
Sa (0.5) (g)	0.087	0.136	0.233
Sa (1.0) (g)	0.044	0.068	0.116
Sa (2.0) (g)	0.020	0.032	0.055
Sa (10.0) (g)	0.0019	0.0032	0.0054



### 6.3 Bridge Foundations - General

#### 6.3.1 Available Foundation Options

To accommodate projected increased traffic demands, a lengthened northbound left hand turn lane on Maitland Avenue to westbound Highway 417 is planned, which requires the widening of the existing Maitland Avenue Underpass Structure to the east by 4.4 m. Only minimal changes to the grades of Highway 417 and the Maitland Avenue approaches are planned as part of this project.

According to the available information, the existing two span bridge structure abutments and wing walls are founded on “perched” foundations on 360 x 109 steel H-piles end bearing on bedrock. The front row of abutment piles are battered towards Highway 417. The central piers are founded on 2.7 by 3.6 m rectangular spread footings on rock. The truncated foreslope retaining walls consist of a concrete-faced soldier pile retaining wall with piles augered and pinned to the bedrock for foundation support, and tie-back anchors extending into the bedrock provide lateral support to the retaining wall.

#### 6.3.2 Bridge Foundation Options

The foundation system for the widening of this bridge should be compatible with the existing bridge foundations and the following options have been considered for the widening:

- Shallow foundations supported on the native silty clay to clayey silt;
- Shallow foundations supported on glacial till soils or bedrock, or on engineered fill supported on the glacial till or bedrock;
- Deep foundations supported on steel H-piles founded on, or socketted into, the bedrock; and,
- Deep foundations supported on caissons founded on, or socketted into, the bedrock.

The first option, using shallow foundations supported on the native silty clay soils, is not considered practical or appropriate for the abutments since the bearing resistance of these low to intermediate plasticity soils would be insufficient for to support of the anticipated abutment loads and the settlement of the foundations would be excessive, given the significant foundation loads. Additionally, the settlement of abutment widenings supported on shallow foundations would be entirely differential with respect to the existing pile supported abutments. Detailing of the structural elements to accommodate the anticipated differential settlements may not be feasible. Spread footings supported on the underlying glacial till or bedrock, or on engineered fill supported on the glacial till or bedrock, have not been considered as a feasible or practical option due to the 5 m deep excavations that would be required.

For support of the central pier, spread footings supported on the underlying glacial till or bedrock, or on engineered fill supported on the glacial till or bedrock, may be feasible. Spread footings supported on bedrock would be consistent with the existing pier foundations. The installation of caissons founded on or socketed into the bedrock would not be feasible for the replacement of the existing pier foundations due to height restrictions imposed by the existing bridge structure.

It is considered that the most feasible and cost-effective options for the bridge abutment widenings are foundations supported on piles or caissons, founded on or socketed into the bedrock. These options are also consistent with the existing bridge abutment foundation construction.

Foundation recommendations for the design of foundations for the bridge abutments are presented in the following sections. A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the foundation options is presented in Table 1 following the text of this report.



### 6.3.3 Feasibility of Integral and Semi-Integral Abutments

As outlined in MTO's report SO-96-01, integral abutment bridges are single span or multiple span continuous deck type bridges with a movement system composed primarily of abutments on flexible integral foundations and approach slabs, in lieu of movable deck expansion joints and bearings at abutments. The feasibility of integral abutments is influenced by a number of factors including geometry and subsurface conditions. The primary criterion is the need to support the abutments on relatively flexible piles. Where the load bearing stratum is near the surface or where the use of short piles or caissons (less than 5 m in length) is planned, the site is not considered suitable for integral abutment bridges. Geometric constraints on the use of integral abutments are also applicable and include: overall bridge length less than 150 m; skew angle less than 35°; and abutment wall heights less than 6 m without a retained soil system.

As outlined in MTO's report BO-99-03, semi-integral abutment bridges are single or multiple span structures of less than 150 m in length with rigid foundations (spread footings) where the concrete deck is continuous with the approach slabs. Expansion joints are eliminated at the end of the deck and the superstructure is supported on movable bearings and is almost independent of the abutment. A expansion joint is provided at the end of the approach slab that is detailed to slide between or over the wingwalls. Unlike integral abutment bridges, there is no limit on skew angle for semi-integral abutments provided that lateral restraint is incorporated in the bridge design to prevent rotation of the superstructure caused by eccentric lateral earth pressures in the horizontal plane acting on both ends of the superstructure and that the movement system at the end of the approach can accommodate deformations associated with skew.

From a foundation perspective, semi-integral abutments are considered most feasible at this location. The existing structure could be modified to a semi-integral bridge by eliminating the expansion joints at the ends of the deck.

### 6.3.4 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the proposed underpass structure and foundation system may be classified as having large traffic volumes and its performance as having potential impacts on other transportation corridors, hence having a "typical" consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of *CHBDC*, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor,  $\Psi$ , and geotechnical resistance factors,  $\phi_{gu}$  and  $\phi_{gs}$ , from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 to 6.7 below.

## 6.4 Shallow Foundations

### 6.4.1 Founding Elevation

For the support of the new pier, and for consistency with the existing pier foundations, spread/strip footings would likely be founded on the bedrock.

The footing may be placed directly on the bedrock surface after excavation of the overlying fill and native overburden soils and any loose or fractured rock, and founded at an elevation of about 77.1 m. All loose or fractured bedrock at the founding level should be removed prior to placing concrete. In addition, the design for the pier footing should be flexible enough to allow for some variation in the bedrock surface elevation.

A working slab could be placed to raise the grade to the founding level and protect the founding bedrock after exposing the bedrock and removing any loosened/fractured bedrock, if required.



### 6.4.2 Factored Geotechnical Axial Resistances

A factored ultimate geotechnical resistances of 4 MPa (at ULS) may be used for design of spread/strip footing founded on the properly prepared bedrock surface. Serviceability Limit States (SLS) conditions do not apply to footings placed on the limestone bedrock which is classified as non-yielding.

The factored geotechnical resistances provided above are given for loads that will be applied perpendicular to the surface of the footings. Where the load is not applied perpendicular to the footing, inclination of the load should be taken into account in accordance with Section 6.10.4 of the CHBDC. The factored geotechnical resistance for sound bedrock given above assumes that the bedrock at and below the founding level has not been fractured, and that no adverse jointing is present below the footings.

### 6.4.3 Resistance to Lateral Loads

Resistance to lateral forces/sliding resistance between cast-in-place concrete footings and the founding soils or bedrock surface should be calculated in accordance with Section 6.10.5 of the CHBDC. An unfactored coefficient of friction,  $\tan \phi' = 0.70$  can be used for the interface between the concrete footing and bedrock.

If necessary, sliding resistance can be supplemented by doweling the footings into bedrock. The horizontal resistance of the dowels will be dependent on the strength of the bedrock, grout and steel. For this site, where the rock mass is essentially as strong as or is stronger than concrete, the design of the dowels in the rock may be handled in the same way as the dowel embedment into the concrete. The dowels should have a minimum embedded length within the sound bedrock of 1 m, and the structural strength of the dowel and compressive strength of the grout should not be exceeded.

For uplift of the dowels, a factored value of 1 MPa may be assumed for the grout-to-rock bond stress for ULS design. The actual bond stress along the rock-grout interface may vary from the design value given and it should therefore be verified in the field by pull-out testing. In this case, a Special Provision will have to be included in the Contract Documents to cover this testing.

### 6.4.4 Frost Protection

The footings should be provided with a minimum 1.8 m of soil cover for frost protection as per Ontario Provincial Standard Drawing (OPSD) 3090.101 (*Frost Penetration Depths for Southern Ontario*), as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the footing.

For spread footings placed on fresh limestone bedrock or mass concrete, frost protection cover is not required.

## 6.5 Driven Steel H-Pile or Steel Pipe (Tube) Pile Foundations

### 6.5.1 Founding Elevation

The perched abutments for the widened underpass structure may be supported on steel H-piles or pipe piles driven to found on the limestone bedrock. Based on the existing grade of the approaches at about Elevation 87 m and the requirement for a minimum 1.8 m of frost cover, it is assumed that the pile cap base will be below Elevation 85 m. The underside of the existing pile caps are perched at Elevation 84 m, approximately 2 m above the adjacent Highway 417 grade. It is understood that the foundations for the widenings will be placed at about the same elevation as the adjacent portions of the existing foundations. As such, based on an approximate bedrock surface at about Elevation 77 m at the abutments, the anticipated pile length will therefore be about 7 m.



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The pile caps should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101, as measured vertically from and perpendicular to the face of the abutment slope to the edge of the underside of the pile cap. If adequate soil cover cannot be provided for the pile cap, rigid insulation (Styrofoam) could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

Pile installation should be in accordance with the Ontario Provincial Standard Specifications, Provincial-Oriented, (OPSS.PROV) 903 (Deep Foundations). For this site, the piles will essentially be driven to practical refusal on (or within) the bedrock. As discussed in Sections 4.2 and 4.4, cobbles are inferred to be present within the fill and glacial till deposits. In this regard, steel H-piles are preferred over closed-ended steel pipe piles, as pipe piles have a higher likelihood of refusing on cobbles and/or boulders or being deflected away from the vertical/battered orientation during installation due to their larger end area. For driven steel H-piles, the piles should be equipped with rock bearing points and should be driven to bedrock. Any battered piles should be equipped with suitable driving points (such as Titus Standard 'H' Bearing Pile Points or equivalent) to ensure adequate seating of the piles on the bedrock. If steel pipe piles are used, driving shoes should be in accordance with Type II reinforcement as per OPSD 3001.100.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A Non Standard Special Provision for vibration monitoring should be included in the contract documents and is provided in Appendix E of this report. A maximum peak particle velocity of 100 mm/s is recommended at the existing abutments and wingwalls. The piles further from the existing structure should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the pile driving criteria for the remaining piles. Due to space constraints at this bridge widening, it may not be feasible to drive the piles furthest from the structure first and if the piles nearest the existing structure are driven first a maximum acceptable peak particle velocity of 50 mm/s is recommended.

The piles for the widened abutments may need to be driven in close proximity to the battered piles supporting the existing abutments and wing walls. These existing piles may be offset from their intended location or alignment and the potential exists for conflicts when driving the new piles. Current construction practice generally limits the acceptable pile offset at the surface to 75 mm and the deviation from the design inclination to 2%. However, even for piles installed meeting these construction limits, the tip offset at depth may be greater and it is considered that, for piles less than 10 m in length such as at this site, the tip offset at depth may be as much as 10% of the pile length. Depending on the final pile configuration, the spacing between the new and existing piles may be less than this tolerance and therefore the potential exists for interference during driving of the new piles. For new piles driven within the potential zone of interference with the existing abutment or wing wall piles (defined as a distance around the existing pile centre equal to 10% of the pile length) the driving operations should be continuously monitored by the QVE and the contractor should cease driving of the pile if the QVE indicates that the driven pile may have come in contact with an existing pile. It may be necessary to extract and re-drive piles if contact between the new and existing piles is believed to exist. Similarly, the new piles for the widened abutments may be in conflict with the rock anchors that were installed as part of the existing retaining walls. A Non Standard Special Provision for driving piles adjacent to existing battered piles or existing rock anchors should be included in the contract documents and is provided in Appendix E of this report.

### 6.5.2 Factored Geotechnical Axial Resistances

The factored axial compressive geotechnical resistances at ULS and SLS (for 25 mm of settlement) that can be used for the existing piles are presented below, and are based on the available design drawings, Geocres information, and the known quality of the steel being used for typical construction projects during that time.





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Pile Size	Factored ULS Resistance	SLS Resistance
14 BP 73 (HP 360 x 109)	1,400 kN	1,150 kN

The factored ultimate and serviceability geotechnical axial resistances (ULS) for new driven steel H-piles and closed-end, concrete-filled 324 mm diameter steel pipe piles having a minimum wall thickness of 13 mm that are successfully driven to found on the bedrock are presented below.

Pile Type	Approximate Length of Driven Pile (m)	Factored Ultimate Geotechnical Axial Resistance (at ULS) (kN)
HP 310 x 110	7.0	2,000
HP 360 x 132	7.0	2,400
HP 360 x 152	7.0	2,750
324 mm OD Pipe Pile	7.0	2,400

Serviceability Limit States (SLS) resistances do not apply to new piles founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

Pile installation should be in accordance with OPSS.PROV 903 (Deep Foundations). The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known. The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) and/or Pile Dynamic Analyzer (PDA) testing during pile installation on selected piles to confirm the design capacity. Considering the relatively short length of piles required for this structure, the use of the Hiley formula is considered sufficient for determining the pile capacity since the full driving energy is likely delivered to the pile tip.

### 6.5.3 Downdrag Load (Negative Skin Friction)

The widening of the embankments up to the new abutment and retaining wall alignments will raise the effective stress level in the silty clay and could generate downdrag forces on the piles. Some limited compression of the deposit is expected (see later discussion in Section 6.9.3), and is estimated at about 25 mm. The elastic shortening of the piles will likely be less than 5 mm under service loads, and therefore the differential settlements would be sufficient to generate downdrag forces.

In calculating the magnitude of the downdrag force, the methods described in both the Canadian Foundation Engineering Manual (CFEM) as well as the US Transportation Research Board's report "Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles" [Briaud and Tucker (1994)] were considered. Considering the larger predicted settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the underside of the silty clay deposit.

Based on the above, and assuming an underside of the pile cap of about Elevation 84 m, the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the silty clay and overlying embankment fill is estimated to be up to about 300 kN, for piles supporting the new abutments.



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The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Section 6.11.4.10 of the CHBDC.

In a closely spaced pile group, the downdrag on individual piles may act independently, however the force may act as a 'block' around the perimeter of the pile group only, as described in Section 18.2.2.2 of the CFEM.

It is not known whether downdrag loads were considered for the design of the existing piles. However, ground movements will occur at the existing piles closest to the widening and will likely be large enough in magnitude to generate downdrag forces on those piles. The unfactored downdrag load acting on a single existing pile (assumed equivalent to a HP 360 x 109) over the length of pile within the native soils is estimated to be 350 kN. In the absence of information on the construction sequence used to design the existing bridges, it can be assumed that this magnitude of loading is already acting on all of the existing piles.

### 6.5.4 Resistance to Lateral Loads

Lateral loading could be resisted fully or partially by the use of battered steel H-piles. Alternatively, the resistance to lateral loading will have to be derived from the soil in front of the piles.

For design of the Maitland Avenue structure, the SLS geotechnical response of the soil in front of the piles under lateral loading may be calculated using subgrade reaction theory where the coefficient of horizontal subgrade reaction,  $k_h$ , is based on the equation given below, as described by Terzaghi (1955) and the Canadian Foundation Engineering Manual (3rd Edition). It may be assumed that this resistance will be nearly the same for vertical and inclined piles. Note: The design of the perched abutments includes a 'truncated toe' configuration in front of the piles, adjacent and parallel to the Highway. Where piles are within 8 diameters (8D) of the retaining walls, the lateral resistances above the roadway level should therefore be ignored.

For cohesionless soils:

$$k_h = \frac{n_h z}{B}$$

Where:  $n_h$  is the constant of horizontal subgrade reaction, as given below;

$z$  is the depth (m); and,

$B$  is the pile diameter/width (m).

For cohesive soils:

$$k_h = \frac{67 s_u}{B}$$

Where:  $s_u$  is the undrained shear strength of the soil (kPa); and,

$B$  is the pile diameter/width (m).

The following ranges for the values of  $n_h$  and  $s_u$  may be used in the preliminary structural analysis. The ranges in values reflect the variability in the subsurface conditions, the soil properties and the approximate nature of the analysis and the non-linear nature of the soil behaviour (such that  $k_h$  is a function of deflection).



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Location	Elevation (m)	Soil Type	$n_h$ (MN/m <sup>3</sup> )	$S_u$ (kPa)
North Abutment (BH 1,6, 98-1, 98-2, 16-403 & 16-405)	82.4 – PCL <sup>1</sup>	Very Stiff Silty Clay to Clayey Silt Embankment Fill	-	50 to 100
	81.4 – 82.4	Loose Silty Sand Embankment FILL	2	-
	79.7 – 81.4	Very Stiff Weathered Silty Clay Crust	-	75 to 125
	78.7 – 79.7	Stiff Silty Clay	-	60 to 100
	77.4 – 78.7	Loose to Dense Silty Sand Glacial Till	1 to 11	-
	77.4	Bedrock	-	-
South Abutment (BH 3 & 4, 98-3, 98-4 16-402 & 16-404)	82.3 – PCL <sup>1</sup>	Very Stiff Silty Clay to Clayey Silt Embankment Fill	-	50 to 100
	80.5 – 82.3	Very Stiff Weathered Silty Clay Crust	-	75 to 125
	79.3 – 80.5	Stiff Silty Clay	-	60 to 100
	78.0 – 79.3	Loose to Dense Silty Sand Glacial Till	1 to 11	-
	78.0	Bedrock	-	-

**Note:** <sup>1</sup> PCL = Pile Cap Level, Understood to be Elevation 84 m.

## 6.6 Drilled Shaft (Caisson) Foundations

### 6.6.1 Founding Elevations

Alternatively, support of the perched abutments may be provided by caisson foundations. Due to the relatively high water table and the difficulty in socketting a liner into the strong bedrock, it may not be feasible to dewater and clean the base of the caisson and, as such, full end-bearing support may not be developed. The axial geotechnical resistance for rock-socketed caissons should be based on the side-wall (shaft) resistance of the rock socket rather than end-bearing. For design purposes, it is recommended that the caissons be founded at about Elevation 75 m (i.e., a rock socket of approximately 2 m).

The native marine (Champlain Sea) clay at this site is a sensitive soil. The disturbed clay could “flow” into the auger hole during caisson installation if left unsupported. The use of a temporary or permanent liner or casing will be required in order to advance the caissons through the overburden with minimal loss of ground. Additionally, it will be difficult to clean the bedrock surface, even with the use of liners, unless the liner is socketed into the bedrock; once disturbed, the sensitive clay soils, as well as the sandy till, could flow under the casings, at the interface with the bedrock. It may therefore be more practical to socket the caissons into the rock, rather than found on the bedrock surface. The casing should be extended so that it is “seated” a minimum of 300 mm into the bedrock.

Casing installation through the fill and glacial till containing cobbles and boulders may be difficult. Churn drilling and possibly rock coring techniques may be required to advance the caissons through the embankment fill and glacial till. In addition, the bedrock at this site is medium strong to strong, and the caisson sockets will likely have to be advanced by rock coring (possibly supplemented with a down-hole hammer) and/or chisel drilling.

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101. If adequate soil cover cannot be provided for the pile cap, rigid styrofoam insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration.





### 6.6.2 Geotechnical Axial Resistance/Reaction

Caissons founded on the surface of the limestone bedrock, or socketed nominally (less than 1 m) into the bedrock, should be designed based on end-bearing resistance and a factored geotechnical resistance at ULS of 4 MPa. Serviceability Limit States (SLS) resistances do not apply to caissons founded on the limestone bedrock, since the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

End-bearing resistance may also be considered in design provided that the base of each caisson is thoroughly cleaned of any cuttings or other material. The *unfactored* geotechnical end-bearing resistance at ULS can be taken as 10 MPa. To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter. The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

End bearing for the caisson relies solely on the quality of the rock surface at the base of the excavation. As such, it is imperative that the rock surface be adequately cleaned of loose soils, rock, and debris prior to construction of the caisson.

In such cases where the basal rock surface cannot be adequately cleaned, the caissons at these locations can be designed solely on shaft resistance within the bedrock. Where suitable end bearing cannot be achieved, the *unfactored* geotechnical sidewall (shaft) resistance at ULS can be taken as 1,500 kPa provided that the caisson socket is formed within competent bedrock (i.e., Rock Quality Designation values greater than 75 percent).

### 6.6.3 Downdrag Load (Negative Skin Friction)

As discussed in Section 6.5.3, the widening of the embankments will raise the effective stress level in the silty clay and could generate downdrag forces on the caissons.

The unfactored downdrag load acting on a single 0.9 m diameter caisson over the length of caisson within the silty clay and overlying embankment fill is estimated to be about 700 kN.

The structural capacity of the caissons must be checked for the factored dead and downdrag loads in accordance with Section 6.11.4.10 of the CHBDC.

### 6.6.4 Resistance to Lateral Loads

For preliminary design purposes, the resistance to lateral loading developed by the soil in front of the caissons may be determined as outlined in Section 6.5.4.

## 6.7 Embankment Toe Retaining Walls

Embankment toe retaining walls are planned to be constructed adjacent to the abutment widenings. The existing truncated toe walls consist of a concrete-faced soldier pile retaining wall with piles augered and pinned to the bedrock for foundation support, and tie-back anchors extending into the bedrock provide lateral support. The choice of retaining wall system will depend on the desired appearance, the anticipated costs, performance, and on other considerations such as constructability. The following options have been considered for the embankment toe (retaining) wall foundations:

- Shallow foundations supported on the native stiff to very stiff weathered silty clay crust;
- Shallow foundations supported on glacial till soils or bedrock, or on engineered fill supported on the glacial till or bedrock;
- Deep foundations supported on steel H-piles founded on, socketed into, or pinned to, the bedrock; and,
- Deep foundations supported on caissons founded on, socketed into, or pinned to, the bedrock.



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Spread footings supported on the glacial till or bedrock, or on engineered fill supported on the glacial till or bedrock, is not considered a practical option due to the 4 metre deep excavation that would be required. Spread footings supported on the native stiff to very stiff weathered silty clay crust is feasible, but the SLS bearing resistances may be too low to support the loads of the retaining walls. The bearing resistance values for the retaining wall foundations are controlled by the criteria of not overstressing the underlying stiff silty clay. The final height and founding level for the retaining wall are therefore important parameters in that assessment. As a preliminary guideline, a factored ULS bearing resistance of 200 kPa can be used for design of the footings for shallow foundations on the stiff silty clay. SLS bearing resistances are not provided at this stage since they are based on several assumptions regarding the subsurface conditions and wall geometry that are currently unknown.

Alternatively, the bridge retaining walls could be supported on piles or caissons, founded on or socketed into the bedrock. Deep foundations are the preferred option for this site.

A summary comparison of the advantages, disadvantages, relative costs, and risks associated with the bridge retaining wall foundation options is presented in Table 2 following the text of this report.

### 6.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the truncated toe walls will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls. Seismic (earthquake) loading must also be taken into account in the design.

The following recommendations are made concerning the design of the abutment walls and associated retaining walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, as in this case, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (Aggregates) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (Compacting). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (Walls, Abutment, Backfill, Minimum Granular Requirement) and OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Other surcharge loadings should be accounted for in the design as required.
- For restrained walls, granular fill should be placed in a zone with the width equal to at least 1.8 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing (in accordance with Figure C6.20(b) of the *Commentary* to the CHBDC). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used for level backfill:



## FOUNDATION REPORT HIGHWAY 417 MAITLAND AVENUE UNDERPASS

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, $K_o$	Active, $K_a$
Granular 'A'	22 kN/m <sup>3</sup>	0.43	0.27
Granular 'B' Type II	21 kN/m <sup>3</sup>	0.43	0.27

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC.

Seismic (earthquake) loading must also be taken into account in the design in accordance with Section 4.6.5 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the abutment stem and/or retaining walls. The walls should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC and its *Commentary*, for structures which allow lateral yielding, the horizontal seismic coefficient,  $k_h$ , used in the calculation of the seismic active pressure coefficient, is taken as 0.5 times the site-specific PGA which is given below for the three design earthquakes. For structures that do not allow lateral yielding,  $k_h$  is taken as equal to the site-specific PGA. For both cases the value of the vertical seismic coefficient  $k_v$  is taken as zero.
- The following seismic active pressure coefficients ( $K_{AE}$ ) for unrestrained walls (in accordance with Section C.4.6.5 of the *Commentary* to the CHBDC) may be used in design; these coefficients reflect the maximum  $K_{AE}$  obtained for each of the earthquake design periods and backfill conditions. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is level.

