

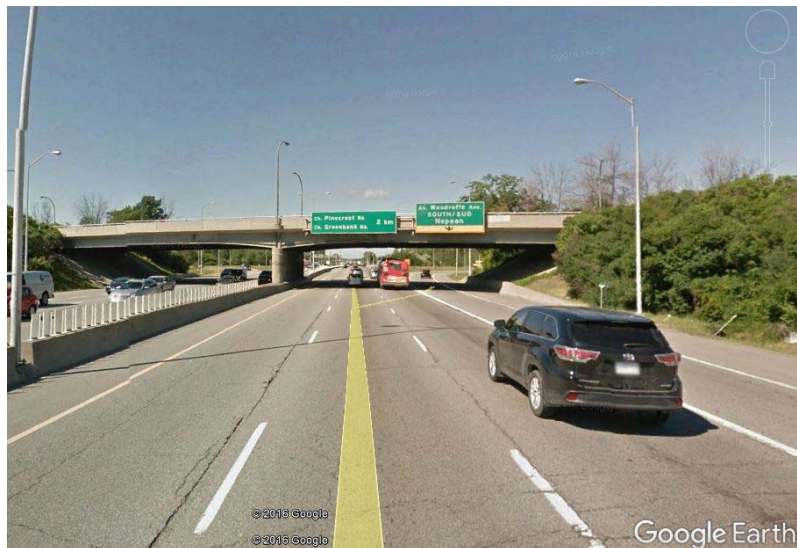


April 2017

REPORT ON

**FOUNDATION INVESTIGATION AND DESIGN
WOODROFFE AVENUE UNDERPASS
SITE NO. 3-041
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



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PART A

**FOUNDATION INVESTIGATION REPORT
WOODROFFE AVENUE UNDERPASS – SITE NO. 3-041
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 Woodroffe 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, and modifications to, the Woodroffe Avenue Underpass (MTO Structure Site No. 3-041) and the associated retaining walls and underpass approach embankments only. The proposed widening of the existing bridge to the east is required to accommodate a new northbound left turn lane on Woodroffe Avenue. Modifications to the foreslope retaining walls at the north and south abutments are required to accommodate the highway widening. The foreslope paving and approach fills in front of both the north and south bridge abutments will be cut back to form a “truncated toe” with a retaining wall.

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 Woodroffe 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 Woodroffe Avenue Underpass is located within a residential area of the City of Ottawa, and is located approximately 4.5 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 is an off-ramp lane with a wide shoulder. In the eastbound direction, there is an on-ramp lane with a wide shoulder.

The existing Woodroffe Avenue Underpass is a two span continuous steel girder bridge. The spans are each approximately 32 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 7 m high relative to the highway profile. The foreslopes of both the north and south abutments are constructed at about 2 Horizontal to 1 Vertical (2H:1V) grade extending down to short toe walls adjacent to the roadway shoulders.

A 200 mm diameter gas main is located about 24 m east of the bridge structure, aligned in a north-south direction.

A previous investigation was 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 Woodroffe Avenue, Bridge No. 2, to Deleuw, Cather and Company of Canada Limited” (Geocres 31G5-021). In late 1988 and early 1989, design memorandums were prepared by McCormick Rankin addressing new retaining walls to support the forward fills of the Woodroffe Avenue Underpass as part of W.P. 118-86-01. The sequence of letters and memos between McCormick Rankin and MTO are included in Geocres 31G5-189.

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, 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 bedrock². 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.

¹ 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.

² 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 15 and July 28, 2016. During that time, a total of 8 boreholes were advanced at the locations shown on Drawing 1. Borehole 16-301 was advanced adjacent to the east pier location, and Boreholes 16-302 and 16-303 were advanced adjacent to the abutments at the east toe of each of the existing embankments. Boreholes 16-304 and 16-305 were advanced through each of the existing approach embankments, and Boreholes 16-310 to 16-312 were advanced within the currently proposed 'Construction Staging Area' within the northwest quadrant of the Woodroffe 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 2.1 m and 10.2 m below present ground surface. Boreholes 16-306 to 16-309 were not drilled as part of the design.

Samples of the overburden were obtained at 0.6 m to 0.8 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 NQ sized core. The bedrock was cored for depths of 3.6 to 5.3 metres, after practical refusal to augering had been reached. One monitoring well was installed (in Boreholes 16-310) to monitor the groundwater level at the site. The monitoring well consists of 30 mm outside diameter PVC pipe with a 1.5 m long slotted tip. The borehole was 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. Unconfined compressive strength testing was carried out on one sample of the bedrock core.

The groundwater level was measured in the monitoring well in Borehole 16-310 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's 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.

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 in Appendix A.



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Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
BH16-301	Pier	5024851.3	362135.6	75.9	7.3
BH16-302	South Embankment Toe	5024821.6	362167.2	76.8	10.2
BH16-303	North Embankment Toe	5024888.8	362143.7	75.7	6.4
BH16-304	South Embankment Crest	5024794.4	362148.3	81.2	8.2
BH16-305	North Embankment Crest	5024885.1	362113.0	81.3	8.2
BH16-310	Staging Area	5024902.8	362037.9	75.1	3.3
BH16-311	Staging Area	5024866.7	362074.2	76.6	3.1
BH16-312	Staging Area	5024850.9	362007.2	75.3	2.1



4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The Record of Borehole sheets and laboratory testing results 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 6 in Appendix B. The results of the in situ field tests (i.e. SPT 'N'-values) carried out during the previous investigation as presented on the Record of Borehole sheets.

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 Woodroffe Avenue Bridge. Six boreholes had been previously advanced at the present bridge abutment and pier locations on behalf of the Ministry in 1958, as previously noted (Geocres No. 31G5-021). The borehole records from this previous investigation are provided for reference in Appendix C. The borehole locations from the current and previous investigations are shown on Drawing 1. The interpreted stratigraphic profile projected along the Woodroffe Avenue centreline is also shown on Drawing 1. A stratigraphic profile projected through the proposed staging area is shown on Drawing 2.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profile and cross-sections 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/or glacial till overlying limestone bedrock with shale interbeds.

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 1958 were advanced prior to the highway construction and therefore the ground conditions shown on those logs are likely different than currently exist, particularly with respect to the composition and thickness of overburden and fill. The previous Geocres information is referenced only in regard to the bedrock surface elevation, which is in general agreement with the results from the current investigation.

4.2 Topsoil

Topsoil exists at ground surface at Boreholes 16-302, 16-303, and 16-310 to 16-312, with thicknesses of about 50 to 200 mm.

4.3 Fill

Asphaltic concrete exists at ground surface at Boreholes 16-301, 16-304 and 16-305, with a thickness of about 200 to 230 mm.

Fill was encountered beneath the topsoil/asphaltic concrete at all borehole locations. Fill materials are quite variable between locations, consisting of sand, gravel, silty sand, and silty clay.

The layer of fill was fully penetrated in all of the boreholes and varies from about 0.3 to 5.9 m in thickness, with the thicker portions located at the embankments. SPT 'N' values obtained within the fill generally range from about 8 to 29 blows per 0.3 m of penetration indicating a loose to compact state of packing. One higher blow count of 74 was encountered within the pavement structure fill at Borehole 16-301), reflecting a very dense state of packing.



Although not encountered in the boreholes, obstructions (e.g., cobbles and boulders) should be anticipated due to the variability of the fill materials.

Grain size distribution testing was carried out on three samples of the fill, the results of which are provided on Figure 1. The measured water content of five samples of the fill ranges from approximately 8 to 30 percent.

4.4 Silty Clay to Clayey Silt

The fill is underlain by a deposit of sensitive silty clay to clayey silt at Boreholes 16-302 to 16-305, and 16-310. The silty clay to clayey silt was fully penetrated by the all of these boreholes and varies from about 0.5 to 2.2 m in thickness.

The full thickness of the silty clay to clayey silt deposit at Boreholes 16-302, 16-304, 16-305 and the upper 1.7 m at Borehole 16-303 has been weathered to form a grey brown crust. The lowest 0.5 m of the deposit at Borehole 16-303 and the entire 0.5 m thickness encountered at Borehole 16-310 is unweathered and grey. Standard penetration tests carried out within the weathered silty clay to clayey silt gave 'N' values ranging from 2 to 26 blows per 0.3 m of penetration, indicating a generally stiff to very stiff consistency. The results of in situ vane testing at one location in the unweathered portion of the deposit gave an undrained shear strength of 46 kPa, indicating a firm consistency. The calculated sensitivity ratio based on a remoulded shear strength of 8 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 two samples of the weathered silty clay to clayey silt are summarized on Figure 2 and indicate plasticity index values generally ranging from 18 to 37 percent and liquid limit values of 34 and 59 percent, reflecting a soil of low to high plasticity. The measured water content of four samples of this deposit ranges from approximately 30 to 51 percent. Grain size distribution testing was carried out on two samples of this deposit, the results of which are provided on Figure 3.

4.5 Till

A deposit of glacial till was encountered below the fill and silty clay to clayey silt at all the borehole locations, at depths of about 0.6 to 7.6 m below the existing ground surface (or between Elevations 73 and 76 m). The till in this area generally consists a heterogeneous mixture of gravel, cobbles, and boulders in a matrix of sandy silt with trace to some clay.

The till was fully penetrated at Boreholes 16-301 to 16-303 and is about 0.3 to 3.0 m in thickness, extending to about 2.9 to 4.4 m depth below the existing ground surface (i.e., Elevations 72.8 to 72.2 m). The till was not fully penetrated in the other boreholes but was proven to extend to depths of about 2.1 to 8.2 m below the ground surface (i.e., Elevations 73.5 to 71.9 m). Standard penetration test 'N' values of 2 to 'in excess of 50' blows (at Borehole 16-310) per 0.3 m of penetration, but typically less than 15, were measured in the glacial till, indicating a very loose to compact state of packing. The higher 'N' values could reflect the presence of cobbles and boulders or the bedrock surface, rather than the state of packing of the soil matrix.

The measured water contents of ten samples of till ranged from approximately 2 to 33 percent. Grain size distribution testing was carried out on five samples of the till, the results of which are provided on Figure 4. The results confirm that the till matrix generally consists of a silty sand to sandy silt with some clay and trace amounts of gravel. The sample tested from Borehole 16-312 had a higher clay and silt content than is typical for glacial till in the Ottawa area. These samples were retrieved using a 50 mm diameter sampler and the results do not reflect the larger gravel, cobble and boulder content of the deposit.



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4.6 Sand and Gravel

A 0.5 m thick layer of sand and gravel was encountered at the bedrock/till interface at Borehole 16-302. One standard penetration test within the sand and gravel deposit gave an SPT 'N' value in excess of 50 blows per 0.3 m of penetration. This high blow count may indicate a very dense state of packing of the sand and gravel deposit, or reflect the bedrock surface.

The measured water content of one sample of the sand and gravel was about 14 percent. Grain size distribution testing was carried out on one sample of this deposit, the result of which are provided on Figure 5.

4.7 Bedrock

The bedrock encountered at the bridge foundation elements at Boreholes 16-301 to 16-303, inclusive, consists of limestone with thin shale interbeds, which is consistent with the previous Geocres investigation. Photos of the bedrock core obtained during the current investigation are provided in Appendix A on Figures A1 to A6, inclusive.

The bedrock is slightly weathered to fresh and medium bedded, and was encountered at depths ranging from about 2.9 to 4.9 m below the existing ground surface (i.e., Elevations 72.8 to 71.8 m) during the current investigation. These boreholes were advanced about 3.6 to 5.3 m into the bedrock.

The bedrock was cored in Boreholes 16-301, 16-302 and 16-303 as part of the current investigation, and in Boreholes 1 through 6 as part of the previous investigation in 1958. The following table summarizes the bedrock surface depths and elevations as encountered at the boreholes advanced as part of the current and previous investigations at the site.

Borehole Location	Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
South Approach Embankment	16-302	76.8	4.9	71.9
	2	75.7	3.7	72.0
	16-304	81.2	> 8.2 ¹	< 73.0 ¹
	4	75.6	5.1	70.5
Pier	16-301	75.9	3.7	72.2
	6	75.5	3.4	72.1
	5	75.6	3.2	72.4
North Approach Embankment	16-303	75.7	2.9	72.8
	1	74.9	2.7	72.1
	16-305	81.3	> 8.2 ¹	< 73.1 ¹
	3	75.3	3.3	72.0
Staging Area (NW)	16-310	75.1	3.3 ²	71.9 ²
	16-311	76.6	3.1 ²	73.5 ²
	16-312	75.3	2.1 ²	73.2 ²

Notes 1: Bedrock was not encountered within the depth of the borehole.

2: Auger refusal on possible bedrock surface.



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The limestone bedrock at the site is mapped as the Rockcliffe Formation. Thin shale interbeds were also present in the rock core. Rock Quality Designation (RQD) values measured on recovered bedrock core samples ranged from about 17 to 100 percent, but more typically above 75 percent. The RQD values for Borehole 16-301, which was advanced at the pier location, were 89 to 100 percent, indicating good to excellent quality rock. The lowest RQD values were recorded as 17 percent for the run of bedrock from 7.2 to 8.0 m depth in Borehole 16-302 (south abutment). It is however noted that the fracturing observed at this depth could be a result of poor drilling techniques/equipment (i.e., mechanical break) rather than the quality of the bedrock.

The result of one unconfined compressive strength test on a sample of the bedrock from Borehole 16-301 was 55 MPa, as shown on Figure 6, indicating a strong rock. 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.

4.8 Groundwater Conditions

A monitoring well was installed in Borehole 16-310. The water levels measured in the well are summarized in the following table:

Borehole Number	Borehole Location	Screened Interval	Date	Depth (m)	Elevation (m)
16-310	Staging Area	Silty Clay/Glacial Till	August 2, 2016	2.4	72.7
			September 30, 2016	1.7	73.4

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.

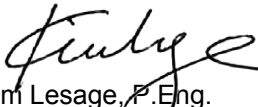


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5.0 CLOSURE


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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PART B

**FOUNDATION DESIGN REPORT
WOODROFFE AVENUE UNDERPASS – SITE NO. 3-041
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 Woodroffe Avenue Underpass structure (MTO Structure Site No. 3-041) 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 Woodroffe Avenue Underpass in Ottawa. The proposed widening of the existing bridge to the east by about 4.9 m is required to accommodate a new northbound left turn lane on Woodroffe Avenue, and foreslope retaining walls to protect the north and south abutments and to accommodate the highway widening.

Based on the General Arrangement (GA) Drawing provided by MMM, the grade of Highway 417 beneath the Woodroffe Avenue Underpass is between about Elevation 75 to 76 m. In comparison, the grade of Woodroffe Avenue where it crosses Highway 417 is at about Elevation 82 m.

The existing Woodroffe Avenue Underpass is a two span continuous steel girder bridge with composite concrete deck. The spans are each approximately 32 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 7 m high relative to the highway profile.

It is understood that widening of the existing Woodroffe 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 the Woodroffe Avenue Underpass will need to accommodate the addition of new travel lanes and that consideration is being given to constructing foreslope retaining walls to accommodate the Highway 417 widening at this location.

This report addresses the proposed widening of the Woodroffe Avenue Underpass (MTO Structure Site No. 3-041) and the associated foreslope 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 in the RFP from 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 1,197 m/s. Despite relatively high average shear wave velocities in the overburden and bedrock, the underside of the footings for the existing 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 Woodroffe Road 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 45.36 and longitude – 75.77), the following are the reference Site Class C (reference) peak seismic hazard values (based on the 5th 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)	0.096	0.155	0.270
PGV (m/s)	0.065	0.107	0.189
Sa (0.2) (g)	0.153	0.243	0.422
Sa (0.5) (g)	0.085	0.133	0.228
Sa (1.0) (g)	0.043	0.067	0.114
Sa (2.0) (g)	0.020	0.032	0.054
Sa (10.0) (g)	0.0019	0.0031	0.0053



6.3 Bridge Foundations - General

6.3.1 Available Foundation Options

To accommodate projected increased traffic demands, a new northbound left hand turn lane on Woodroffe Avenue to westbound Highway 417 is planned, which requires the widening of the existing Woodroffe Avenue Underpass Structure to the east by about 4.9 m. Only minimal changes to the grades of Highway 417 and the Woodroffe Avenue approach embankments 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.

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 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 for the abutments due to the 7 to 9 m deep excavations that would be required.

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.

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 (founded on 2.7 by 3.6 m rectangular footings). Roadway protection would be required for the construction of spread footings for the central pier due to the limited space available in the median (i.e., about 5 to 6 m between the eastbound and westbound driving lanes). Based on discussions with a local contractor, it is anticipated that roadway protection could take up to about 1.5 m of space on either side of the median (i.e., 3 m in total) and only leave about 2 to 3 m of space for footing construction. It is therefore anticipated that lane closures may be required (in addition to roadway protection) for the removal of the old footings and for construction of the new footings.



The installation of caissons founded on or socketed into the bedrock would not be preferred for the replacement of the existing pier foundations due to vertical clearance restrictions imposed by the existing bridge structure. Locally available low headroom equipment for caisson installation would limit the caisson diameter to no more than 0.9 m. Piles could be driven in short sections underneath the existing bridge structure.

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. An 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, Ψ , and geotechnical resistance factors, ϕ_{gu} and ϕ_{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.

Footings may be placed directly on the bedrock surface after excavation of the overlying fill and native overburden soils and any loose or highly fractured rock, and founded at an elevation of about 71.2 m to match the existing pier foundations. 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 3 MPa (at ULS) may be used for design of spread/strip footing founded on the properly prepared bedrock surface at the pier. 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. This value is based on a fraction of the unconfined compressive strength of the rock and a resistance factor of 0.4, in accordance with the CHBDC. 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 (free of any soil seams) or mass concrete, frost protection cover is not required. No soil seams were noted within the boreholes advanced at the central pier location.

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 82 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 80 m. The underside of the existing pile caps are perched at Elevation 77.6 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, and based on approximate bedrock surface elevations obtained during the current and previous investigations, the anticipated pile length will be as follows:

Location	Approximate Bedrock Surface Elevation (m)	Anticipated Pile Length (m)
North Abutment	72.0 to 72.8	4.8 to 5.6
South Abutment	70.5 to 72.0	5.6 to 7.1

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 4.3 and 4.5, 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. 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



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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. 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 (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. A Non Standard Special Provision for driving piles adjacent to existing battered piles 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.

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

The above values are limited by the structural resistances for the piles. 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 short length of piles required for this structure, the use of the Hiley formula is sufficient for determining the pile capacity since the full driving energy is likely delivered to the pile tip (i.e., the driving energy is not absorbed by the thin overburden).

6.5.3 Downdrag Load (Negative Skin Friction)

The widening of the embankments up to the new abutment and foreslope retaining wall alignments will raise the effective stress level in the weathered silty clay deposit, however only very limited compression of the deposit is expected and therefore downdrag forces are not considered to be an issue on the 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 preliminary design of the Woodroffe 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 will likely include a 'truncated toe' configuration in front of the piles, adjacent and parallel to the Highway. Where piles are within 8 diameters (8D) of the foreslope 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:

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

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).

Location	Elevation (m)	Soil Type	n_h (MN/m ³)	s_u (kPa)
North Abutment (BH 1 & 3, 16-303 & 16-305)	75.0 – PCL ¹	Compact Sand and Silt Embankment FILL	6 to 7	-
	73.6 – 75.0	Firm to Very Stiff Weathered Silty Clay Crust	-	45 to 125
	72.3 – 73.6	Very Loose to Dense Silty Sand Glacial Till	1 to 11	-
	72.3	Bedrock	-	-



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Location	Elevation (m)	Soil Type	n_h (MN/m ³)	S_u (kPa)
South Abutment (BH 2 & 4, 16-302 & 16-304)	77.4 – PCL ¹	Compact Sand Embankment FILL	6 to 7	-
	75.5 – 77.4	Very Stiff Silty Clay Embankment Fill	-	50 to 100
	74.5 – 75.5	Very Stiff to Stiff Weathered Silty Clay Crust	-	75 to 125
	73.4 – 74.5	Very Loose to Compact Silty Sand Glacial Till (above water level)	2 to 7	-
	70.5/72 – 73.4	Very Loose to Compact Silty Sand Glacial Till (below water level)	1 to 4	-
	70.5 to 72	Bedrock	-	-

Note: ¹ PCL = Pile Cap Level, understood to be Elevation 77.6 m.

