



May 2017

REPORT ON

FOUNDATION INVESTIGATION AND DESIGN RICHMOND ROAD UNDERPASS SITE NO. 3-039 HIGHWAY 417 WIDENING AND REHABILITATION FROM WEST OF HIGHWAY 416 TO EAST OF RICHMOND ROAD G.W.P. 4124-14-00

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REPORT





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PART A

**PRELIMINARY FOUNDATION INVESTIGATION REPORT
RICHMOND ROAD UNDERPASS
SITE NO. 3-039
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 preliminary design for the replacement of the Richmond Road Underpass at Highway 417 in the City of Ottawa. The proposed work is part of the preliminary 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 rapid replacement of the Richmond Road Underpass (MTO Structure Site No. 3-039). No widening of Highway 417 or Richmond Road will take place at this location.

The terms of reference and scope of work for the foundation investigation at the staging area 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 Richmond Road Underpass widening is contained in Table 17.8.3 of MMM's Technical Proposal for this assignment. Subsequent to that technical proposal, Golder was requested by MTO to carry out additional seismic investigations for this structure under Retainer Agreement 4014-E-0012.

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 Richmond Road Underpass is located within a mixed use (commercial-residential) area of the City of Ottawa, and is located approximately 800 meters east of the junction of Highway 416 and Highway 417. At this location, Highway 417 is a divided highway with three full eastbound travel lanes and four full westbound travel lanes, separated by a concrete median. In the westbound direction, there is a lane on the left side splitting off in preparation for the off-ramp onto Highway 416 southbound. This lane reaches full width just beyond the underpass. Additionally, there is a lane reduction in the westbound right lane, tapering off completely just beyond the underpass.

The existing Richmond Road Underpass is two separate, two span, continuous steel plate girder bridges with composite concrete decks. The spans are each approximately 35 m in length for a total length of 70 m. The central piers and the bridge abutments are supported on foundations with a combination of vertical and battered piles driven to bedrock. The front row of the abutment piles are battered towards Highway 417, while the piers have a row of battered piles in each direction. The northbound structure was built in 1959, and is supported on BP 14 x 73 (HP 360 x 108) piles, while the southbound structure was built in 1966 on BP 12 x 53 (HP 310 x 79) piles.

The existing approach embankments are about 6 metres high relative to the highway profile. The foreslopes of both the north and south abutments for the northbound structure were originally constructed at a 2 Horizontal to 1 Vertical grade extending down to the roadway shoulders. In 1966, the southbound structure was constructed with foreslopes of 1.5 Horizontal to 1 Vertical, and the foreslopes of the northbound structure were re-graded to match. In 1989, the north side foreslopes of both structures were replaced with a roughly 3.4 m tall vertical retaining wall in order to widen the 417 (WP 124-87-01). This retaining wall included sheet piles extending downwards to an elevation of 60.8 m.

Previous investigations were carried out for the design of the existing structures by McRostie & Associates (McRostie) in 1959. The results of that investigation are contained in the report titled "Foundation Investigation Highway 15 at Trans-Canada" (Geocres 31G5-008). A subsequent investigation was carried out by The Department of Highways Ontario in 1966 for the construction of the southbound structure. The results of that investigation are contained in the report titled "Foundation Investigation Report for Proposed Structure Widening, Richmond Road Underpass, Ottawa Queensway, Dist. #9" on WP 909-64 (Geocres No. 31G5-007). A third investigation was carried out in 1989 for the modification of the north abutments and the installation of a retaining wall. The report was titled: "Richmond Road Underpass, North and South Abutments Reconstruction Hwy. 416-417, WP-124-87-01, Site 3-39, District 9, Ottawa" (Geocres 31G5-151).

2.2 Regional Geology

As delineated in *The Physiography of Southern Ontario*¹, this section of Highway 417 lies on the border of the minor physiographic regions known as the Russell and Prescott Sand Plains and the Ottawa Valley Clay Plains, which lie within the major physiographic region of the Ottawa-St. Lawrence Lowland.

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.



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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.

The Russell and Prescott Sand Plain is a deltaic deposit built by the Ottawa River and its tributaries flowing into the Champlain Sea. The region is characterized by fluvial deposits of sand and fine sand, up to roughly 10 m in thickness, that overlie the Champlain Sea clay deposits.

These regions are 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.

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 November 15, 2016. During this time, two boreholes were advanced at the locations adjacent to the underpass structure and three boreholes were advanced at the locations within the proposed staging area shown on Drawing 1. Boreholes 16-101, 16-102, and 16-103 were advanced in the currently proposed 'Construction Staging Area', located in the grassy area inside the existing N-E ramp, to the southwest of the underpass. Borehole 16-105, Seismic Cone Penetration Test (SCPT) 16-104 and Augerhole 16-104 (which was pre-drilled to penetrate the pavement structure and fill in advance of SCPT 16-104) were advanced in the median of the eastbound 417, adjacent to the western edge of the structure center pier. Borehole 16-105 was advanced as deep as was possible during a complete night shift closure of the inside lane of eastbound Highway 417, while carrying out SPTs at regular depth intervals as outlined in our Understanding of Scope for the work. The boreholes were advanced using a combination of track mounted drill rigs supplied and operated by CCC Geotechnical and Environmental Drilling Ltd, and truck mounted drill rigs supplied and operated by Downing George Estate Drilling Ltd. CCC Geotechnical and Environmental Drilling Ltd. is operated out of Ottawa, Ontario, while Downing George Estate Drilling Ltd. is operated out of Grenville-sur-la-Rouge, Quebec. In addition, SCPT16-104 was advanced, after pre-drilling of the testhole location, using a truck mounted SCPT rig, supplied and operated by ConeTec out of Toronto, Ontario. The testholes were advanced to between 3.1 and 29.4 m below present ground surface.

Samples of the overburden were obtained at 0.6 m to 1.5 m intervals of depth using 50 mm outside diameter split-spoon samplers in accordance with the Standard Penetration Test (SPT) procedure. In boreholes 16-101 and 16-102, Dynamic Cone Penetration Testing (DCPT) was used to assess the density of the soils at depths below 10 m, beyond which depths no further SPT testing was carried out. No coring of bedrock was carried out in the current investigation. Monitoring wells were installed in boreholes 16-103 and 16-105 to monitor the groundwater levels at the site. The monitoring wells consist of 50 mm outer diameter PVC tubing with a 1.5 m long slotted tip. Boreholes were backfilled with bentonite mixed with soil cuttings. The site conditions were restored following completion of the field work.

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

The groundwater levels in the monitoring wells in Boreholes 16-103 and 06-105 were measured on August 2 and November 23, 2016, respectively. The groundwater levels at SCPT 06-104 were also inferred based on pore water pressure measurements taken during the SCPT advance on November 14, 2016.

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



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In addition to the MASW, shear wave velocity testing was carried out as part of the Seismic Cone Penetration Testing. A built-in geophone within the cone penetration probe recorded seismic wave traces from a surface source as the penetration test was advanced. Measurements were recorded at roughly 1 m intervals from depths between 2.7 and 17.6 m. A more detailed description of the test methodology is provided in Conetec's report in Appendix E.

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.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
BH16-101	Staging Area	5022946.5	359053.8	65.2	29.4
BH16-102	Staging Area	5022997.5	359067.7	66.1	29.2
BH16-103	Staging Area	5023017.2	359135.6	66.6	9.8
SCPT16-104	Eastbound Median at Pier	5023063.1	359185.5	65.5	17.8
BH16-105	Eastbound Median at Pier	5023062.5	359180.4	65.5	15.2



4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 General

The Record of Testhole sheets, including the record for Augerhole 16-104 pre-drilled for SCPT 16-104, 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 Record of Borehole sheets from the previous investigations at the site (Geocres No. 31G5-008, 31G5-007, 31G5-151) are provided for reference in Appendix C.

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

The results of the Seismic Cone Penetration Testing are provided in the report in Appendix E and the seismic shear wave velocity (Vs) values obtained from that testing are also provided in Appendix F.

The borehole locations from the current and previous investigations are shown on Drawings 1 to 3. Interpreted stratigraphic profiles at the underpass structures, projected along Highway 417, are also shown on Drawings 1 and 2, and the interpreted stratigraphic profile at the proposed staging area is shown on Drawing 3.

The stratigraphic boundaries shown on the Record of Borehole sheets and on the interpreted stratigraphic profiles 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. The stratigraphy from previous investigations may have been altered over time as a result of man-made activities.

In general, the subsurface conditions at the underpass and staging area sites are very similar (except the depth to bedrock) and consist of a layer of pavement structure or topsoil, and fill at some locations, underlain by interlayered clay, silt and sand deposits, containing varying proportions of each material.

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, unless otherwise noted.