**Seismic Active Pressure Coefficients,  $K_{AE}$**

Wall Type	Design Earthquake	Site PGA	$K_{AE}$ for Granular 'A'	$K_{AE}$ for Granular 'B' Type II
Yielding Wall	475 Year	0.099	0.30	0.30
	975 Year	0.159	0.32	0.32
	2,475 Year	0.275	0.35	0.35
Non-Yielding Wall	475 Year	0.099	0.33	0.33
	975 Year	0.159	0.37	0.37
	2,475 Year	0.275	0.46	0.46



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The  $K_{AE}$  value for a yielding wall is applicable provided that the wall can move up to  $250k_h$  mm, where  $k_h$  is the site specific PGA as given in the table above. This corresponds to displacements of 25, 40, and 70 mm for the 475-year, 975-year, and 2,475-year design earthquakes at this site.

The earthquake-induced dynamic pressure distribution, which is to be added to the static earth pressure distribution, is a linear distribution with maximum pressure at the top of the wall and minimum pressure at its toe (i.e. an inverted triangular pressure distribution). The total pressure distribution (static plus seismic) may be determined as follows:

For Yielding Walls:  $\delta_{h(z)} = K_a \gamma z + (K_{AE} - K_a) \gamma (H-z)$

For Non-Yielding Walls:  $\delta_{h(z)} = K_o \gamma z + (K_{AE} - K_o) \gamma (H-z)$

Where:

- $\sigma_{h(z)}$  is the lateral earth pressure at depth 'z' (kPa);
- $K_a$  is the static active earth pressure coefficient;
- $K_o$  is the static at-rest earth pressure coefficient;
- $K_{AE}$  is the seismic earth pressure coefficient;
- $\gamma$  is the unit weight of the backfill soil ( $\text{kN/m}^3$ ), as given previously;
- $z$  is the depth below the top of the wall (m); and,
- $H$  is the total height of the wall (m).

## 6.9 Approach Embankment Design and Construction

Embankment widening beyond the ends of the retaining walls will likely be accomplished using conventional 2H:1V embankment side slopes.

Based on the borehole results, the embankment widening subgrade soils will consist of fill materials, generally comprising of silty sand to sand. The fill is underlain by silty clay to clayey silt to clay, which is in turn underlain by loose to very dense silty sand till.

### 6.9.1 Subgrade Preparation and Embankment Construction

It is recommended that all topsoil/organic soil or soil containing organic matter be stripped from the footprint of the approaches for the widened Maitland Avenue approach embankments. The existing fill material within the footprint of the widening can generally be left in place beneath the embankment widening, provided some modest settlement (i.e., less than 15 mm) of the subgrade can be tolerated. However the subgrade surface should be proof rolled and compacted to 95 percent of the standard Proctor maximum dry density. The type of material placed in the frost zone (up to about 1.8 metres depth) should match, as closely as possible, the existing embankment fill for frost heave compatibility purposes.

The embankment fill for the bridge replacement should be placed and compacted in accordance with OPSS.PROV 206 (Grading) and OPSS.PROV 501 (Compacting). Benching of the existing embankment side slopes should be carried out to "key in" the new fill materials in areas where the embankment is widened, in accordance with OPSS 208.010 (Benching of Earth Slopes).

To reduce erosion of the embankment side slopes due to surface water runoff, placement of topsoil and seeding or pegged sod is recommended as soon as practicable after construction of the embankments. The erosion protection should be in accordance with OPSS 804.PROV (Seed and Cover).



### 6.9.2 Approach Embankment and Retaining Wall Stability

“Limit equilibrium” stability analyses were carried out to assess the factor of safety against deep-seated global instability of the approach embankments and retaining walls (based on a rotational shear failure through the underlying silty clay). Those analyses were carried out for the critical ‘undrained’ conditions which would exist during and immediately following construction, the ‘drained’ conditions that would exist in the long term, and also possible seismic loading conditions. The Slope/W commercial software was used to determine the factor of safety based on the Morgenstern-Price methodology.

With appropriate subgrade preparation and proper placement of earth or granular soils, the 6 m high approach embankments, beyond the ends of the wingwalls, with side slopes maintained at 2 horizontal to 1 vertical, founded on the existing fill materials and native soils, will have a factor of safety greater than 1.3 against deep seated slope instability (as shown on Figure F1 in Appendix F) and a factor of safety greater than 1.1 against deep-seated global instability under seismic loading, based on an acceleration of 0.14g (which corresponds to half the PGA, as per the CHBDC). The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. That sloughing would not however impair the short term use of the structure and is mainly a maintenance/repair issue. The potential for sloughing could be reduced by providing well vegetated side slopes, as mentioned above in Section 6.9.1.

Embankment toe retaining walls up to 4 m in height will have a factor of safety of greater than 1.3 against deep-seated global instability. The stability of the retaining walls against sliding, overturning, and bearing failure will however need to be checked by the designers.

The following soil parameters were used in the stability analysis:

Material	Bulk Unit Weight (kN/m <sup>3</sup> )	Effective Friction Angle (degrees)	Undrained Shear Strength (kPa)
New Granular Embankment Fill	21	32	-
Existing Sand and Silty Sand Fill	19	28	-
Weathered Silty Clay (Drained)	18	35	7.5
Weathered Silty Clay (Undrained)	18	-	80
Silty Clay (Drained)	16.8	35	5
Silty Clay (Undrained)	16.8	-	65
Till	22	32°	-
Bedrock	Impenetrable		

The preliminary assessment of the stability of the approach embankments and retaining walls should be reviewed and confirmed based on the design drawings and subsoil conditions encountered within the proposed approach embankment and retaining wall footprints.

### 6.9.3 Settlement under Widened Embankment Loading

It is understood that an approximately 4.4 m of eastward widening is planned for the existing 6 m high approach embankments.



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Settlement assessments have been completed for the eastward widening using estimated consolidation parameters, based on correlations with the shear strength and SPT “N” values and engineering judgement from experience with similar soils in this region of Ontario. Based on this assessment, the recompression settlement of the subgrade soils under the 4.4 m eastward widening (i.e., beneath the existing 2H:1V slopes) is estimated to be about 25 mm, mainly associated with the silty clay deposit. It is expected that most of this settlement would take place during the construction of the embankment.

The above estimates do not include compression of the fill itself, which would occur essentially during the construction. The magnitude of settlement of the fill will depend on the type of fill placed and on the method and sequence of placement and compaction, but is expected to range from 0.5 to 1.0 percent of the height of the embankment.

### 6.10 Construction Considerations

The following sections identify future construction considerations that may impact the future design and construction.

#### 6.10.1 Existing Utilities

There are several utilities in the area of the proposed bridge widenings. If the settlements discussed in Section 6.9.3 will not be mitigated by the use of lightweight fill or structural support, the impact of the potential settlements to the existing utilities will need to be considered. The impact should be limited to a distance of about 3 m (in plan) from the footprint of the widening.

#### 6.10.2 Open-Cut Excavations

Excavations should be carried out in accordance with the guidelines outlined in the latest edition of the Occupational Health and Safety Act (OHSA) for Construction Activities.

The construction of new spread/strip footings and/or pile caps will require excavations to depths up to about 5 m below the existing grade and will be made through the existing fill, and native clay and till deposits. The groundwater level at the pier is indicated to be at a depth of about 2 to 3 m (i.e., about Elevation 79 to 80 m). The soils at the site are generally classified as Type 3 soils according to the OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper than 1 horizontal to 1 vertical (1H:1V).

#### 6.10.3 Temporary Protection Systems

It is anticipated that temporary roadway protection will be required along Maitland Avenue and along Highway 417 to permit construction of the abutment widenings and toe walls. For the pier footing construction, temporary excavation support may be required due to space restrictions. It is considered that the temporary support system could consist of internally braced soldier piles and lagging; the internal bracing may reduce the need for sockets and tie-backs which would have to be extended into the bedrock.

The design of the shoring will be entirely the responsibility of the contractor. Where required, temporary protection systems should be designed and constructed in accordance with OPSS.PROV 539 (Temporary Protection System), and the lateral movement should meet Performance Level 2 provided that any existing adjacent utilities can tolerate this magnitude of deformation. Traffic loading should be included as a surcharge.



#### **6.10.4 Obstructions**

It should be noted that obstructions (inferred as cobbles) were encountered within the fill and till deposit in the area of the proposed foundations. The presence of such obstructions could affect excavation works, installation of temporary protection systems as well as construction of deep foundation.

#### **6.10.5 Groundwater and Surface Water Control**

The groundwater level at the pier is about 2 to 3 m depth below the highway grade. Excavations for the construction of the pier foundations on bedrock will likely involve minimal groundwater and surface water control. It should be possible to handle ground and surface water inflows by pumping from well filtered sumps established in the floor of the excavations.



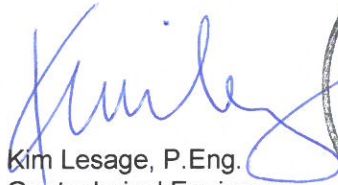


## FOUNDATION REPORT HIGHWAY 417 MAITLAND AVENUE UNDERPASS


### 7.0 CLOSURE


This report was prepared by Ms. Kim Lesage, P.Eng., and was reviewed by Ms. Erin O'Neill, P.Eng., a senior geotechnical engineer and Associate of Golder. Mr. Fintan Heffernan, P.Eng., a Senior Consultant with Golder and the Designated MTO Foundations Contact, conducted an independent quality control review of this report.

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Designated MTO Foundations Contact



KSL/ESO/FJH/ob

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## FOUNDATION REPORT HIGHWAY 417 MAITLAND AVENUE UNDERPASS

**TABLE 1 – COMPARISON OF BRIDGE ABUTMENT FOUNDATION ALTERNATIVES**

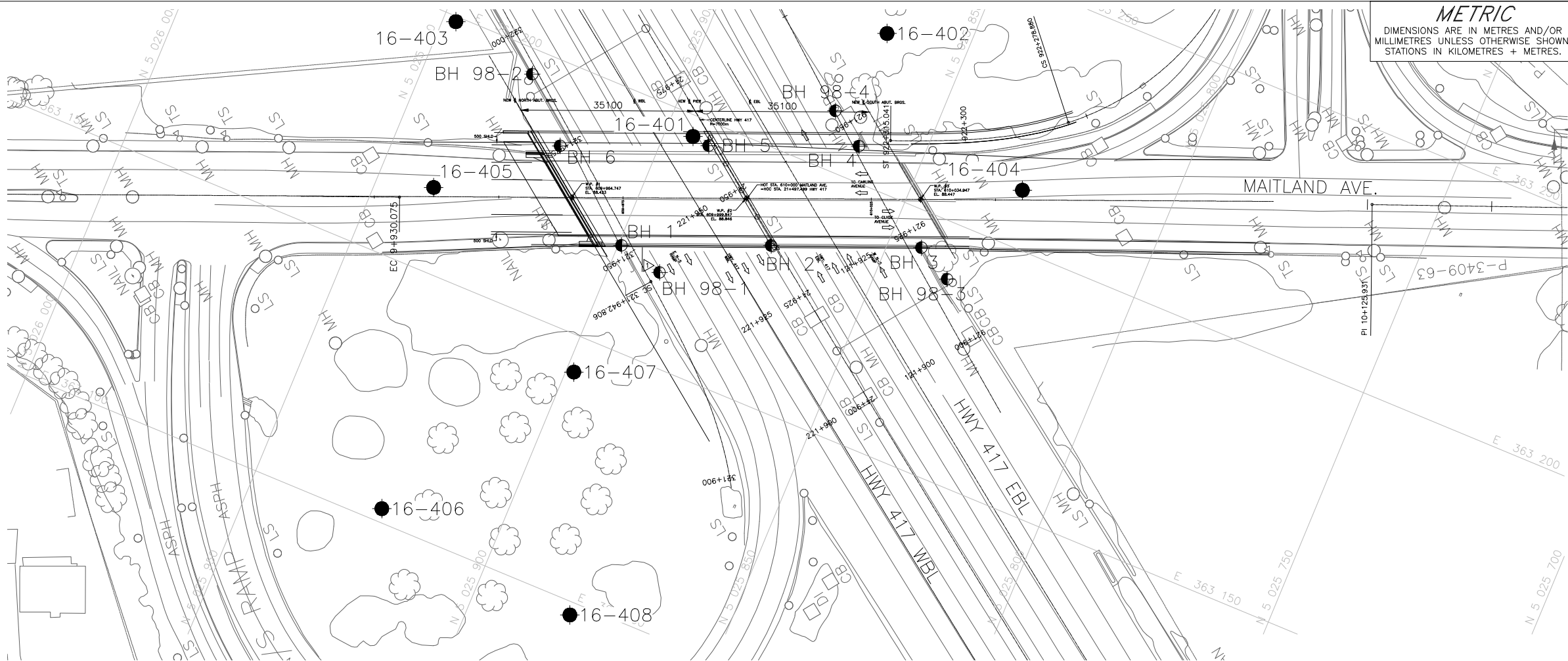
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/ Risks
1. Steel H-piles or pipe piles founded on or socketed into bedrock	<ul style="list-style-type: none"> <li>Feasible for the support of new abutments with pile cap “perched” within the approach embankments.</li> </ul>	<ul style="list-style-type: none"> <li>High bearing resistance.</li> <li>Negligible settlement.</li> <li>Compatible with existing bridge foundations.</li> <li>Conventional construction methods for H-pile or steel pipe pile foundations.</li> <li>Steel H-piles allow for integral abutment configuration.</li> </ul>	<ul style="list-style-type: none"> <li>Piles may refuse above design tip elevation due to the bouldery till, especially pipe piles which have a larger displacement base.</li> <li>If lateral / seismic loading conditions merit, pile toe may have to be socketed into medium strong to strong bedrock, which would require coring or churn drilling</li> <li>If sockets required, temporary liner necessary</li> <li>Piles may interfere with existing battered piles.</li> <li>Pipe piles not readily accepted for integral abutment construction; allow for semi-integral abutment configuration.</li> </ul>	<ul style="list-style-type: none"> <li>May be less expensive than caisson option.</li> </ul>	<ul style="list-style-type: none"> <li>Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving.</li> <li>Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving.</li> <li>Potential for conflict with existing rock anchors for retaining walls.</li> </ul>
2. Drilled shaft (caissons) founded on or socketed into bedrock	<ul style="list-style-type: none"> <li>Feasible, but not compatible with the existing foundations.</li> </ul>	<ul style="list-style-type: none"> <li>High bearing resistance.</li> <li>Negligible settlement.</li> <li>Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles.</li> </ul>	<ul style="list-style-type: none"> <li>Temporary liners may be required.</li> <li>Possibility of encountering cobbles or boulders during installation.</li> <li>Socketting of liner may be required to permit cleaning and inspection.</li> <li>If rock socket required, coring or churn drilling will be required to form rock socket in medium strong to strong bedrock.</li> </ul>	<ul style="list-style-type: none"> <li>May be more expensive than steel H-pile option, particularly if rock sockets are necessary, due to larger socket diameter.</li> </ul>	<ul style="list-style-type: none"> <li>May not be able to dewater socket for cleaning and inspection.</li> <li>Potential for conflict with existing rock anchors for retaining walls.</li> </ul>
Spread/strip footings supported on native soil	<ul style="list-style-type: none"> <li>Not Feasible</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>	<ul style="list-style-type: none"> <li>N/A</li> </ul>



## FOUNDATION REPORT HIGHWAY 417 MAITLAND AVENUE UNDERPASS

**TABLE 2 – COMPARISON OF EMBANKMENT TOE RETAINING WALL FOUNDATION ALTERNATIVES**

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/ Risks
Shallow foundations supported on native soil	<ul style="list-style-type: none"> <li>Feasible, but not consistent with existing design.</li> </ul>	<ul style="list-style-type: none"> <li>Walls would be constructed in conjunction with overall widening, so excavations into slope would only need to be deep enough to provide sufficient frost protection.</li> </ul>	<ul style="list-style-type: none"> <li>Temporary roadway protection may be required.</li> <li>Differential settlement with respect to existing retaining walls because of different foundations.</li> <li></li> </ul>	<ul style="list-style-type: none"> <li>May be less expensive than deep foundations.</li> </ul>	<ul style="list-style-type: none"> <li>Conventional excavation and construction techniques.</li> </ul>
Steel H-pile foundations founded on, socketted into, or pinned to bedrock	<ul style="list-style-type: none"> <li>Feasible for support of all foundation elements.</li> </ul>	<ul style="list-style-type: none"> <li>High bearing resistance.</li> <li>Negligible settlement.</li> <li>Consistent with existing wall foundation system.</li> </ul>	<ul style="list-style-type: none"> <li>If lateral / seismic loading conditions merit, pile toe may have to be pinned to (as was done for the existing wall) or socketted into medium strong to strong bedrock, which would require coring or churn drilling.</li> <li>Possibility of encountering cobbles or boulders during installation.</li> <li>If sockets required, temporary liner necessary.</li> </ul>	<ul style="list-style-type: none"> <li>Possibility of piles being driven mis-aligned due to boulders in glacial till.</li> </ul>	<ul style="list-style-type: none"> <li>May be less expensive than caisson option.</li> </ul>
Caissons founded on, socketted into, or pinned to bedrock	<ul style="list-style-type: none"> <li>Feasible for support of all foundation elements.</li> </ul>	<ul style="list-style-type: none"> <li>Very high bearing resistance.</li> <li>Negligible settlement.</li> </ul>	<ul style="list-style-type: none"> <li>Temporary liners required to minimize disturbance to surrounding soils.</li> <li>Possibility of encountering cobbles or boulders during installation.</li> <li>If rock socket required, coring or churn drilling will be required to form rock socket in medium strong to strong bedrock.</li> </ul>	<ul style="list-style-type: none"> <li>May not be able to dewater socket for cleaning and inspection.</li> </ul>	<ul style="list-style-type: none"> <li>More expensive than steel H-pile option and spread footings.</li> </ul>



**METRIC**  
DIMENSIONS ARE IN METRES AND/OR  
MILLIMETRES UNLESS OTHERWISE SHOWN.  
STATIONS IN KILOMETRES + METRES.

CONT No.  
WP No.4015-E-0017



HIGHWAY 417 REHABILITATION  
AND WIDENING  
MAITLAND AVENUE UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEY PLAN  
SCALE  
1 0 1 2 km

LEGEND

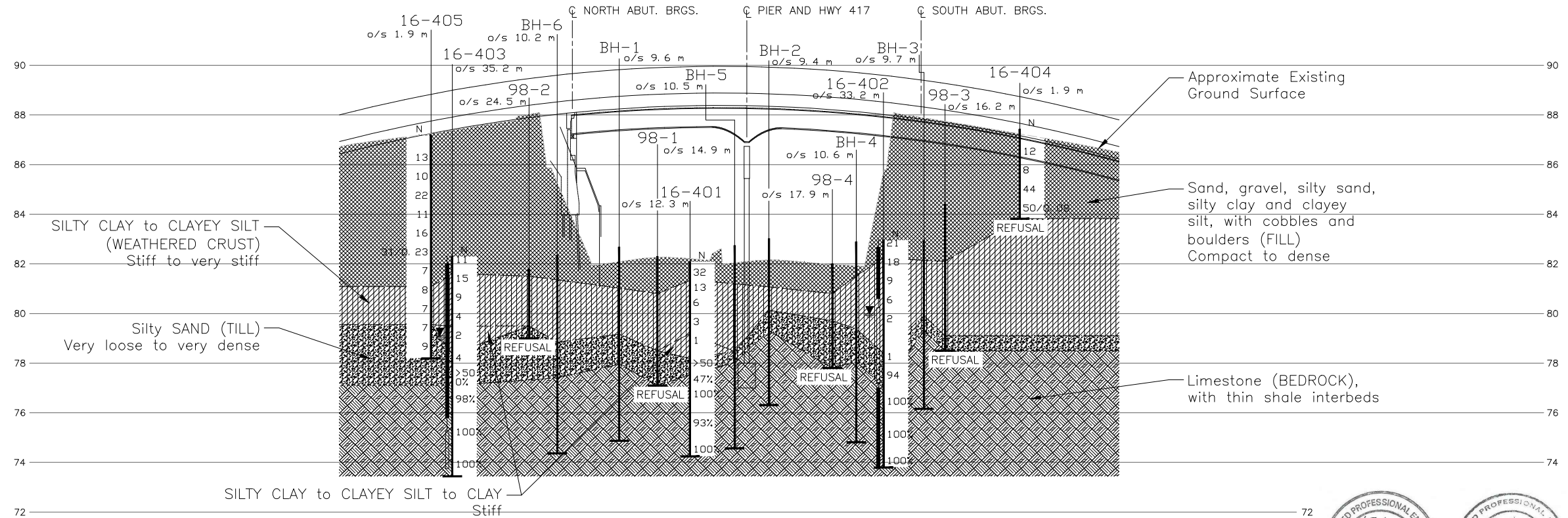
- Borehole - Current Investigation
- Borehole - Previous Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Total Core Recovery (REC)
- ⊥ WL in piezometer, measured on Sept. 30, 2016
- ⊥ WL upon completion of drilling

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-401	82.1	5025894.5	363195.9
16-402	83.0	5025866.3	363230.0
16-403	82.3	5025947.4	363199.0
16-404	87.4	5025829.0	363211.3
16-405	87.2	5025938.7	363166.4
16-406	83.5	5025923.6	363102.7
16-407	82.9	5025898.4	363142.9
16-408	82.9	5025880.4	363097.4
98-1	82.3	5025890.2	363168.0
98-2	81.8	5025929.1	363195.1
98-3	84.4	5025836.2	363189.0
98-4	82.0	5025869.9	363211.7
BH-1	82.7	5025899.3	363170.1
BH-2	83.0	5025871.5	363181.7
BH-3	83.0	5025843.4	363192.8
BH-4	82.9	5025862.8	363206.9
BH-5	82.7	5025890.8	363195.4
BH-6	82.4	5025918.4	363183.9

NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.



PROFILE ALONG MAITLAND AVE.