6.6 Drilled Shaft (Caisson) Foundations

Alternatively, support of the perched abutments may be provided by caisson foundations. However, based on the low RQD values obtained during the field investigations, deep sockets would be required and therefore caissons are not recommended.

6.7 Foreslope Retaining Walls

To accommodate a widened Highway 417, the foreslope paving and approach fills in front of both the north and south bridge abutments will need to be cut back and retained, as was completed previously at the Maitland Avenue Underpass. The choice of retaining wall system will depend on the desired appearance, the anticipated costs, performance, and on other considerations such as constructability.

6.7.1 Retaining Wall Options

The following technically feasible options for the proposed 'truncated' foreslope retaining walls in front of the abutments have been considered:

- Permanent soldier pile and lagging (SPL) wall with tie-backs and concrete facing;
- Bin-wall supported on native soils;
- Reinforced soil system (RSS) walls founded on the native soils; and,
- Reinforced concrete retaining wall founded on deep foundations.

It is understood that the walls constructed for the nearby Maitland Avenue Underpass consist of concrete-faced soldier pile retaining walls with piles augered and pinned to the bedrock for foundation support, and that tie-back anchors extending into the bedrock provide lateral support to the retaining walls. Similar walls could be considered for this structure. It is further understood that the existing toe walls beneath the Woodroffe Avenue structure are founded on caissons. Reinforced concrete retaining walls founded on deep foundations would also be feasible to retain the 'truncated' foreslope, however likely not economical. For deep foundations, the recommendations provided in Section 6.5 would apply. Recommendations for lateral earth pressures are provided further in Section 6.8.

Bin-walls are feasible for the proposed 'truncated' foreslope retaining walls. If bin-walls are selected to retain the foreslopes, the borehole results show that the subgrade would consist of very stiff clay over loose glacial till for both the north and south wall locations. Bin-walls, installed in sections, and backfilled for lateral support would be sufficiently flexible to found on the native soils. ULS and SLS values of 200 and 150 kPa, respectively, may be used for the design of bin walls founded on the very stiff silty clay over loose till.



RSS walls are likely not be feasible considering the need to install the lateral strips into the existing slopes and between the existing battered piles.

Shallow foundations for the reinforced concrete retaining wall supported on the loose till (below the stiff clay for adequate frost protection) are not considered practical or appropriate for this site since the bearing resistance of these soils would likely be insufficient for support of the retaining wall using standard backfill, and the total and differential settlement of the foundations during and after construction would likely be greater than the normally accepted values of 25 mm total and 15 mm differential settlement.

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

Based on the site constraints, relative costs and estimated settlement at the site due to the widened embankment loading, the preferred foundation option for the retaining walls is an SPL wall with tie-backs and concrete facing. The next preferred option may be a bin-wall.

The design of the SPL wall may be carried out according to the guidance provided in Section 6.5. Furthermore, due to the limited vertical clearance beneath the existing bridge deck, it is anticipated that the piles would be constructed in short sections welded together and installed using either a vibratory or drop hammer, or an auger piling rig, requiring minimal head room. The location of the existing battered piles supporting the abutments will have to be considered in the design. Additional guidance for design of the anchors for an SPL wall is provided in Section 6.7.3.

6.7.2 Frost Protection

Frost protection is not required for SPL walls, bin-walls, or RSS walls.

The footings for reinforced concrete walls 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. If adequate soil cover cannot be provided for the footings, rigid insulation (Styrofoam) could be installed to compensate for the lack of soil cover and provide protection from frost penetration.

6.7.3 Rock Anchors

A permanent soldier pile and lagging wall will likely require tie-backs to resist the lateral loads from the backfill. This lateral restraint may be provided by tie-backs installed into the bedrock. Rock anchors will need to consider the location of the existing and new piles supporting the abutments.

Anchors should be installed in accordance with OPSS 942 and should be provided with double corrosion protection which typically consists of a corrugated PVC sleeve sheathing placed over the full length of the anchor which is in turn encased within a smooth PVC sheathing over the length of the free stressing zone. The space between the anchor and sheathing is grouted. Corrosion inhibiting wax or grease and a PVC cap should be used to protect the anchor head.

Prestressing of the tie-backs prior to loading will minimize anchor movement due to service loads.

The capacity of the rock anchor will be the lesser of the bond strength or the resistance calculated based on the buoyant weight of the potential mass of rock which could be mobilized by the anchor. This is typically considered as the mass of rock included within a cone having an apex at the tip of the anchor and having an apex angle of 60 degrees.



For a group of anchors or for a line of closely spaced anchors the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. The calculated resistance for a rock anchor group should not be greater than the sum of the individual rock anchor capacities calculated without group effects.

The factored bond stress at the concrete/rock interface may be taken as 1 MPa for ultimate limit state (ULS) design purposes.

It is recommended that proof load tests be carried out on anchors to confirm their performance at the time of construction. The proof load tests should be carried out to 1.5 times the anchor service loads, and at least 10 percent of the anchors should be tested in this manner.

6.8 Lateral Earth Pressures for Design

The lateral earth pressures acting on the abutment walls and foreslope retaining 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.

Where there is sloping ground behind the retaining walls, 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 (i.e., abutment stems or tied-back retaining 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). The pressures are based on the existing embankment fill materials and the following parameters (unfactored) may be used:

Material:	Embankment Fill	
Unit Weight:	19 kN/m ³	
Backslope:	Flat	2H:1V
Coefficients of static lateral earth pressure:		
Active, Ka	0.33	0.54
At rest, Ko	0.50	0.50 ¹

Note 1: The lateral earth pressures under at-rest conditions should be calculated by treating the weight of the sloping backfill located above the top of the wall as a surcharge.



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- For unrestrained retaining 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 fill material behind the wall and the following parameters (unfactored) may be used:

Material:	Granular A or Granular B Type II	
Unit Weight:	22 kN/m ³	
Backslope:	Flat	2H:1V
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.39
At rest, K_o	0.43	0.43 ¹

Note 1: The lateral earth pressures under at-rest conditions should be calculated by treating the weight of the sloping backfill located above the top of the wall as a surcharge.

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. Where sloping backfill is present above the top of the wall, as is expected to be the case at this site, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.



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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	0.096	0.30	0.30
	975 Year	0.155	0.31	0.31
	2,475 Year	0.270	0.35	0.35
Non-Yielding Wall	475 Year	0.096	0.33	0.33
	975 Year	0.155	0.36	0.36
	2,475 Year	0.270	0.45	0.45

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 24, 39, and 68 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: $\delta_{h(z)} = K_{a1} \gamma z + (K_{AE} - K_{a2}) \gamma (H-z)$

For Non-Yielding Walls: $\delta_{h(z)} = K_o \gamma z + (K_{AE} - K_{a2}) \gamma (H-z)$

Where:

- $\sigma_{h(z)}$ is the lateral earth pressure at depth 'z' (kPa);
- K_{a1} is the static active earth pressure coefficient for sloping backfill;
- K_{a2} is the static active earth pressure coefficient for flat non-sloping backfill;
- K_o is the static at-rest earth pressure coefficient;
- K_{AE} is the seismic earth pressure coefficient;
- γ is the unit weight of the backfill soil (kN/m³), 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, which is in turn underlain by loose to compact 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 Woodroffe 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 OPSD 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 Foreslope 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 7 m high approach embankments, beyond the ends of the retaining walls, 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 on 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.

Foreslope retaining walls up to 3 m in height with a 2H:1V backslope will have a factor of safety of greater than 1.3 against deep-seated global instability, as shown on Figure F2 in Appendix F. Pseudo-static seismic stability analyses also indicate that the retaining walls will have factors of safety greater than 1.1 against deep-seated global instability, based on an acceleration of 0.14g. The stability of the retaining walls against sliding, overturning, and bearing failure will, however, need to be checked by the designers.



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Material	Bulk Unit Weight (kN/m ³)	Effective Friction Angle (degrees)	Undrained Shear Strength (kPa)
New Granular Embankment Fill	21	32	-
Existing Sand, Silt and Clay 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	-	46
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 detail design drawings and subsoil conditions encountered within the proposed approach embankment and retaining wall footprints during detail design.

6.9.3 Settlement under Widened Embankment Loading

It is understood that an approximately 4.9 m of eastward widening is planned for the existing approach embankments which are up to 7 m high.

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 the settlement assessment, the settlement of the subgrade soils under the 4.9 m eastward widening is estimated to be less than about 15 mm, mainly associated with the silty clay deposit (thicker for the north abutment). 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. It is not considered that the construction will impact the identified utility (i.e., gas main), which is located over 20 m east of the existing structure.



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, and removal of the existing piers 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 expected to be at a depth of about 3 m (i.e., about Elevation 73 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). Below the groundwater level, sand in loose till could slough.

6.10.3 Temporary Protection Systems

It is anticipated that temporary roadway protection will be required along Woodroffe Avenue and along Highway 417 to permit construction of the abutment widenings and foreslope retaining walls. For the pier footing construction, temporary excavation support may be required due to space restrictions. It is considered that the temporary support system for this site could consist of soldier piles and lagging. At the pier, internal bracing could be used to 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.

The temporary protection system under the existing bridge structure will have to be installed using equipment suitable for work in areas with limited vertical clearance.

6.10.4 Obstructions

It should be noted that obstructions (inferred as cobbles) should be anticipated 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.

It is understood that the existing toe walls beneath the Woodroffe Avenue structure are founded on caissons. During construction of the new 'truncated' foreslope retaining walls, associated with the widening of the Highway 417 platform, the existing caissons would need to be removed to below proposed pavement subgrade level.

6.10.5 Groundwater and Surface Water Control

The groundwater level at the pier is about 3 m depth below the highway grade. Excavations for the construction of the pier foundations on bedrock will likely involve 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.



6.11 Recommendations for Further Work

The design-build proponent will be responsible for the detail design of the widened bridge structure and retaining walls, and assessing additional requirements for investigations to suit the design and for mitigating any identified constructions risks. It is anticipated that additional boreholes could be required during the design-build stage of investigation, to further assess and/or confirm the subsurface conditions and the recommendations provided in this report for the foreslope retaining walls.

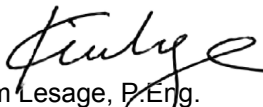


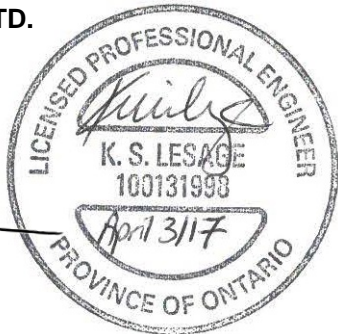
FOUNDATION REPORT - HIGHWAY 417 WOODROFFE AVENUE UNDERPASS

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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SM/KSL/ESO/FJH/ob

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FOUNDATION REPORT - HIGHWAY 417 WOODROFFE AVENUE UNDERPASS

TABLE 1 – COMPARISON OF BRIDGE ABUTMENT FOUNDATION ALTERNATIVES

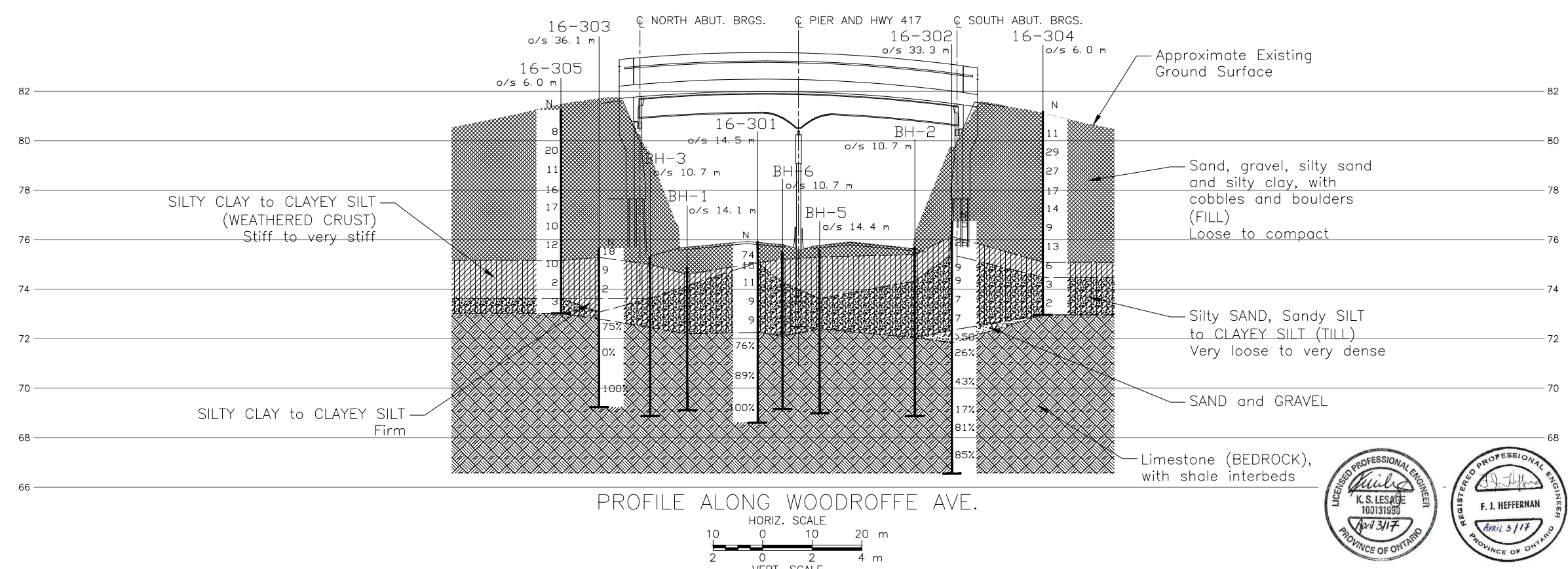
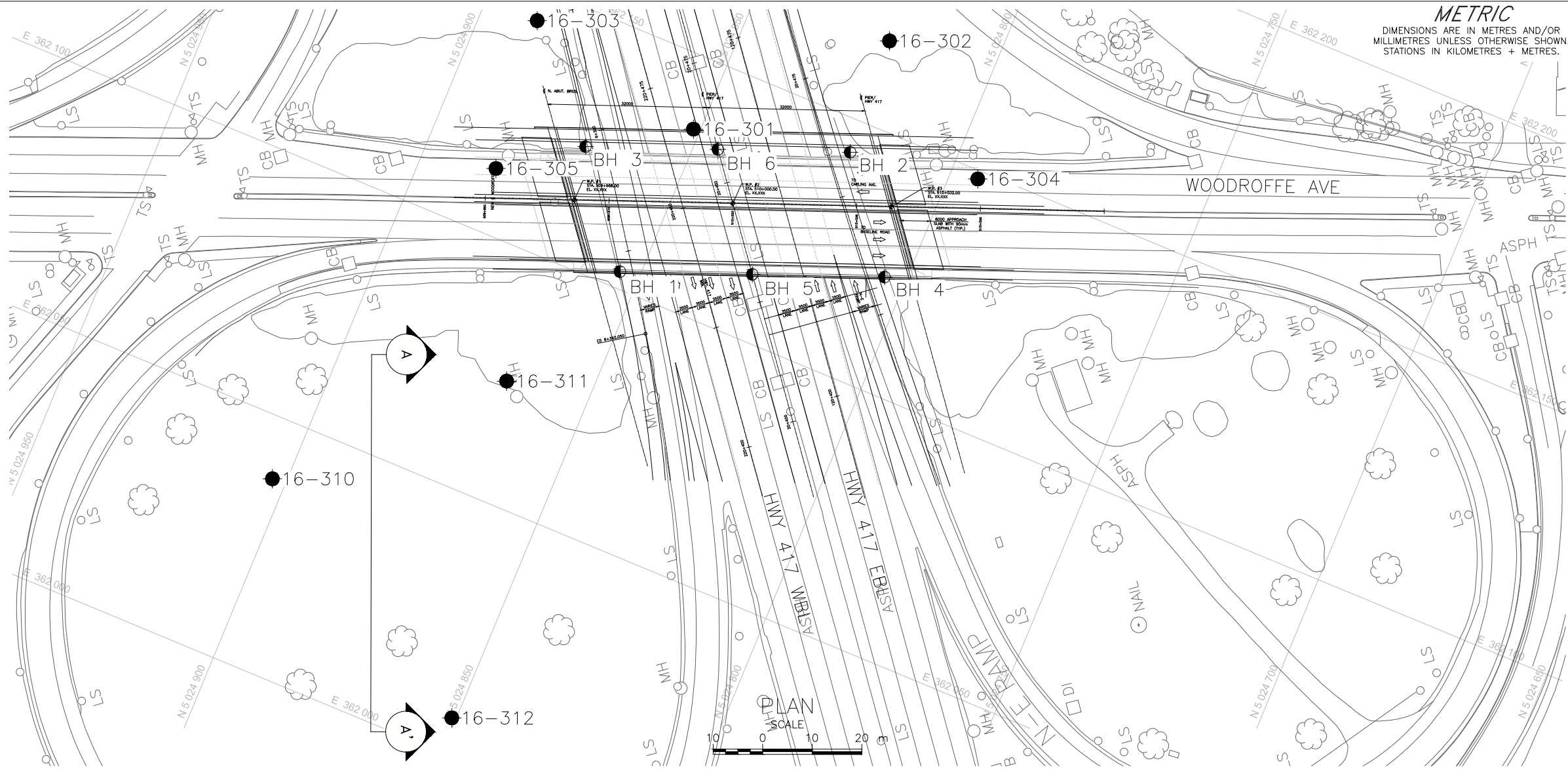
Foundation Option	Feasibility	Advantages	Disadvantages	Estimated Costs	Risk / Consequences
1. Steel H-piles or pipe piles founded on or socketed into bedrock	<ul style="list-style-type: none"> Feasible for the support of new abutments with pile cap “perched” within the approach embankments. 	<ul style="list-style-type: none"> High bearing resistance. Negligible settlement. Compatible with existing bridge foundations. Conventional construction methods for H-pile or steel pipe pile foundations. Steel H-piles allow for integral and semi-integral abutment configuration. 	<ul style="list-style-type: none"> If lateral / seismic loading conditions merit, pile toe may have to be socketed into strong bedrock, which would require coring or churn drilling If sockets required, temporary liner necessary Piles may interfere with existing battered piles. Pipe piles not readily accepted for integral abutment construction; allow for semi-integral abutment configuration. 	<ul style="list-style-type: none"> May be less expensive than caisson option. 	<ul style="list-style-type: none"> Minor potential for pile damage / deflection if cobbles and boulders are encountered during pile driving. Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving.
2. Drilled Shaft (caissons) founded on or socketed into bedrock	<ul style="list-style-type: none"> Feasible, but not recommended or compatible with the existing foundations. 	<ul style="list-style-type: none"> High bearing resistance. Negligible settlement. Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles. 	<ul style="list-style-type: none"> Deep sockets required. Possibility of encountering cobbles or boulders during installation. Socketting of liner required to permit cleaning and inspection. Coring or churn drilling will be required to form rock socket in strong bedrock. 	<ul style="list-style-type: none"> May be more expensive than steel H-pile option, particularly for deep rock sockets, due to larger socket diameter. 	<ul style="list-style-type: none"> May not be able to dewater socket for cleaning and inspection if no temporary liner.
Spread/strip footings supported on native soil	<ul style="list-style-type: none"> Not Feasible 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A