4.2 Pavement Structure, Topsoil and Fill

Silty sand topsoil was noted at ground surface in the staging area at boreholes 16-101, 16-102, and 16-103, with thicknesses of about 200 mm.

Silty sand fill, about 0.4 m in thickness, was encountered beneath the topsoil at borehole 16-103 in the staging area and the measured water content of one sample of the fill was 12 percent.

Asphaltic concrete was encountered at the underpass structure at Borehole 16-105 and SCPT 16-104, and is about 160 and 155 mm in thickness, respectively. Portland cement concrete, about 600 mm in thickness, was encountered at borehole 6, at the north structure approach during the 1988 investigation.

Gravelly sand and sand and gravel fill, about 1.4 m in thickness, was encountered beneath the asphaltic concrete at Borehole 16-105. SPT 'N' values within this material of 49 and 88 blows per 0.3 m of penetration indicate a dense to very dense state of packing. The measured water content of one sample of this fill was 5 percent.



Previous investigations encountered fill at some locations (i.e., Boreholes 7, 8, 10 11 from the 1966 investigation and Boreholes 5 and 6 from the 1988 investigation). The fill ranged roughly from about 1.1 to 3 m in thickness, except in borehole 6 from 1988, where the fill was about 6 m thick. The fill composition is not fully described in the 1966 records, being described only as sandy, and at Boreholes 5 and 6 from the 1988 investigation clayey silt fill and sand fill was encountered, respectively. SPT 'N' values obtained within the fill materials at the time of investigation ranged from about 8 to 38 blows per 0.3 m of penetration, indicating a loose to dense state of packing. The measured water contents ranged from about 5 to 35%.

4.3 Interlayered Silty Clay, Clayey Silt and Sandy Silt to Sand

The pavement structure, topsoil and fill is underlain by interlayered deposits of silty clay, clayey silt and sandy silt to sand. These deposits were sampled during the current investigation to depths ranging between 9.8 and 15.2 m (i.e., elevations ranging from 56.9 to 50.3 m). Where penetrated, the layer thicknesses range from about 0.2 to 3.6 m. Layers that were not penetrated were proven to have thicknesses of up to 5 m.

The continuous measurements of the Seismic Cone Penetration Testing also witness the interlayered nature of the overburden soil at this site. The results of the SCPT, provided in Appendix E, indicate that the layer thicknesses range from less than about 0.1 m to 1.2 m at that testhole location.

In general, the corrected piezocone tip resistance (q_t) recorded during advancement of the SCPT in these interlayered deposits were between about 2,000 to 5,000 kPa, but ranged up to about 15,000 kPa in the layers inferred to have a higher sand content.

4.3.1 Silty Clay to Clayey Silt

SPT's carried out within the silty clay and clayey silt gave 'N' values ranging from 3 to 14, indicating a stiff to very stiff consistency. In addition, at Borehole 16-101 a layer of silty clay with a 'N' value of "weight of hammer" was encountered, possibly indicating a firm consistency. The higher measured 'N' values may be the result of interbedding of thin sand layers within the clayey silt regions. Cone penetration testing of the silty clay and clayey silt deposits indicate undrained shear strength values generally between 100 and 250 kPa and cone tip resistances ranging from about 2,000 to 4,000 kPa.

Field shear vane tests were not carried out as a part of the current investigation but two in-situ tests carried out during the 1966 investigation measured undrained shear strengths of 65 and 100 kPa within the clayey silt.

Laboratory penetrometer testing on samples of the clayey silt and silty clay obtained during the 1959 investigation indicated that the shear strengths of those deposits may generally range between about 60 and 300 kPa, although some values potentially as low as 40 kPa were recorded.

The results of Atterberg limit testing carried out on five samples of the silty clay and clayey silt from the current investigation are summarized on Figure 1 and indicate plasticity index values ranging from 16 to 29 percent and liquid limit values ranging from 33 to 43 percent, reflecting intermediate plasticity. The measured water content of the silty clay and clayey silt generally ranges from approximately 19 to 31 percent. The measured water content of the firm silty clay encountered in borehole 16-101 had a higher water content of 40 percent. Grain size distribution testing was carried out on five samples of the silty clay to clayey silt, the results of which are provided on Figure 2.

As a part of the 1988 investigation a consolidation test was carried out on a representative sample of the silty clay to clayey silt. The results of this test indicated a preconsolidation pressure of 645 kPa with an initial void ratio of 0.79 and a compression index of 0.43.



4.3.2 Sandy Silt to Sand to Sand and Gravel

Sandy silt, silty clayey sand and silty sand to sand were encountered at relatively shallow depths of 2 to 3 m, interlayered with the silty clay and clayey silt at Boreholes 16-102 and 16-103. Silty sand to sand, about 2.1 m in thickness, overlies clayey silty sand, about 0.6 m in thickness at Borehole 16-102 and sandy silt, about 0.6 m in thickness, underlies the fill and native sand at Borehole 16-103. SPT's carried out within the sandy silt and silty sand yielded 'N' values between 4 and 8 blows, indicating loose states of packing. An 'N' value of 2 within the clayey silty sand indicates a very loose state of packing. The measured water content of one sample of the silty sand was 16 percent and the measured water content of one sample of the silty clayey sand was 28 percent. Grain size distribution testing was carried out on one sample of the clayey silty sand and two samples of the silty sand, the results of which are provided on Figures 3 and 4, respectively.

A layer of silt and sand was encountered at Borehole 16-105 at deeper depths of about 14 to 15 m. The measured water content of one sample of the silt and sand was 32 percent.

Sand, containing varying amounts of gravel and silt, and sand and gravel was encountered throughout the boreholes. The sand and sand and gravel deposits range in thickness from about 0.1 m to 5.6 m, where penetrated. The 'N' values generally range from about 10 to 29, indicating a compact state of packing. Lower 'N' values of 9 at borehole 16-101 at about 1.7 m depth, of 6 at borehole 16-103 at a relatively shallow depth of about 2.5 m and of 4 at borehole 16-105 at a deeper depth of about 5.3 m indicate layers with loose states of packing within the sand deposits. One 'N' value of 50 was also obtained within the sand and gravel deposit at borehole 16-102, indicating a compact to dense state of packing. The measured water contents ranged between 5 and 22 percent and the results of grain size distribution testing on the sand and sand and gravel are provided on Figure 5 and Figure 6, respectively.

The results of the previous investigations are generally consistent with the results described above for the current investigation, with some exceptions. Standard penetration test 'N'-values obtained within the sandy deposits during the previous investigations range from 27 to 68 blows but at Borehole 2 from the 1966 investigation (along the centre pier), three consecutive SPT's returned 'N' values of "weight of hammer" within the silty sand to sand, between about elevations 52.5 and 48.5 m, indicating a very loose state of packing. Also, Borehole 5 from the 1988 investigation (near the north abutment) indicates 'N' values of 4 and 8 and Borehole 7 from the 1959 investigation (at the centre pier) indicates 'N'-values of 7 and 10, between about elevations 46 and 52 m at both boreholes, indicating very loose to loose states of packing.

4.4 Dynamic Cone Penetration Testing

Dynamic Cone Penetration Testing (DCPT) was carried out in Boreholes 16-101 and 16-102 below 10 m depth, after completion of sampling. The blow counts during the DCPT ranged from about 15 to 124, rapidly increasing with depth below about elevation 55 m.

DCPT's were also carried out at Boreholes 1 to 3 during the 1966 investigation and at Borehole 5 during the 1988 investigation. A DCPT was also carried out at Borehole 4 during the 1966 investigation, but no sampling was undertaken at that borehole locations. The DCPT's were advanced to depths ranging from about 7 to 22 m below ground surface and the blow counts recorded ranged from about 6 to 150, generally increasing with depth.



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4.5 Bedrock

The bedrock encountered at the bridge foundation elements during the previous investigations consists of interbedded shale, sandstone and dolostone. No coring took place as a part of the current investigation.

The following table summarizes the bedrock surface depths and elevations encountered at the previously advanced Boreholes 6 to 14 (1959: Geocres 31G5-008), 1 to 3 (1966: Geocres 31G5-007), and 5 to 6 (1988: Geocres 31G5-151). The bedrock was cored in all these boreholes.