HORIZ. SCALE  
10 0 10 20 m  
VERT. SCALE  
2 0 2 4 m

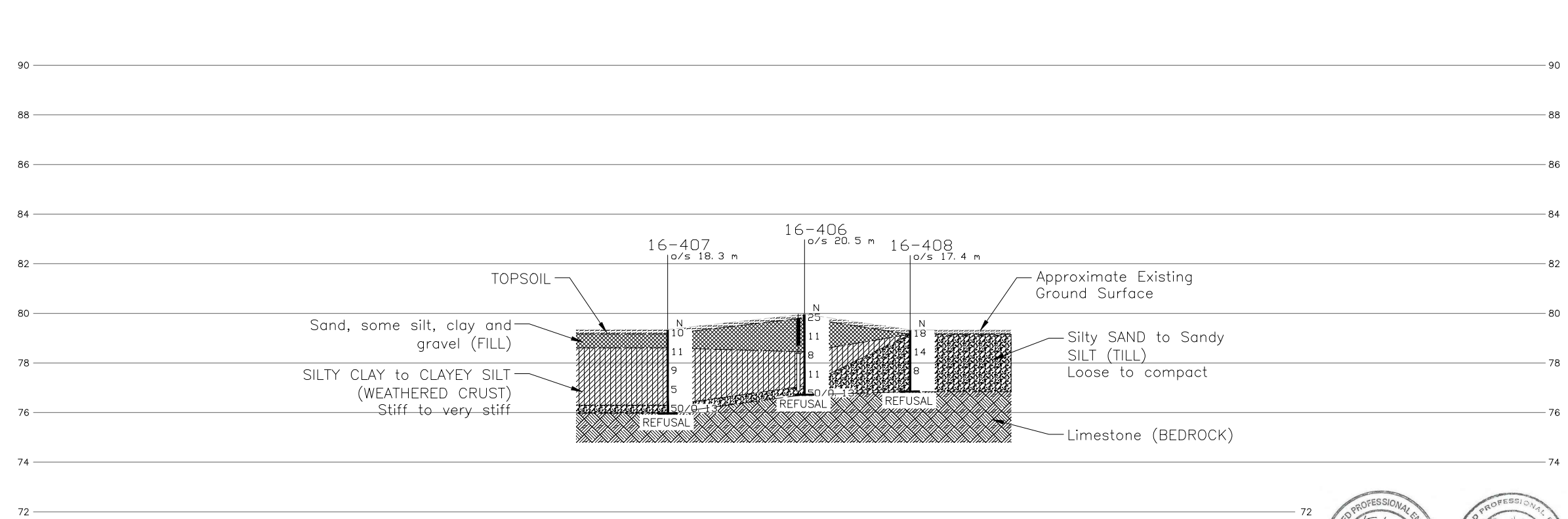
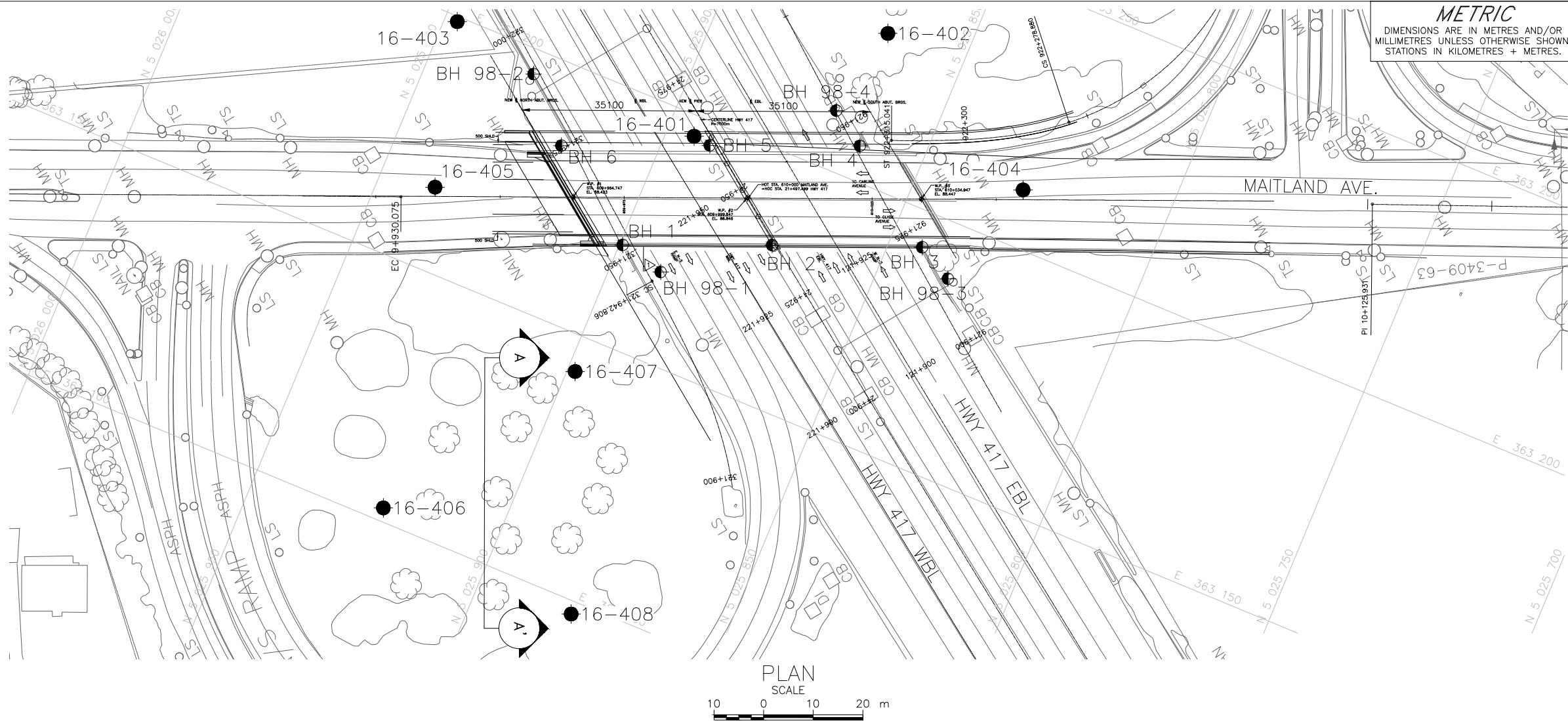
REFERENCE

Base plans provided in digital format by MMM Group, drawing file no. GG\_3416012-305-001\_General-Arrangement.dwg, received October 26, 2016.



NO.	DATE	BY	REVISION
Geocres No. 3165-275			
HWY. 417		PROJECT NO. 1546542-1040	
SUBM'D. KSL		DATE: 11/01/2016	SITE: 3-042
DRAWN: JM		CHKD. ESO	APPD. FJH
			DWG. 1





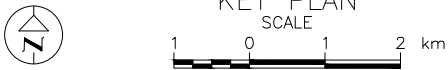
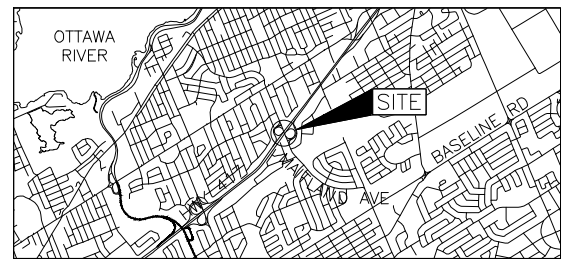
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CONT No. WP No.4015-E-0017

HIGHWAY 417 REHABILITATION AND WIDENING  
MAITLAND AVENUE UNDERPASS  
BOREHOLE LOCATIONS AND SOIL STRATA



### LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation
- ⊥ Seal
- ⊥ Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- 100% Total Core Recovery (REC)
- ⊥ WL in piezometer, measured on Sept. 30, 2016
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16-406	83.5	5025923.6	363102.7
16-407	82.9	5025898.4	363142.9
16-408	82.9	5025880.4	363097.4
BH-1	82.3	5025890.2	363168.0
BH-2	81.8	5025929.1	363195.1
BH-3	84.4	5025836.2	363189.0
BH-4	82.0	5025869.9	363211.7
BH-5	82.7	5025899.3	363170.1
BH-6	82.4	5025918.4	363183.9

### NOTES

This drawing is for subsurface information only. The proposed structure details/works are shown for illustration purposes only and may not be consistent with the final design configuration as shown elsewhere in the Contracts Documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

NO.	DATE	BY	REVISION
1	11/22/2016	J.M.	1

Geocres No. 3165-275

HWY. 417	PROJECT NO. 1546542-1040	DIST. EASTERN
SUBM'D. KSL	CHKD. KSL	DATE: 11/22/2016
DRAWN: JM	CHKD. ESO	APPD. FJH



# **APPENDIX A**

## **Borehole and Drillhole Records, Current Investigation**

**Lists of Abbreviations and Symbols**

**Lithological and Geotechnical Rock Description Terminology**

**Records of Boreholes 16-401 to 16-408**

**Bedrock Core Photographs, Figures A1 to A6**



## LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

### I. GENERAL

$\pi$	3.1416
$\ln x$ ,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

### II. STRESS AND STRAIN

$\gamma$	shear strain
$\Delta$	change in, e.g. in stress: $\Delta \sigma$
$\varepsilon$	linear strain
$\varepsilon_v$	volumetric strain
$\eta$	coefficient of viscosity
$\nu$	Poisson's ratio
$\sigma$	total stress
$\sigma'$	effective stress ( $\sigma' = \sigma - u$ )
$\sigma'_{vo}$	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
$\sigma_{oct}$	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
$\tau$	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

### III. SOIL PROPERTIES

#### (a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
$\gamma'$	unit weight of submerged soil ( $\gamma' = \gamma - \gamma_w$ )
$D_R$	relative density (specific gravity) of solid particles ( $D_R = \rho_s / \rho_w$ ) (formerly $G_s$ )
e	void ratio
n	porosity
S	degree of saturation

#### (a) Index Properties (continued)

w	water content
$w_l$ or LL	liquid limit
$w_p$ or PL	plastic limit
$I_p$ or PI	plasticity index = $(w_l - w_p)$
$w_s$	shrinkage limit
$I_L$	liquidity index = $(w - w_p) / I_p$
$I_C$	consistency index = $(w_l - w) / I_p$
$e_{max}$	void ratio in loosest state
$e_{min}$	void ratio in densest state
$I_D$	density index = $(e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

#### (b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

#### (c) Consolidation (one-dimensional)

$C_c$	compression index (normally consolidated range)
$C_r$	recompression index (over-consolidated range)
$C_s$	swelling index
$C_\alpha$	secondary compression index
$m_v$	coefficient of volume change
$C_v$	coefficient of consolidation (vertical direction)
$C_h$	coefficient of consolidation (horizontal direction)
$T_v$	time factor (vertical direction)
U	degree of consolidation
$\sigma'_p$	pre-consolidation stress
OCR	over-consolidation ratio = $\sigma'_p / \sigma'_{vo}$

#### (d) Shear Strength

$\tau_p, \tau_r$	peak and residual shear strength
$\phi'$	effective angle of internal friction
$\delta$	angle of interface friction
$\mu$	coefficient of friction = $\tan \delta$
$c'$	effective cohesion
$c_u, s_u$	undrained shear strength ( $\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
$p'$	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
$q_u$	compressive strength $(\sigma_1 - \sigma_3)$
$S_t$	sensitivity

\* Density symbol is  $\rho$ . Unit weight symbol is  $\gamma$  where  $\gamma = \rho g$  (i.e. mass density multiplied by acceleration due to gravity)

Notes: 1  
2

$$\tau = c' + \sigma' \tan \phi'$$

$$\text{shear strength} = (\text{compressive strength})/2$$



## LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

### I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

### II. PENETRATION RESISTANCE

#### Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

#### Dynamic Cone Penetration Resistance; $N_d$ :

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60° cone attached to "A" size drill rods for a distance of 300 mm (12 in.).

**PH:** Sampler advanced by hydraulic pressure

**PM:** Sampler advanced by manual pressure

**WH:** Sampler advanced by static weight of hammer

**WR:** Sampler advanced by weight of sampler and rod

#### Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm<sup>2</sup> pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance ( $Q_t$ ), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

### III. SOIL DESCRIPTION

#### (a) Non-Cohesive (Cohesionless) Soils

Density Index	N
Relative Density	Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

#### (b) Cohesive Soils Consistency

	$c_u, s_u$	
	kPa	psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

### IV. SOIL TESTS

w	water content
w <sub>p</sub>	plastic limit
w <sub>l</sub>	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test <sup>1</sup>
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement <sup>1</sup>
D <sub>R</sub>	relative density (specific gravity, $G_s$ )
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO <sub>4</sub>	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
$\gamma$	unit weight

**Note:** 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

### V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand



## LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY

### WEATHERINGS STATE

**Fresh:** no visible sign of weathering

**Faintly weathered:** weathering limited to the surface of major discontinuities.

**Slightly weathered:** penetrative weathering developed on open discontinuity surfaces but only slight weathering of rock material.

**Moderately weathered:** weathering extends throughout the rock mass but the rock material is not friable.

**Highly weathered:** weathering extends throughout rock mass and the rock material is partly friable.

**Completely weathered:** rock is wholly decomposed and in a friable condition but the rock and structure are preserved.

### BEDDING THICKNESS

Description	Bedding Plane Spacing
Very thickly bedded	Greater than 2 m
Thickly bedded	0.6 m to 2 m
Medium bedded	0.2 m to 0.6 m
Thinly bedded	60 mm to 0.2 m
Very thinly bedded	20 mm to 60 mm
Laminated	6 mm to 20 mm
Thinly laminated	Less than 6 mm

### JOINT OR FOLIATION SPACING

Description	Spacing
Very wide	Greater than 3 m
Wide	1 m to 3 m
Moderately close	0.3 m to 1 m
Close	50 mm to 300 mm
Very close	Less than 50 mm

### GRAIN SIZE

Term	Size*
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns

Note: \* Grains greater than 60 microns diameter are visible to the naked eye.

### CORE CONDITION

#### Total Core Recovery (TCR)

The percentage of solid drill core recovered regardless of quality or length, measured relative to the length of the total core run.

#### Solid Core Recovery (SCR)

The percentage of solid drill core, regardless of length, recovered at full diameter, measured relative to the length of the total core run.

#### Rock Quality Designation (RQD)

The percentage of solid drill core, greater than 100 mm length, recovered at full diameter, measured relative to the length of the total core run. RQD varied from 0% for completely broken core to 100% for core in solid sticks.

### DISCONTINUITY DATA

#### Fracture Index

A count of the number of discontinuities (physical separations) in the rock core, including both naturally occurring fractures and mechanically induced breaks caused by drilling.

#### Dip with Respect to Core Axis

The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

#### Description and Notes

An abbreviation description of the discontinuities, whether naturally occurring separations such as fractures, bedding planes and foliation planes or mechanically induced features caused by drilling such as ground or shattered core and mechanically separated bedding or foliation surfaces. Additional information concerning the nature of fracture surfaces and infillings are also noted.

#### Abbreviations

JN Joint	PL Planar
FLT Fault	CU Curved
SH Shear	UN Undulating
VN Vein	IR Irregular
FR Fracture	K Slickensided
SY Stylolite	PO Polished
BD Bedding	SM Smooth
FO Foliation	SR Slightly Rough
CO Contact	RO Rough
AXJ Axial Joint	VR Very Rough
KV Karstic Void	
MB Mechanical Break	



PROJECT		1546542-1040		RECORD OF BOREHOLE No 16-401		SHEET 1 OF 2		METRIC									
W.P.		4015-E-0017		LOCATION		N 5025894.5 ; E 363195.9		ORIGINATED BY									
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		COMPILED BY									
DATUM		Geodetic		DATE		June 22, 2016		CHECKED BY									
								KSL									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
82.1	GROUND SURFACE							20	40	60	80	100					
0.0	ASPHALTIC CONCRETE							20	40	60	80	100					
81.8																	
0.3	Gravelly sand (FILL) Dense Grey Moist		1	SS	32												
81.3																	
0.8	SILTY CLAY to CLAYEY SILT (WEATHERED CRUST) Stiff to very stiff Grey brown Moist		2	SS	13												
			3	SS	6												
			4	SS	3												
79.2																	
2.9	SILTY CLAY to CLAYEY SILT, some sand Stiff Grey Wet		5	SS	1												
78.2																	
3.9	Silty SAND, some clay, trace gravel (TILL) Dense Wet		6	SS	>50												
77.6																	
4.6	Limestone (BEDROCK)  Bedrock cored from depths of 4.6 m to 7.9 m  For bedrock coring details refer to Record of Drillhole 16-401		1	RC	REC 80%											RQD = 47%	
			2	RC	REC 100%												RQD = 100%
			3	RC	REC 100%												RQD = 93%
			4	RC	REC 100%												RQD = 100%
74.3																	
7.9	END OF BOREHOLE																

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SHEET 2 OF 2

DATUM: Geodetic

DRILLING CONTRACTOR: CCC

[illegible]

DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: KSL

<b>PROJECT</b> 1546542-1040		<b>RECORD OF BOREHOLE No 16-402</b>		SHEET 1 OF 3		<b>METRIC</b>	
<b>W.P.</b> 4015-E-0017		<b>LOCATION</b> N 5025866.3 ; E 363230.0		<b>ORIGINATED BY</b> DG			
<b>DIST</b> Eastern HWY 417		<b>BOREHOLE TYPE</b> Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		<b>COMPILED BY</b> ZS			
<b>DATUM</b> Geodetic		<b>DATE</b> June 27, 2016		<b>CHECKED BY</b> KSL			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W <sub>p</sub>	W	W <sub>L</sub>		
83.0	GROUND SURFACE							20	40	60	80	100					GR SA SI CL
0.0	Silty sand (TOPSOIL)																
0.1	Brown Dry		1	SS	21												
	Silty sand (FILL)																
82.3	Compact Brown Moist																
0.7	SILTY CLAY to CLAYEY SILT, trace sand (WEATHERED CRUST)		2	SS	18												
	Very stiff Grey-brown Moist																
			3	SS	9												0 3 52 45
			4	SS	6												
80.0	SILTY CLAY to CLAYEY SILT																
3.1	Stiff Grey Wet		5	SS	2												
78.4	Silty SAND, some clay, trace gravel (TILL)		6	SS	1												
4.6	Very loose Grey Wet																
77.7	Silty SAND, some clay, trace gravel (TILL)		7	SS	94												
5.3	Very dense Grey Moist																
77.0	Limestone (BEDROCK)																
6.0	Bedrock cored from depths of 6.0 m to 9.8 m		1	RC	REC 100%												RQD = 100%
	For bedrock coring details refer to Record of Drillhole 16-402																
			2	RC	REC 100%												RQD = 100%
			3	RC	REC 100%												RQD = 100%
73.8																	
9.2																	

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○ 3% STRAIN AT FAILURE

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PROJECT <u>1546542-1040</u>		<b>RECORD OF BOREHOLE No 16-402</b>				SHEET 2 OF 3		<b>METRIC</b>	
W.P. <u>4015-E-0017</u>		LOCATION <u>N 5025866.3 ;E 363230.0</u>				ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core</u>				COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>June 27, 2016</u>				CHECKED BY <u>KSL</u>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W <sub>p</sub>	W			W <sub>L</sub>
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> <span>20   40   60   80   100</span> </div> <div style="display: flex; justify-content: space-between;"> <span>20   40   60   80   100</span> </div>										
	END OF BOREHOLE  NOTES:  1. Water level in well screen at a depth of 3.0 m below ground surface (Elev. 79.9 m), measured on September 30, 2016.																

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PROJECT: 1546542-1040

**RECORD OF DRILLHOLE: 16-402**

SHEET 3 OF 3

LOCATION: N 5025866.3 ;E 363230.0

DRILLING DATE: June 27, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
						FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 <sup>-9</sup>	10 <sup>-8</sup>	10 <sup>-7</sup>	10 <sup>-6</sup>	W1		W2	W3	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
6		Continued from Record of Borehole 16-402		77.00																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								</

DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: KSL

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PROJECT		1546542-1040		RECORD OF BOREHOLE No 16-403		SHEET 1 OF 2		METRIC										
W.P.		4015-E-0017		LOCATION		N 5025947.4 ; E 363199.0		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		COMPILED BY										
DATUM		Geodetic		DATE		July 5, 2016		CHECKED BY										
								KSL										
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	20 40 60 80 100	25 50 75	W <sub>p</sub>	W	W <sub>L</sub>			
82.3	0.0	GROUND SURFACE																
	0.1	Silty sand (TOPSOIL) Brown Dry		1	SS	11		82										
		Sand, trace silt (FILL) Compact Brown Moist																
	0.7	SILTY CLAY to CLAYEY SILT (WEATHERED CRUST) Stiff to very stiff Grey-brown		2	SS	15		81										
				3	SS	9		80										
				4	SS	4												
	79.5	Silty CLAY to CLAY Stiff Grey		5	SS	2		79										
	2.8																	
	78.4	Silty SAND, some clay, trace gravel, contains cobbles and boulders (TILL) Loose to dense Grey Wet		6	SS	4		78										3 51 34 12
	3.9			7	SS	>50												
				1	RC	REC 58%												RQD = 0%
	77.1	Limestone (BEDROCK)						77										
	5.2	Bedrock cored from depths of 5.2 m to 8.9 m  For bedrock coring details refer to Record of Drillhole 16-403		2	RC	REC 100%		76										RQD = 98%
				3	RC	REC 100%		75										RQD = 100%
				4	RC	REC 100%		74										RQD = 100%
	73.4	END OF BOREHOLE																
	8.9	NOTES:  1. Water level in well screen at a depth of 3.2 m below ground surface (Elev. 79.1 m), measured on September 30, 2016.																

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PROJECT: 1546542-1040

**RECORD OF DRILLHOLE: 16-403**

SHEET 2 OF 2

LOCATION: N 5025947.4 ; E 363199.0

DRILLING DATE: July 5, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 750

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY															FEATURES	
				ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX				
							TOTAL CORE % 000000 000000 000000	SOLID CORE % 000000 000000 000000				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 <sup>-9</sup> 10 <sup>-8</sup> 10 <sup>-7</sup> 10 <sup>-6</sup>	10 <sup>-9</sup> 10 <sup>-8</sup> 10 <sup>-7</sup> 10 <sup>-6</sup>	W1 W2 W3 W4 W5 W6			
		Continued from Record of Borehole 16-403		77.09																
		Limestone (BEDROCK) Fresh Grey Medium bedded Medium strong to strong			5.21															
6				2	100															
7	Rotary Drill NQ Core			3	100															
8				4	100															
9		END OF DRILLHOLE		73.44 8.86																
10																				
11																				
12																				
13																				
14																				
15																				

DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: KSL

GTA-RCK 031 N:\ACTIVE\SPATIAL\_IMMTO\HWY417\REHAB&amp;WIDENING\02\_DATA\GINT\1546542.GPJ GAL-MISS.GDT 03/09/17 JM

PROJECT		1546542-1040		RECORD OF BOREHOLE No 16-404		SHEET 1 OF 1		METRIC							
W.P.		4015-E-0017		LOCATION		N 5025829.0 ; E 363211.3		ORIGINATED BY							
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY							
DATUM		Geodetic		DATE		July 27, 2016		CHECKED BY							
								KSL							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
87.4	GROUND SURFACE														
0.0 87.2	ASPHALTIC CONCRETE														
86.9	Sandy gravel (FILL) Brown Dry														
86.7 0.7	Sand (FILL) Brown Moist														
	Gravel and silty clay, some sand, contains sand seams, cobbles and boulders (FILL) Grey to grey-brown		1	SS	12										
			2	SS	8										
			3	SS	44										
			4	SS	50/0.08										
83.8 3.6	END OF BOREHOLE AUGER REFUSAL														

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMTOHWY417REHAB&amp;WIDENING\02\_DATA\GINT\1546542.GPJ GAL-GTA.GDT 03/09/17 JM

PROJECT		1546542-1040		RECORD OF BOREHOLE No 16-405		SHEET 1 OF 1		METRIC								
W.P.		4015-E-0017		LOCATION		N 5025938.7 ; E 363166.4		ORIGINATED BY								
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY								
DATUM		Geodetic		DATE		July 26, 2016		CHECKED BY								
								KSL								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
87.2	GROUND SURFACE															
0.0	ASPHALTIC CONCRETE															
87.0																
0.2	Sandy gravel (FILL) Compact Brown-grey Dry															
86.6																
86.4	Sand (FILL) Compact Brown Moist		1	SS	13											
0.8	Silty clay to clayey silt, some sand, trace gravel, contains sand seams, asphaltic concrete fragments and cobbles (FILL) Grey-brown		2	SS	10											8 24 34 34
			3	SS	22											
			4	SS	11											
			5	SS	16											
82.6	Silty sand to sand, some clay and gravel, contains cobbles and boulders (FILL) Loose Grey-brown Moist		6	SS	31/0.23											
4.6																
			7	SS	7											12 43 27 18
81.1	SILTY CLAY to CLAYEY SILT, trace sand (WEATHERED CRUST) Very stiff Grey-brown Moist		8	SS	8											
6.1																
			9	SS	7											
79.6	Silty SAND, some clay and gravel (TILL) Loose Grey Wet		10	SS	7											
7.6																
			11	SS	9											
78.2	END OF BOREHOLE															
9.0																

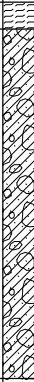
GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMTOHWY417REHAB&amp;WIDENING\02\_DATA\GINT\1546542.GPJ GAL-GTA.GDT 03/09/17 JM

PROJECT		1546542-1040		RECORD OF BOREHOLE No 16-406		SHEET 1 OF 1		METRIC								
W.P.		4015-E-0017		LOCATION		N 5025923.6 ; E 363102.7		ORIGINATED BY								
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY								
DATUM		Geodetic		DATE		July 14, 2016		CHECKED BY								
								KSL								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
83.5	GROUND SURFACE															
0.0	Silty sand (TOPSOIL) Dark brown Moist															
0.2	Silt and sand, some clay, trace gravel (FILL) Brown to black Moist		1	SS	25											
			2	SS	11											3 43 38 16
82.0																
1.5	SILTY CLAY to CLAYEY SILT (WEATHERED CRUST) Very stiff Grey-brown Moist		3	SS	8											
			4	SS	11											
80.6																
2.9	Silty SAND, some clay, trace gavel (TILL) Very dense Grey-brown Moist		5	SS	50/0.13											
80.3																
3.3	END OF BOREHOLE AUGER REFUSAL															
NOTES: 1. Well screen dry on September 30, 2016.																