FOUNDATION REPORT - HIGHWAY 417 WOODROFFE AVENUE UNDERPASS

TABLE 2 – COMPARISON OF RETAINING WALL FOUNDATION ALTERNATIVES

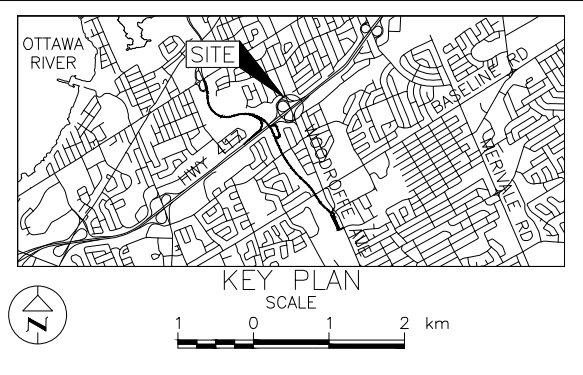
Retaining Wall Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risks/ Consequences
1. Soldier pile and concrete lagging (SPL) wall with tie-back anchors, Preferred option	■ Feasible	<ul style="list-style-type: none"> ■ Limited excavation of existing embankment required ■ Negligible settlement ■ Minimal excavation since frost protection not required 	<ul style="list-style-type: none"> ■ Low vertical clearance for tie-back installation 	<ul style="list-style-type: none"> ■ Low cost, less expensive than reinforced concrete wall 	
2. Bin-Walls	■ Feasible	<ul style="list-style-type: none"> ■ Simple to construct ■ Minimal excavation since frost protection not required 	<ul style="list-style-type: none"> ■ Excavation required into existing foreslopes ■ Backfill behind walls would be replaced with granular fill ■ Adequate founding depth required for lateral restraint 	<ul style="list-style-type: none"> ■ Low cost, likely least expensive option 	<ul style="list-style-type: none"> ■ Shorter time for construction ■ Potential for conflict with existing battered piles
3. RSS wall	■ Feasible, but not recommended	<ul style="list-style-type: none"> ■ Minimal excavation since frost protection not required 	<ul style="list-style-type: none"> ■ Extensive excavation required into existing foreslopes and temporary protection for installation of tie back strips 	<ul style="list-style-type: none"> ■ Moderate cost 	
4. Reinforced concrete wall supported on deep foundations	■ Feasible, but not recommended	<ul style="list-style-type: none"> ■ Negligible settlement. 	<ul style="list-style-type: none"> ■ Excavation required into existing foreslopes ■ Low vertical clearance for piling ■ More involved construction 	<ul style="list-style-type: none"> ■ Higher cost 	<ul style="list-style-type: none"> ■ Longer time for construction (i.e., concrete pouring and curing in sections)
Reinforced concrete wall supported on shallow foundations on till	■ Not feasible	<ul style="list-style-type: none"> ■ N/A 	<ul style="list-style-type: none"> ■ Loose till subgrade could lead to higher than tolerable settlements ■ Frost protection required 	<ul style="list-style-type: none"> ■ Moderate cost, less costly than concrete wall on deep foundations 	<ul style="list-style-type: none"> ■ Predicted settlements exceed tolerance for structure



CONT No.
WP No.4015-E-0017

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

SHEET



LEGEND

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-301	75.9	5024851.3	362135.6
16-302	76.8	5024821.6	362167.2
16-303	75.7	5024888.8	362143.7
16-304	81.2	5024794.4	362148.3
16-305	81.2	5024885.1	362113.0
16-310	75.1	5024902.8	362037.9
16-311	76.6	5024866.7	362074.2
16-312	75.3	5024850.9	362007.2
BH-1	74.9	5024854.0	362103.4
BH-2	75.7	5024820.3	362143.4
BH-3	75.3	5024870.1	362124.0
BH-4	75.6	5024804.2	362122.8
BH-5	75.6	5024829.1	362113.1
BH-6	75.5	5024845.2	362133.7

REFERENCE

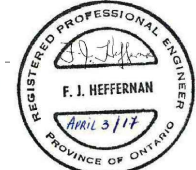
Base plans provided in digital format by MMM Group, drawing file no. TN_3416012-304-001_General-Arrangement-I.dwg, received October 26, 2016.

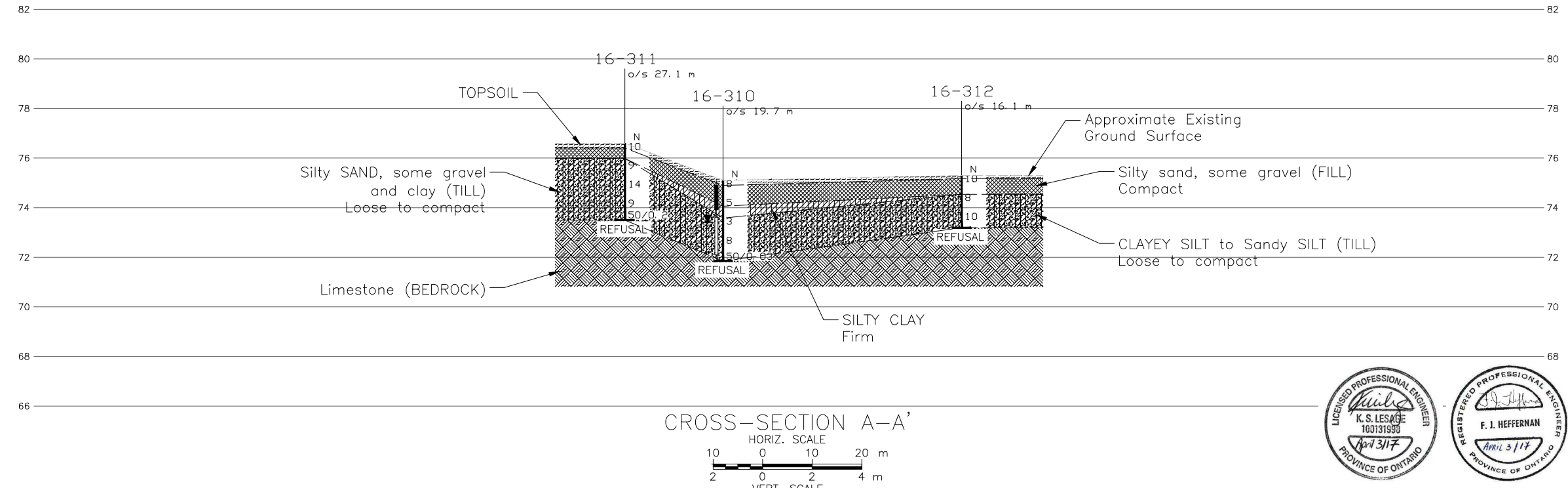
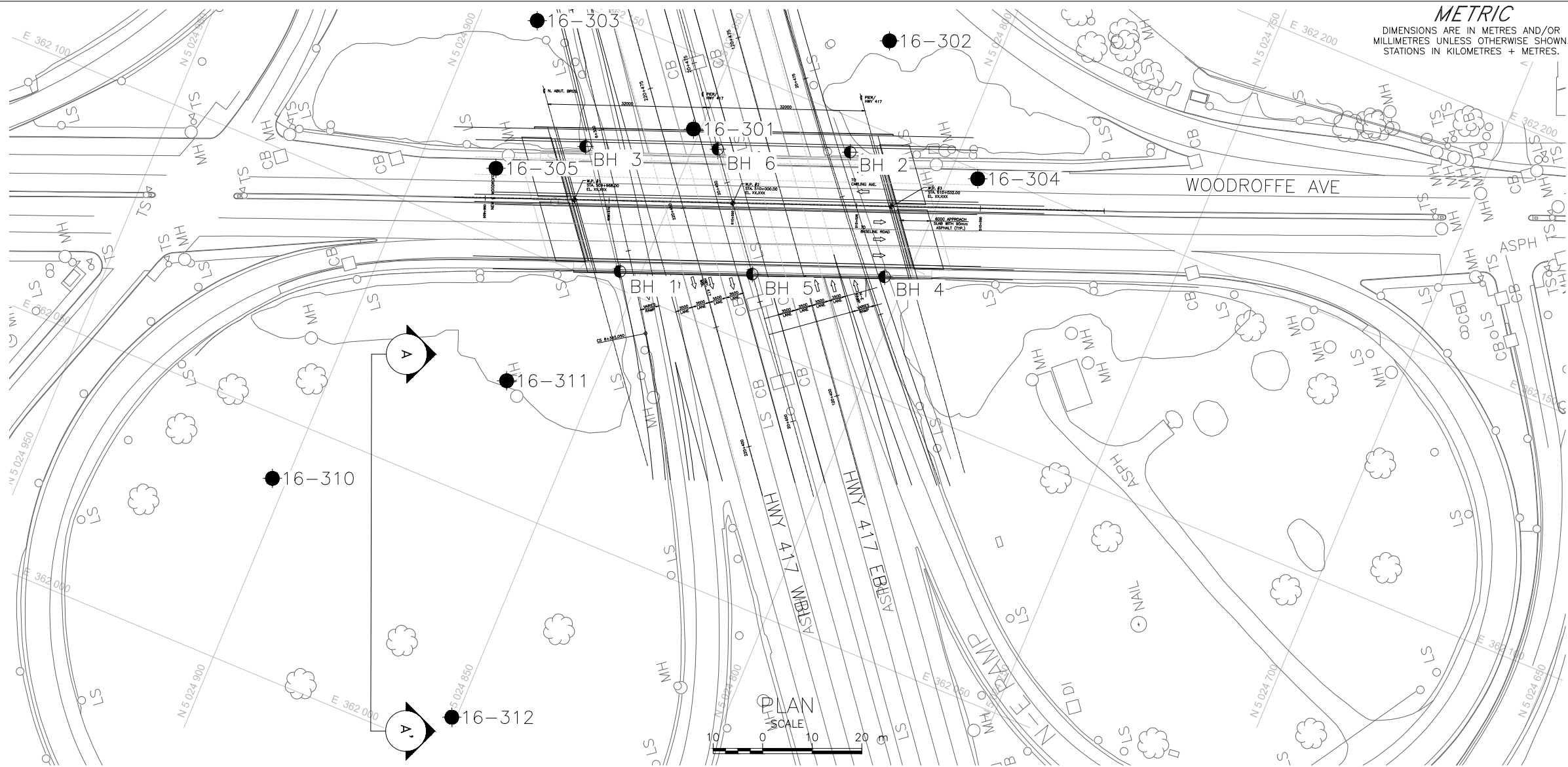
NOTES

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
Geocres No. 3165-278			
HWY. 417	PROJECT NO. 1546542-1030		DIST. EASTERN
SUBM'D. KSL	CHKD. KSL	DATE: 11/14/2016	SITE: 3-041
DRAWN: JM	CHKD. ESO	APPD. FJH	DWG. 1





CONT No.
WP No.4015-E-0017

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

KEY PLAN
SCALE
1 0 1 2 km

LEGEND

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

BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-301	75.9	5024851.3	362135.6
16-302	76.8	5024821.6	362167.2
16-303	75.7	5024888.8	362143.7
16-304	81.2	5024794.4	362148.3
16-305	81.2	5024885.1	362113.0
16-310	75.1	5024902.8	362037.9
16-311	76.6	5024866.7	362074.2
16-312	75.3	5024850.9	362007.2
BH-1	74.9	5024854.0	362103.4
BH-2	75.7	5024820.3	362143.4
BH-3	75.3	5024870.1	362124.0
BH-4	75.6	5024804.2	362122.8
BH-5	75.6	5024829.1	362113.1
BH-6	75.5	5024845.2	362133.7

REFERENCE

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NO.	DATE	BY	REVISION

Geocres No. 31G5-278

HWY. 417	PROJECT NO. 1546542-1030	DIST. EASTERN
SUBM'D. KSL	CHKD. KSL	DATE: 11/24/2016
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 16-301 to 16-306 and 16-310 to 16-312

Bedrock Core Photographs, Figures A1 to A6



LIST OF SYMBOLS

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

I. GENERAL

π	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

γ	shear strain
Δ	change in, e.g. in stress: $\Delta \sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
ν	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	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
γ'	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_α	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
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio = σ'_p / σ'_{vo}

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	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 ρ . Unit weight symbol is γ 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 cm² 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 _p	plastic limit
w _l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test ¹
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement ¹
D _R	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 ₄	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	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	

+3, ×3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

PROJECT: 1546542-1030

RECORD OF DRILLHOLE: 16-301

SHEET 2 OF 2

LOCATION: N 5024851.3 ; E 362135.6

DRILLING DATE: June 23, 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					
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 100	DIP W/L CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX											
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr													
		Continued from Record of Borehole 16-301		72.22																					
4		Limestone (BEDROCK), with shale interbeds Fresh Medium bedded Grey Medium strong to strong		3.68	1	100																			
5					2	100																			
6					3	100																			
7																									
8		END OF DRILLHOLE		68.58																					
9				7.32																					
10																									
11																									
12																									
13																									

DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: KSL

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PROJECT <u>1546542-1030</u>		RECORD OF BOREHOLE No 16-302		SHEET 1 OF 3		METRIC	
W.P. <u>4015-E-0017</u>		LOCATION <u>N 5024821.6 ; E 362167.2</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>			

[illegible]

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

○ 3% STRAIN AT FAILURE



+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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PROJECT: 1546542-1030

RECORD OF DRILLHOLE: 16-302

SHEET 3 OF 3

LOCATION: N 5024821.6 ;E 362167.2

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					
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 100	DIP W/L CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX											
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr													
5	Rotary Drill NQ Core	Continued from Record of Borehole 16-302		71.87																					
		Limestone (BEDROCK) Slightly weathered to fresh Medium bedded Grey Medium strong		4.93	1	0																			
6					2	0																			
7					3	0																			
8		Shale (BEDROCK) Fresh Black Medium strong		68.80 8.00 68.50 8.30	4	0																			
9		Limestone (BEDROCK) with shale interbeds Fresh Medium bedded Grey Medium strong			5	0																			
10		END OF DRILLHOLE		66.59 10.21																					
11		NOTES: 1. Fractured rock for Run 1 and 3 may be a result of drilling techniques/equipment (i.e., mechanical breaks).																							
12																									
13																									
14																									

DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: KSL

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PROJECT		RECORD OF BOREHOLE		No 16-303		SHEET 1 OF 2		METRIC											
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE											
4015-E-0017		N 5024888.8 ; E 362143.7		DG		Eastern		Power Auger 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core											
DATUM		DATE		CHECKED BY		COMPILED BY		ZS											
Geodetic		June 26, 2016		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	W _p	W	W _L	25 50 75			
75.7	0.0	GROUND SURFACE																	
75.3	0.1	Silty sand (TOPSOIL) Brown Dry		1	SS	18													
75.3	0.4	Silty sand (FILL) Compact Brown Dry																	
		SILTY CLAY to CLAYEY SILT (WEATHERED CRUST) Very stiff to stiff Grey-brown Wet		2	SS	9													
				3	SS	2													
73.6	2.1	SILTY CLAY to CLAYEY SILT Firm Grey Wet																	
73.1	2.6	Probable TILL																	
72.8	2.9	Limestone (BEDROCK)																	
		Bedrock cored from depths of 2.9 m to 6.4 m		1	RC	REC 100%													RQD = 75%
		For bedrock coring details refer to Record of Drillhole 16-303		2	RC	REC 88%													RQD = 60%
				3	RC	REC 100%													RQD = 100%
69.3	6.4	END OF BOREHOLE																	

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PROJECT: 1546542-1030

RECORD OF DRILLHOLE: 16-303

SHEET 2 OF 2

LOCATION: N 5024888.8 ; E 362143.7

DRILLING DATE: June 26, 2016

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 45

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					
						RECOVERY		R.Q.D. %	FRACT. INDEX PER 100	DIP W/L CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX											
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr													
		Continued from Record of Borehole 16-303		72.83																					
3		Limestone (BEDROCK), with shale interbeds Fresh Medium bedded Grey Medium Strong		2.87	1	100																			
4					2	100																			
5					3	100																			
6																									
7		END OF DRILLHOLE		69.27																					
		NOTES: 1. Fractured rock for Run 2 may be a result of drilling techniques/equipment (i.e., mechanical breaks).		6.43																					
8																									
9																									
10																									
11																									
12																									

DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: KSL

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PROJECT		1546542-1030		RECORD OF BOREHOLE No 16-304		SHEET 1 OF 1		METRIC								
W.P.		4015-E-0017		LOCATION		N 5024794.4 ; E 362148.3		ORIGINATED BY								
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY								
DATUM		Geodetic		DATE		July 28, 2016		CHECKED BY								
								KSL								
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			SHEAR STRENGTH kPa								WATER CONTENT (%)
81.2	GROUND SURFACE							20	40	60	80	100				
0.0	ASPHALTIC CONCRETE															
0.2	Sandy gravel (FILL) Grey-brown Moist						81									
80.6	Sand, some silt, trace clay and gravel, contains silty clay pockets (FILL) Compact Brown Moist		1	SS	11		80									
0.6																
			2	SS	29											
							79									
			3	SS	27											8 56 29 7
			4	SS	17		78									
77.4	Silty clay, some sand, some gravel (FILL) Dark grey-brown Moist		5	SS	14		77									
3.8																
			6	SS	9											
							76									
75.9	Sand and silt, some clay, trace gravel, contains rootlets (FILL) Compact Dark grey-brown Moist		7	SS	13											
5.3																
75.1	SILTY CLAY to CLAY, some sand (WEATHERED CRUST) Very stiff Grey-brown Wet		8	SS	6		75									1 14 37 48
6.1																
74.5	Silty SAND, some clay and gravel (TILL) Very loose Grey-brown Wet		9	SS	3		74									
6.7																
			10	SS	2											
73.0	END OF BOREHOLE						73									
8.2																

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PROJECT <u>1546542-1030</u>		RECORD OF BOREHOLE No 16-305		SHEET 1 OF 1		METRIC	
W.P. <u>4015-E-0017</u>		LOCATION <u>N 5024885.1 ; E 362113.0</u>		ORIGINATED BY <u>JD</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>July 26, 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 γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					W _p W W _L				
						20 40 60 80 100					25 50 75						
81.3	GROUND SURFACE																
0.0	ASPHALTIC CONCRETE																
81.1																	
0.2	Sandy silt, some gravel and clay (FILL)																
80.8	Grey-brown																
0.5	Moist																
	Sand, contains silty clay pockets (FILL)																
	Loose to compact		1	SS	8												
	Brown																
	Moist																
			2	SS	20												
			3	SS	11												
78.2																	
3.1	Sand and silt, some clay, trace gravel, contains wood fragments (FILL)		4	SS	16												
	Compact																
	Dark grey-brown		5	SS	17												
	Moist to wet																
			6	SS	10												
			7	SS	12												
75.2																	
6.1	Silty CLAY, some sand (WEATHERED CRUST)		8	SS	10												
	Very stiff to stiff																
	Grey-brown		9	SS	2												
	Wet																
73.7																	
7.6	Sandy SILT, some gravel and clay (TILL)		10	SS	3												
	Very loose																
	Brown																
	Wet																
73.1																	
8.2	END OF BOREHOLE																

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PROJECT		1546542-1030		RECORD OF BOREHOLE No 16-310		SHEET 1 OF 1		METRIC										
W.P.		4015-E-0017		LOCATION		N 5024902.8 ; E 362037.9		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY										
DATUM		Geodetic		DATE		June 15, 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	20 40 60 80 100	20 40 60 80 100	W _p	W	W _L	γ	GR	SA	SI	CL
75.1	0.0	GROUND SURFACE																
74.9	0.2	Silty sand (TOPSOIL) Dark brown Moist		1	SS	8		75										
74.7	0.4	Sand (FILL) Light brown Dry																
74.1	1.0	Silt, some sand and clay (FILL) Loose Grey-brown Moist		2	SS	5		74										0 23 49 28
73.6	1.5	SILTY CLAY Firm Grey Moist to wet																
		Silty SAND, some gravel, trace clay, with cobbles and boulders (TILL) Loose Grey Wet		3	SS	3		73										
				4	SS	8												
				5	SS	50/0.03		72										
71.9	3.3	END OF BOREHOLE AUGER REFUSAL																
		NOTES: 1. Water level in well screen at a depth of 1.7 m below ground surface (Elev. 73.4 m), measured on September 30, 2016.																