Borehole Location	Borehole Number	Ground Surface Elevation (m)	Depth to Bedrock Surface (m)	Bedrock Surface Elevation (m)
South Abutment	1 (1966)	65.8	26.8	39.0
	8 (1959)	67.5	27.6	39.8
	9 (1959)	65.5	26.5	39.0
	5 (1988)	65.9	24.1	41.8
Pier	2 (1966)	65.6	23.7	41.9
	10 (1959)	66.9	24.0	42.8
	7 (1959)	67.8	22.8	45.0
North Abutment	3 (1966)	65.9	21.1	44.8
	6 (1988)	72.5	14.6	57.9
	6 (1959)	67.8	6.9	60.9
	11 (1959)	67.9	20.9	47.0
	12 (1959)	67.8	21.7	46.1
	13 (1959)	67.8	15.0	52.8
	14 (1959)	67.7	8.7	59.0

During the previous investigations, bedrock was encountered between about 61 to 39 m elevation, (i.e., at depths ranging from about 7 to 27 m). The bedrock surface generally drops by about 2 to 3 metres from north to south across the site, except in the vicinity of the north-east corner where the bedrock appears to rise by about 15 m quite abruptly.

At the central pier, the bedrock was encountered in boreholes put down as part of the 1959 investigation (Borehole 7 and 10) and 1966 investigation (Borehole 2) at Elevations ranging from about 42 m to 45 m. The core recoveries measured at these boreholes ranged from 93 to 97 percent with the exception of the upper 0.3 m of bedrock at Borehole 7 where 60 percent core recovery was recorded. RQD was not recorded at these boreholes.

Core samples indicate that the upper 1 to 4 meters of bedrock are typically broken and weathered, with core recoveries ranging from 44 to 100 percent. RQD values recorded in borehole 5 from the 1988 investigation were 56 and 75 percent and the RQD values for two cores taken in Borehole 6 from the 1988 investigation were 8 percent.



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4.6 Groundwater Conditions

Monitoring wells were installed during the current investigation in Boreholes 16-103 and 16-105. The water levels in the wells and the open hole water levels in the boreholes without monitoring wells are provided in the table below.

Borehole Number	Borehole Location	Date	Depth (m)	Elevation (m)
5 (1988)	South Abutment	August 3, 1988	3.2 ¹	62.7 ¹
1 (1966)		March 2, 1966	1.3 ¹	64.5 ¹
8 (1959)		July 10, 1959	3.3 ¹	64.2 ¹
9 (1959)		July 23, 1959	8.2 ¹	57.3 ¹
16-104	Pier	November 14, 2016	3.6 ²	61.9 ²
16-105		November 23, 2016	3.6	61.9
10 (1959)		July 17, 1959	5.1 ¹	61.8 ¹
7 (1959)		July 2, 1959	2.9 ¹	64.8 ¹
6 (1988)	North Abutment	August 9, 1988	7.5 ¹	65.0 ¹
6 (1959)		June 30, 1959	2.7 ¹	65.1 ¹
11 (1959)		July 21, 1959	5.1 ¹	62.8 ¹
12 (1959)		July 29, 1959	2.5 ¹	65.3 ¹
13 (1959)		July 27, 1959	1.6 ¹	66.2 ¹
14 (1959)		July 24, 1959	2.8 ¹	65 ¹
16-101	Staging Area (SW)	June 20, 2016	6.3 ¹	58.9 ¹
16-102		June 16, 2016	7.9 ¹	58.2 ¹
16-103		August 2, 2016	4.6	61.9

Note: ¹ Water level measured in open borehole

² Level inferred from SCPT measurements

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.



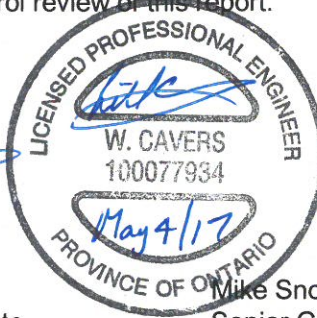
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5.0 CLOSURE

The field operations were supervised by Mr. Doug Grylls and Ms. Kim MacDonald. This preliminary report was prepared by Mr. Bill Cavers, P.Eng., and was reviewed by Mr. Mike Snow, P.Eng., a senior geotechnical engineer and Principal of Golder. Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for this project, conducted an independent quality control review of this report.

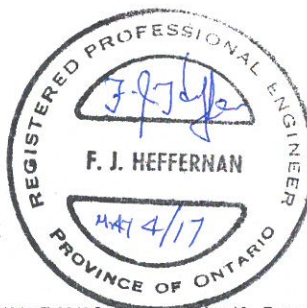
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**FOUNDATION REPORT - HIGHWAY 417
RICHMOND ROAD UNDERPASS**

PART B

**PRE-DRAFT PRELIMINARY FOUNDATION INVESTIGATION REPORT
RICHMOND ROAD UNDERPASS
SITE NO. 3-039
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 Richmond Road Underpass structure (MTO Structure Site No. 3-039) 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 aspects for the design of the proposed rapid replacement of the Richmond Road Underpass in Ottawa. This work will not include any widening of Highway 417 at this location.

Based on the General Arrangement (GA) Drawing for the existing bridges (prepared by De Leuw Cather & Co of Canada Ltd., dated July 16, 1969) provided by MMM, the grade of Highway 417 beneath the Richmond Road Underpass is at about Elevation 65.5 m and the bridge deck is at about Elevation 72.4 m.

The existing Richmond Road Underpass bridges are two span continuous steel girder bridges with a reinforced concrete deck. The spans are each approximately 35 m in length.

The original bridge, which now carries the Northbound lanes of Richmond Road, was constructed in 1961 and the underpass bridge carrying the Richmond Road southbound lanes was constructed later, in 1968.

The existing abutments, which are 'perched' foundations, and the central pier of the Northbound underpass bridge are supported on piles (BP 14x73 in size) end bearing on bedrock. The front row of piles at the abutments are battered towards Highway 417 and the outer rows of piles along the central piers are also battered outwards.

The existing abutments, which are 'perched' foundations, and the central pier of the Southbound underpass bridge are supported on smaller piles (BP 12x53 in size) end bearing on bedrock. The front row of piles at the abutments are battered towards Highway 417 and the outer rows of piles along the central piers are also battered outwards.

The existing approach embankments are about 7 m high relative to the highway profile.

This report addresses the proposed replacement of the Richmond Road Underpass (MTO Structure Site No. 3-039) only.

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



6.1 Seismic Design

6.1.1 Site Seismicity 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.

6.1.2 Seismic Site Classification

Subsurface ground conditions for seismic site characterization were established based on the results of the current field investigation and laboratory testing. The shear wave velocity results from the MASW and SCPT work were used to select a seismic site classification in accordance with Table 4.1 of the CHBDC. The shear wave velocities measured at the two locations are in general agreement ranging from about 160 m/s to 260 m/s between Elevation 65 m and 57 m. The shear wave velocities diverge between about Elevation 57 m and 52 m, where the measured values range from about 220 m/s to 315 m/s, but are in agreement below Elevation 52 m where they generally increase with depth.

Table 4.1 of the CHBDC also specifies circumstances for which a Site Class of F is applicable and a site-specific response evaluation must be carried out; the presence of liquefiable soils is one of those conditions. As presented below in Section 6.1.4, this site, at the location of the central pier, is underlain by soils which may undergo liquefaction under the design earthquake events.

As outlined in Section 4.4.3.3 of the CHBDC, where the fundamental period of the structure is less than 0.5 s then the non-liquefied site class may be used. A non-liquefied site class may also be used if the liquefaction can be prevented (i.e., by ground improvement, see Section 6.3).

Section 6.1.4 outlines the liquefaction assessment which indicates the potential for liquefaction at the central pier, while at the abutments the potential for liquefaction is mitigated as a result of the additional confinement offered by the embankments. Section 6.3 outlines a recommendation to undertake compaction grouting at the central pier to mitigate the liquefaction risk at that location.

Given that the liquefaction potential will be mitigated through a ground improvement process at the central pier, a seismic site classification of Site Class D can be used for design of the bridge.



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6.1.3 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 45.345N and longitude 75.806W), 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.

Seismic Hazard Values for Reference Ground Condition Site Class C

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)	0.095	0.153	0.267
PGV (m/s)	0.065	0.106	0.187
Sa (0.2) (g)	0.151	0.240	0.417
Sa (0.5) (g)	0.084	0.132	0.226
Sa (1.0) (g)	0.043	0.067	0.113
Sa (2.0) (g)	0.020	0.032	0.054
Sa (5.0) (g)	0.0046	0.0078	0.014
Sa (10.0) (g)	0.0018	0.0031	0.0053

The values given above are for the reference ground condition Site Class C and must be modified to the site-specific seismic site classification given in Section 6.2.2 (Site Class D) in accordance with Section 4.4.3.3 of the CHBDC. As indicated in Section 4.4.3.3 the value of PGA_{ref} for use with Tables 4.2 to 4.9 of CHBDC shall be taken as 80 percent of PGA for Site Class C where $Sa(0.2)/PGA$ is less than 2.0. Based on this requirement PGA_{ref} values of 0.214, 0.122 and 0.076 for the 2,475, 975 and 475 year return periods, respectively, were used. The corresponding site-specific seismic hazard values given in the table below can be used for preliminary design.