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMTOHWY417REHAB&amp;WIDENING\02\_DATA\GINT\1546542.GPJ GAL-GTA.GDT 03/09/17 JM

PROJECT		1546542-1040		RECORD OF BOREHOLE No 16-407		SHEET 1 OF 1		METRIC										
W.P.		4015-E-0017		LOCATION		N 5025898.4 ; E 363142.9		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY										
DATUM		Geodetic		DATE		June 14, 2016		CHECKED BY										
								KSL										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
82.9	GROUND SURFACE							20	40	60	80	100						
82.7	Silty sand (TOPSOIL) Dark brown Moist		1	SS	10													
82.2	SAND, some silt, clay and gravel (FILL) Compact Brown Moist		2	SS	11													
81.7	SILTY CLAY to CLAYEY SILT, some sand (WEATHERED CRUST) Stiff to very stiff Grey-brown Moist		3	SS	9													
81.2			4	SS	5													
80.7																		
79.9	Silty SAND, some clay, trace gravel (TILL) Very dense Brown Wet		5	SS	50/0.13													
79.5	END OF BOREHOLE AUGER REFUSAL																	
79.4																		

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMTOHWY417REHAB&amp;WIDENING\02\_DATA\GINT\1546542.GPJ GAL-GTA.GDT 03/09/17 JM

PROJECT		1546542-1040		RECORD OF BOREHOLE No 16-408		SHEET 1 OF 1		METRIC										
W.P.		4015-E-0017		LOCATION		N 5025880.4 ;E 363097.4		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY										
DATUM		Geodetic		DATE		June 14, 2016		CHECKED BY										
								KSL										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
82.9	GROUND SURFACE							20	40	60	80	100						
82.7	Sandy silt (TOPSOIL) Dark brown Moist		1	SS	18													
0.2	Silty SAND to Sandy SILT, some clay, trace gravel (TILL) Loose to compact Brown Moist																	
			2	SS	14													
			3	SS	8													
80.4	END OF BOREHOLE AUGER REFUSAL																	
2.5																		

GTA-MTO 001 N:\ACTIVE\SPATIAL\_IMMTOHWY417REHAB&amp;WIDENING\02\_DATA\GINT\1546542.GPJ GAL-GTA.GDT 03/09/17 JM



BH 16-401 (Wet)  
Cored Length of 4.55 to 7.85 metres  
Core Box 1 to 2 of 2

4.55 m Top of bedrock



7.85 m EOH

CLIENT  
MMM Group Limited

PROJECT  
MMM/ MTO 4014-E-0017 HIGHWAY 417/ OTTAWA  
MAITLAND UNDERPASS

CONSULTANT



YYY/MM/DD	3/2/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE  
**BOREHOLE 16-401 (WET)  
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1040	0a	A2

BH 16-401 (Dry)  
Cored Length of 4.55 to 7.85 metres  
Core Box 1 to 2 of 2

4.55 m Top of bedrock



7.85 m EOH

CLIENT  
MMM Group Limited

PROJECT  
MMM/ MTO 4014-E-0017 HIGHWAY 417/ OTTAWA  
MAITLAND UNDERPASS

CONSULTANT



YYY/MM/DD	3/2/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE  
**BOREHOLE 16-401 (DRY)  
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1040	0a	A1



BH 16-402 (Wet)  
Cored Length of 5.97 to 9.17 metres  
Core Box 1 to 2 of 2

5.97 m Top of bedrock



9.17 m EOH

CLIENT  
MMM Group Limited

CONSULTANT



YYY/MM/DD	3/2/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

PROJECT  
MMM/ MTO 4014-E-0017 HIGHWAY 417/ OTTAWA  
MAITLAND UNDERPASS

TITLE  
**BOREHOLE 16-402 (WET)  
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1040	0a	A4

BH 16-402 (Dry)  
Cored Length of 5.97 to 9.17 metres  
Core Box 1 to 2 of 2

5.97 m Top of bedrock



9.17 m EOH

CLIENT  
MMM Group Limited

PROJECT  
MMM/ MTO 4014-E-0017 HIGHWAY 417/ OTTAWA  
MAITLAND UNDERPASS

CONSULTANT



YYY/MM/DD	3/2/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE  
**BOREHOLE 16-402 (DRY)  
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1040	0a	A3



5.21 m Top of bedrock

BH 16-403 (Wet)  
Cored Length of 5.21 to 8.86 metres  
Core Box 1 to 2 of 2



CLIENT  
MMM Group Limited

PROJECT  
MMM/ MTO 4014-E-0017 HIGHWAY 417/ OTTAWA  
MAITLAND UNDERPASS

CONSULTANT



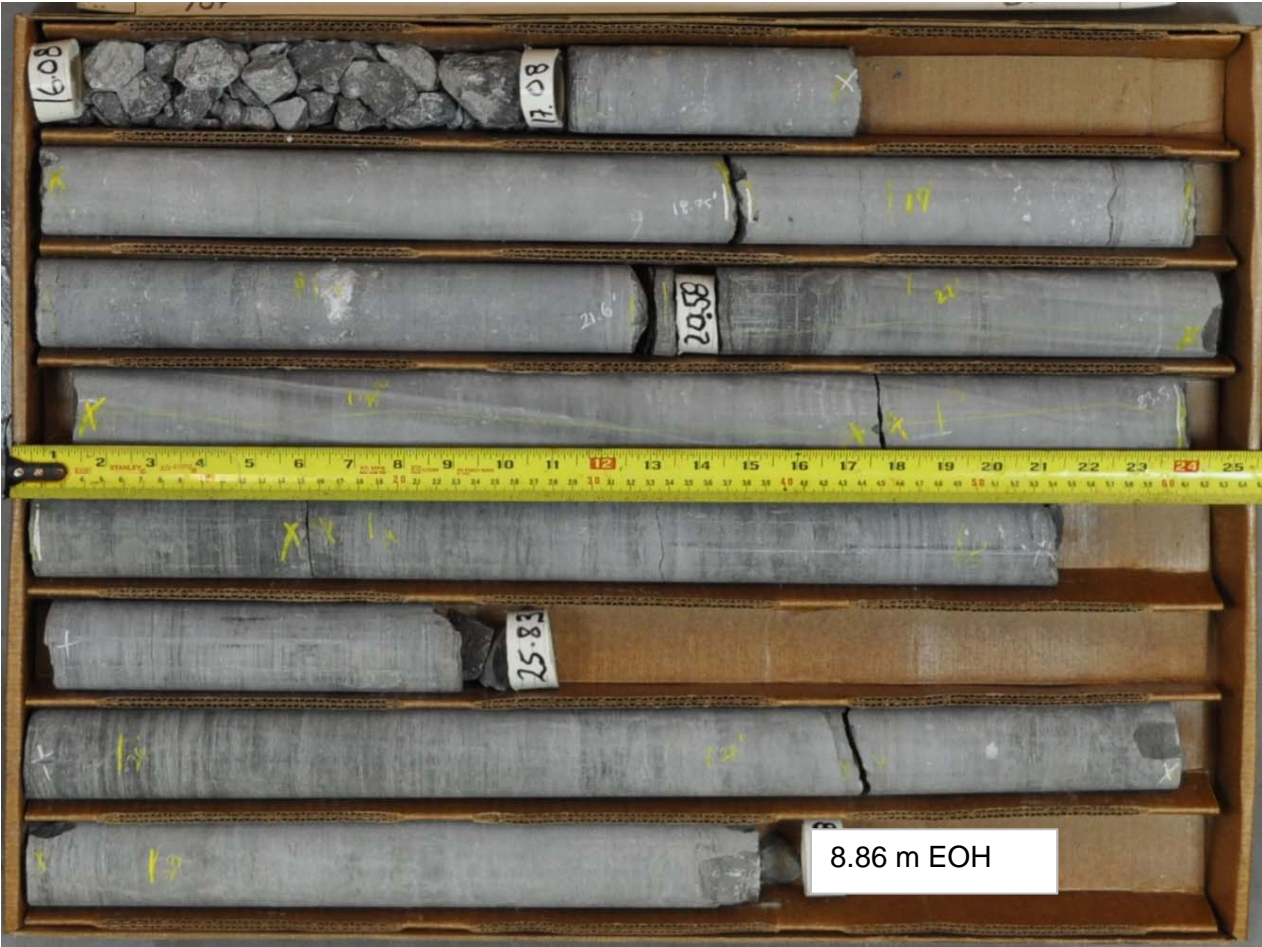
YYY/MM/DD	3/2/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE  
**BOREHOLE 16-403 (WET)  
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1040	0a	A6

5.21 m Top of bedrock

BH 16-403 (Dry)  
Cored Length of 5.21 to 8.86 metres  
Core Box 1 to 2 of 2



CLIENT  
MMM Group Limited

PROJECT  
MMM/ MTO 4014-E-0017 HIGHWAY 417/ OTTAWA  
MAITLAND UNDERPASS

CONSULTANT



YYY/MM/DD	3/2/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE  
**BOREHOLE 16-403 (DRY)  
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1040	0a	A5





## **APPENDIX B**

### **Laboratory Test Results, Current Investigation**

**Figure 1 - Grain Size Distribution Test Results – Silty Sand to Sand (FILL)**

**Figure 2 - Plasticity Chart – Silty Clay (Fill)**

**Figure 3 - Plasticity Chart – Silty Clay to Clayey Silt (Weathered Crust)**

**Figure 4 - Grain Size Distribution Test Results – Silty Clay to Clayey Silt (Weathered Crust)**

**Figure 5 - Plasticity Chart – Silty Clay to Clayey Silt to Clay**

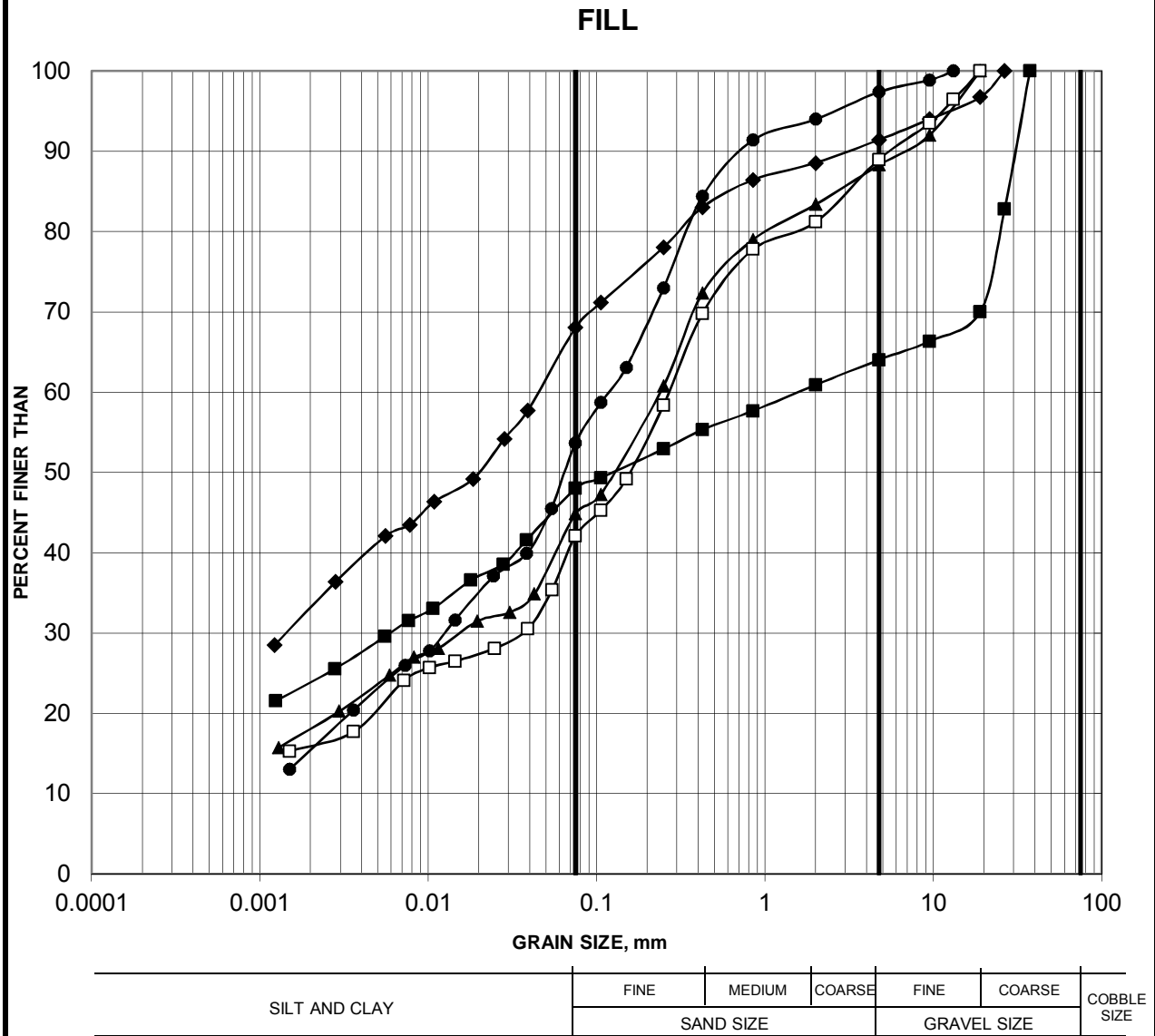
**Figure 6 - Grain Size Distribution Test Results – Silty Clay to Clayey Silt**

**Figure 7 - Grain Size Distribution Test Results – Silty Sand (Till)**

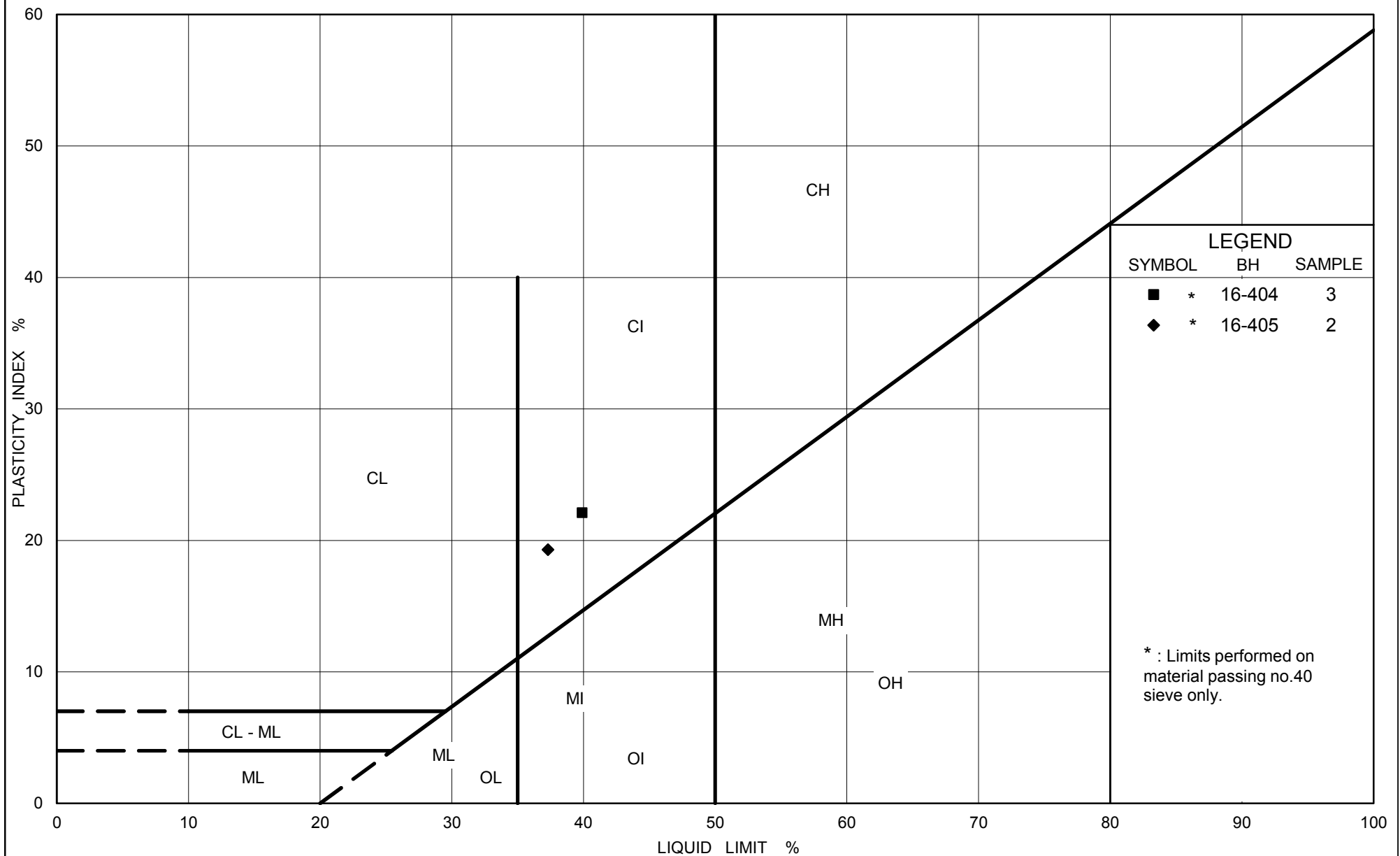
**Figure 8 - Summary of Laboratory Compressive Strength Unconfined Compression Tests**

# GRAIN SIZE DISTRIBUTION

FIGURE 1



Borehole	Sample	Depth (m)
16-404	3	2.29-2.90
16-405	2	1.52-2.13
16-405	7	4.82-5.18
16-406	2	0.76-1.37
16-407	1B	0.18-0.61