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PROJECT		RECORD OF BOREHOLE		No 16-311		SHEET 1 OF 1		METRIC	
W.P.		LOCATION		ORIGINATED BY		DIST		BOREHOLE TYPE	
DATE		COMPILED BY		CHECKED BY		DATUM		DATE	
1546542-1030		N 5024866.7 ;E 362074.2		DG		Eastern HWY 417		Power Auger 200 mm Diam. (Hollow Stem)	
Geodetic		June 15, 2016		ZS		KSL			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100	W _p	W	W _L		
76.6	GROUND SURFACE																
76.0	Silty sand (TOPSOIL)																
76.4	Dark brown																
0.2	Moist		1	SS	10												
76.0	Silty sand, some gravel (FILL)																
0.6	Compact Brown																
	Moist																
	Silty SAND, some gravel and clay (TILL)		2	SS	9												
	Loose to compact																
	Grey-brown to grey																
	Wet																
			3	SS	14												
			4	SS	9												
73.5	END OF BOREHOLE		5	SS	50/0.2												
3.1	AUGER REFUSAL																

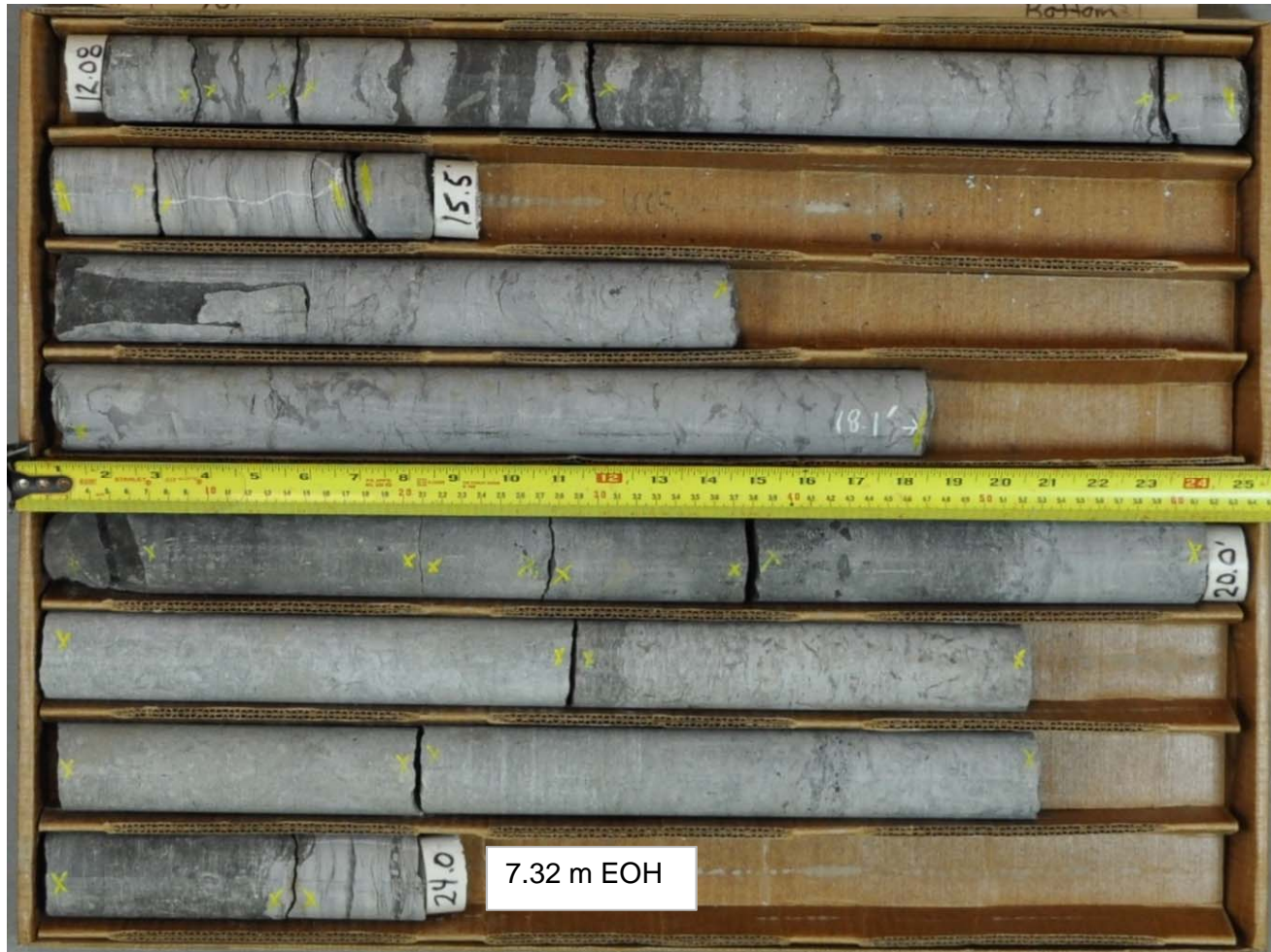
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PROJECT <u>1546542-1030</u>		RECORD OF BOREHOLE No 16-312		SHEET 1 OF 1		METRIC	
W.P. <u>4015-E-0017</u>		LOCATION <u>N 5024850.9 ; E 362007.2</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem)</u>		COMPILED BY <u>ZS</u>			
DATUM <u>Geodetic</u>		DATE <u>June 15, 2016</u>		CHECKED BY <u>KSL</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE LIQUID CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE	20	40	60	80	100	● QUICK TRIAXIAL × REMOULDED	WATER CONTENT (%)						
75.3	GROUND SURFACE																				
0.0	Silty sand (TOPSOIL) Dark brown Moist		1	SS	10																
0.1	Silty sand, some gravel (FILL) Compact Brown Moist																				
74.5																					
0.8	CLAYEY SILT to Sandy SILT (TILL) Loose to compact Grey Moist		2	SS	8																
			3	SS	10																
73.2																					
2.1	END OF BOREHOLE AUGER REFUSAL																				

3.68 m Top of bedrock

BH 16-301 (Dry)
Cored Length of 3.68 to 7.32 metres
Core Box 1 to 2 of 2



7.32 m EOH

CLIENT
MMM Group Limited

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

CONSULTANT



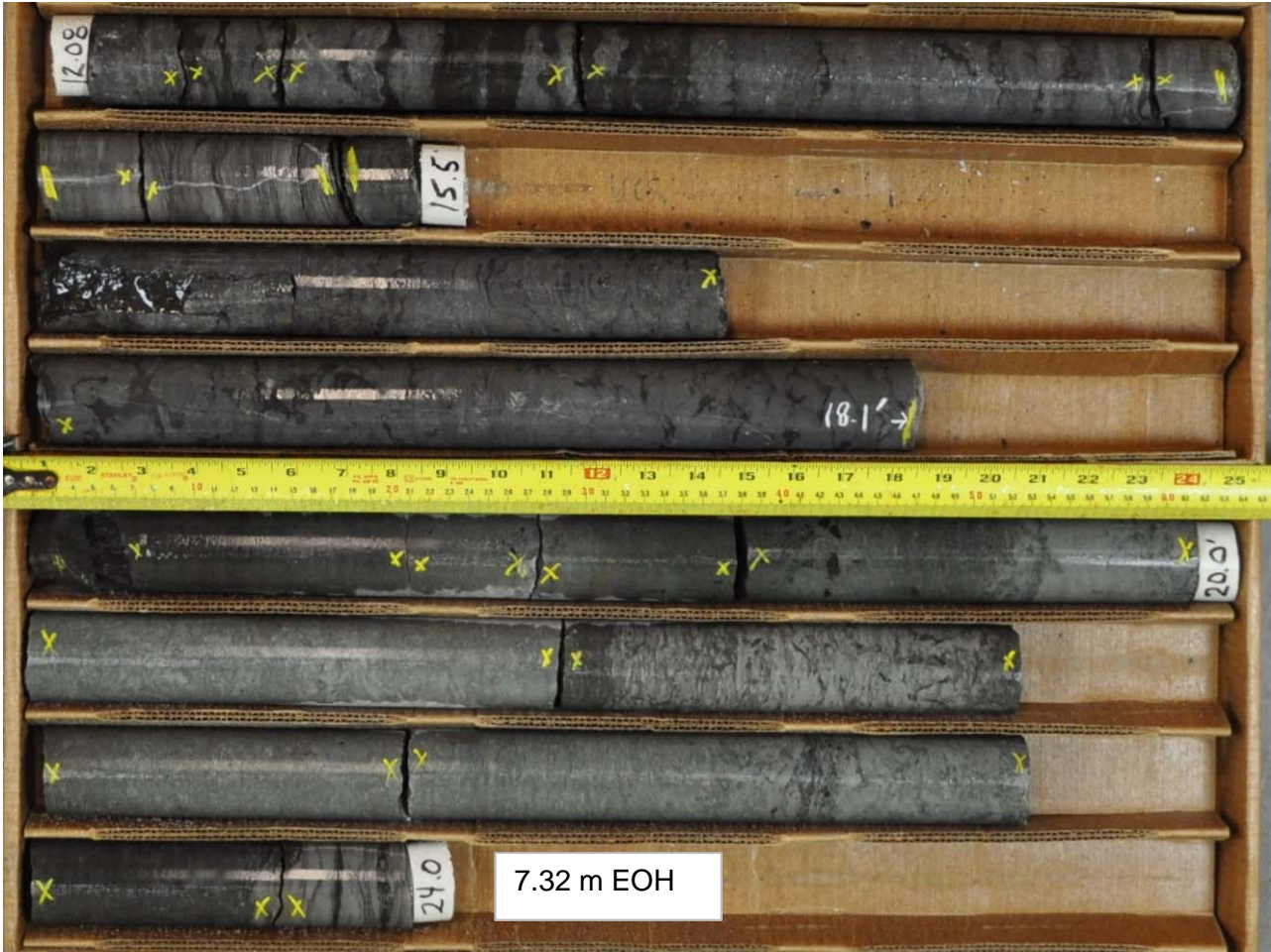
YYY/MM/DD	2/23/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE
**BOREHOLE 16-301 (DRY)
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1030	0a	A1

BH 16-301 (Wet)
Cored Length of 3.68 to 7.32 metres
Core Box 1 to 2 of 2

3.68 m Top of bedrock



7.32 m EOH

CLIENT
MMM Group Limited

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

CONSULTANT



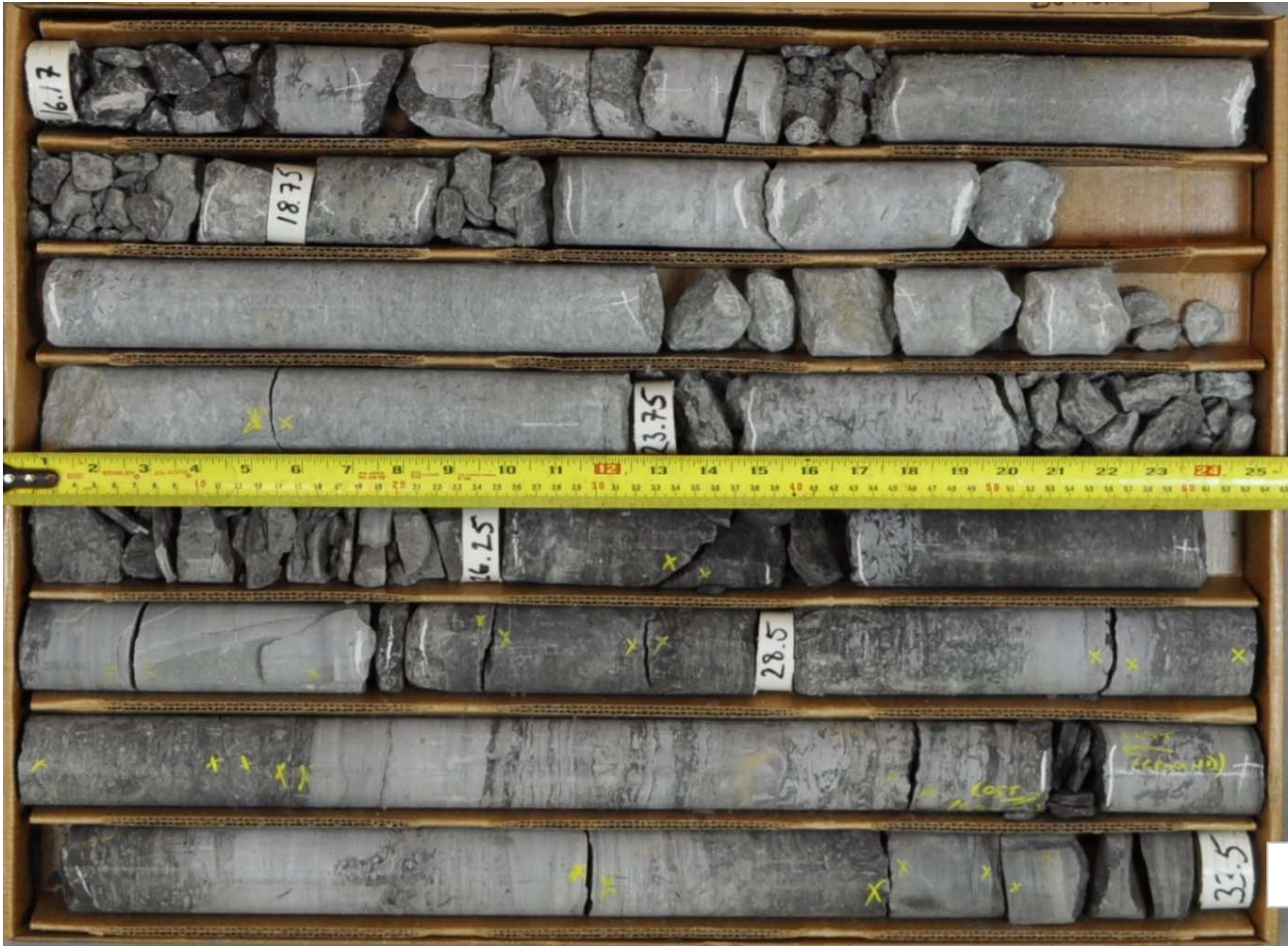
YYY/MM/DD	2/23/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE
**BOREHOLE 16-301 (WET)
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1030	0a	A2

4.93 m Top of bedrock

BH 16-302 (Dry)
Cored Length of 4.93 to 10.21 metres
Core Box 1 to 2 of 2



10.21 m EOH

CLIENT
MMM Group Limited

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

CONSULTANT



YYY/MM/DD	2/23/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE
**BOREHOLE 16-302 (DRY)
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1030	0a	A3

4.93 m Top of bedrock

BH 16-302 (Wet)
Cored Length of 4.93 to 10.21 metres
Core Box 1 to 2 of 2



10.21 m EOH

CLIENT
MMM Group Limited

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

CONSULTANT



YYY/MM/DD	2/23/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE
**BOREHOLE 16-302 (WET)
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1030	0a	A4

BH 16-303 (Dry)
Cored Length of 2.87 to 6.43 metres
Core Box 1 to 2 of 2

2.87 m Top of bedrock



6.43 m EOH

CLIENT
MMM Group Limited

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

CONSULTANT



YYY/MM/DD	2/23/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE
**BOREHOLE 16-303 (DRY)
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1030	0a	A5

BH 16-303 (Wet)
Cored Length of 2.87 to 6.43 metres
Core Box 1 to 2 of 2

2.87 m Top of bedrock



6.43 m EOH

CLIENT
MMM Group Limited

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

CONSULTANT



YYY/MM/DD	2/23/2017
PREPARED	KM
DESIGN	KM
REVIEW	KSL
APPROVED	ESO

TITLE
**BOREHOLE 16-303 (WET)
CORE PHOTOGRAPHS**

PROJECT No.	PHASE	Rev.	FIGURE
1546542	1030	0a	A6



APPENDIX B

Laboratory Test Results, Current Investigation

Figure 1 - Grain Size Distribution Test Results – Sand and Silt (FILL)

Figure 2 - Plasticity Chart – Silty Clay to Clayey Silt (Weathered Crust)

**Figure 3 - Grain Size Distribution Test Results – Silty Clay to Clayey Silt
(Weathered Crust)**

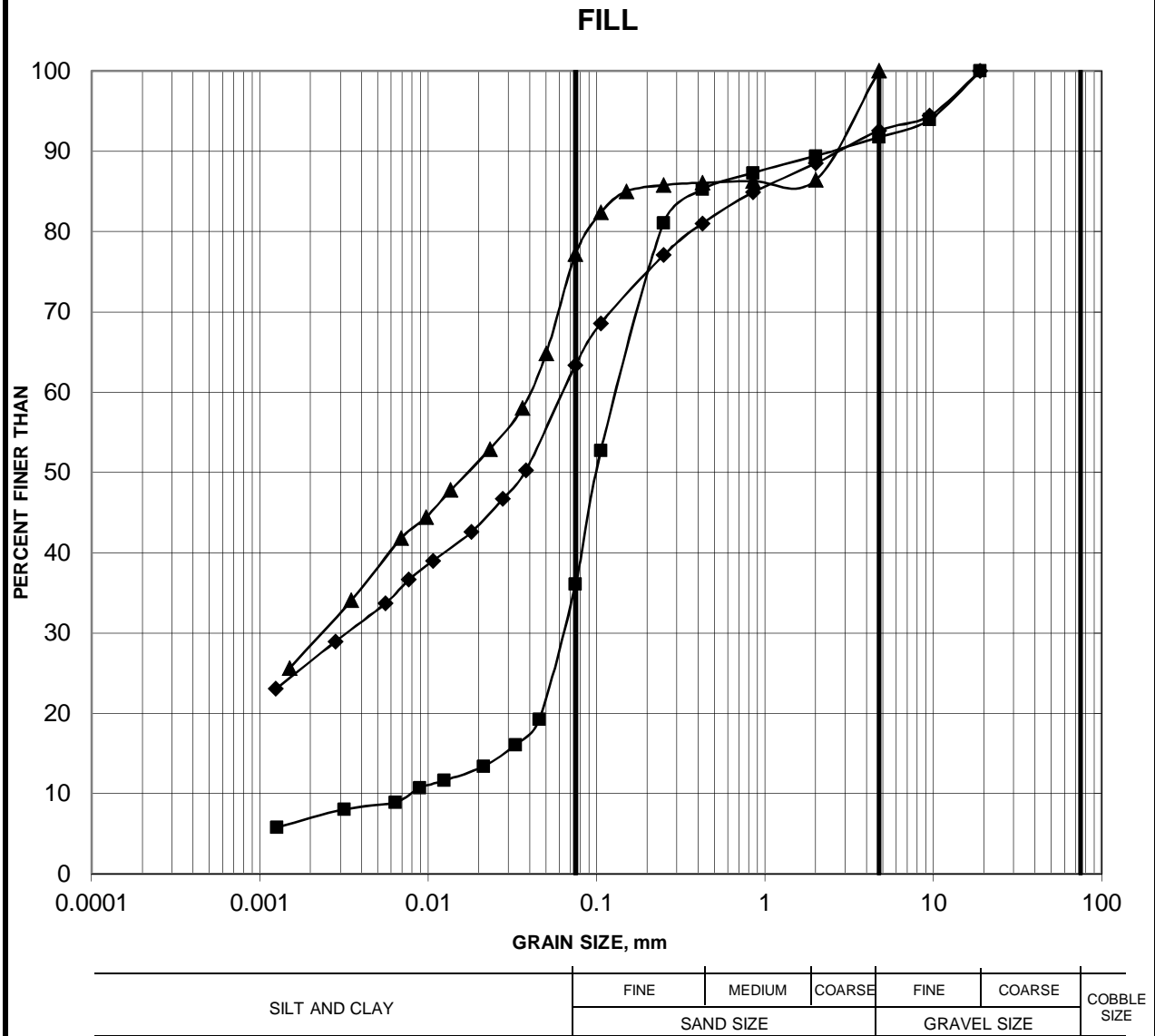
Figure 4 - Grain Size Distribution Test Results – Till