Seismic Hazard Values for Reference Ground Condition Site Class D

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)	0.123	0.191	0.290
PGV (m/s)	0.096	0.152	0.241
Sa (0.2) (g)	0.187	0.290	0.449
Sa (0.5) (g)	0.123	0.189	0.291
Sa (1.0) (g)	0.067	0.101	0.156
Sa (2.0) (g)	0.031	0.049	0.077
Sa (5.0) (g)	0.007	0.012	0.021
Sa (10.0) (g)	0.003	0.005	0.007



As indicated in Section 6.2.2, the fundamental period of the rehabilitated/replaced structure has yet to be confirmed and will depend on the design modifications to the superstructure and foundation elements which will be finalized during detail design, but it is understood that preliminary analysis indicates the bridge will have a fundamental period greater than 0.5 s, which in consideration of its *major route* importance category and the site-specific seismic hazard values given above, would indicate that the bridge structure falls in Seismic Performance Category 2 in accordance with Table 4.10 of the CHBDC. Based on this Seismic Performance Category and the *irregular* geometry of the bridge (since its skew angle exceeds 20 degrees), it is understood that the structure will be designed using a “performance based approach” as defined in the CHBDC.

The Seismic Performance Category and bridge design approach will need to be confirmed when the fundamental period of the structure has also been confirmed.

6.1.4 Liquefaction Assessment

Liquefaction is a phenomenon whereby seismically-induced shaking generates shear stresses within the soil under undrained conditions. These stresses tend to densify the soil (i.e., leading to potentially large surface settlements) and under undrained conditions generate excess pore pressures. The excess pore pressures also lead to sudden temporary losses in strength. Where existing static shear stresses are present, the loss of strength can lead to significant lateral movements (i.e., analogous to a slope failure) often referred to as “lateral spreading” or under certain conditions even catastrophic failure of the slope often referred to as “flow slides”. Lateral spreading and flow slides often accompany liquefaction along rivers and other shorelines.

The liquefaction susceptibility of granular soils was evaluated by comparing the penetration resistance required to trigger liquefaction with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with that presented in Section C4.6.6 of the CHBDC Commentary (as presented from Idriss and Boulanger, 2008). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out for the staging area and the bridge abutments and piers. Those analyses used the in-situ testing data collected at the boreholes advanced during the previous and current investigations adjacent to the existing abutments and central pier and within the staging area.

The design groundwater level was established based on the measured groundwater level in the standpipe piezometer installed in Borehole 16-105 at Elevation 62 m for the bridge location and in the piezometer installed in Borehole 16-103 at Elevation 62 m.

The CRR with depth was calculated at each borehole/SCPT location using the parameter, $(N_1)_{60cs}$ or $(q_c/N)_{cs}$, that is based on the values obtained in the field and corrected for various conditions (i.e., overburden stress, rod length during sampling, hammer energy efficiencies, and fines content).

The results of the liquefaction assessment indicate that the soils at the bridge abutments and staging area are not liquefiable during the design earthquake. However, the results also indicate that discrete layers of the very loose to loose silty sand and sand deposits at the central bridge piers are potentially liquefiable during the 975 and 2,475-year design earthquake. The CRR values from SCPT 16-104 and Borehole 16-105 are plotted with the CSR profiles for the 475, 975, and 2,475-year design earthquakes on Figure G1 in Appendix G. As shown on Figure G1, the overburden is potentially liquefiable between about Elevations 62 m and 49 m (i.e. depths ranging from about



3 to 16 m below existing ground surface) during the 2,475-year event, and about Elevations 55.5 m and 49.5 m (i.e., depths ranging from about 10 to 16 m below existing ground surface) during the 975-year event.

Post-liquefaction ground surface settlements are estimated to range from about 100 to 500 mm at the centre piers.

6.1.5 Consequences of Liquefaction

In summary, the potential consequences of liquefaction of the very loose to loose silty sand and sand deposits are that, following the 2,475-year earthquake, *total* vertical settlements of up to about 500 mm may occur at the ground surface at the central bridge piers. This settlement will be entirely differential with respect to the abutments, which would not settle during the design earthquake.

The estimated total and differential vertical settlements would result in significant downdrag loads, following a seismic event, on the existing pile foundations and the structural capacity of the foundations to support those loads would need to be evaluated. The deep foundations would also be subjected to lateral loading and deformation, due to both the inertial forces on the structure. Guidelines on this issue are provided in Section 6.3.

Based on discussions with the structural engineers, it is understood that the existing pile foundations may not have either the vertical or lateral capacity to withstand the additional loading imposed during the design earthquake as a result of the liquefaction of the soils. However, this is still to be confirmed.

6.2 Foundations – General

The existing Richmond Road Underpass bridges are two span continuous steel girder bridges with a reinforced concrete deck. The spans are each approximately 35 m in length. The original bridge, which now carries the Northbound lanes of Richmond Road, was constructed in 1961 and the underpass bridge carrying the Richmond Road southbound lanes was constructed later, in 1968.

The existing abutments, which are 'perched' foundations, and the central pier of the Northbound underpass bridge are supported on piles (BP 14x73 in size) end bearing on bedrock. The piles at the abutments were installed in 2 rows, with 12 piles in the front row (i.e., closest to the travelled highway lane) and 9 in the back row. There are 45 piles, installed in three rows of 15 piles, at the centre pier. The front row of piles at the abutments are battered towards Highway 417 at 1H:3V and the outer rows of piles along the central piers are also battered outwards at 1H:6V.

The existing abutments, which are 'perched' foundations, and the central pier of the Southbound underpass bridge are supported on smaller piles (BP 12x53 in size) end bearing on bedrock. The piles at the abutments were installed 2 rows, with 6 piles in the front row (i.e., closest to the travelled highway lane) and 9 in the back row. The central pier has an irregular shape, with a wider footprint over the eastern third of the pile cap, which has four rows of 2 piles, instead of the 3 rows of 6 piles over the remainder of the pier footprint. The front row of piles at the abutments are battered towards Highway 417 at 1H:3V and the outer rows of piles along the central piers are also battered outwards at 1H:6V.

Guidance is provided below for foundation options and assessment of the existing bridge foundations.



6.3 Foundation Options

Based on the conditions encountered and the results of the assessment, the existing abutment foundations are likely suitable for supporting the replacement bridge decks, pending confirmation by the structural engineers. However, the piles supporting the central piers at each bridge will experience additional downdrag loading and lateral forces, due to liquefaction resulting from a seismic event, and the existing piles at the central piers may not be capable of withstanding those additional loads (also to be confirmed by the structural engineers). The remainder of this report therefore addresses the foundation options at the central piers only.

The foundation options for the central piers and a summary of the disadvantages and advantages for each option, is provided below. The options provided assume that mitigation to address the potential liquefaction impacts will be required at the central piers. A comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **Underpinning of existing foundations:** Deep foundation elements (i.e., driven steel H-piles or micro-piles) driven to refusal on the limestone bedrock or socketed into the rock could be feasible for underpinning of the central piers to provide additional foundation capacity. Typical H-pile types or micro-piles may not have sufficient stiffness in order to resist the expected seismic lateral loads associated with liquefaction for this site (i.e., a larger more robust pile type would likely be required and micro-piles with sufficient lateral resistance may not be available). It would also be difficult, although it is conceptually possible, to install these deep foundation elements without removal of the bridge decks which would result in significant and prolonged traffic disruptions, effectively eliminating rapid replacement of the bridge decks. Liquefaction, associated downdrag loads and low lateral resistance would remain considerations.
- **Construction of new central piers:** Deep foundation elements (i.e., driven steel H-piles) driven to refusal on the limestone bedrock would be feasible for construction of new central piers for support of the bridge deck. Typical H-pile types may not have sufficient stiffness in order to resist the expected seismic lateral loads associated with liquefaction for this site (i.e., a larger more robust pile type would likely be required). In addition, providing rock sockets for toe 'fixity' of such piles can be quite challenging. Furthermore, H-Piles may susceptible to weak axis buckling through the liquefiable layers under seismic loading. Removal of the bridge decks during the construction period for the piers would be required and would result in significant and prolonged traffic disruptions, effectively eliminating rapid replacement of the bridge decks.
- **Ground Improvement – Compaction grouting:** Compaction grouting involves drilling holes (about 250 to 300 mm in diameter) to the required depth and then pumping a cement based low slump grout under relatively high pressures (i.e., up to 3,500 kPa) in stages to compact the soil between the relatively tightly spaced grout columns. This may be a suitable technique at this location, possibly in combination with vertical drains, since it is possible to carry out this operation below the pile caps as required and the operation can be managed to target specific depths. Compaction grouting can also be carried out with the bridge deck in place, although it will be slower.
- **Ground Improvement – Vertical Drains:** Vertical drains could be installed and below the pier pile caps to relieve the pore water pressures induced during a seismic event. The use of vertical drains is not a widely accepted technique for liquefaction mitigation, particularly in North America, and is more suited for liquefaction mitigation in relation to slope instability. The drains could be relatively easily installed however,



but the design of the drains and construction of the drains to provide acceptable and effective long term performance is difficult and not well understood. While vertical drains are potentially a very economical solution, the risks of unacceptable performance during a seismic event are relatively high in comparison to other alternatives.