Ministry of Transportation

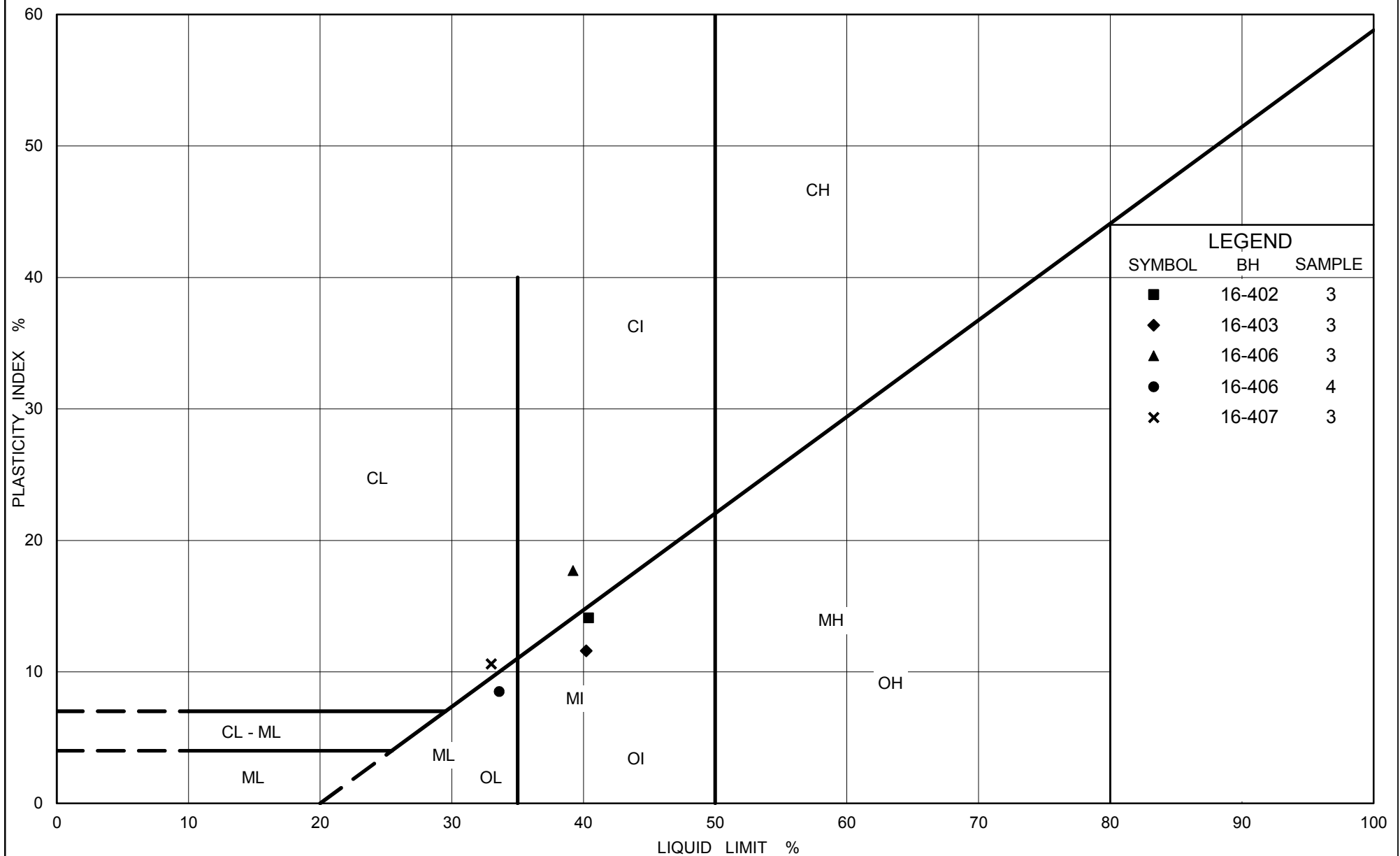
Ontario

## PLASTICITY CHART Silty Clay (FILL)

FIG No. 2

Project No. 1546542-1040

Compiled By : MI    Checked By : CNM



Ontario

Ministry of Transportation

# PLASTICITY CHART SILTY CLAY to CLAYEY SILT (WEATHERED CRUST)

FIG No. 3

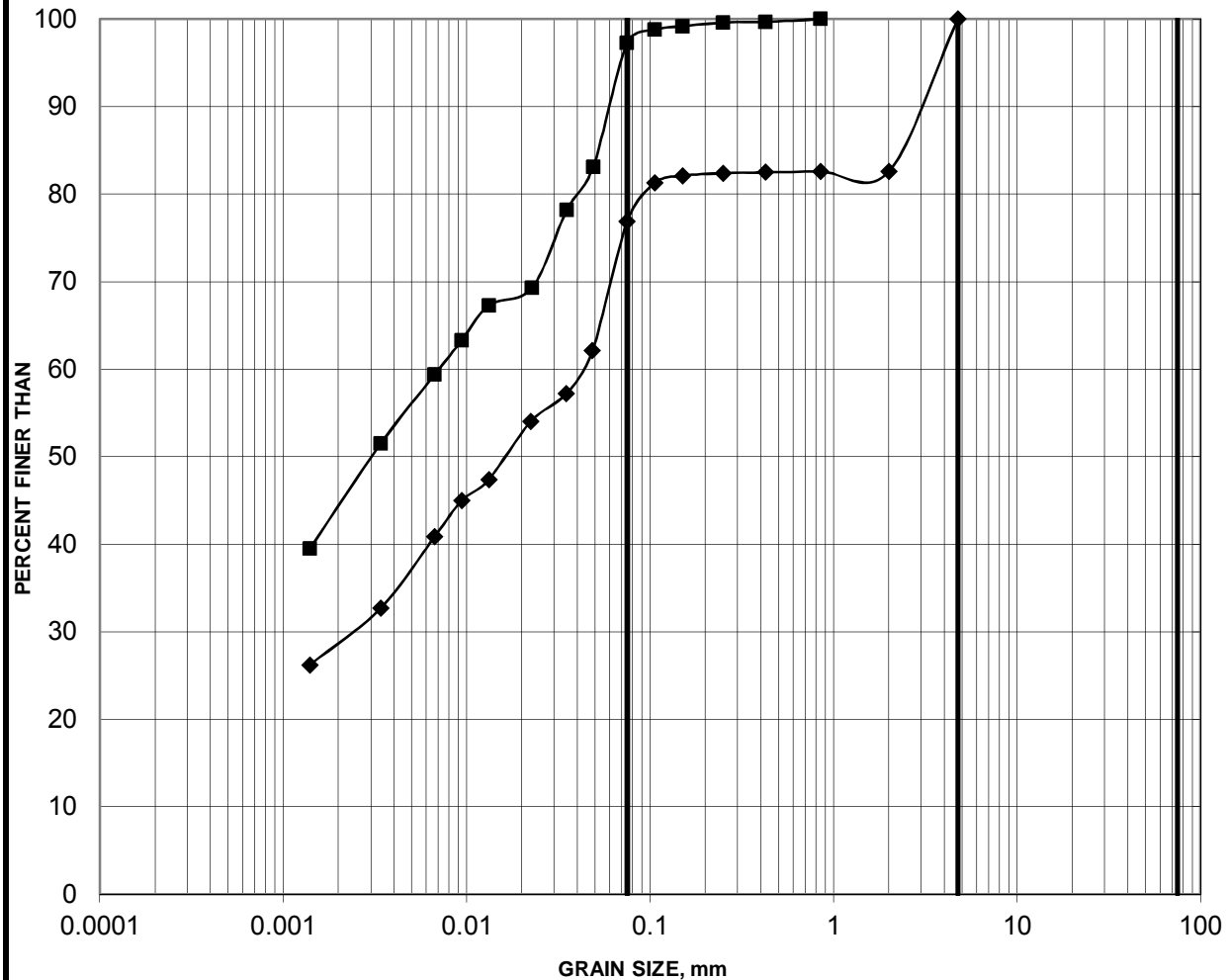
Project No. 1546542-1040

Compiled By : MI      Checked By : CNM

# GRAIN SIZE DISTRIBUTION

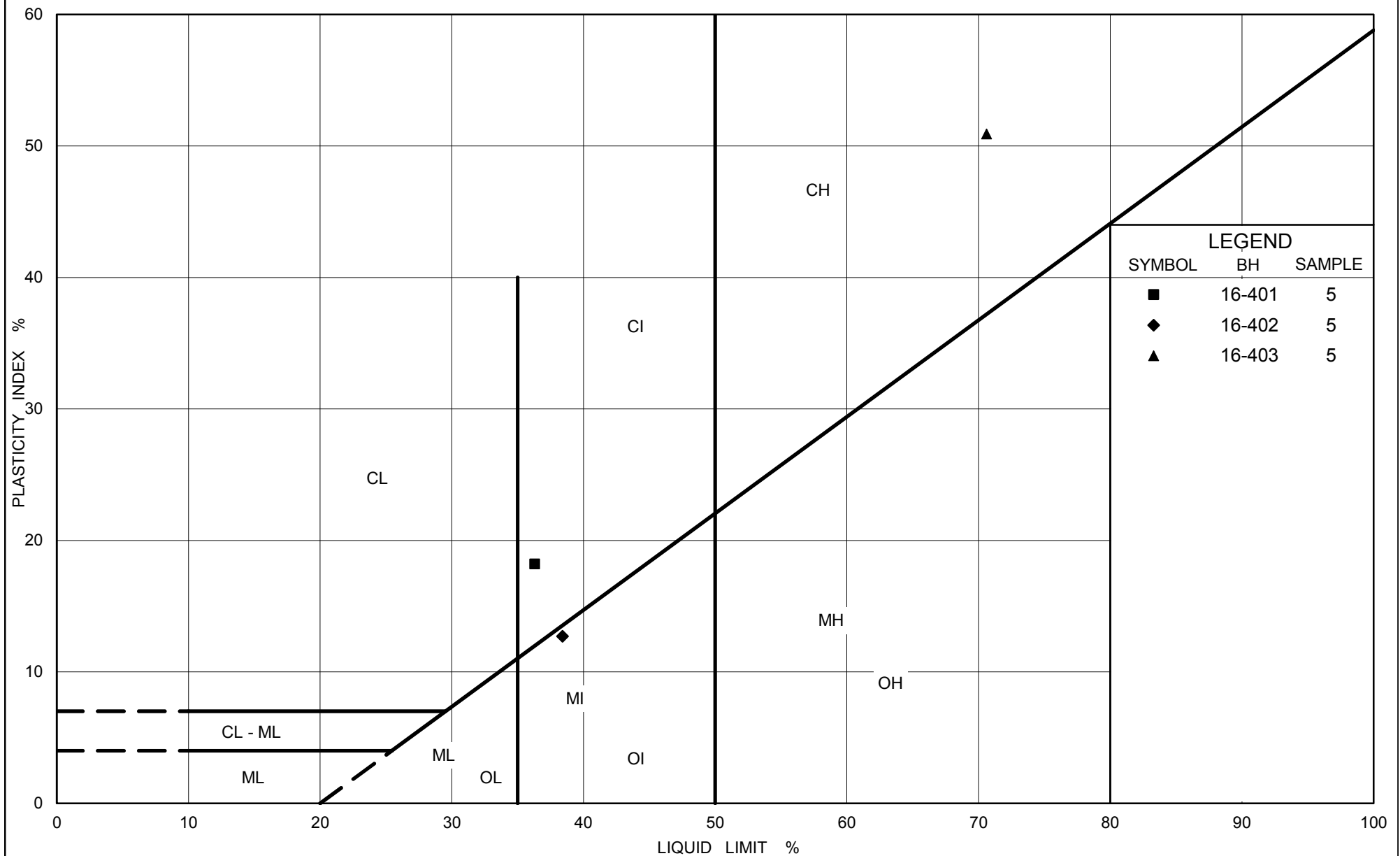
FIGURE 4

## SILTY CLAY TO CLAYEY SILT (WEATHERED CRUST)



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-402	3	1.52-2.13
16-407	3	1.52-2.13



Ontario

Ministry of Transportation

## PLASTICITY CHART SILTY CLAY to CLAYEY SILT to CLAY

FIG No. 5

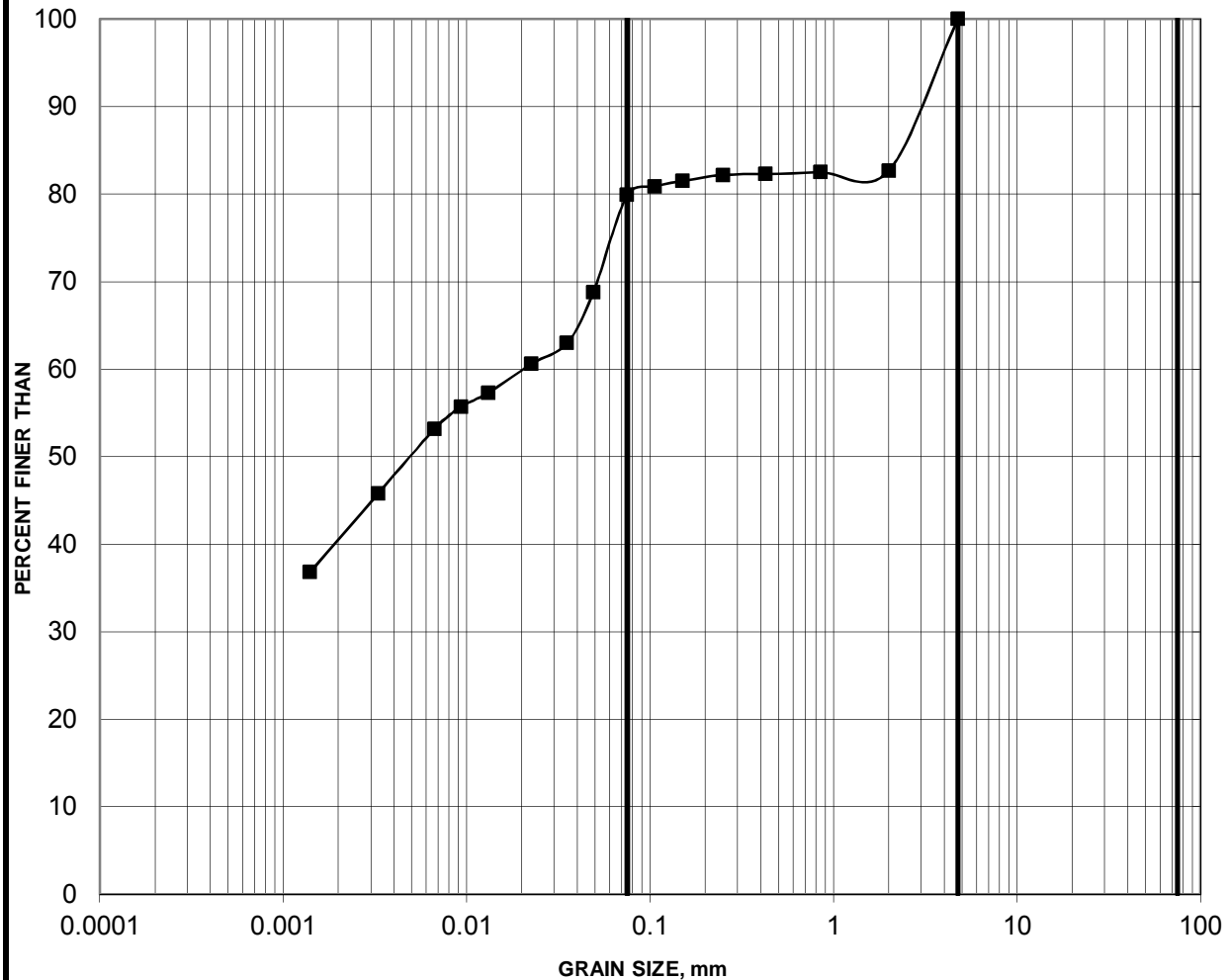
Project No. 1546542-1040

Compiled By : MI    Checked By : CNM

# GRAIN SIZE DISTRIBUTION

FIGURE 6

## SILTY CLAY TO CLAYEY SILT



SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

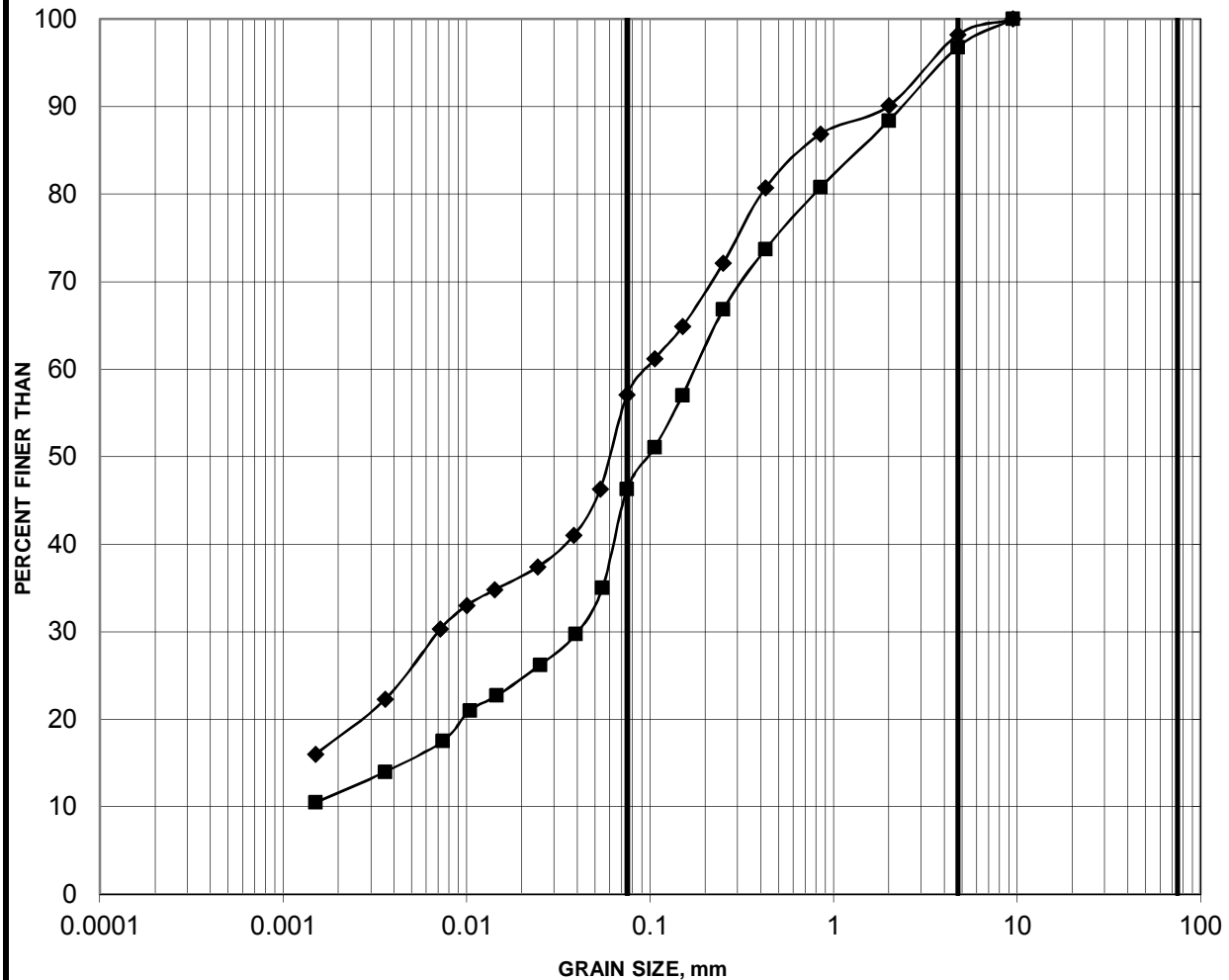
Borehole	Sample	Depth (m)
16-401	5	3.05-3.66



# GRAIN SIZE DISTRIBUTION

FIGURE 7

## SILTY SAND (TILL)

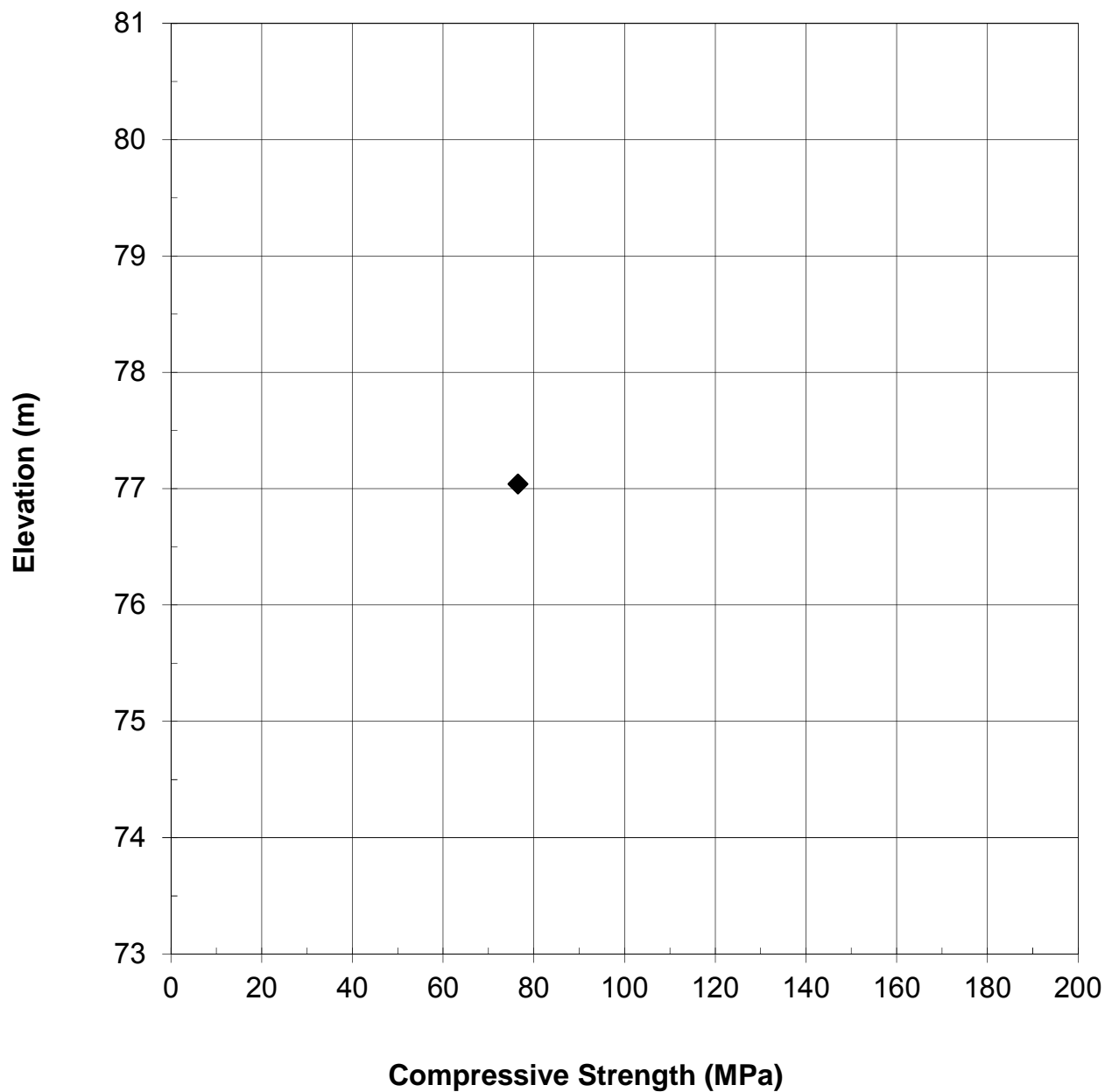


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-403	6	3.96-4.57
16-408	3	1.52-2.13

**SUMMARY OF LABORATORY COMPRESSIVE STRENGTH  
UNCONFINED COMPRESSION TESTS**

**FIGURE 8**





# **APPENDIX C**

## **Previous Borehole Records**

**Records of Previous Boreholes 1 to 6 (Geocres No. 31G5-022)**

**Records of Previous Boreholes 98-1 to 98-4 (Geocres No. 31G5-190)**











# McROSTIE & ASSOCIATES

## CONSULTING ENGINEERS

### OTTAWA CANADA

# SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY AT MAITLAND AVE.  
BRIDGE No. 3

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 271.3 GEODETIC  
REMARKS SEE PLATE 2

HOLE No.

5

DATE MAY 14-1958

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. <sup>2</sup>	SMALL SCALE PENETROMETER KIPS/FT. <sup>2</sup>	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PERCENTAGE WATER CONTENT			
				GROUND SURFACE						
				TOP SOIL	0	271.3				
				HARD, FISSURED, SILTY	2	270.3				
3.7 R-1.8	8.2 8.3 8.4 9.0 9.0 9.0 9.0 9.0 9.0		5-1	BROWNISH-GRAY CLAY	4	266.8				
				VERY STIFF	6					
1.8 R-1.2	5.6 5.6 7.0 7.2 7.0 7.6 6.8 6.8 6.4		5-2	FISSURED, SILTY	8					
3.0 R-0.9	5.0 5.4 5.0 6.2 6.0 6.2 5.6 6.0 5.8		5-3	BROWNISH-GRAY CLAY	10	261.8				
0.9 R-0.3	2.4 2.0 2.2 3.0 3.2 3.0 3.4 3.2 3.2		5-4	STIFF, FISSURED, SILTY	12	259.3				
				BROWNISH-GRAY CLAY	14	257.3				
				SOFT, VERY SILTY	16	255.8				
				GRAY CLAY	18	254.3				
				DENSE TILL	20	251.3				
				LIMESTONE (DRILLED)	22	246.1				
				CORE RECOVERY 60%	24	244.5				
				LIMESTONE (DRILLED)	26					
				CORE RECOVERY 83%	28					
				BEDDING THICKNESS 3"						
				LIMESTONE (DRILLED)						
				CORE RECOVERY 87%						
				BEDDING THICKNESS 3"						
				ONE 80° JOINT IN CORE BREAK						
				LIMESTONE (DRILLED)						
				CORE RECOVERY 100%						
				BEDDING THICKNESS 3"						
				BOTTOM OF HOLE						

NATURAL  
% WATER CONTENT  
SHOWN THUS ○

PLATE  
6

# McROSTIE & ASSOCIATES

## CONSULTING ENGINEERS

### OTTAWA CANADA

#### SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY AT MAITLAND AVE.  
BRIDGE No. 3

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 270.0 - GEODETIC

HOLE NO.

REMARKS SEE PLATE 2

6

DATE MAY 13-14, 1958

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. <sup>2</sup>	SMALL SCALE PENETROMETER KIPS/FT. <sup>2</sup>	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST				
							LE. HAMMER		NO. CASING		
							INCH DROP		INCH DIA. ROD		
GROUND SURFACE							BLOWS PER FOOT				
				TOP SOIL	0	270.0					
					1.0	269.0					
				HARD, FISSURED, SILTY	2						
2.5	8.4-7.8 7.4		6-1	BROWNISH-GRAY CLAY	4						
	9.0 9.0 9.0				4.5	265.5					
4.7	6.2 7.2 8.0		6-2	VERY STIFF, FISSURED SILTY	6						
	7.4 7.6 7.0			BROWNISH-GREY CLAY	7.0	263.0					
	8.0 8.0 8.0										
1.8	3.6 3.4 3.2		6-3	STIFF, FISSURED, SILTY	8						
	3.2 3.5 3.8										
	4.0 3.7 3.4										
2.0	3.4 3.6 3.4		6-4	BROWNISH-GRAY CLAY	10						
	3.6 4.0 3.8				11.5	258.5					
			7	LOOSE TILL	12						
			33 for 6"		13.5	256.5					
			44 for 3"	DENSE TILL	14						
					16.2	253.8					
				LIMESTONE (DRILLED)	18						
				CORE RECOVERY 93%							
				BEDDING THICKNESS 2"	19.7	250.3					
				LIMESTONE (DRILLED)	22						
				CORE RECOVERY 94%							
				BEDDING THICKNESS 3"	24.2	245.8					
				LIMESTONE (DRILLED)	26	243.8					
				CORE RECOVERY 95%							
				BEDDING THICKNESS 3"	26.2						
				BOTTOM OF HOLE							

0 20 40 60 80 100

NATURAL  
% WATER CONTENT  
SHOWN THUS ①

PLATE

7

RECORD OF BOREHOLE No 98-1

1 OF 1

METRIC

W.P. 203-86-02 LOCATION Hwy 417 WBL at Maitland Avenue ORIGINATED BY LP  
DIST 429 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PC  
DATUM Geodetic DATE 98.08.27 & 98.08.28 CHECKED BY PC

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100					
82.3																
0.0	Brown, silty sand, trace organics: FILL		1	SS	5											
81.6																
0.8	Brown, silty clay, some gravel, trace organics: FILL		2	SS	8											
80.8																
1.5	Very stiff, brown, SILTY CLAY, occasional sand seams		3	SS	7											
			4	SS	4											
79.3																
3.0	Stiff		5	SS	3											
78.5																
3.8	Loose to compact, gray, silty sand with gravel, trace clay; TILL		6	SS	7											
			7	SS	16											
77.1																
5.2	End of Borehole  Auger Refusal on Inferred Bedrock  - standpipe installed  - standard penetration tests carried out using 45 lb hammer and 30 inch freefall  max. field vane capacity was 150 kPa which was exceeded by soil resistance.															

x<sup>3</sup>, x<sup>2</sup>: Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE

# RECORD OF BOREHOLE No 98-2

1 OF 1

METRIC

W.P. 203-86-02 LOCATION Hwy 417 WBL at Maitland Avenue ORIGINATED BY LP  
 DIST 428 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PC  
 DATUM Geodetic DATE 98.06.27 & 98.06.28 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa											
81.8								20	40	60	80	100							
0.0	Dark brown, silty sand: FILL		1	SS	3		81												
81.5																			
0.3	Very stiff, brown, SILTY CLAY		2	SS	8			80											
			3	SS	7														
79.5					*REF	79													
2.3	Compact, brown, silty sand, some gravel, trace clay: TILL	4	SS																
79.0																			
2.8	End of Borehole																		
	Auger Refusal on Inferred Bedrock																		
	max. field vane capacity was 150 kPa which was exceeded by soil resistance.																		
	*REF = split spoon refusal																		

$\times^3, \times^1$ : Numbers refer to  
Sensitivity

20  
15  
10  
(%) STRAIN AT FAILURE



# RECORD OF BOREHOLE No 98-3

1 OF 1

METRIC

W.P. 203-86-02

LOCATION Hwy 417 EBL at Maitland Avenue

ORIGINATED BY LP

DIST 429 HWY 417

BOREHOLE TYPE Hollow Stem Augers

COMPILED BY PC

DATUM Geodetic

DATE 98.08.26 & 98.08.27

CHECKED BY PC






SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20	40	60	80	100		
84.4													
0.0	Brown, sand, some silt, trace organics, clayey pockets: FILL		1	SS	6								
			2	SS	3								
			3	SS	6								
82.1													
2.3	Very stiff, brown, SILTY CLAY, occasional sand seams		4	SS	8								
			5	SS	6								
80.6													
3.8	stiff		6	SS	3								
			7	SS	5								
79.2													
5.3	Loose, brown, clayey sand, some silt, trace gravel: TILL		8	SS	7								
78.6													
5.9	End of Borehole												
	Auger Refusal on Inferred Bedrock												
	max. field vane capacity was 150 kPa which was exceeded by soil resistance.												