Figure 5 - Grain Size Distribution Test Results – Sand and Gravel

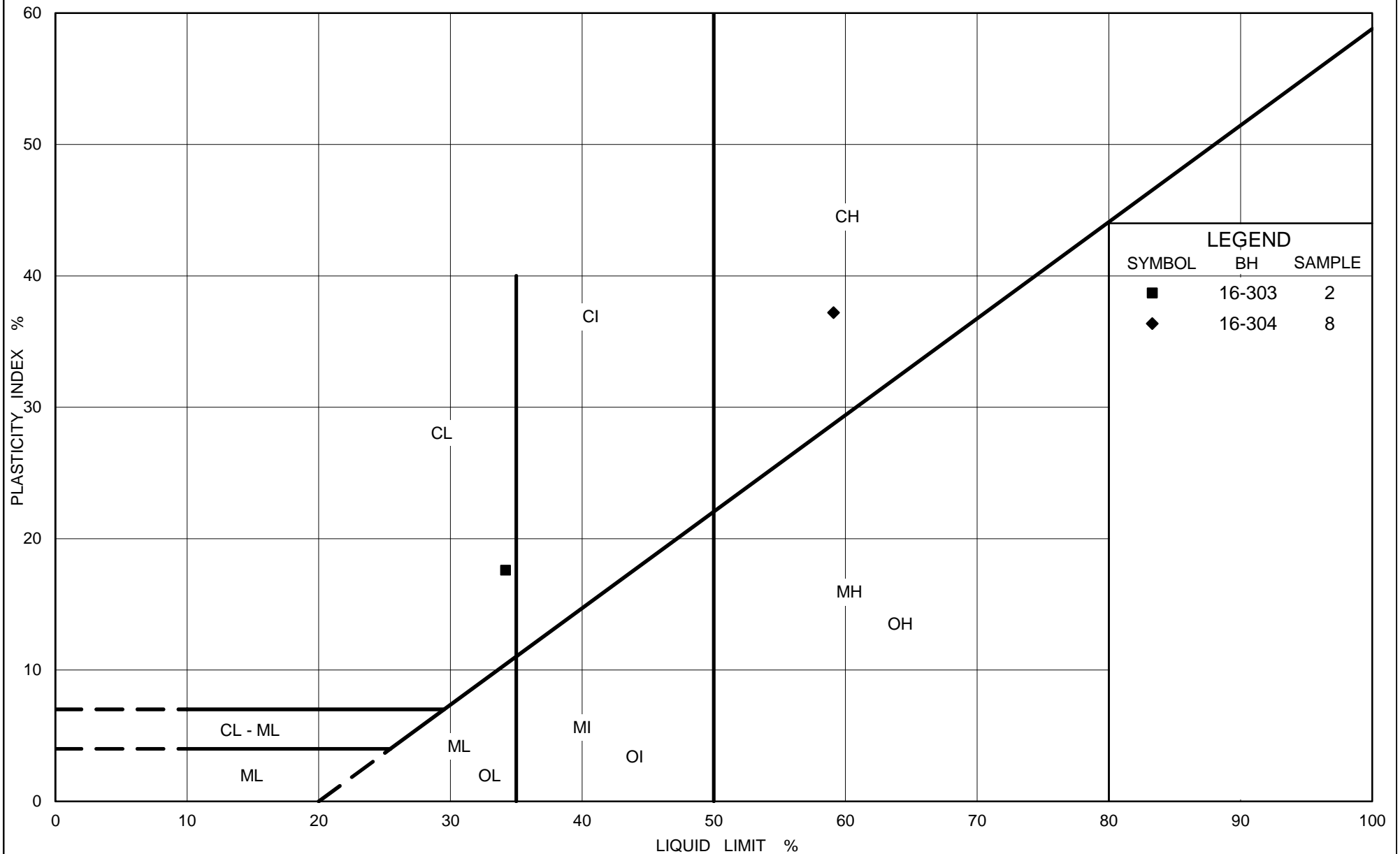
Figure 6 - UCS Test Result – Bedrock

GRAIN SIZE DISTRIBUTION

FIGURE 1



Borehole	Sample	Depth (m)
16-304	3	2.29-2.90
16-305	5	3.81-4.42
16-310	2A	0.76-1.04



Ontario

Ministry of Transportation

PLASTICITY CHART SILTY CLAY TO CLAYEY SILT (WEATHERED CRUST)

FIG No. 2

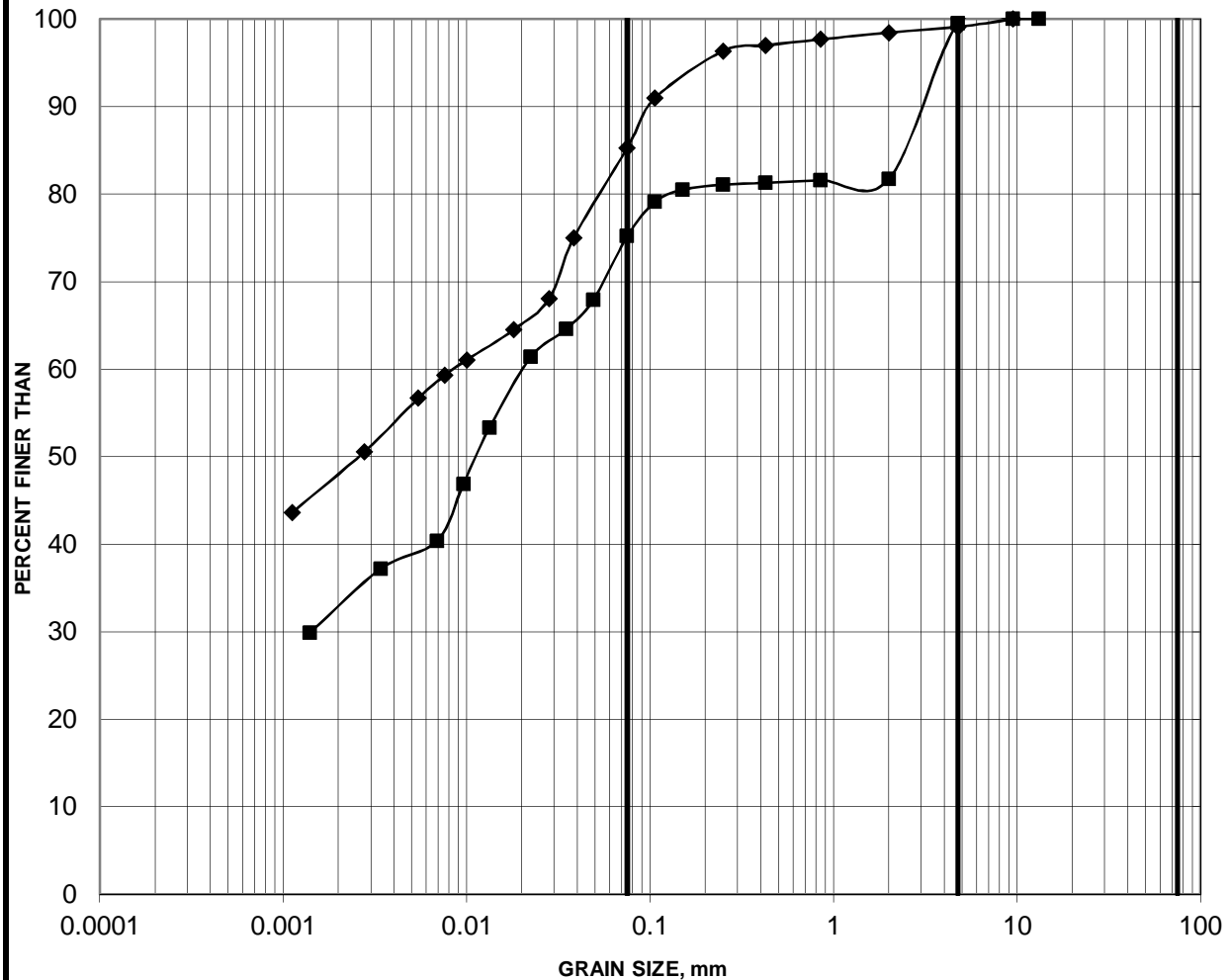
Project No. 1546542-1030

Compiled By : CW Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE 3

SILTY CLAY TO CLAYEY SILT (WEATHERED CRUST)



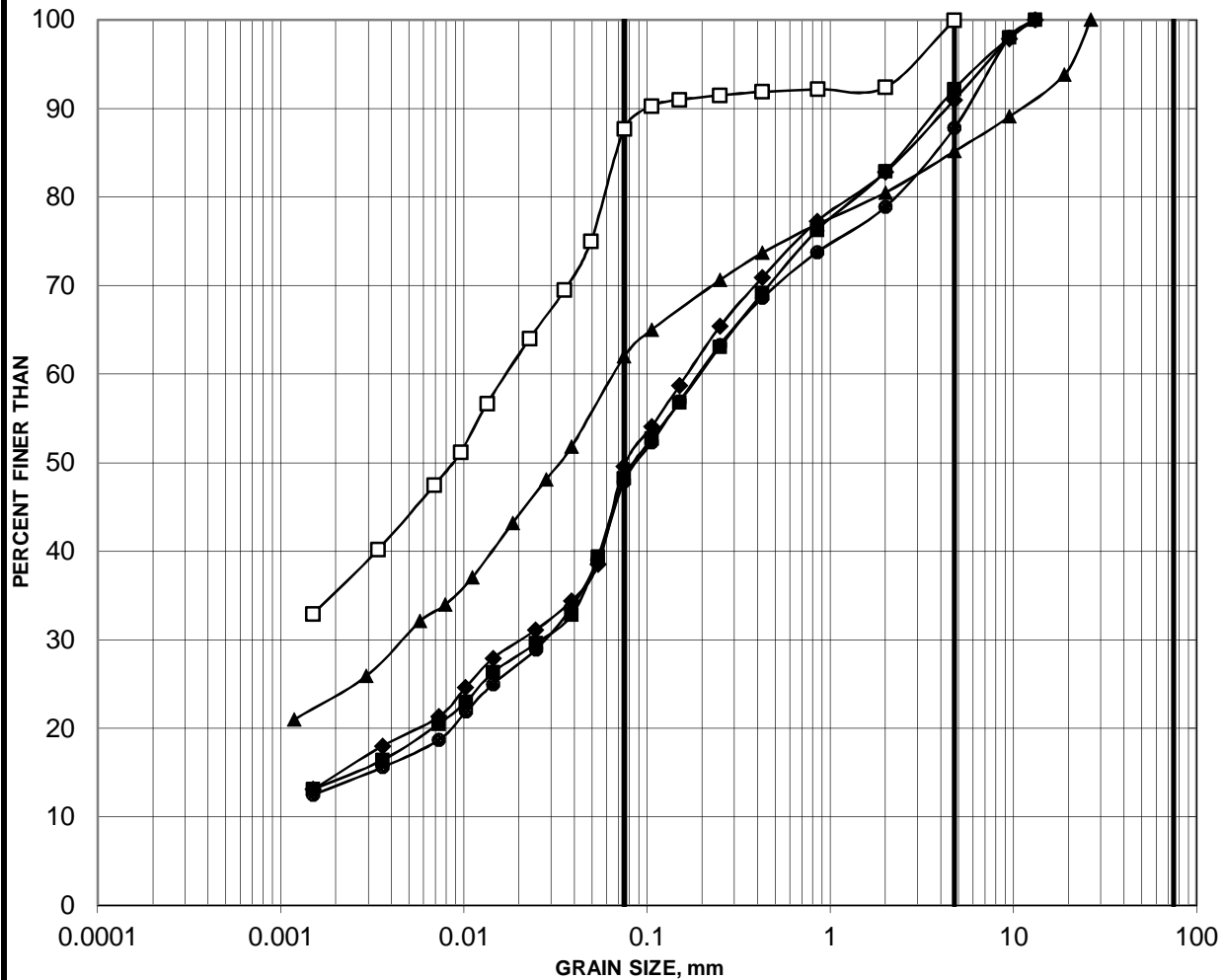
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-303	3	1.52-2.13
16-304	8	6.10-6.71

GRAIN SIZE DISTRIBUTION

FIGURE 4

SILTY SAND TO SANDY SILT TO CLAYEY SILT (TILL)



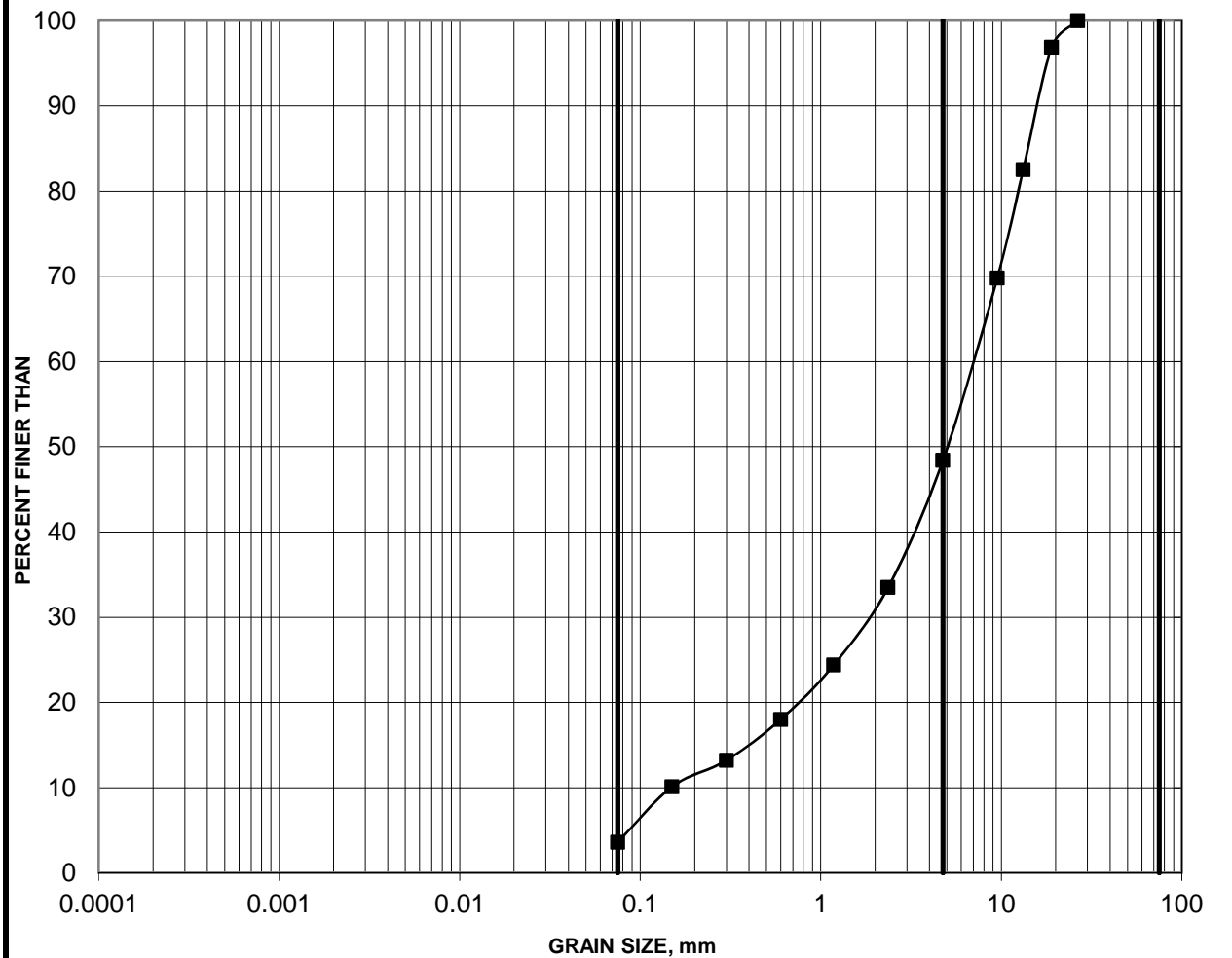
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-301	4	2.29-2.90
16-302	5	3.05-3.66
16-305	10	7.62-8.23
16-311	4	2.29-2.90
16-312	2	0.76-1.37

GRAIN SIZE DISTRIBUTION

FIGURE 5

SAND AND GRAVEL

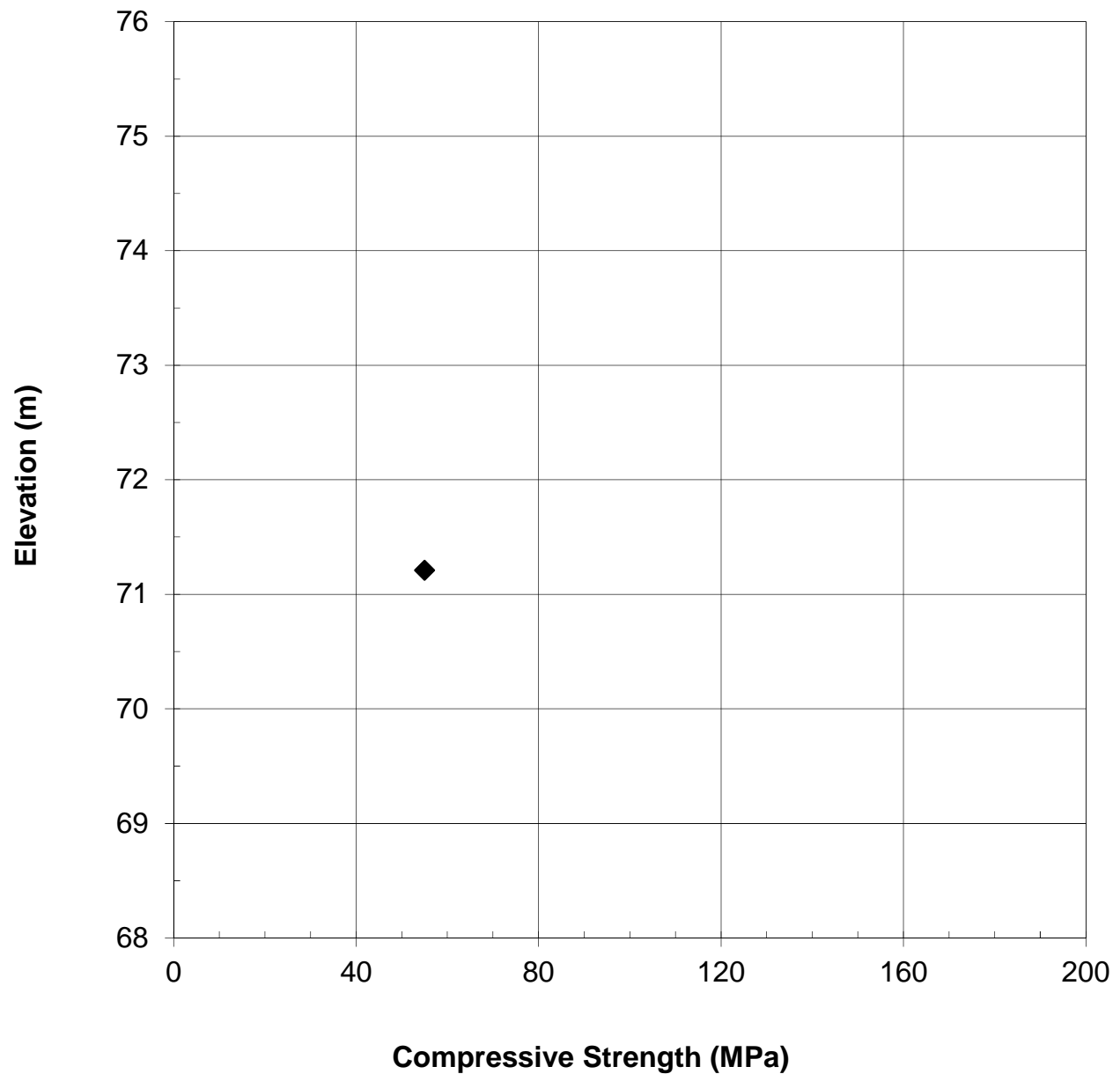


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-302	7	4.57-4.85

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

FIGURE 6





APPENDIX C

Records of Previous Boreholes 1 to 6 (Geocres No. 31G5-021)

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY AT WOODROFFE
BRIDGE No. 2

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 245.7 - GEODETIC

HOLE No. 1

REMARKS REF. B.M. No. 11-G PL. 252-3

DATE MAY 16, 1958

UNCONFIRMED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST							
							LB. HAMMER INCH DROP	NO CASING INCH DIA. ROD						
							BLOWS PER FOOT							
GROUND SURFACE							NO OVERNIGHT WATER							
				TOP SOIL	0	245.7								
				CLAY	1.5	244.7								
					2	243.2								
				LOOSE TILL	4									
		8	1-1											
					6.5	239.2								
				DENSE TILL	8									
		50	1-2		10	236.7								
				SHALY LIMESTONE (DRILLED)	12									
				CORE RECOVERY 25%	14	231.7								
				BEDDING THICKNESS 2"	16	229.2								
				(DRILLED)	18	226.7								
				SHALY LIMESTONE	20									
				CORE RECOVERY 70%										
				LIMESTONE (DRILLED)										
				CORE RECOVERY 96%										
				BEDDING THICKNESS 4"										
				BOTTOM OF HOLE										
							% WATER CONTENT							
							PLATE							
							2							

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY AT WOODROFFE
BRIDGE No. 2

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 247.9 - GEODETIC
REMARKS SEE PLATE 2

HOLE NO.