- **Ground Improvement – Dynamic Compaction:** Dynamic compaction involves using large weights dropped from a fixed height to compact the soil. This is not a suitable technique at this location since the technique can induce downdrag on existing piles during construction; would not result in ground improvement under the pier pile caps; could de-structure the clayey soils layer, leading to long term post-construction settlements; and, is not effective to the required depths (about 15 m). This work could also not be carried out without removal of the bridge decks. Furthermore, this technique is not well suited to soils with high fines contents.
- **Ground Improvement – Stone Columns:** Stone columns, consisting of bored or driven columns of compacted aggregate installed within the affected soil, are not considered to be suitable for this site. They cannot be installed below the existing pile caps, as would be needed, and the depth of improvement required is greater than the depths to which these elements can be installed effectively. Furthermore, this technique is not well suited to soils with high fines contents.
- **Ground Improvement – Deep soil mixing:** Deep soil mixing involves driving a cutter to the required depth and then mixing the native soil with cement as the cutter is withdrawn. This is not a suitable technique at this location since it would be very difficult to carry out this operation below the pile caps as required (and may not be possible) and the equipment is large and cumbersome so this work could also not be carried out without removal of the bridge decks.
- **Ground Improvement – Jet grouting:** Jet grouting involves driving a specialized drill head to the required depth and then eroding and mixing the native soil with cement, using a combination of water and air injection, as the drill head is withdrawn. This may be a suitable technique at this location since it is possible to carry out this operation below the pile caps as required and the operation effectively treats the entire soil volume at the required depths. However, the equipment is large and cumbersome so this work could likely also not be carried out without removal of the bridge decks. This technique also results in a significant volume of spoil that is difficult to manage effectively and without some spillage, particularly immediately next to highway lanes carrying a large volume of traffic moving at speed.

Based on the above considerations, the preferred option from a geotechnical/foundations perspective may be to carry out ground improvement using compaction grouting, possibly with wick drains. Conceptually, the compaction grouting would need to be designed to densify the soil underneath the pier pile caps and surrounding the existing piles to a depth of about 18 m and extend at least 9 m from the pile cap edge outwards. Further guidance can be provided as the design progresses. Successful compaction grouting would eliminate the potential for liquefaction, the liquefaction induced downdrag and result in higher lateral soil resistance (i.e., p-y curves) along the piles.

A specialty contractor would need to be retained to further assess the feasibility and costs for ground improvement and to design and carry out the improvement works.



6.4 Assessment of Existing Foundations

6.4.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the 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 Section 6.3 below.

6.4.2 Geotechnical Capacity – Existing Piles

Based on the available design drawings, Geocres information, and anticipated subsurface conditions, the factored axial geotechnical resistance at Ultimate Limit States (ULS) for the BP 12x53 (HP 310 x 79) steel H-piles driven to bedrock at the southbound structure may be taken as 1,250 kN.

Similarly, the factored axial geotechnical resistance at Ultimate Limit States (ULS) for the BP 14x73 (HP 360 x 108) steel H-piles driven to bedrock at the northbound structure may be taken as 1,500 kN.

SLS resistances would not apply to the piled foundations based on the assumption that they are driven to found on the bedrock.

6.4.3 Downdrag (Negative Skin Friction)

The post-liquefaction settlement will be in the order of 100 to 500 mm. The elastic shortening of the piles will likely be less than 5 mm under service loads, and therefore the differential settlements would be sufficient to generate downdrag forces.

In calculating the magnitude of the potential downdrag forces due to liquefaction, the methods described in both the Canadian Foundation Engineering Manual (using a combination of the “ β method” for cohesionless soils and the “ α method” for cohesive soils outlined in Section 18.2.1 of the CFEM) as well as the US Transportation Research Board’s report “Design and Construction Manual For Downdrag on Uncoated and Bitumen-Coated Piles” [Briaud and Tucker (1994)] were considered.

The soil strengths used in the downdrag analyses are estimated however based on the post-liquefaction residual strength using the methods outlined in Section C4.6.6.1 of the Commentary on CSA S6-14, CHBDC. Considering the settlement of the silty clay deposit versus the elastic shortening of the pile, the neutral plane used in those analyses was assumed to be at the bottom of the lowest layer that would undergo liquefaction under the design earthquake. Based on the above, and assuming an underside of the pile cap of about Elevation 63 m, the unfactored downdrag load acting on a single existing BP 12x53 or a BP 14x73 pile, at the central piers, is estimated to be up to about 1225 kN after a design earthquake with a 975 year return period and about 275 kN after a design earthquake with a 2,475 year return period. The downdrag is greater for the design earthquake with the lower return period since only the lower portions of the soil deposits liquefy under that reduced shaking and the upper portions of the soil deposits in contact with the pile retain their strength and therefore are able to transfer greater downdrag loading to the pile.

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



If ground improvement is undertaken that eliminates the risk of liquefaction, then downdrag will not occur and no additional loading will be applied to the piles.

6.4.4 Pile Uplift Resistance

The uplift resistances of the existing piles were calculated using a similar methodology to that described above for downdrag, under a liquefied state, and include the shaft resistance available below the depth of liquefaction. The following unfactored ULS uplift resistances may be assumed for the existing piles:

Foundation Element	Unfactored Uplift Resistance (kN)	
	BP 12x53 (HP 310 x 79)	BP 14x73 (HP 360 x 108)
Central Pier	590	695
North Abutment	775	915
South Abutment	3,230	3,820

The uplift resistance at the central pier indicated in the above table assumes that liquefaction occurs during the 2,475-year design earthquake event. If ground improvement is implemented at the central pier, the uplift resistance will need to be evaluated based on the type of ground improvement and the density and strength of the resulting soil matrix.

In accordance with the CHBDC, a resistance factor of 0.3 is to be applied to the values in the above table.

6.4.5 Pile Lateral Resistance

The foundation lateral soil-structure interaction springs required for the dynamic analysis of the bridge pier and abutments were computed based on the available subsurface information on the soil layers surrounding the foundations and the pile dimensions.

The soil-structure interaction between the bridge foundations and the surrounding soils was modeled using the load transfer method. The lateral load-displacement behaviour of the piles can be modeled using p-y curves (CFEM, 2006). P-y curves relate the lateral deflection of a single pile to the corresponding soil and bedrock reactions at any depth below ground surface. The p-y curves were generated internally using the commercially available software programs LPILE Plus (Version 5.0.29), produced by ENSOFT Inc.

For all loading conditions, a pinned connection was assumed between the pile head and the pile cap. Cyclical loading conditions were assumed for all the lateral analyses; therefore, cyclic p-y curves were generated for all foundations as described in the LPILE Plus (Version 5.0) Technical Manual (2004) which is based on the studies of Wang (1982) and Long (1984). Where appropriate, post-earthquake residual shear strengths were assigned to the soil layers that were identified as potentially liquefiable as recommended in Section C4.6.6.2 of the Commentary to the CHBDC.

The family of cyclic p-y curves calculated at 0.5 to 1.0 m increments of depth for a single, vertical 360x108 steel H-pile and a single, vertical 310x79 steel H-pile (equivalent respectively to BP 14x73 and BP 12x53 H-piles) at the pier (assuming liquefaction under the design 2475-year event) are shown in tabular format and graphically in Appendix H. The cyclic p-y curves for the pier (assuming no liquefaction) and at the abutments are provided in Appendix I. If ground improvement is implemented at the central pier, p-y curves will need to be estimated for the resulting 'improved' soil matrix.



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For piles arranged in closely spaced groups, the pile-soil-pile interaction causes the individual piles in a group to be less effective than a single pile. These “group effects” can be incorporated into the design using a method that modifies the single pile p-y curves by some factor (i.e. a p-reduction factor). Generalized p-multipliers (i.e. p-reduction factors) for a range of pile spacings are provided in Section C6.11.3.4 of CHBDC.