# RECORD OF BOREHOLE No 98-4

1 OF 1

METRIC

W.P. 203-86-02 LOCATION Hwy 417 EBL at Maitland Avenue ORIGINATED BY LP  
 DIST 428 HWY 417 BOREHOLE TYPE Hollow Stem Augers COMPILED BY PC  
 DATUM Geodetic DATE 98.08.26 & 98.08.27 CHECKED BY PC

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					
82.0								20 40 60 80 100					
0.0	Brown, silty sand: FILL		1	SS	16								
80.8			2	SS	5								
1.2	Very stiff to stiff, brown, SILTY CLAY		3	SS	4								
78.7													
2.3	Loose, brown-grey, silty sand with gravel, trace clay: TILL		4	SS	8								
79.0													
3.1	Grey, silty clay, some gravel: TILL		5	SS	4								
77.8			6	SS		*REF							
4.2	End of Borehole												
	Auger Refusal on inferred Bedrock												
	- standpipe installed												
	- standard penetration tests carried out using 45 lb hammer and 30 inch freefall												
	*REF=split spoon refusal												



# **APPENDIX D**

## **Results of MASW TESTING**



**DATE** February 15, 2017**PROJECT No.** 1546542**TO** Kim Lesage  
Golder Associates Ltd.**CC****FROM** Stephane Sol, Christopher Phillips**EMAIL** ssol@golder.com;cphillips@golder.com**NBCC SEISMIC SITE CLASS TESTING RESULTS - HWY417 WIDENING PROJECT  
4 LOOP RAMP LOCATIONS ALONG HIGHWAY417, OTTAWA, ONTARIO**

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This technical memorandum presents the results of four Multichannel Analysis of Surface Waves (MASW) tests performed for the purpose of the 2010 National Building Code of Canada (NBCC2010) Seismic Site Classification for a Highway 417 widening project located along HWY417, Ottawa, Ontario. Site 1 is located within the HWY417 southwest loop ramp just west of Richmond Road (Richmond Site - Figure 1). Site 2 is located within the HWY417 southwest loop ramp just west of Pinecrest/ Greenbank Road (Pinecrest Site - Figure 2). Site 3 is located within the HWY417 northwest loop ramp just west of Woodroffe Avenue (Woodroffe Site - Figure 3). Site 4 is located within the HWY417 northwest loop ramp just west of Maitland Avenue (Maitland Site - Figure 4).

The geophysical testing was performed by Golder personnel on May 26 and 27, 2016.





Figure 1: MASW Location Site Map at the Richmond site (MASW Line 1 in red)



Figure 2: MASW Location Site Map at the Pinecrest site (MASW Line 2 in red)





Figure 3: MASW Location Site Map at the Woodroffe site (MASW Line 3 in red)

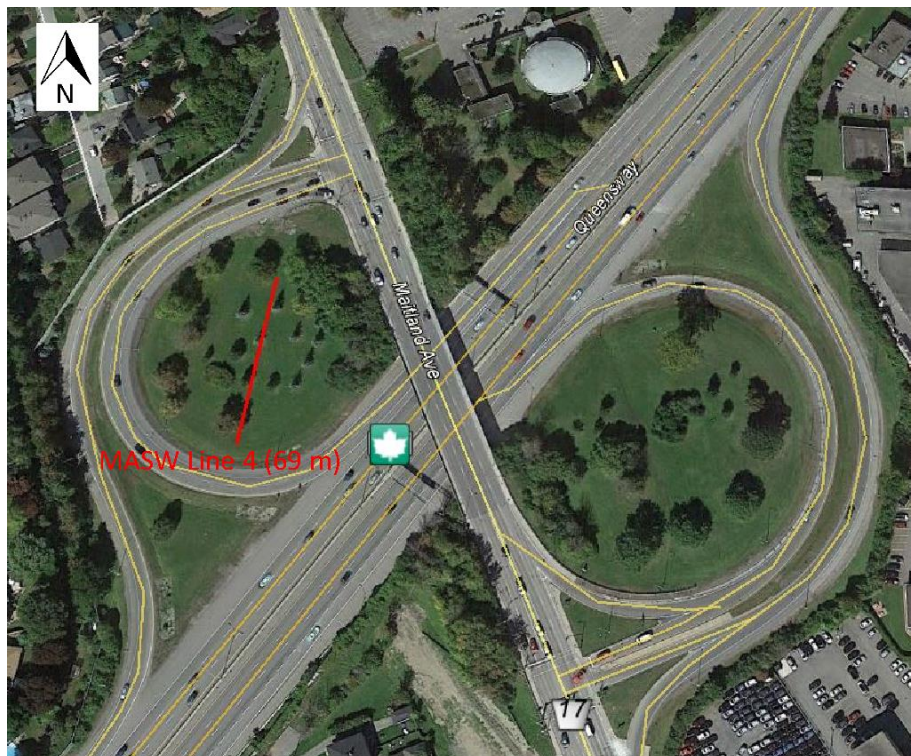


Figure 4: MASW Location Site Map at the Maitland site (MASW Line 4 in red)

## Methodology

The 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 May 26 and 27 by personnel from the Golder Mississauga and Ottawa offices. One MASW line was collected at each of the four sites (Figures 1, 2, 3 and 4).

For each survey line a series of 24 low frequency (4.5 Hz) geophones were laid out at 3-metre intervals. Both active and passive readings were recorded along each MASW lines. For the active investigation, a seismic drop of 45 kg and a 9.9 kg sledge hammer were used as seismic sources. Active seismic records were collected with seismic sources located 5, 10, 15, and 20 metres from and collinear to the geophone array. An example of active seismic records collected for MASW Lines 1, 2 and 3 are shown in Figures 5, 6, 7 and 8, respectively below. MASW Line 4 located west of Maitland Avenue had a higher noise level due to large amount of road traffic.

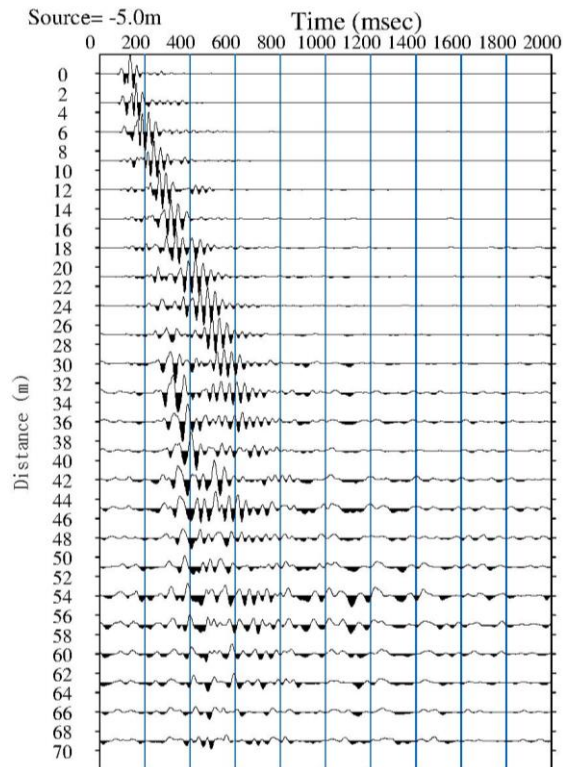


Figure 5: Typical seismic record collected along MASW Line 1

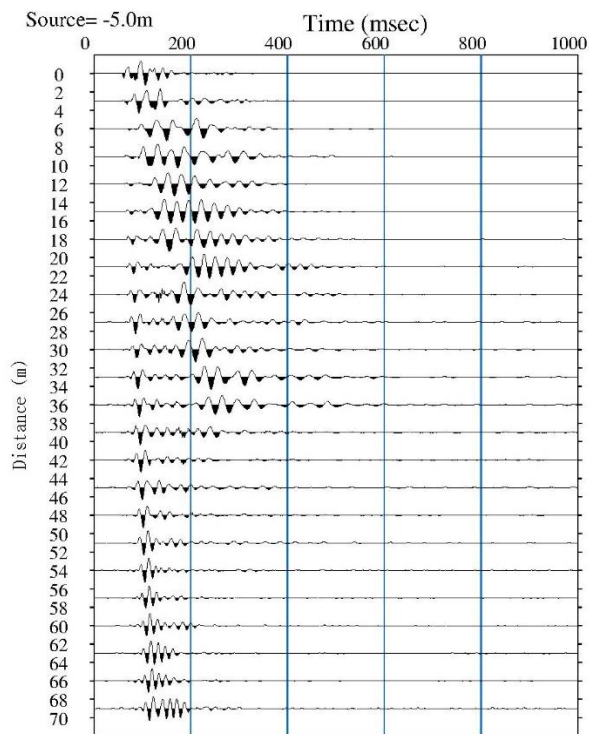


Figure 6: Typical seismic record collected along MASW Line 2



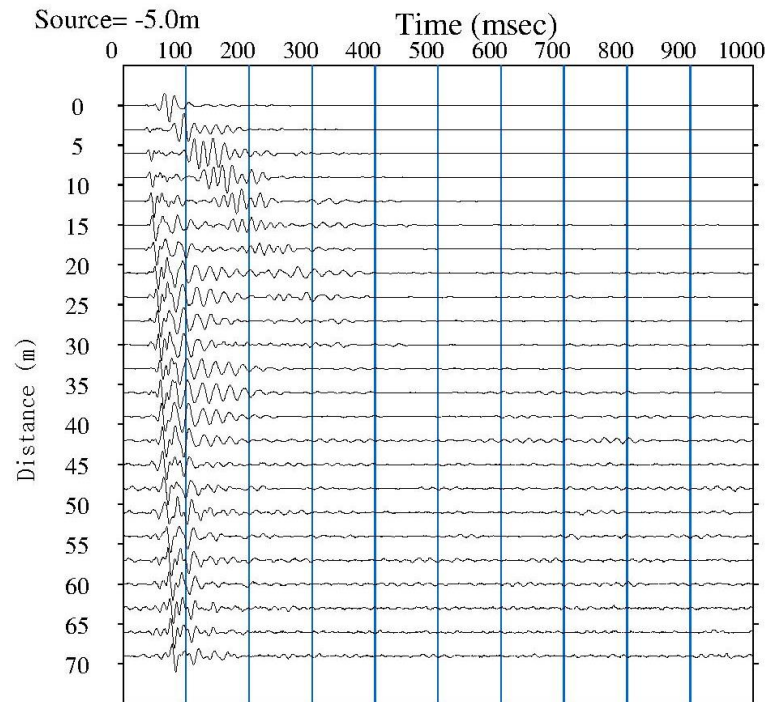


Figure 7: Typical seismic record collected along MASW Line 3

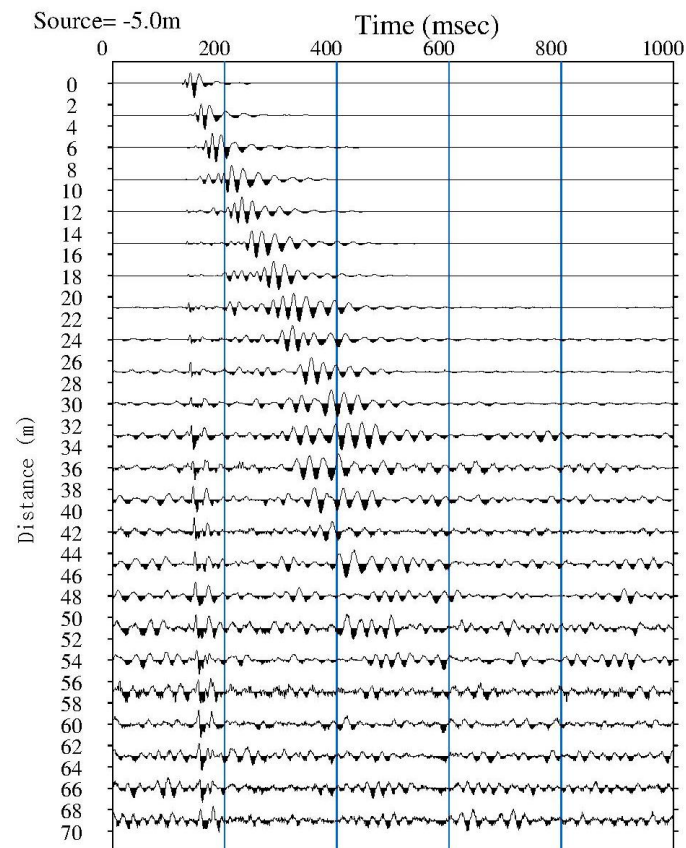


Figure 8: Typical seismic record collected along MASW Line 4

## 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 9, 10, 11 and 12. Shear-wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves.

Along MASW Line 1, the active survey provided a dispersion curve with a suitable frequency range (5 to 27 Hz), providing information for both shallow and deeper depths. The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 5 Hz.

Along MASW Line 2, the active survey provided a dispersion curve with a suitable frequency range (30-150 Hz), providing information for both shallow and deeper depths. The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 30 Hz.

Along MASW Line 3, the active survey provided a dispersion curve with a suitable frequency range (35-135 Hz), providing information for both shallow and deeper depths. The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 35 Hz.

Along MASW Line 4, the active survey provided a dispersion curve with a suitable frequency range (17-58 Hz), providing information for both shallow and deeper depths. The minimum measured surface-wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 17 Hz.

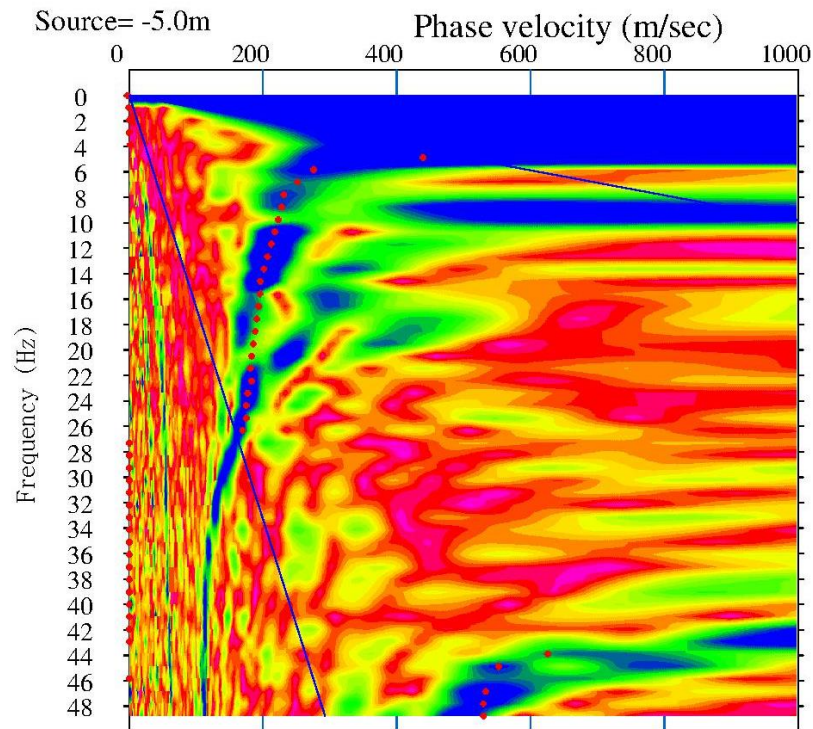


Figure 9: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1 (Richmond)

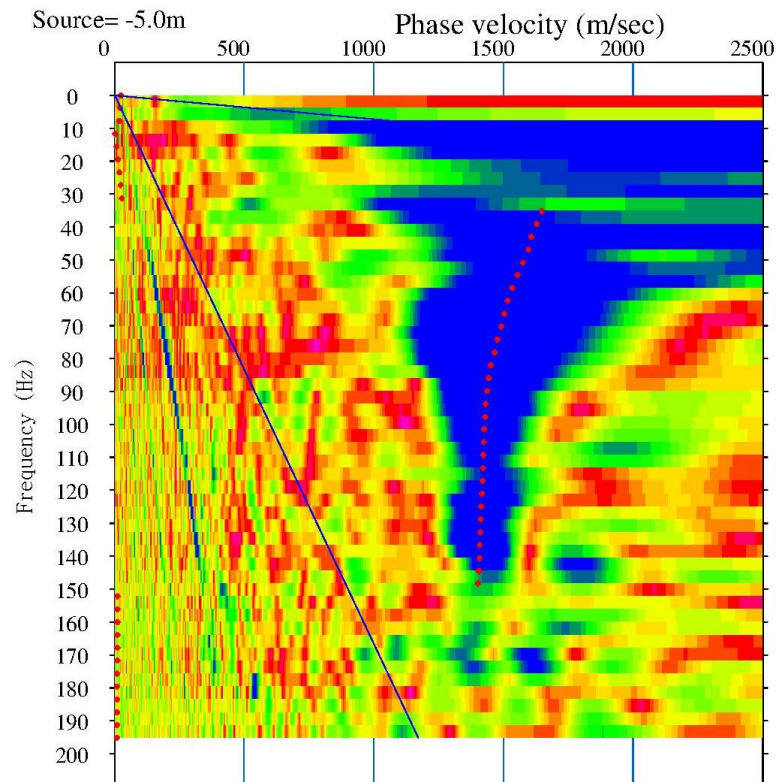


Figure 10: Active MASW Dispersion Curve Picks (red dots) along MASW Line 2 (Pinecrest).



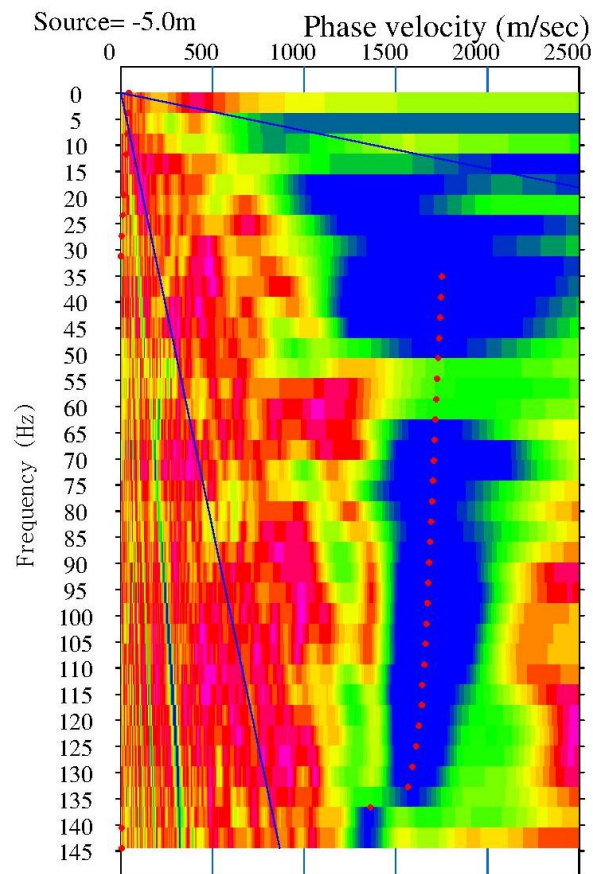


Figure 11: Active MASW Dispersion Curve Picks (red dots) along MASW Line 3 (Woodroffe).

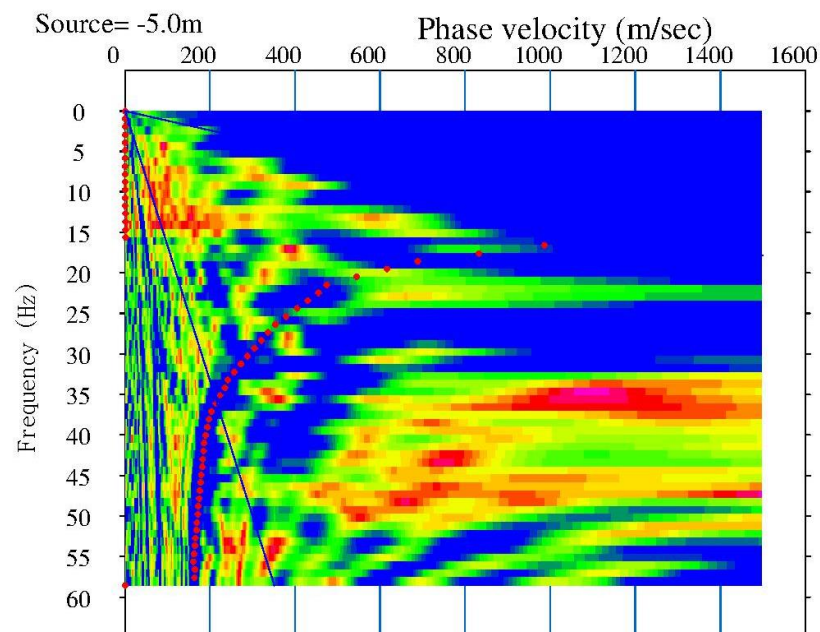


Figure 12: Active MASW Dispersion Curve Picks (red dots) along MASW Line 4 (Maitland).

## Results

The MASW test results are presented in Figure 13 (MASW Line 1 - Richmond), Figure 14 (MASW Line 2 - Pinecrest), Figure 15 (MASW Line 3 - Woodroffe) and Figure 16 (MASW Line 4 - Maitland), which present the calculated shear wave velocity profile derived from the field testing. The results along MASW Lines 1, 2, and 3 have been calculated using weight-drop located at 5 metres from the last geophone, respectively. The field collected dispersion curves are compared with the model generated dispersion curves on Figures 17, 18, 19 and 20. There is a satisfactory correlation between the field collected and model calculated dispersion curves, with a root mean squared error of less than 4% along the three MASW lines.