4

DATE MAY 21-22, 1958

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST			
							LB. HAMMER		NO CASING	
							INCH DROP		INCH DIA. ROD	
							BLOWS PER FOOT			
				GROUND SURFACE						
				TOP SOIL	0	247.9				
				VERY STIFF FISSURED SILTY	2	246.9				
6.0	53	5	4-1	BROWNISH-GRAY CLAY	4	243.9				
24	2.2	3	4-2	STIFF FISSURED SILTY BROWNISH-GRAY CLAY	6.5	241.4				
	2.0	8	4-3	LOOSE TILL	8					
		19	4-4	MEDIUM DENSE TILL	10	237.9				
		25 for 6		DENSE SANDY TILL	11.7					
		90	4-5	LIMESTONE (DRILLED)	16.7	231.2				
				CORE RECOVERY 95%	20					
				BEDDING THICKNESS 3"	20.8	226.1				
				SHALY LIMESTONE						
				DRILLED-CORE RECOVERY 68%	24					
				PORTION OF CORE COULD NOT BE RETRIEVED	26					
				BEDDING THICKNESS 2"	27.0	220.9				
				BOTTOM OF HOLE	28					
							← OVERNIGHT WATER LEVEL 11.3 FT.			
								</		

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY AT WOODROFFE
BRIDGE No. 2

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 247.8

REMARKS SEE PLATE 2

HOLE NO.

5

DATE MAY 22-25 1958

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST	
							LB. HAMMER INCH DROP	NO CASING INCH DIA. ROD
							BLOWS PER FOOT	
GROUND SURFACE							NO OVERNIGHT WATER	
				TOP SOIL	0	247.8		
					1.5	246.8		
6.0	5.6	6	5-1	VERY STIFF, FISSURED	2			
	5.4			SILTY	4			
4.8	4.4	4	5-2	BROWNISH-GRAY CLAY	6	241.3		
	4.2				6.5			
		2007	5-3	DENSE	8			
				SANDY TILL	10	237.3		
		115			10.5			
				SHALY LIMESTONE (DRILLED)	12			
				CORE RECOVERY 93%	14			
				BEDDING THICKNESS 2"	14			
					15.8	232.0		
				LIMESTONE (DRILLED)	16			
				CORE RECOVERY 91%	18			
				BEDDING THICKNESS 4"	20			
					21.7	226.1		
				BOTTOM OF HOLE	22			
							% WATER CONTENT	
							PLATE 6	

McROSTIE & ASSOCIATES

CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF LABORATORY TESTS

QUEENSWAY AT WOODROFFE
BRIDGE No. 2

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 247.7 - GEODETIC
REMARKS SEE PLATE 2

HOLE NO.

6

DATE MAY 20-21-1958

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PENETRATION TEST			
							LB. HAMMER		NO CASING	
							INCH DROP		INCH DIA. ROD	
GROUND SURFACE							BLOWS PER FOOT			
					0	247.7				
				TOP SOIL	1.0	246.7				
				VERY STIFF, FISSURED SILTY	2					
6.2	6.3	6	6-1	BROWNISH-GRAY CLAY	4.0	243.7				
		14	6-2	MEDIUM DENSE	6					
		13	6-3	TILL	8					
		15	6-4		10					
		3 for 2			11.2	236.5	← OVERNIGHT WATER LEVEL 11.2 FT.			
				SHALY LIMESTONE (DRILLED) CORE RECOVERY 78% BEDDING THICKNESS 2"	12					
					14					
				SHALY LIMESTONE 1" DRP - WATER LOST AT 16"-4"	15.2	232.4				
				BEDDING THICKNESS 2"	16					
				LIMESTONE BEDDING THICKNESS 2"	18					
					20					
					20.5	226.9				
				BOTTOM OF HOLE	22					
							% WATER CONTENT			
							PLATE 7			



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**

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)

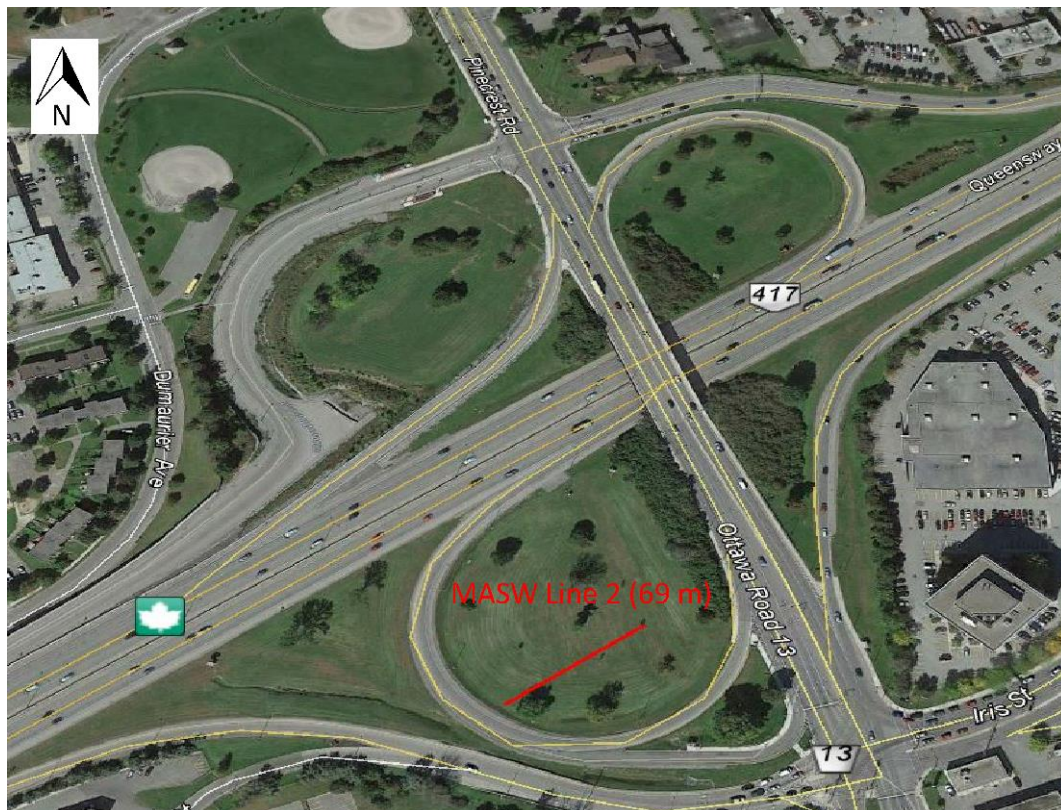


Figure 2: MASW Location Site Map at the Pinecrest site (MASW Line 2 in red)



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

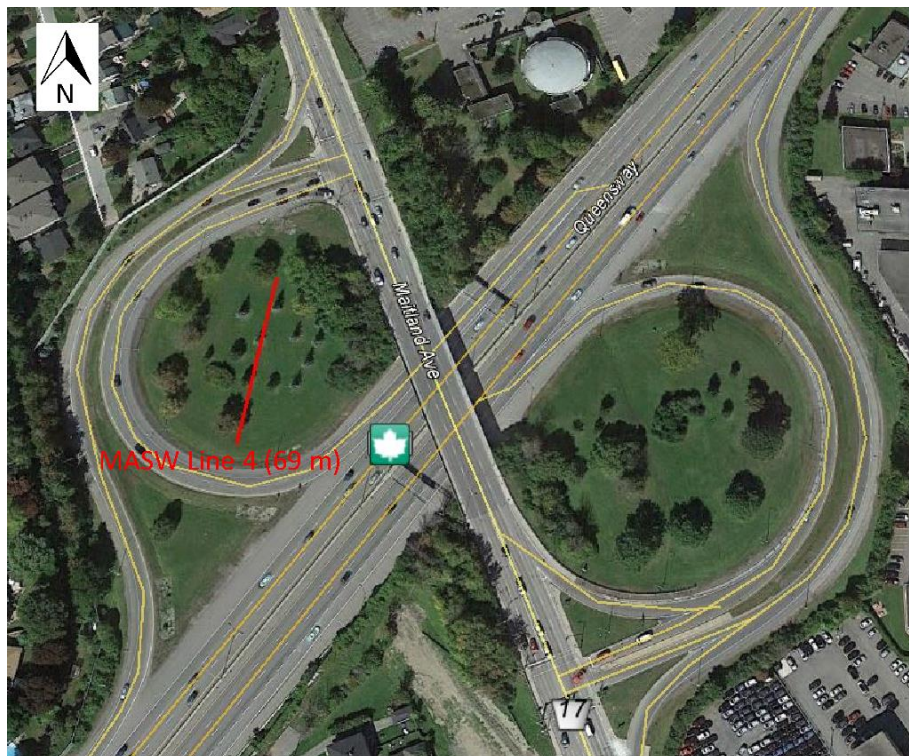


Figure 4: MASW Location Site Map at the Maitland site (MASW Line 4 in red)

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.

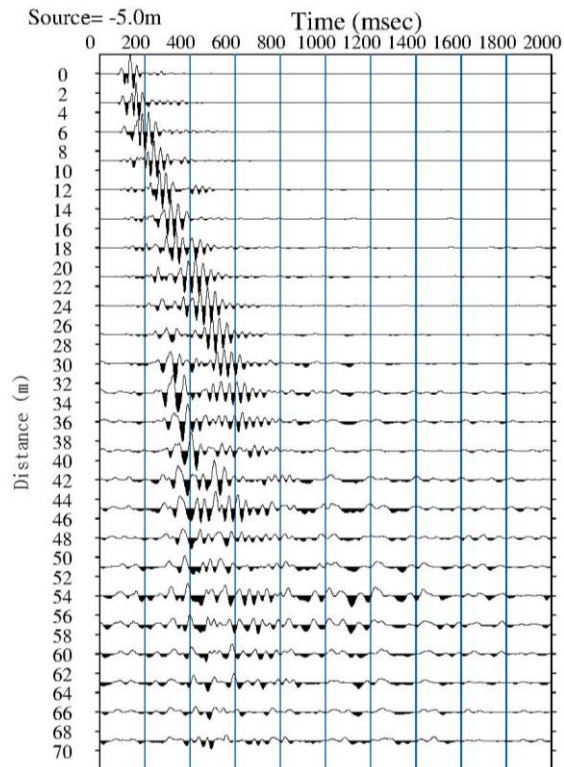


Figure 5: Typical seismic record collected along MASW Line 1

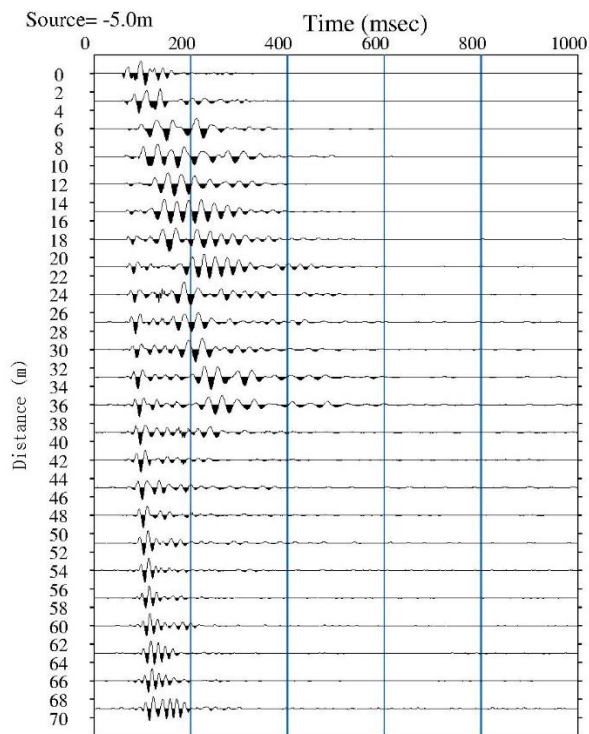


Figure 6: Typical seismic record collected along MASW Line 2

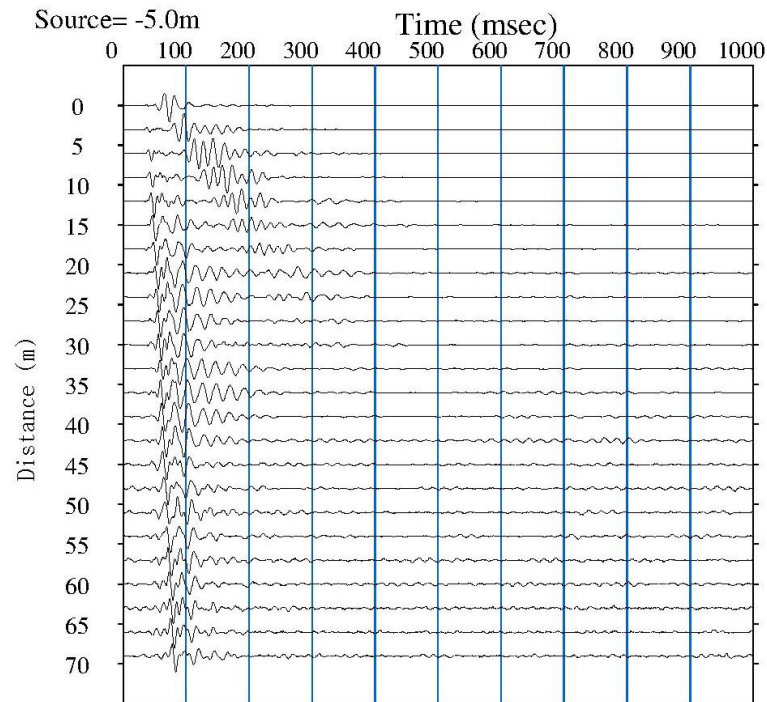


Figure 7: Typical seismic record collected along MASW Line 3

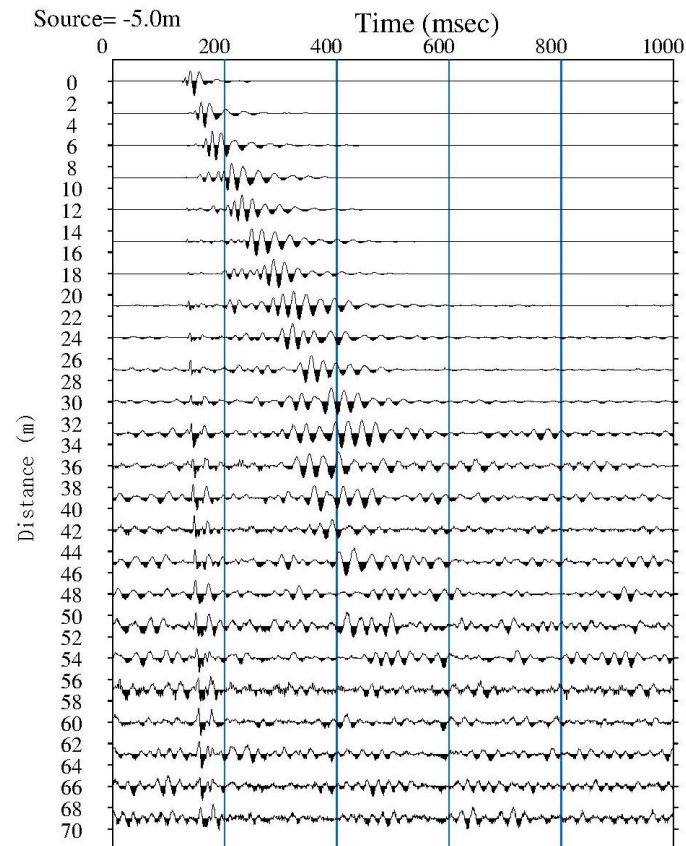


Figure 8: Typical seismic record collected along MASW Line 4

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.

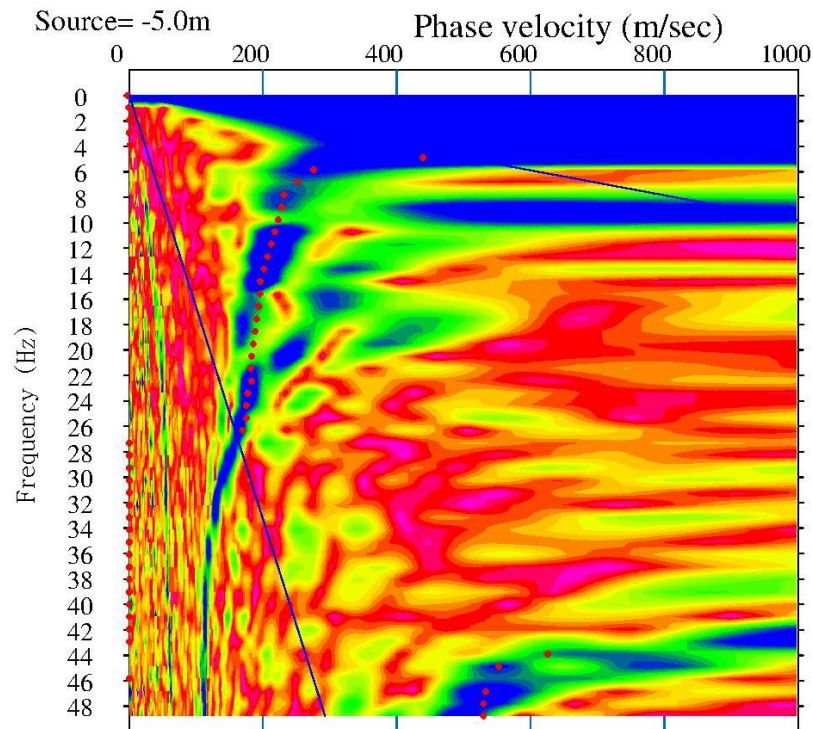


Figure 9: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1 (Richmond)

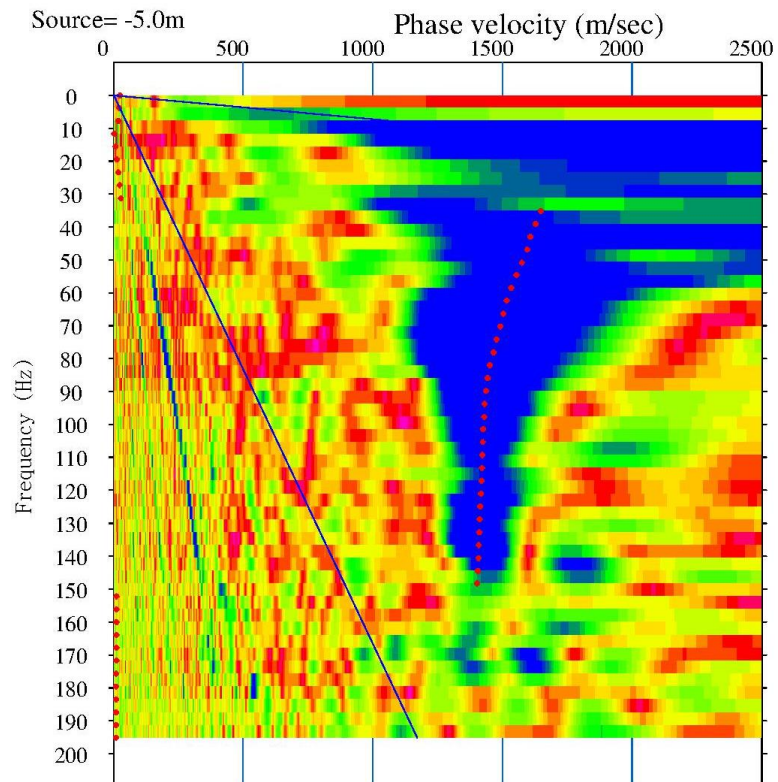


Figure 10: Active MASW Dispersion Curve Picks (red dots) along MASW Line 2 (Pinecrest).