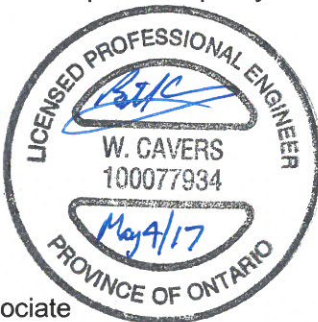
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7.0 CLOSURE

This report was prepared by Mr. Bill Cavers, P.Eng., and was reviewed by Mr. Michael Snow, P.Eng., a senior geotechnical engineer and Principal of Golder. Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for this project, conducted an independent quality control review of this report.

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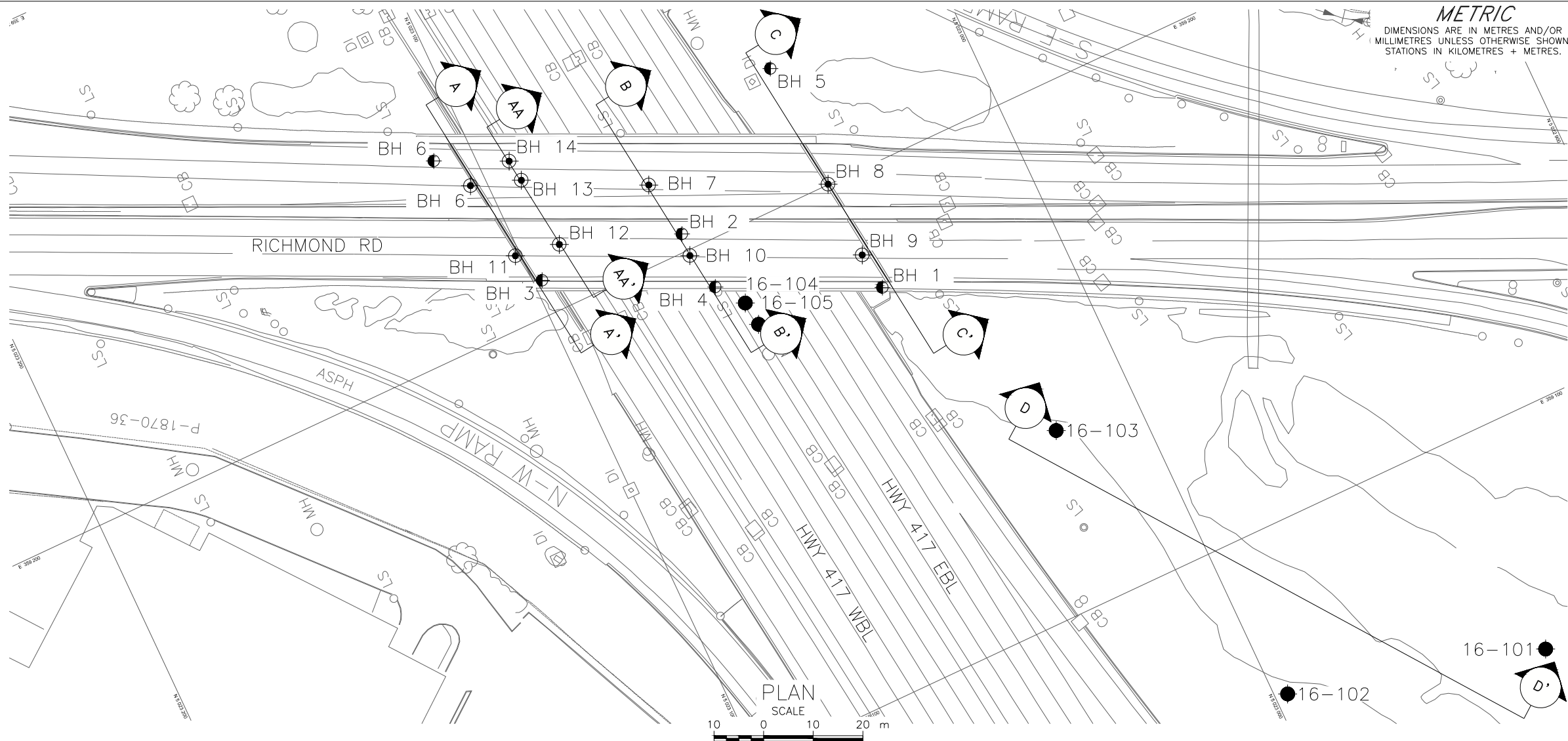
TABLE 1 – COMPARISON OF BRIDGE CENTRE PIER FOUNDATION ALTERNATIVES

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/ Risks
Underpinning of Existing Foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Structural solution that is possibly less risky than ground improvement. Compatible with existing bridge foundations. 	<ul style="list-style-type: none"> Difficult to carry out with the bridge deck in place. 	<ul style="list-style-type: none"> May be more expensive than ground improvement considering the costs of carrying it out with the bridge deck in place or removing the deck and detouring traffic. 	<ul style="list-style-type: none"> Extended periods of traffic closures/diversions on both Richmond Road and Highway 417. May be difficult to drive piles (if bridge deck left in place) and achieve design pile resistance.
Construction of Replacement Piers with New Foundations	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> New construction that does not rely on existing components. 	<ul style="list-style-type: none"> Would require removal of bridge decks at each bridge for construction. 	<ul style="list-style-type: none"> Likely more expensive than ground improvement or underpinning considering the costs of removing the deck and detouring traffic. 	<ul style="list-style-type: none"> Extended periods of traffic closures/diversions on both Richmond Road and Highway 417.
Ground Improvement – Compaction Grouting	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> Improved soil strength that reduces the downdrag and lateral loading forces due to liquefaction. Can treat targeted depths without treating all soils above that depth Negligible settlement. Compatible with existing bridge foundations. 	<ul style="list-style-type: none"> Difficult to carry out with bridge decks in place Higher risk of inadequate performance than with underpinning or new foundations. 	<ul style="list-style-type: none"> May be less expensive than underpinning or new foundations. 	<ul style="list-style-type: none"> Requires a specialist contractor and extreme care to achieve design performance. Pre and post-construction verification testing required.
Ground Improvement – Vertical Drains	<ul style="list-style-type: none"> May Be Feasible 	<ul style="list-style-type: none"> Relatively easy to install with bridge decks in place. Compatible with existing bridge foundations. 	<ul style="list-style-type: none"> Not widely accepted as effective liquefaction mitigation, particularly in North America. Difficult to design and maintain for effective long term performance 	<ul style="list-style-type: none"> Likely lowest cost option. 	<ul style="list-style-type: none"> Requires a specialist contractor and extreme care to achieve design performance.
Ground Improvement – Dynamic Compaction	<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 RICHMOND ROAD UNDERPASS

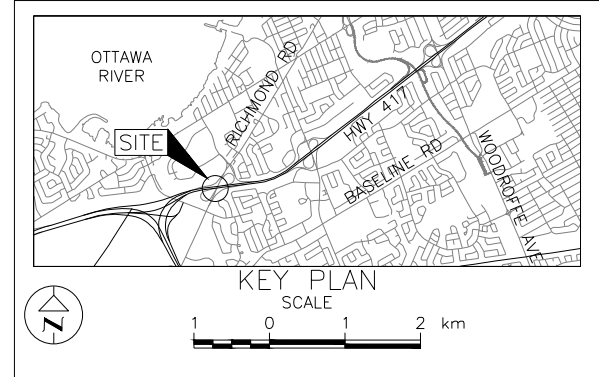
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/ Risks
Ground Improvement – Stone Columns	<ul style="list-style-type: none">• Not Feasible	<ul style="list-style-type: none">• N/A	<ul style="list-style-type: none">• N/A	<ul style="list-style-type: none">• N/A	<ul style="list-style-type: none">• N/A
Ground Improvement – Deep Soil Mixing	<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
Ground Improvement – Jet Grouting	<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



CONT No.
GWP No.4124-14-00

HIGHWAY 417 REHABILITATION
AND WIDENING
RICHMOND ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation, Geocres No. 31G05-007
- Borehole - Previous Investigation, Geocres No. 31G05-008
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer