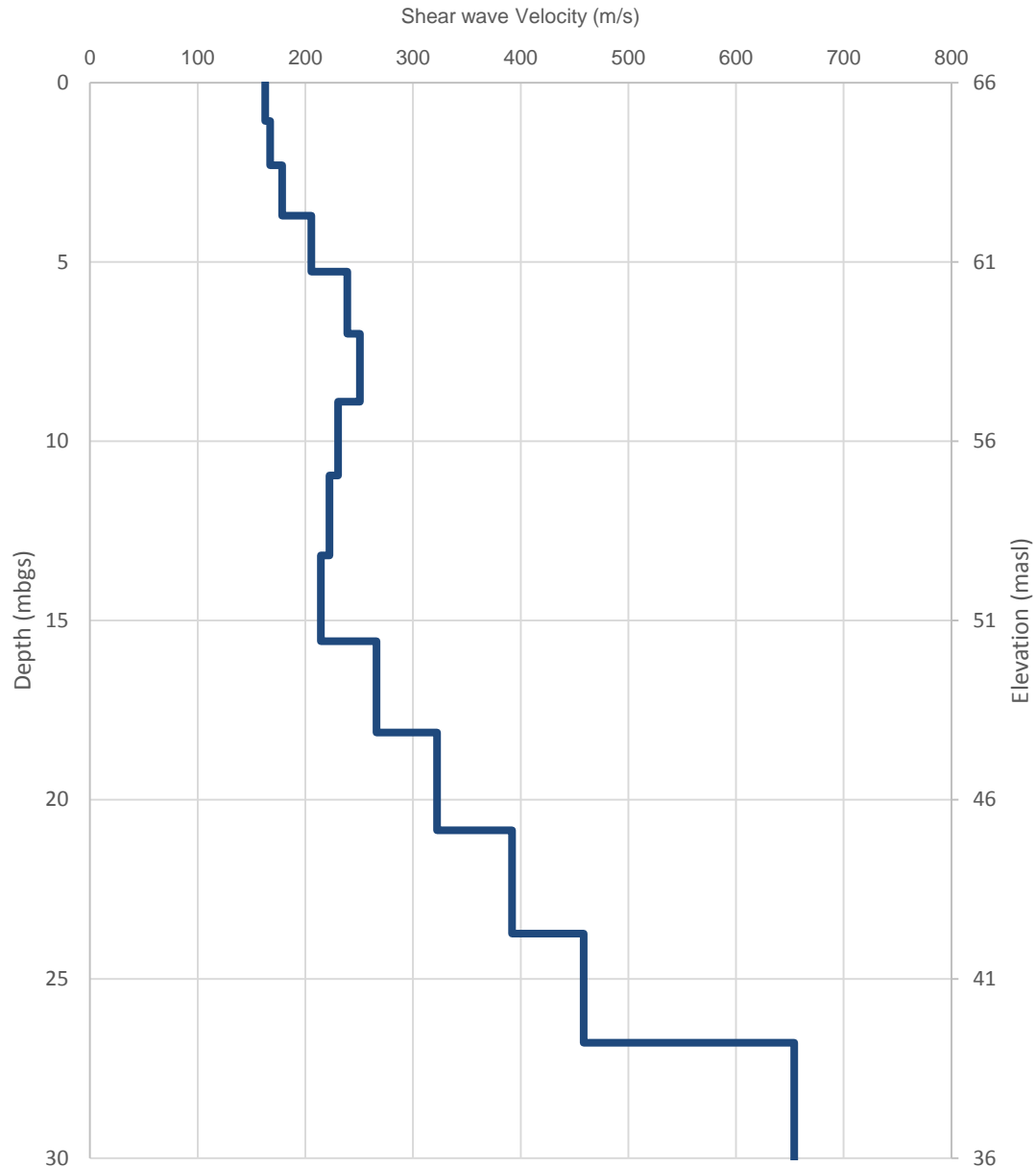


Figure 13: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

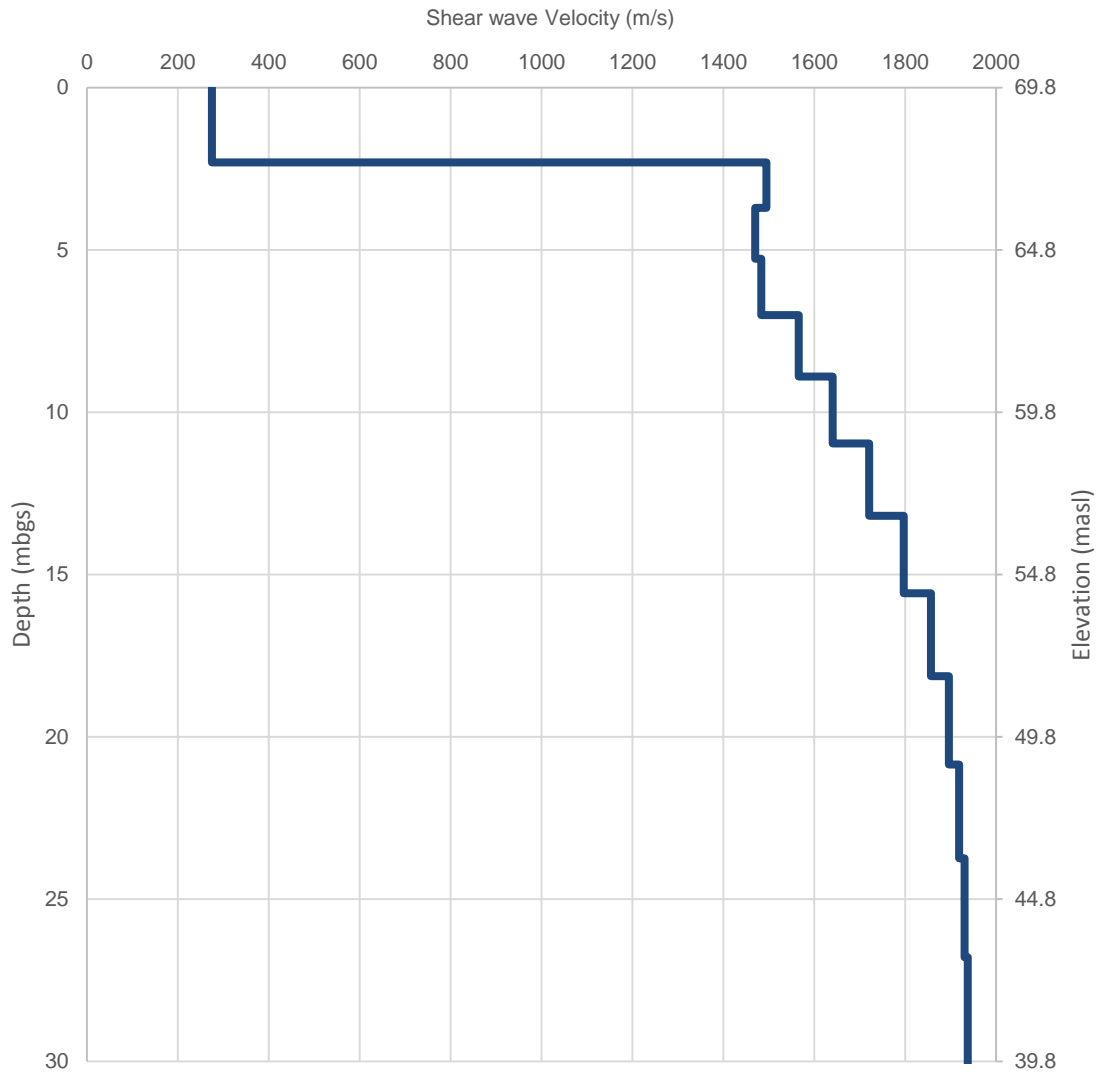


Figure 14: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 2

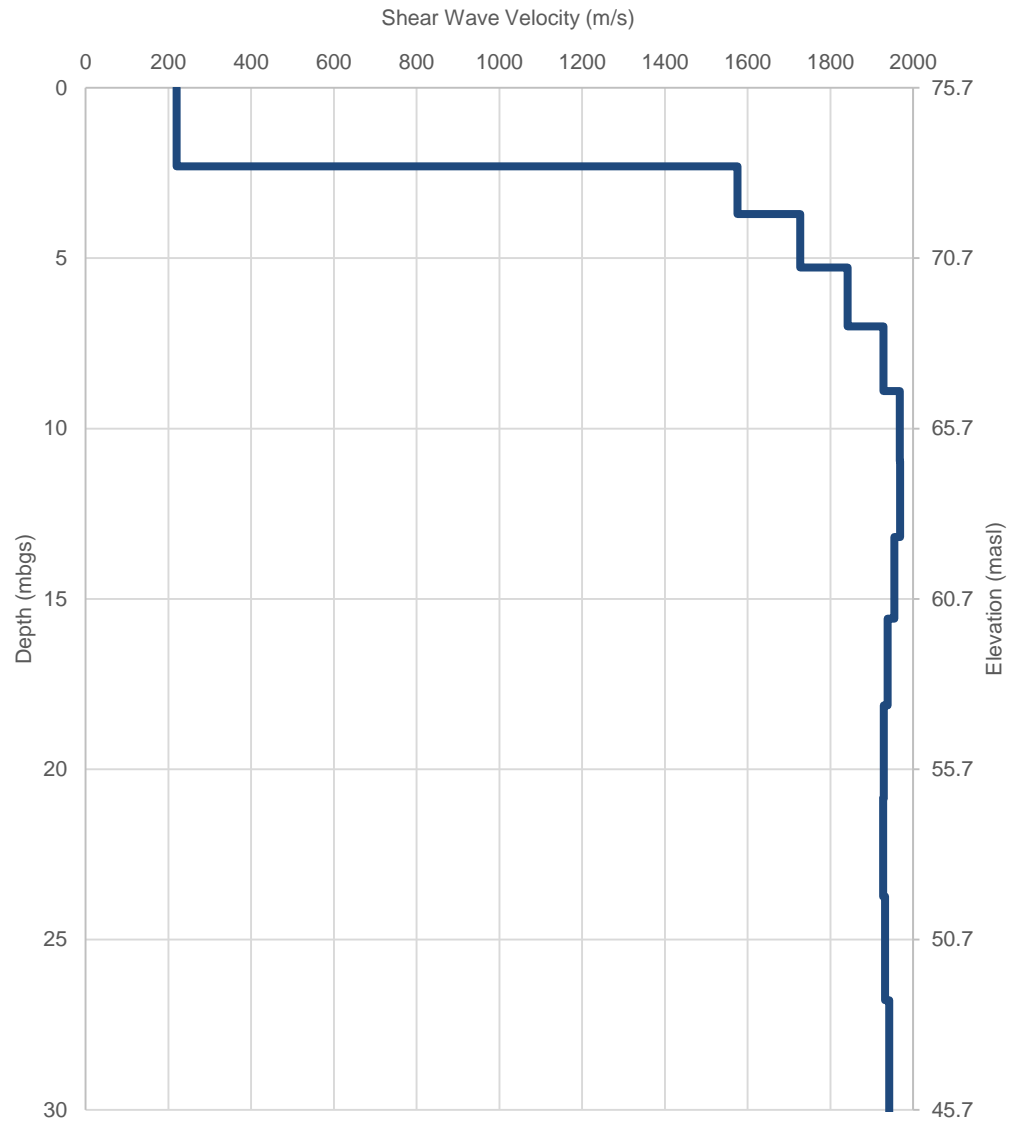


Figure 15: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 3

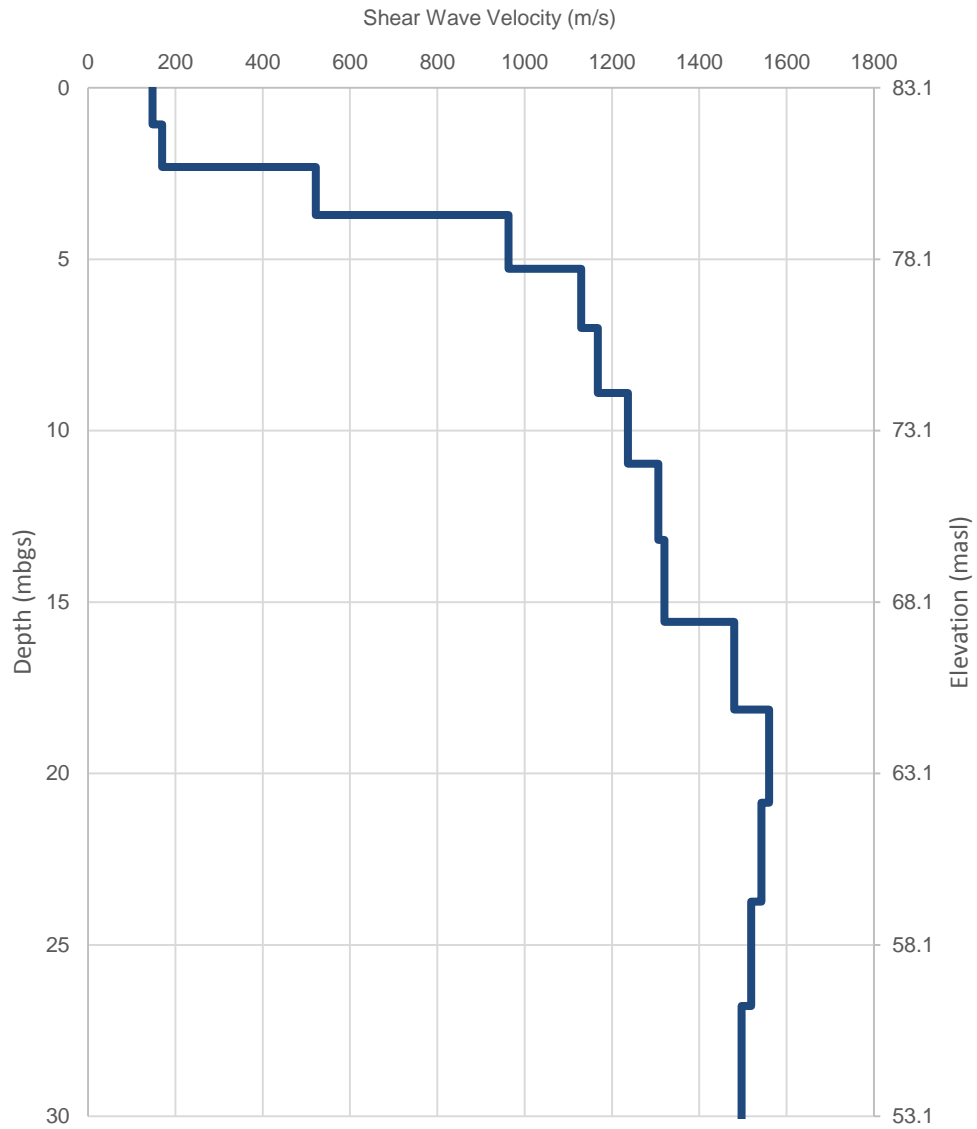


Figure 16: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 4

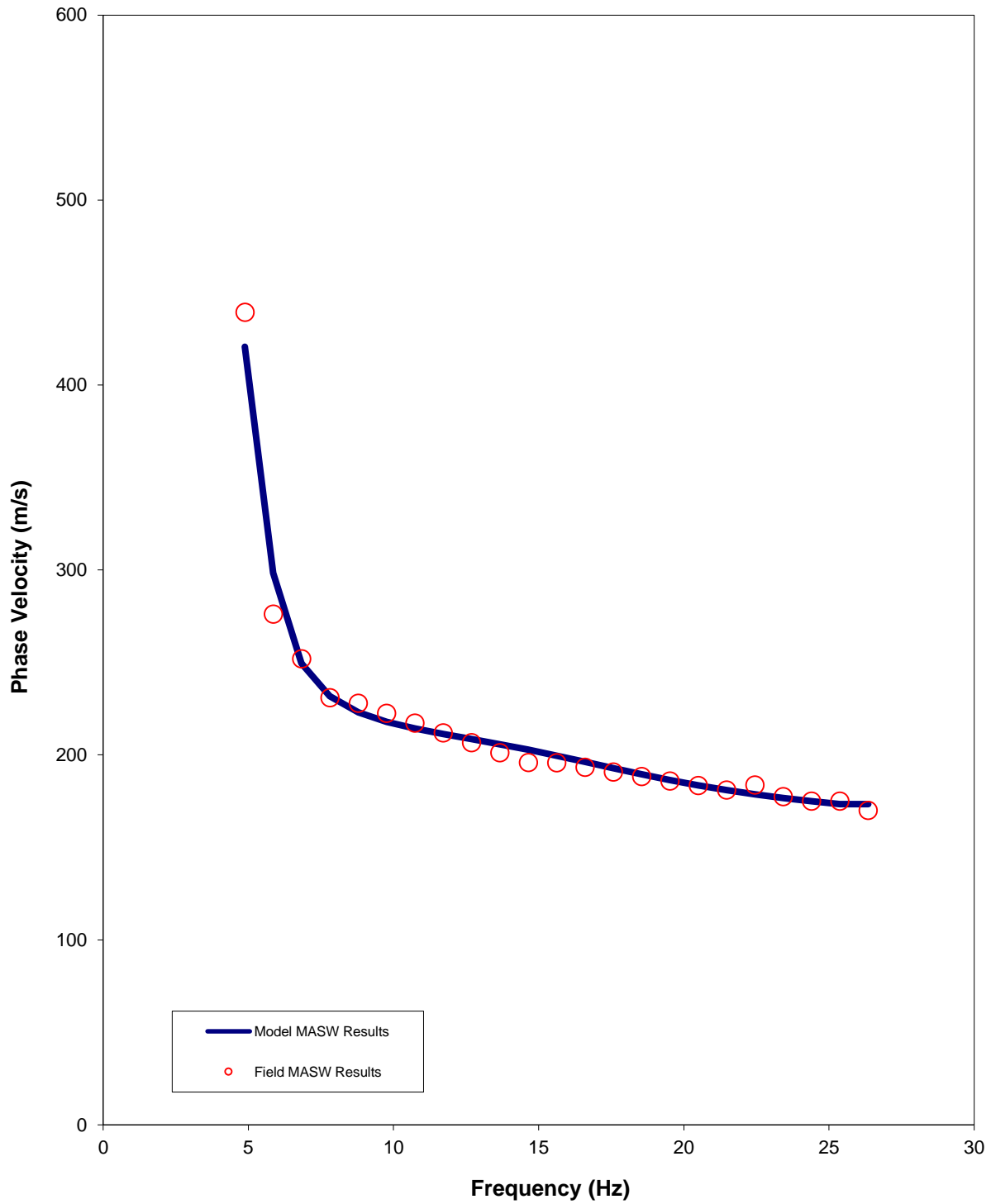


Figure 17: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

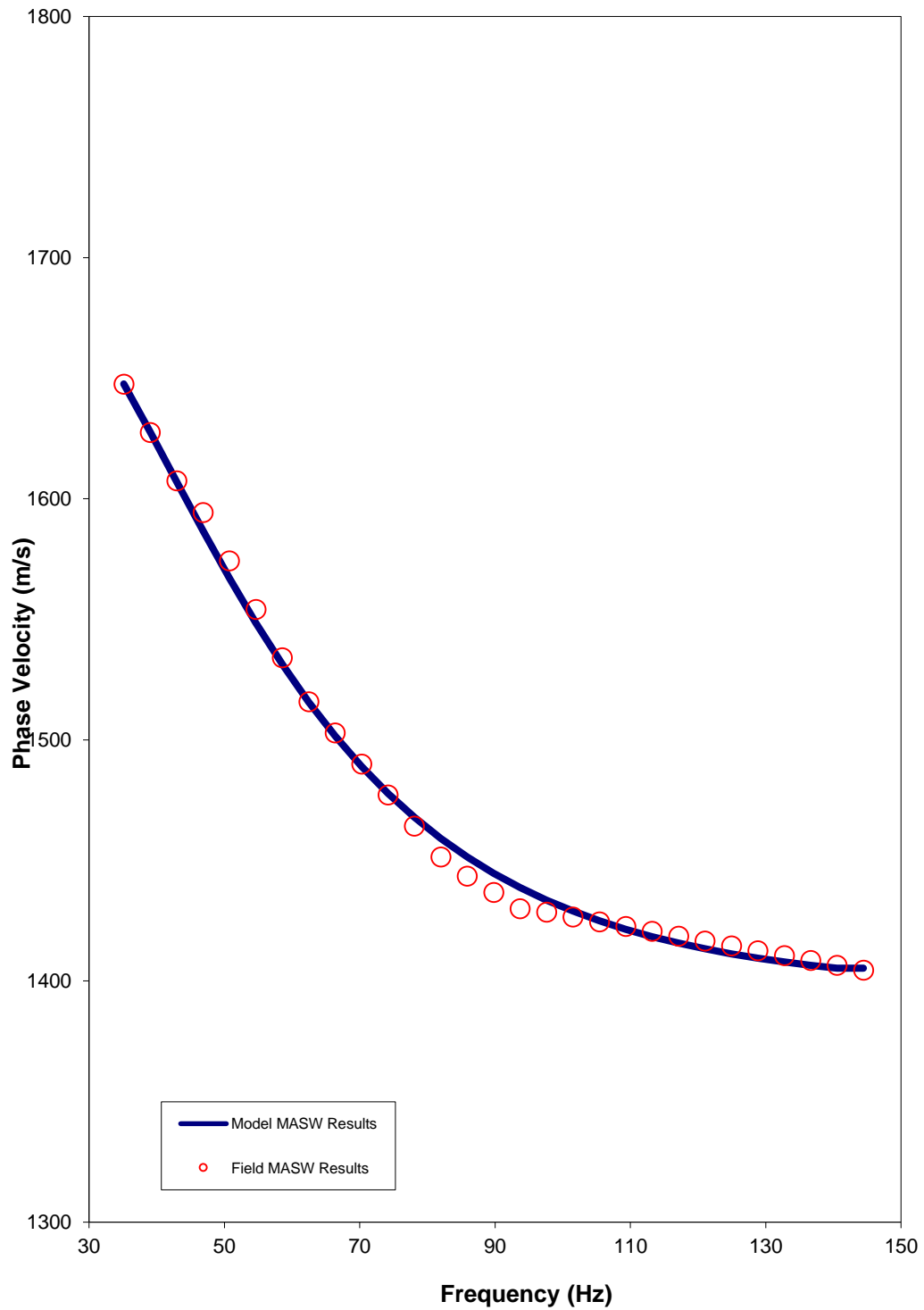


Figure 18: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 2



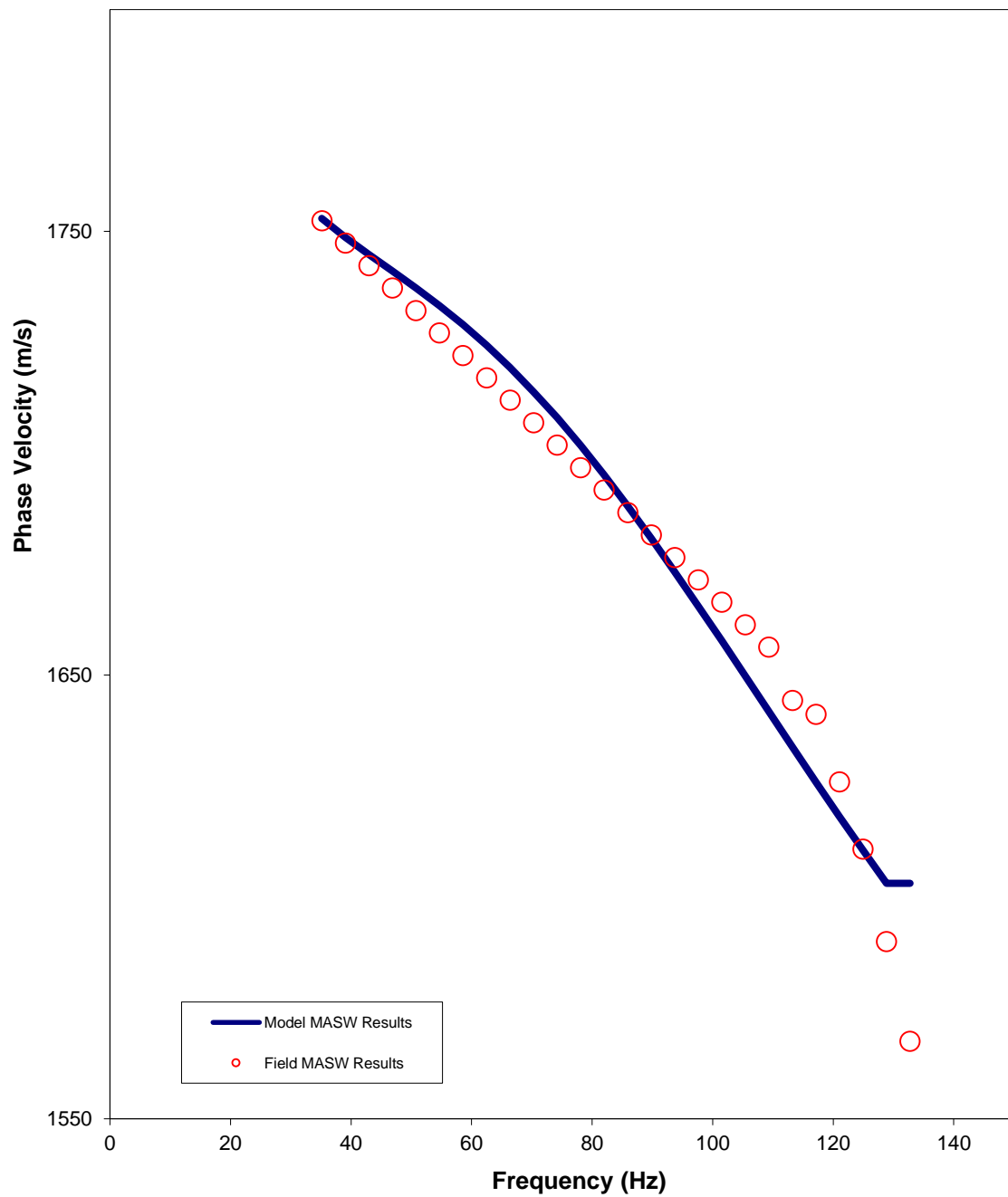


Figure 19: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

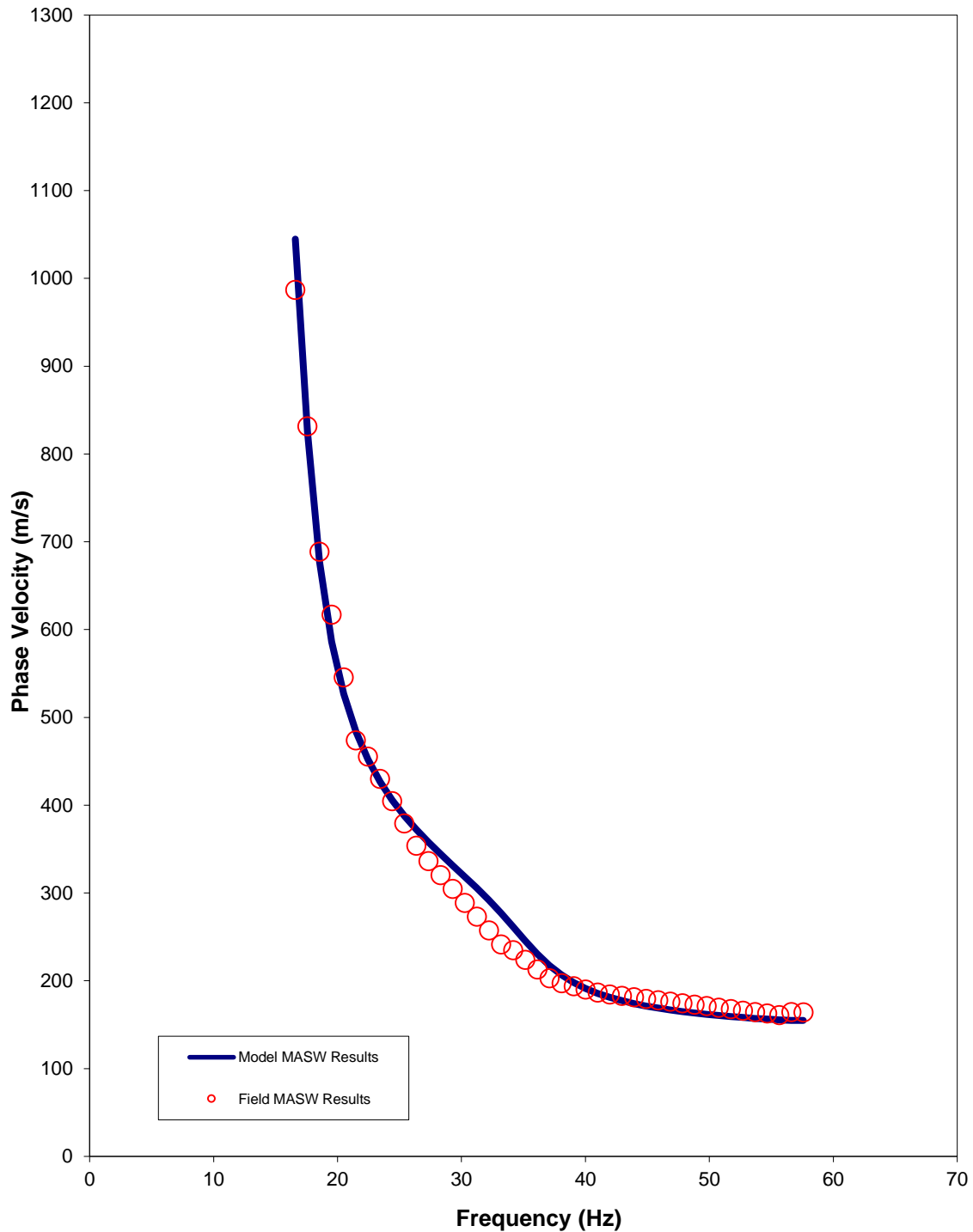


Figure 20: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

To calculate the average shear-wave velocity as required by the NBCC2010, the results were modelled to 30 metres below ground surface. The average shear-wave velocity along MASW Line 1 (Richmond) was found

to be 270 m/s (Table 1). The average shear-wave velocity along MASW Line 2 (Pinecrest) was found to be 1,239 m/s (Table 2). The average shear-wave velocity along MASW Line 3 (Woodroffe) was found to be 1,197 m/s (Table 3). The average shear-wave velocity along MASW Line 4 (Maitland) was found to be 818 m/s (Table 4).

The NBCC2010 requires special site specific evaluation if certain soil types are encountered on the site, so the site classification stated here should be reviewed, and modified if necessary, according to borehole stratigraphy, standard penetration resistance results, and undrained shear strength measurements, if available for this site.

**Table 1: Shear-Wave Velocity Profile along MASW Line 1 (Richmond)**

Model Layer Depth (mbgs)		Model Layer Elevation (masl)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom	Top	Bottom			
0.00	1.07	66.00	64.93	1.07	163	0.006580
1.07	2.31	64.93	63.69	1.24	167	0.007384
2.31	3.71	63.69	62.29	1.40	178	0.007852
3.71	5.27	62.29	60.73	1.57	206	0.007613
5.27	7.01	60.73	58.99	1.73	239	0.007239
7.01	8.90	58.99	57.10	1.90	251	0.007562
8.90	10.96	57.10	55.04	2.06	230	0.008945
10.96	13.19	55.04	52.81	2.23	223	0.010001
13.19	15.58	52.81	50.42	2.39	214	0.011154
15.58	18.13	50.42	47.87	2.55	266	0.009603
18.13	20.85	47.87	45.15	2.72	322	0.008436
20.85	23.74	45.15	42.26	2.88	392	0.007361
23.74	26.79	42.26	39.21	3.05	458	0.006652
26.79	30.00	39.21	36.00	3.21	654	0.004914
<b>Vs Average to 30 mbgs (m/s)</b>						<b>270</b>

**Table 2: Shear-Wave Velocity Profile along MASW Line 2 (Pinecrest):**

Model Layer Depth (mbgs)		Model Layer Elevation (masl)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom	Top	Bottom			
0.00	1.07	69.80	68.73	1.07	275	0.003896
1.07	2.31	68.73	67.49	1.24	275	0.004496
2.31	3.71	67.49	66.09	1.40	1495	0.000937
3.71	5.27	66.09	64.53	1.57	1470	0.001065
5.27	7.01	64.53	62.79	1.73	1484	0.001167
7.01	8.90	62.79	60.90	1.90	1566	0.001211
8.90	10.96	60.90	58.84	2.06	1641	0.001256
10.96	13.19	58.84	56.61	2.23	1720	0.001293
13.19	15.58	56.61	54.22	2.39	1797	0.001330
15.58	18.13	54.22	51.67	2.55	1857	0.001376
18.13	20.85	51.67	48.95	2.72	1896	0.001434
20.85	23.74	48.95	46.06	2.88	1918	0.001504
23.74	26.79	46.06	43.01	3.05	1930	0.001580
26.79	30.00	43.01	39.80	3.21	1938	0.001659
Vs Average to 30 mbgs (m/s)						1239

**Table 3: Shear-Wave Velocity Profile along MASW Line 3 (Woodroffe)**

Model Layer Depth (mbgs)		Model Layer Elevation (masl)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom	Top	Bottom			
0.00	1.07	75.70	74.63	1.07	220	0.004870
1.07	2.31	74.63	73.39	1.24	220	0.005619
2.31	3.71	73.39	71.99	1.40	1575	0.000889
3.71	5.27	71.99	70.43	1.57	1727	0.000907
5.27	7.01	70.43	68.69	1.73	1841	0.000940
7.01	8.90	68.69	66.80	1.90	1929	0.000983
8.90	10.96	66.80	64.74	2.06	1968	0.001047
10.96	13.19	64.74	62.51	2.23	1969	0.001130
13.19	15.58	62.51	60.12	2.39	1955	0.001223
15.58	18.13	60.12	57.57	2.55	1939	0.001318
18.13	20.85	57.57	54.85	2.72	1929	0.001410
20.85	23.74	54.85	51.96	2.88	1928	0.001496

Model Layer Depth (mbgs)		Model Layer Elevation (masl)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom	Top	Bottom			
23.74	26.79	51.96	48.91	3.05	1932	0.001578
26.79	30.00	48.91	45.70	3.21	1942	0.001655
Vs Average to 30 mbgs (m/s)						1197

**Table 4: Shear-Wave Velocity Profile along MASW Line 4 (Maitland)**

Model Layer Depth (mbgs)		Model Layer Elevation (masl)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom	Top	Bottom			
0.00	1.07	83.10	82.03	1.07	148	0.007243
1.07	2.31	82.03	80.79	1.24	169	0.007299
2.31	3.71	80.79	79.39	1.40	521	0.002687
3.71	5.27	79.39	77.83	1.57	963	0.001626
5.27	7.01	77.83	76.09	1.73	1129	0.001533
7.01	8.90	76.09	74.20	1.90	1167	0.001624
8.90	10.96	74.20	72.14	2.06	1236	0.001667
10.96	13.19	72.14	69.91	2.23	1307	0.001703
13.19	15.58	69.91	67.52	2.39	1320	0.001810
15.58	18.13	67.52	64.97	2.55	1480	0.001726
18.13	20.85	64.97	62.25	2.72	1560	0.001744
20.85	23.74	62.25	59.36	2.88	1542	0.001870
23.74	26.79	59.36	56.31	3.05	1519	0.002008
26.79	30.00	56.31	53.10	3.21	1497	0.002148
Vs Average to 30 mbgs (m/s)						818

## Limitations

This technical memorandum is based on data and information collected by Golder Associates Ltd. and is based solely on the conditions of the properties at the time of the work, supplemented by historical information and data obtained by Golder Associates Ltd. as described in this memo.

Golder Associates Ltd. has relied in good faith on all information provided and does not accept responsibility for any deficiency, misstatements, or inaccuracies contained in the reports as a result of omissions, misinterpretation, or fraudulent acts of the persons contacted or errors or omissions in the reviewed documentation.

The services performed, as described in this memo, were conducted in a manner consistent with that level of care and skill normally exercised by other members of the engineering and science professions currently practicing under similar conditions, subject to the time limits and financial and physical constraints applicable to the services.

Any use which a third party makes of this memo, or any reliance on, or decisions to be made based on it, are the responsibilities of such third parties. Golder Associates Ltd. accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this memo.

The findings and conclusions of this memo are valid only as of the date of this memo. If new information is discovered in future work, including excavations, borings, or other studies, Golder Associates Ltd. should be requested to re-evaluate the conclusions of this memo, and to provide amendments as required.

## Closure

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

### GOLDER ASSOCIATES LTD.



Stephane Sol, Ph.D, P. Geo.  
Senior Geophysicist

SS/CRP/jl



Christopher Phillips, M. Sc., P. Geo.  
Senior Geophysicist, Principal

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# **APPENDIX E**