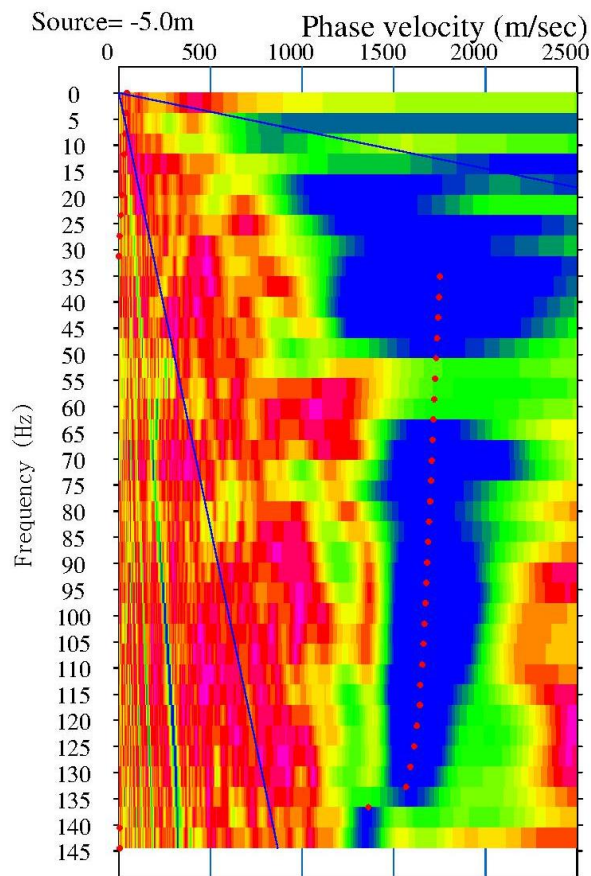


Figure 11: Active MASW Dispersion Curve Picks (red dots) along MASW Line 3 (Woodroffe).

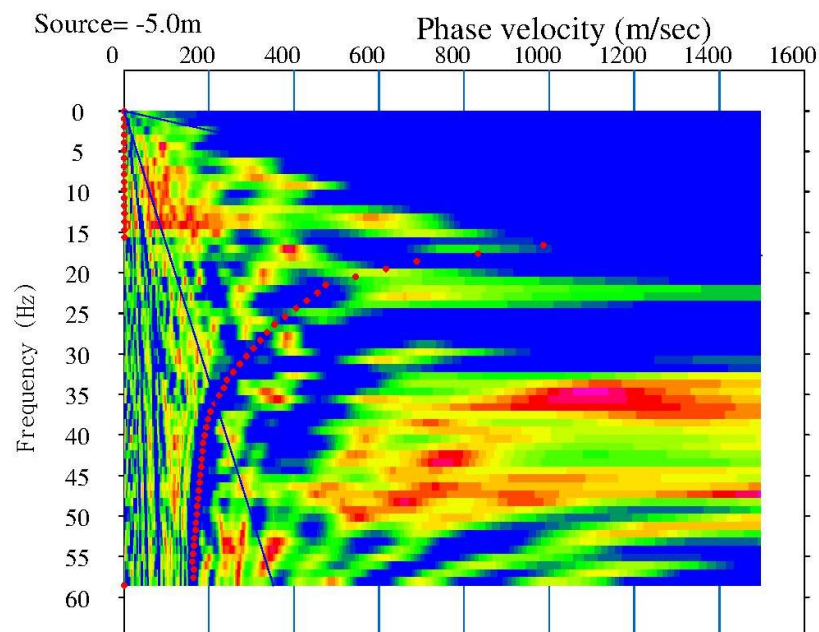


Figure 12: Active MASW Dispersion Curve Picks (red dots) along MASW Line 4 (Maitland).

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.