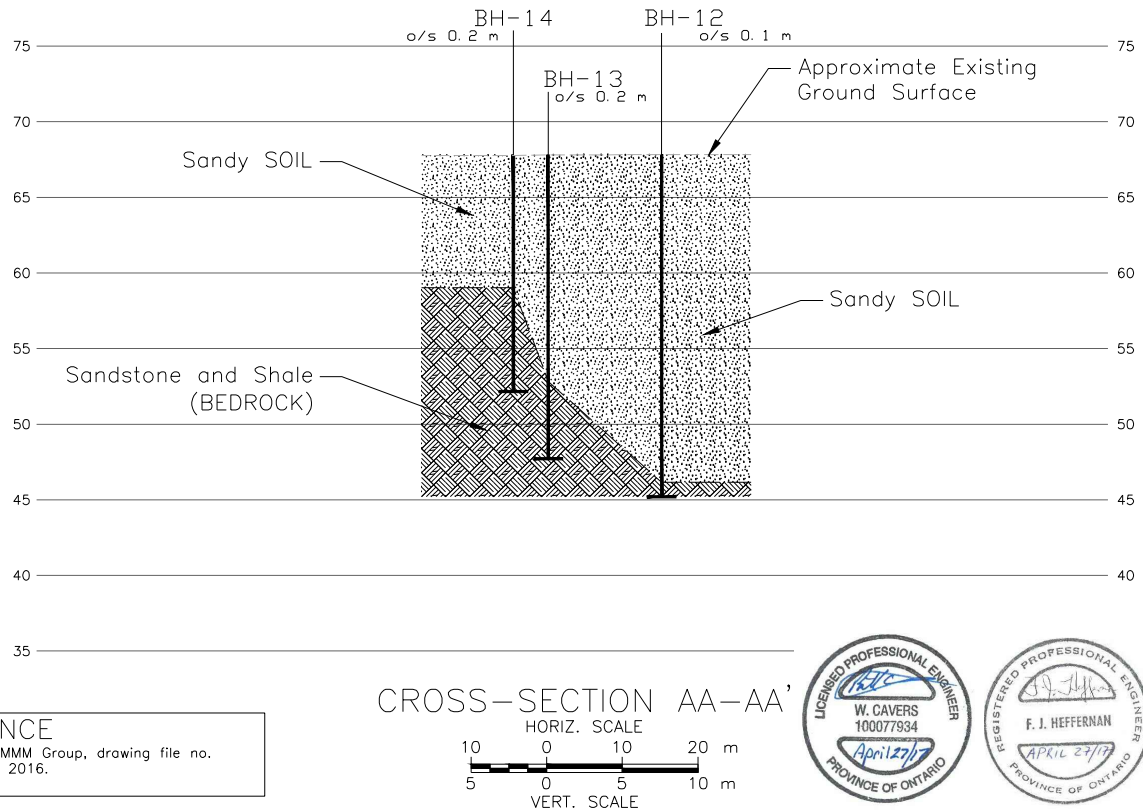
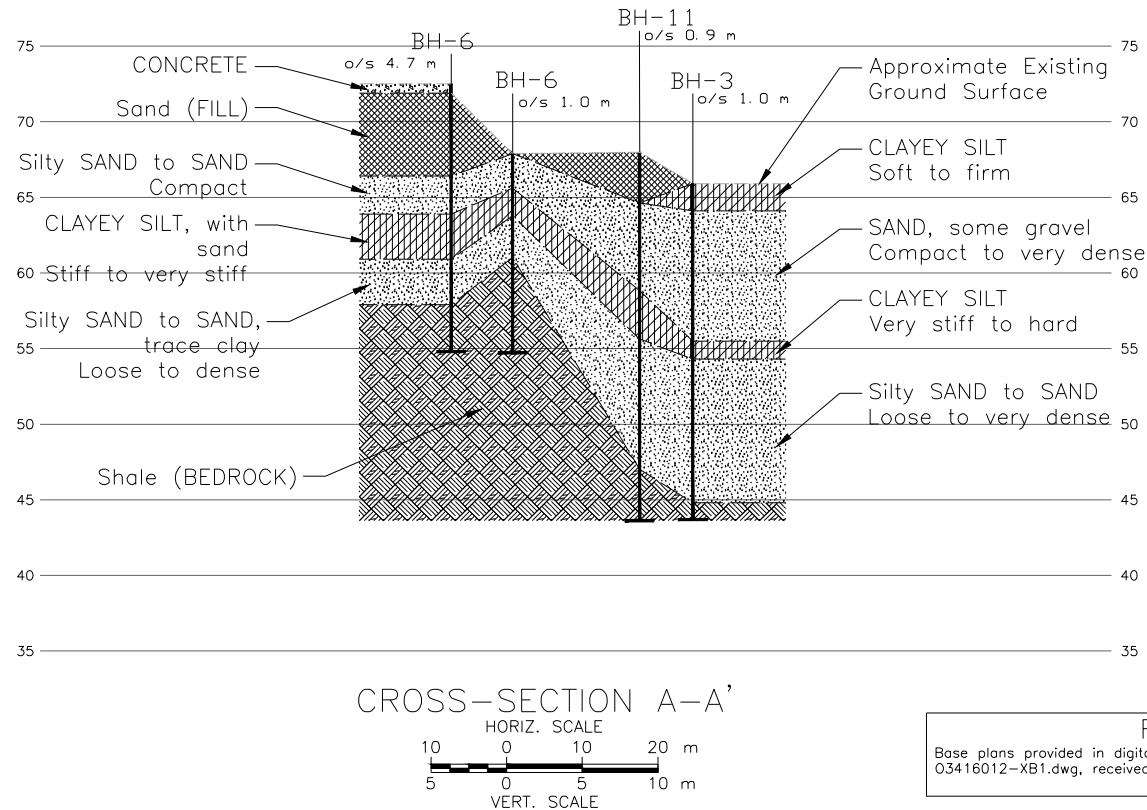
BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-101	65.2	5022946.5	359053.8
16-102	66.1	5022997.5	359067.7
16-103	66.6	5023017.2	359135.6
16-104	65.5	5023063.1	359185.5
16-105	65.5	5023062.5	359180.4
BH-1	65.8	5023036.7	359176.6
BH-2	65.6	5023068.8	359203.5
BH-3	65.9	5023098.3	359207.0
BH-4	65.6	5023067.2	359190.9
BH-5	65.9	5023038.4	359226.2
BH-6	72.5	5023107.8	359238.0
BH-10	66.9	5023069.1	359198.9
BH-11	68.0	5023101.0	359213.8
BH-12	67.8	5023092.0	359212.1
BH-13	67.8	5023093.5	359227.0
BH-14	67.8	5023094.1	359231.6
BH-6	67.9	5023103.2	359230.4
BH-7	67.8	5023070.6	359215.3
BH-8	67.5	5023037.8	359200.1
BH-9	65.6	5023037.6	359184.3

NOTES

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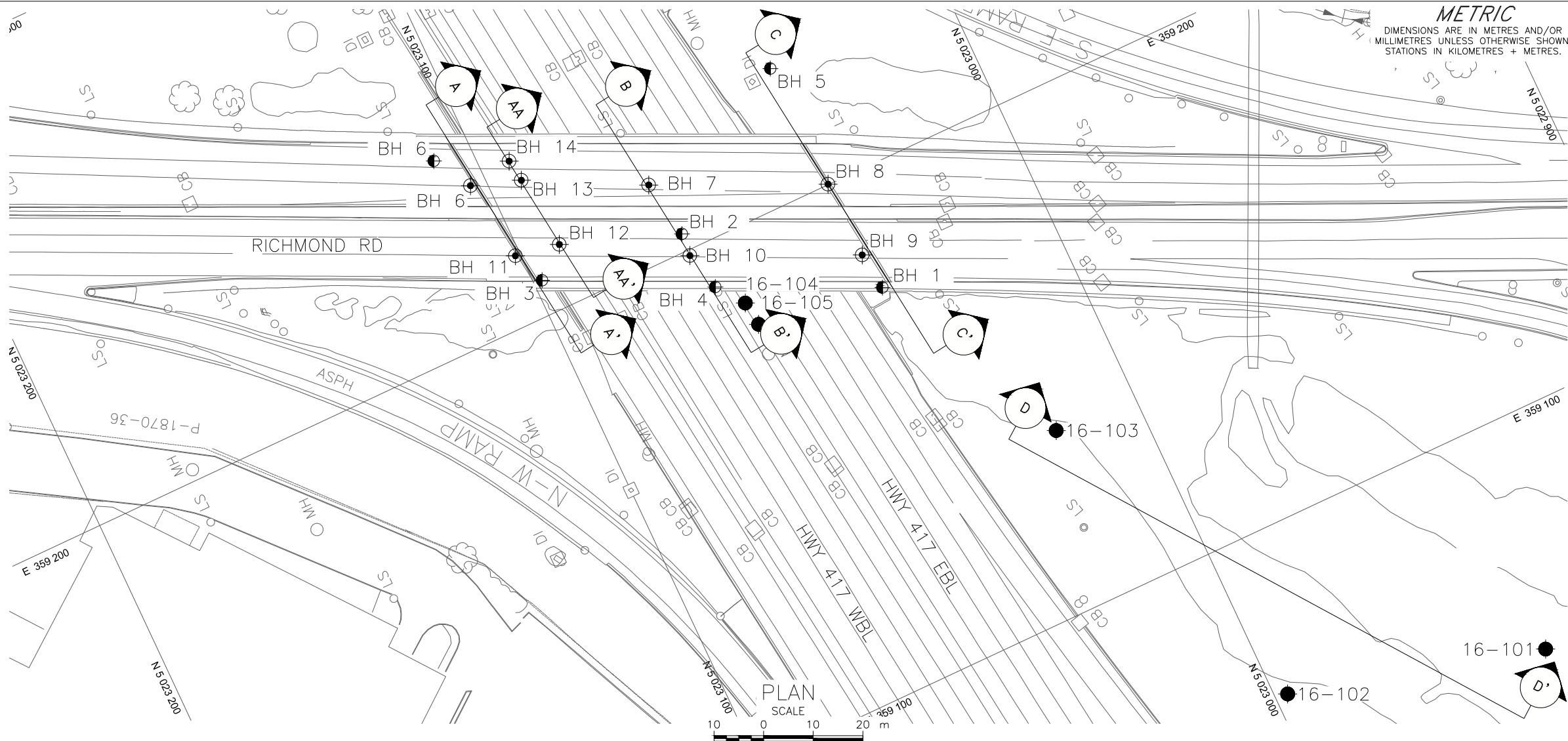


REFERENCE

Base plans provided in digital format by MMM Group, drawing file no. 03416012-XB1.dwg, received October 26, 2016.