## **Non Standard Special Provisions**

**Vibration Monitoring**

**Driving Piles Adjacent to Existing Battered Piles**



## **VIBRATION MONITORING - Item No.**

---

Special Provision

---

### ***Scope***

This special provision describes requirements for vibration monitoring during pile installation works.

### ***Definitions***

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling and vibration monitoring or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and shall issue certificate(s) of conformance.

### ***Submission Requirements***

The Contractor shall submit details of the vibration monitoring plan to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

- Qualifications of vibrations monitoring specialist.
- Proposed instrumentation.
- Proposed location of instruments.
- Proposed frequency of readings.
- Proposed methods for adjusting piling methods if readings show vibrations exceeding tolerable levels.

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractor's Quality Verification Engineer.

### ***Monitoring***

The Contractor shall take readings during driving of each pile. The readings should be taken and recorded during the entire length of driving and during seating of the pile on the bedrock.

The pile(s) furthest from the monitored structure or utility should be driven first to assess the vibration level at the existing structures. If necessary, the contractor must alter the pile driving procedures for the remaining piles. The revised procedure shall be submitted to the Contract Administrator for approval prior to driving the remaining piles.

The measured vibrations shall not exceed 100 mm/s (peak particle velocity).

If it is not practical to drive the piles furthest from the existing structure first due to space constraints, the piles nearest the existing structure may be driven first but the measured vibrations in that case shall not exceed 50 mm/s.

The results shall be submitted to the Contract Administrator after each pile has been driven and prior to continuing with the subsequent piles. As a minimum, the pile number, location, set criteria and driving log must be submitted with vibration monitoring results.

If the vibration monitoring results are acceptable, the Contractor may continue with the next piles with readings taken during driving of each pile. The results of subsequent piles should be submitted to the Contract Administrator after each pile has been driven.

If the readings are not within the limits stated above, the Contractor must alter the driving procedures until the vibrations are within acceptable levels. The above process must be repeated for each pile.

### **Basis of Payment**

Payment at the lump sum contract price for this tender item shall be full compensation for all labour, equipment and materials for completion of the work.

END OF SECTION

## **Driving Piles Adjacent to Existing Battered Piles – Item No.**

### **Non-Standard Special Provision**

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#### **Scope of Work**

This Special Provision covers the requirements for driving piles within close proximity to existing battered piles (i.e., where the anticipated distance between the new pile tip at depth and the existing battered pile tip at depth is less than 20% of the existing pile length.)

#### **Definitions**

Quality Verification Engineer (QVE): An Engineer with a minimum of five (5) years experience in the field of installation of piling or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and shall issue certificate(s) of conformance.

#### **Construction**

Work under this item shall adhere to the following requirements:

- For new piles driven within the potential zone of interference with the existing abutment or wing wall piles (defined as a distance around the existing pile tip at depth equal to 20% of the pile length) the driving operations shall be continuously monitored by the QVE.
- The contractor shall cease driving of the pile if the QVE indicates that the driven pile may have come in contact with an existing pile.
- If contact between the new and existing piles is believed to exist, the contractor shall take remedial action as directed by the Contract Administrator, which may include extracting the pile and re-driving or replacing the pile.

#### **Basis of Payment**

Payment at the contract price for the above noted Tender Item includes full compensation for all labour, equipment and materials to do the required work.



# **APPENDIX F**

## **Results of Slope Stability Analysis**

**Figure F1 – Embankment Slope Stability Analysis Results**

**Figure F2 – Retaining Wall Stability Analysis Results**

Title: 1546542-1040 Maitland Avenue Overpass  
 Name: Static Undrained  
 Date: 3/2/2017  
 Method: Morgenstern-Price  
 Direction of movement: Left to Right  
 Horz Seismic Load: 0

Name: New Fill  
 Model: Mohr-Coulomb  
 Unit Weight: 21 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 32 °

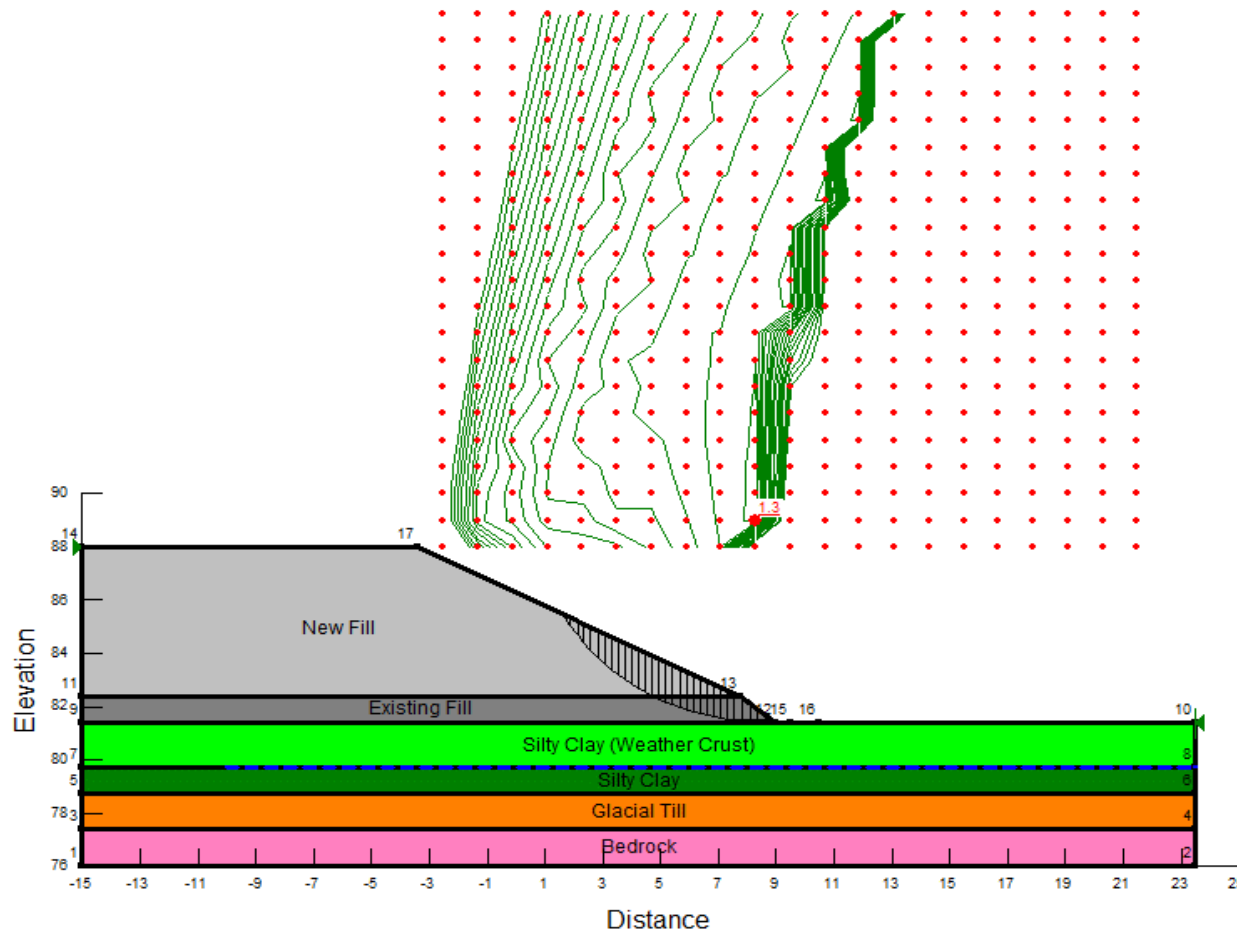
Name: Existing Fill  
 Model: Mohr-Coulomb  
 Unit Weight: 19 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 28 °

Name: Silty Clay (Weather Crust)  
 Model: Undrained (Phi=0)  
 Unit Weight: 17.5 kN/m<sup>3</sup>  
 Cohesion: 80 kPa

Name: Silty Clay  
 Model: Undrained (Phi=0)  
 Unit Weight: 16 kN/m<sup>3</sup>  
 Cohesion: 65 kPa

Name: Glacial Till  
 Model: Mohr-Coulomb  
 Unit Weight: 21 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Phi: 32 °

Name: Bedrock  
 Model: Bedrock (Impenetrable)

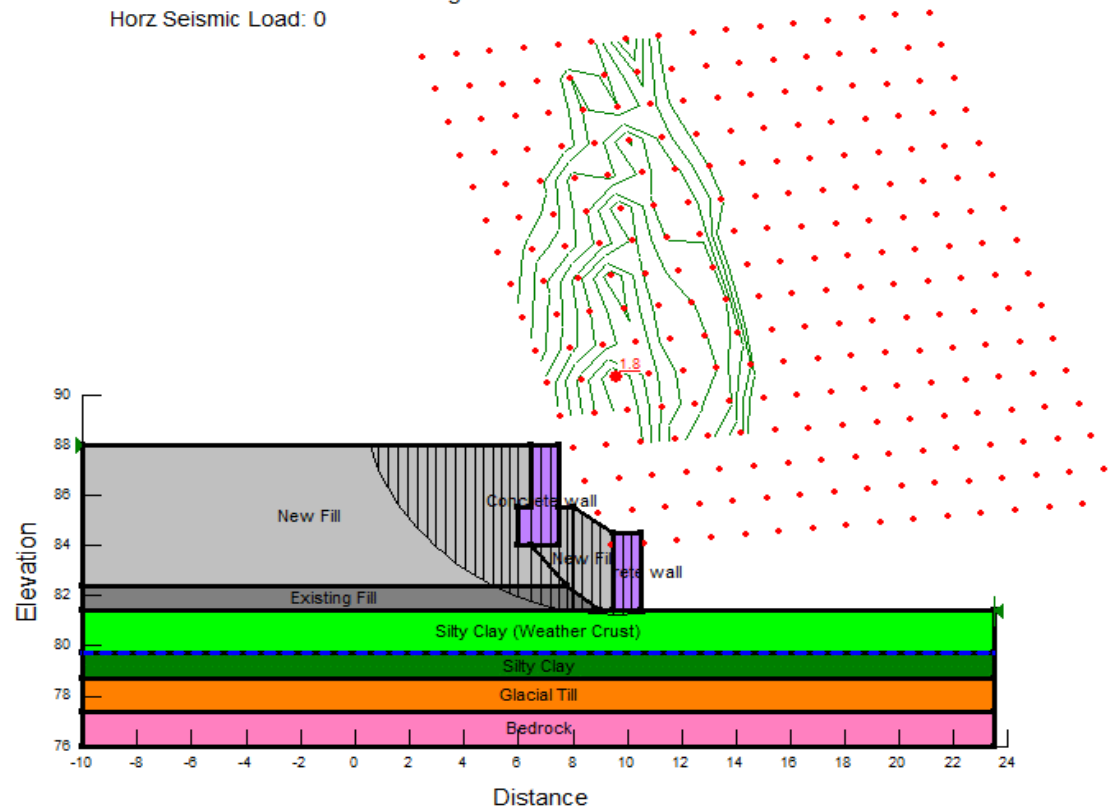


**Maitland Avenue Overpass**  
**Embankment Slope Stability Analysis Results**  
**Ottawa, Ontario**

Project No.	1546542-1040
Drawn:	WAM
Date:	3/3/2017
Checked:	
Review:	

**Figure F1**

Title: 1546542-1040 Maitland Avenue Overpass  
 Name: Static Undrained  
 Date: 3/2/2017  
 Method: Morgenstern-Price  
 Direction of movement: Left to Right  
 Horz Seismic Load: 0



Name: New Fill  
 Model: Mohr-Coulomb  
 Unit Weight: 21 kN/m³  
 Cohesion: 0 kPa  
 Phi: 32 °  
 Piezometric Line: 1

Name: Existing Fill  
 Model: Mohr-Coulomb  
 Unit Weight: 19 kN/m³  
 Cohesion: 0 kPa  
 Phi: 28 °  
 Piezometric Line: 1

Name: Silty Clay (Weather Crust)  
 Model: Undrained (Phi=0)  
 Unit Weight: 17.5 kN/m³  
 Cohesion: 80 kPa  
 Piezometric Line: 1

Name: Silty Clay  
 Model: Undrained (Phi=0)  
 Unit Weight: 16 kN/m³  
 Cohesion: 65 kPa  
 Piezometric Line: 1

Name: Glacial Till  
 Model: Mohr-Coulomb  
 Unit Weight: 21 kN/m³  
 Cohesion: 0 kPa  
 Phi: 32 °  
 Piezometric Line: 1

Name: Bedrock  
 Model: Bedrock (Impenetrable)  
 Piezometric Line: 1

Name: Concrete wall  
 Model: Undrained (Phi=0)  
 Unit Weight: 23 kN/m³  
 Cohesion: 15000 kPa



**Maitland Avenue Overpass**  
**Retaining Wall Slope Stability Analysis Results**  
**Ottawa, Ontario**

Project No.	1546542-1040
Drawn:	WAM
Date:	3/3/2017
Checked:	
Review:	

**Figure F2**



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