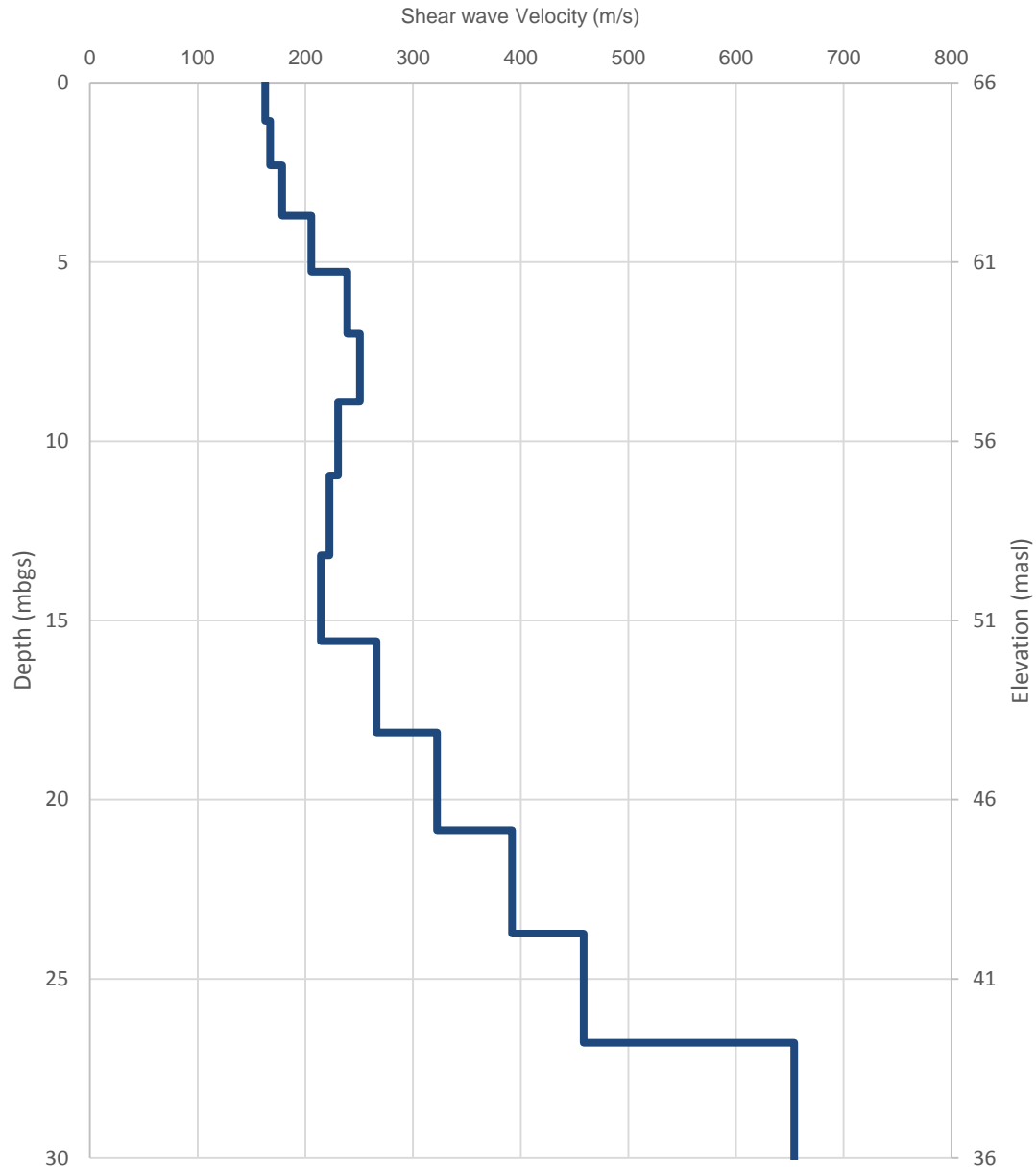


Figure 13: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

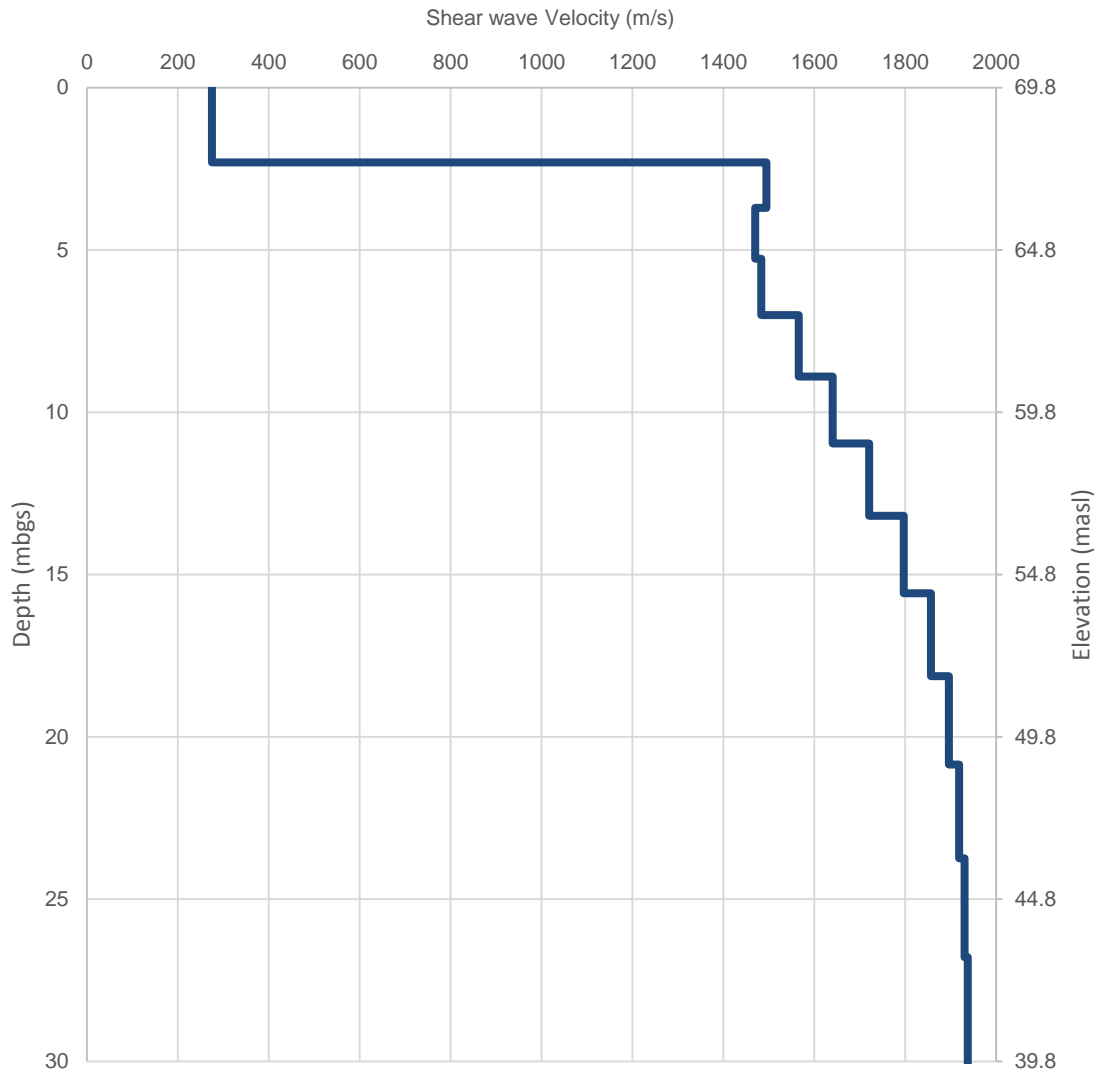


Figure 14: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 2

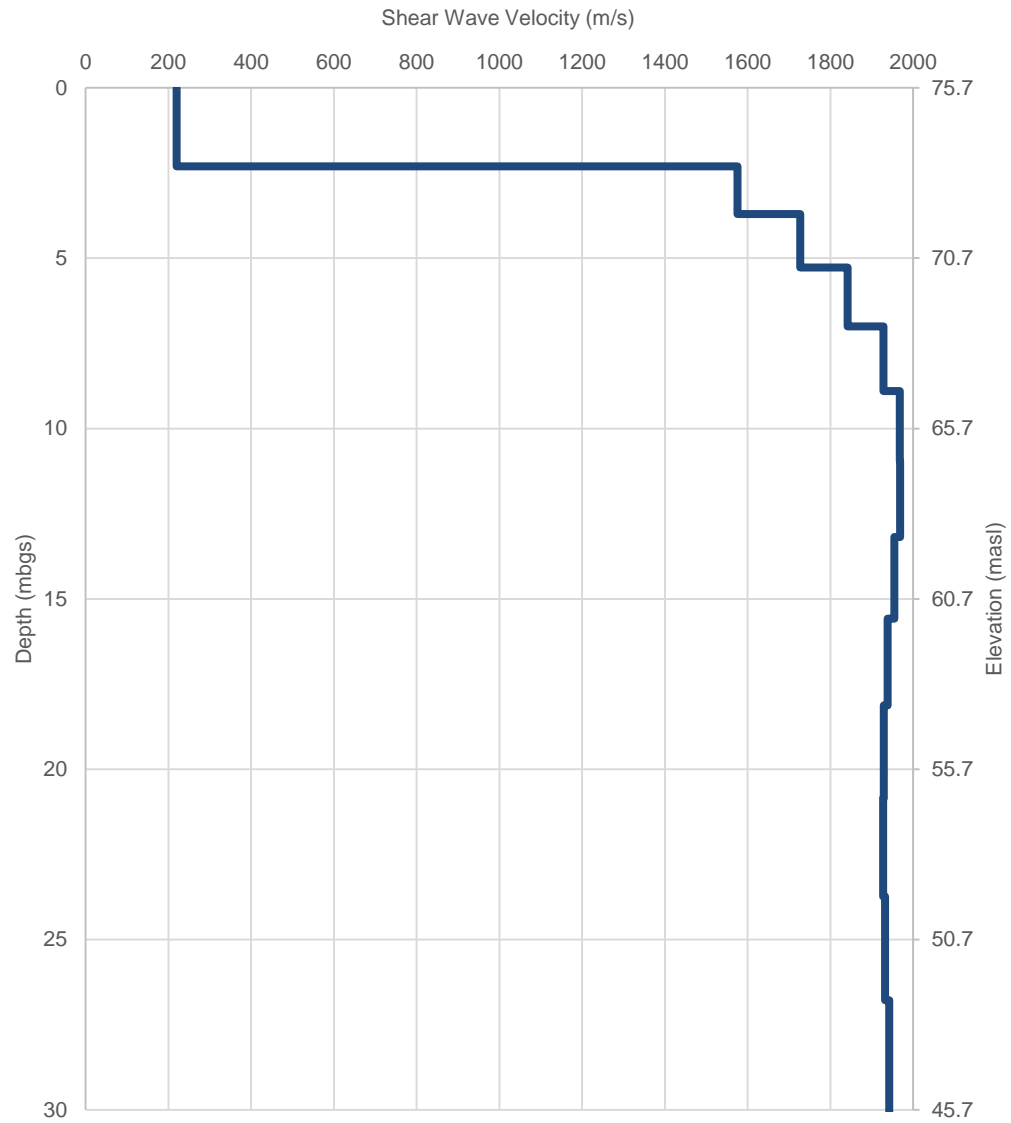


Figure 15: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 3

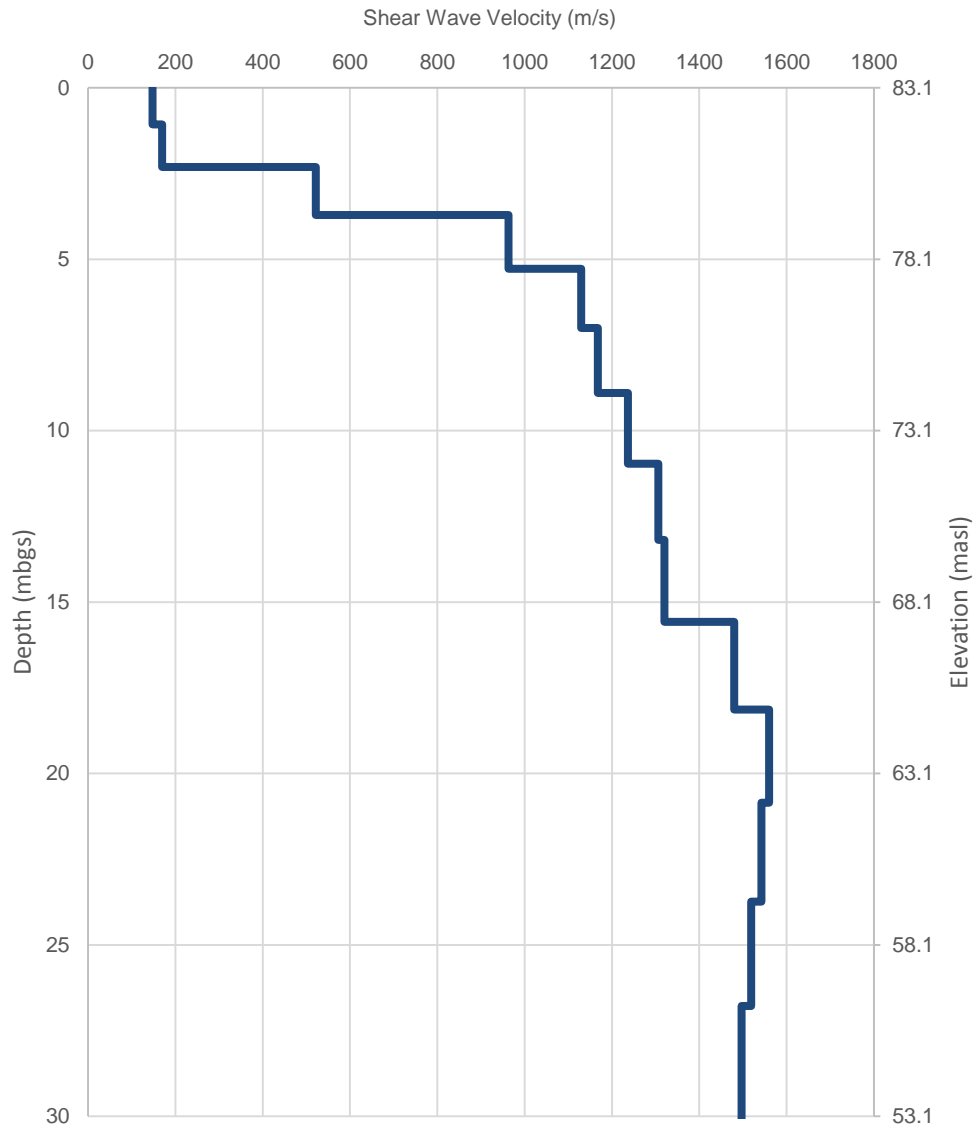


Figure 16: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 4

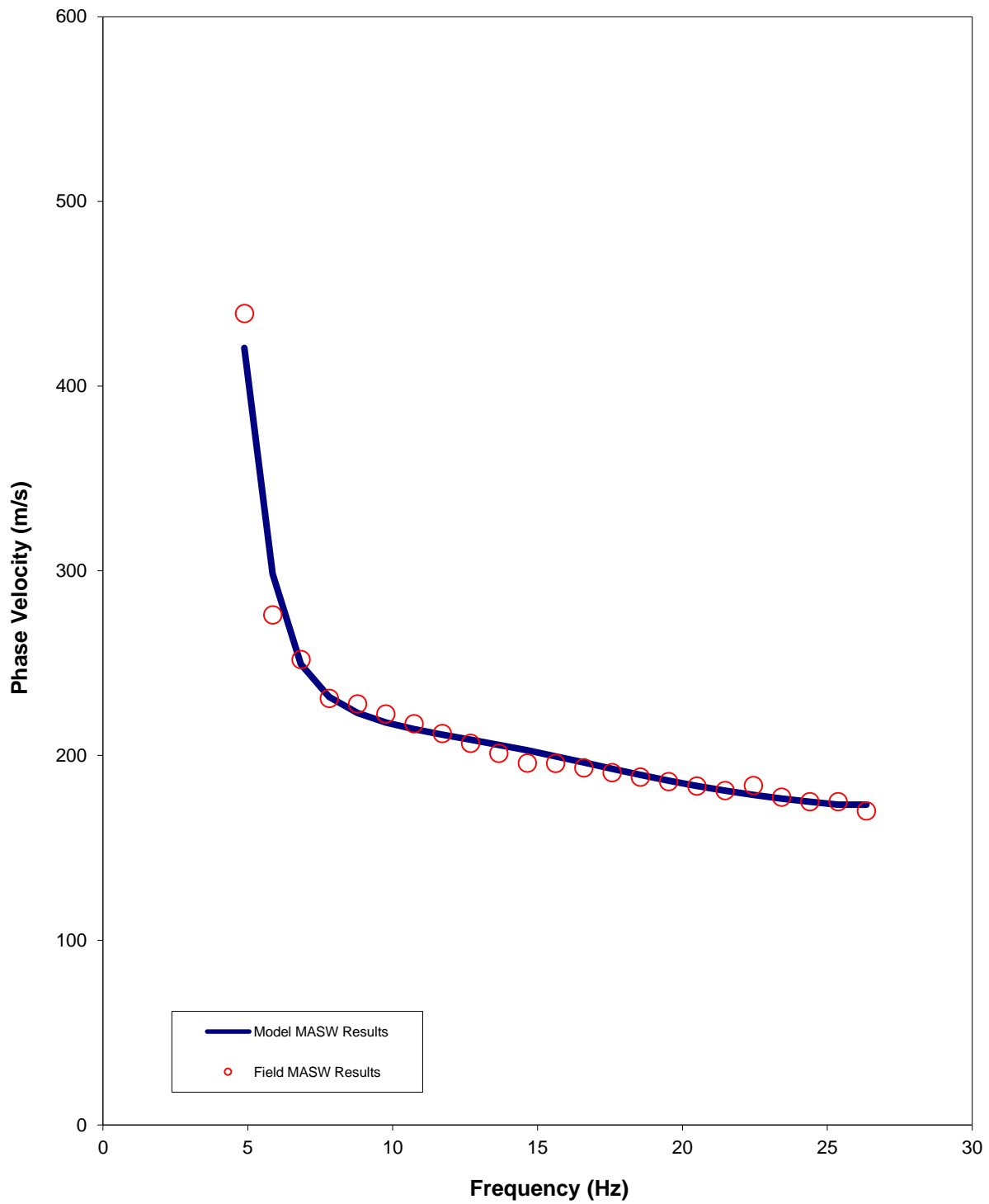


Figure 17: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

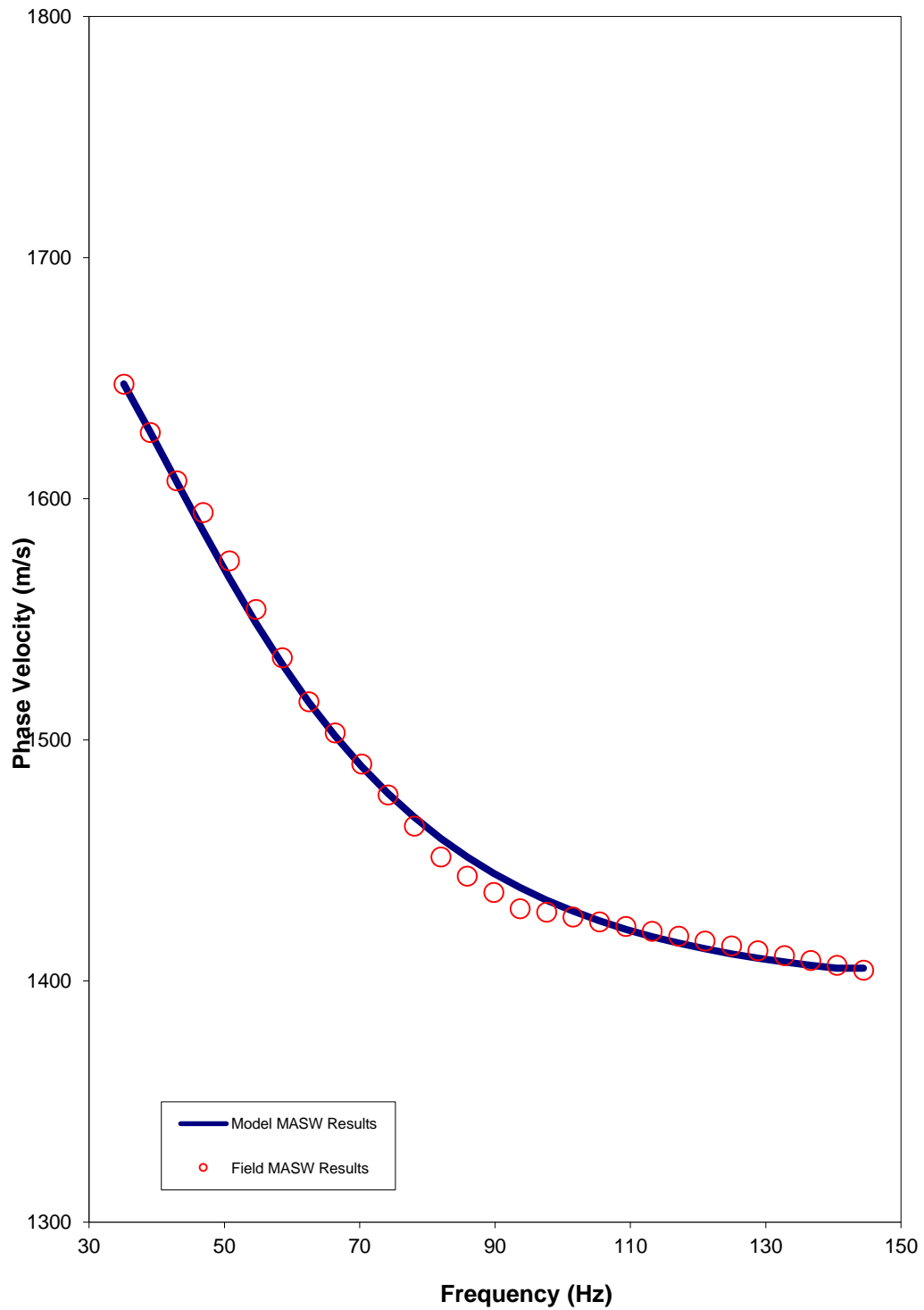


Figure 18: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 2

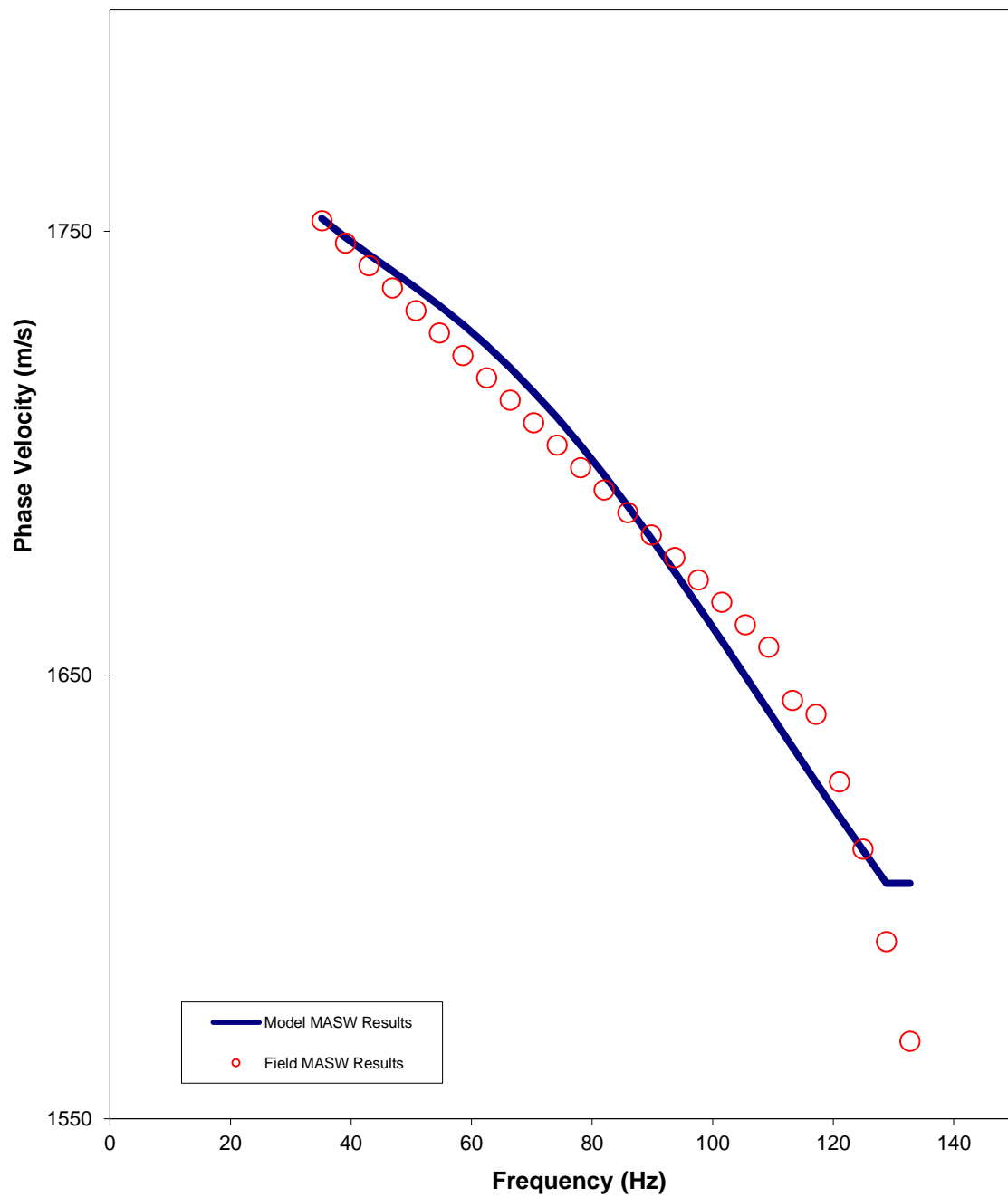


Figure 19: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

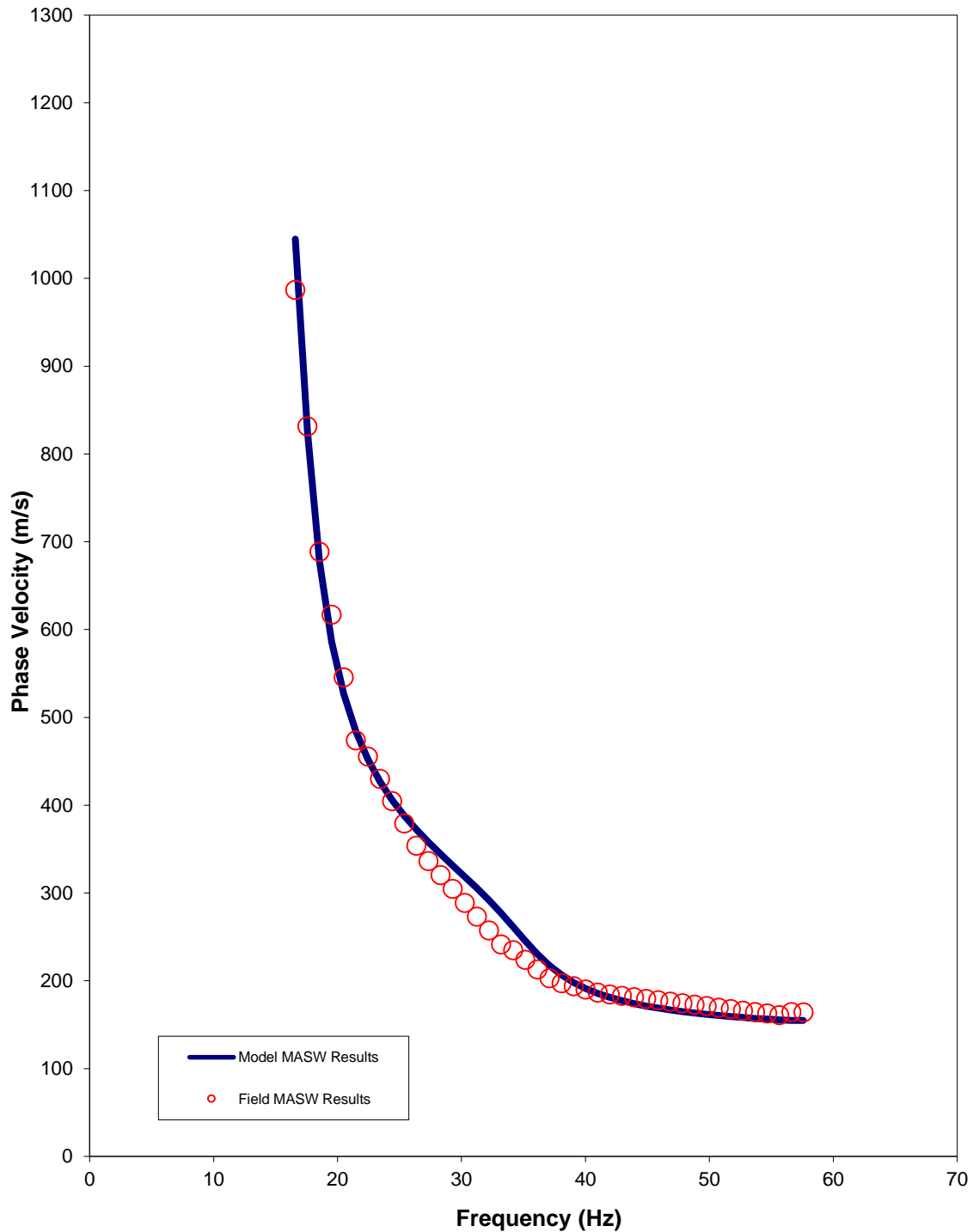


Figure 20: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

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
Vs Average to 30 mbgs (m/s)						270

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.



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Senior Geophysicist

SS/CRP/jl



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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

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

File Name: 1546542 Woodroffe Slope Stability - Retaining Wall.gsz
 Date: 2/21/2017
 Name: 1. Static Drained
 Method: Morgenstern-Price
 Direction of movement: Left to Right
 Horz Seismic Load: 0

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

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

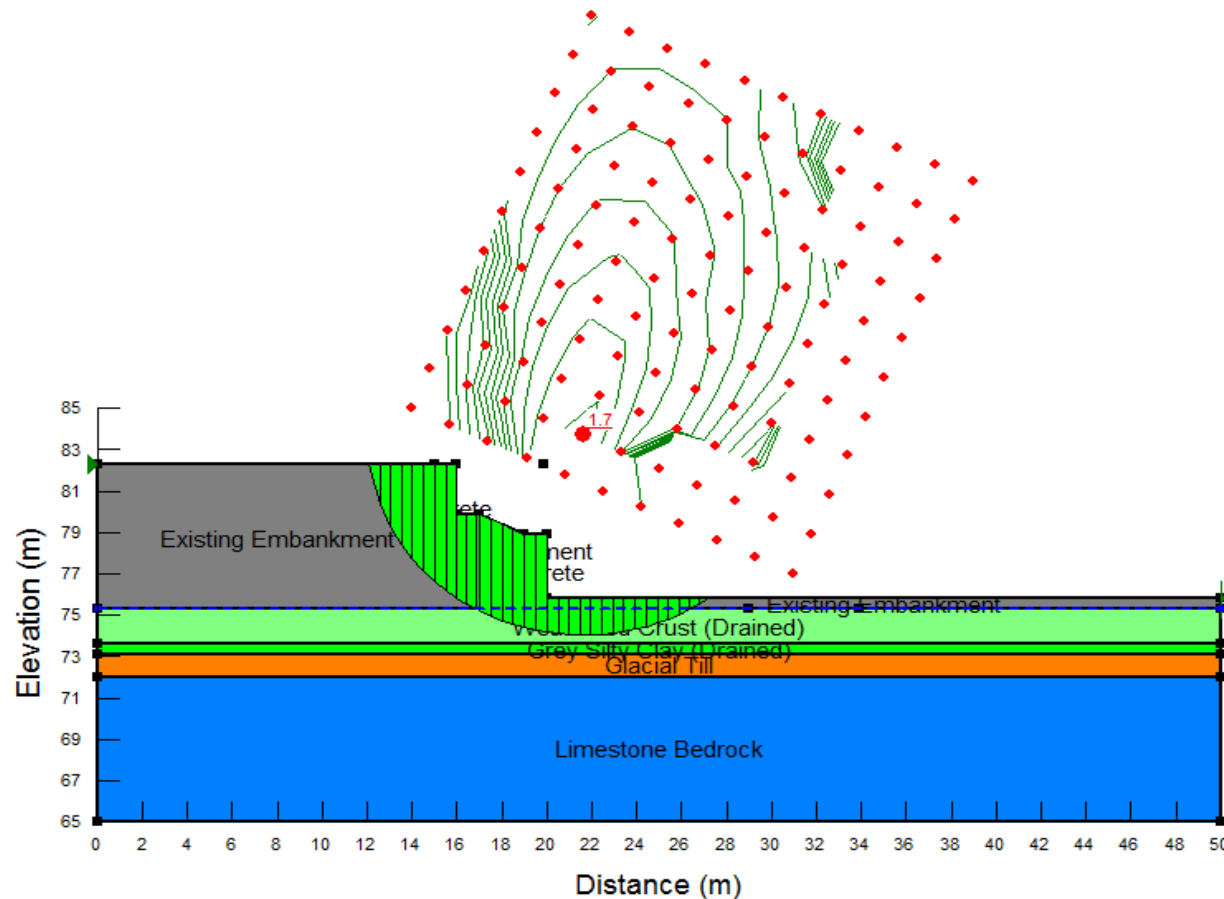
Name: Weathered Crust (Drained)
 Model: Mohr-Coulomb
 Unit Weight: 18 kN/m³
 Cohesion: 7.5 kPa
 Phi: 35 °
 Piezometric Line: 1

Name: Grey Silty Clay (Drained)
 Model: Mohr-Coulomb
 Unit Weight: 16.8 kN/m³
 Cohesion: 5 kPa
 Phi: 35 °
 Piezometric Line: 1

Name: Glacial Till
 Model: Mohr-Coulomb
 Unit Weight: 22 kN/m³
 Cohesion: 0 kPa
 Phi: 32 °
 Piezometric Line: 1

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

Name: Concrete
 Model: Mohr-Coulomb
 Unit Weight: 24 kN/m³
 Cohesion: 15000 kPa
 Phi: 90 °
 Piezometric Line: 1



Woodroffe Avenue Overpass
Retaining Wall Slope Stability Analysis Results
Ottawa, Ontario

Project No.	1546542-1030
Drawn:	WAM
Date:	2/22/2017
Checked:	KSL
Review:	ESO

Figure F2

At Golder Associates we strive to be the most respected global company providing consulting, design, and construction services in earth, environment, and related areas of energy. Employee owned since our formation in 1960, our focus, unique culture and operating environment offer opportunities and the freedom to excel, which attracts the leading specialists in our fields. Golder professionals take the time to build an understanding of client needs and of the specific environments in which they operate. We continue to expand our technical capabilities and have experienced steady growth with employees who operate from offices located throughout Africa, Asia, Australasia, Europe, North America, and South America.

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