NO.	DATE	BY	REVISION
Geocres No. 31G5-276			
HWY. 417	PROJECT NO. 1546542		DIST. EASTERN
SUBM'D. MJK	CHKD. WC	DATE: 12/21/2016	SITE: 3-039
DRAWN: JM	CHKD. WC	APPD. FJH	DWG. 1



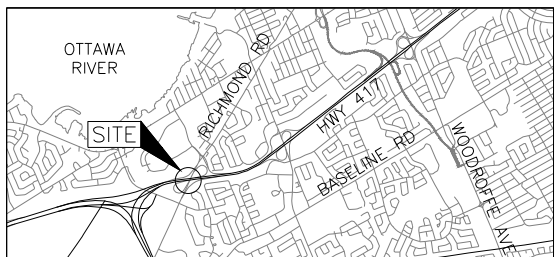
METRIC
DIMENSIONS ARE IN METRES AND/OR
MILLIMETRES UNLESS OTHERWISE SHOWN.
STATIONS IN KILOMETRES + METRES.

CONT No.
GWP No.4124-14-00



HIGHWAY 417 REHABILITATION
AND WIDENING
RICHMOND ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



KEY PLAN
SCALE
1 0 1 2 km

LEGEND

- Borehole - Current Investigation
- ⊙ Borehole - Previous Investigation, Geocres No. 31G05-007
- ⊕ Borehole - Previous Investigation, Geocres No. 31G05-008
- N Standard Penetration Test Value
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- Seal
- Piezometer
- WL in piezometer, measured on NOV 23, 2016

BOREHOLE CO-ORDINATES

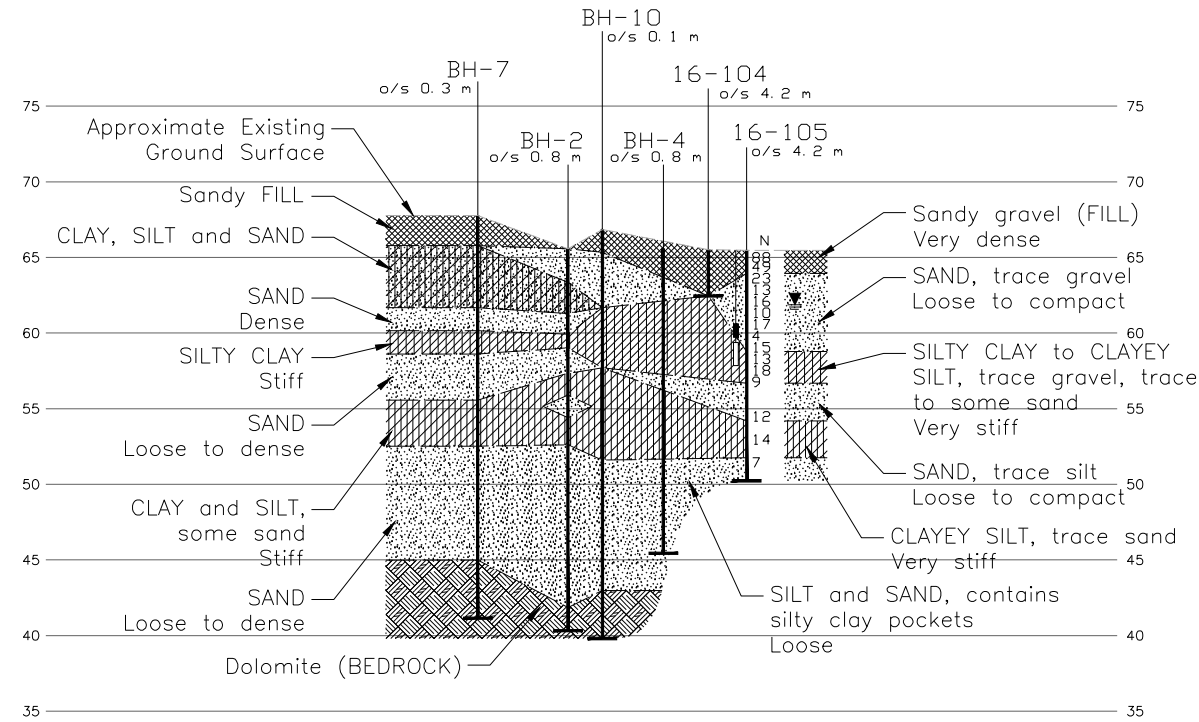
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16-102	66.1	5022997.5	359067.7
16-103	66.6	5023017.2	359135.6
16-104	65.5	5023063.1	359185.5
16-105	65.5	5023062.5	359180.4
BH-1	65.8	5023036.7	359176.6
BH-2	65.6	5023068.8	359203.5
BH-3	65.9	5023098.3	359207.0
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BH-9	65.6	5023037.6	359184.3

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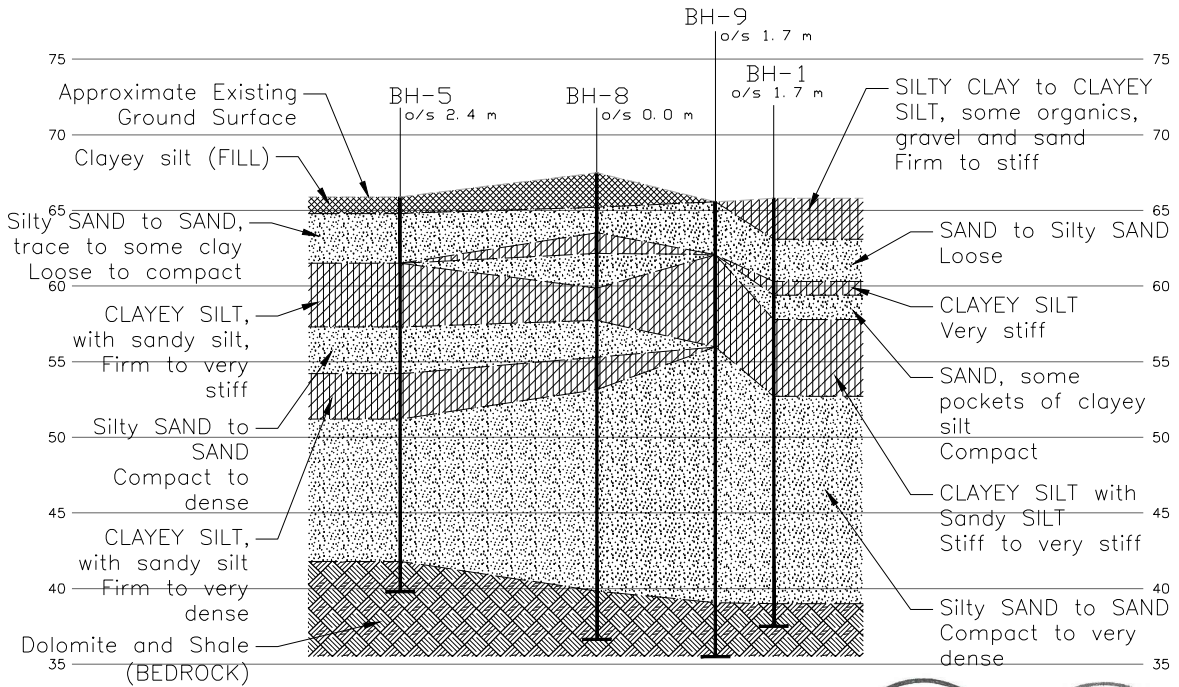
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CROSS-SECTION B-B'

HORIZ. SCALE
10 0 10 20 m
VERT. SCALE
5 0 5 10 m



CROSS-SECTION C-C'

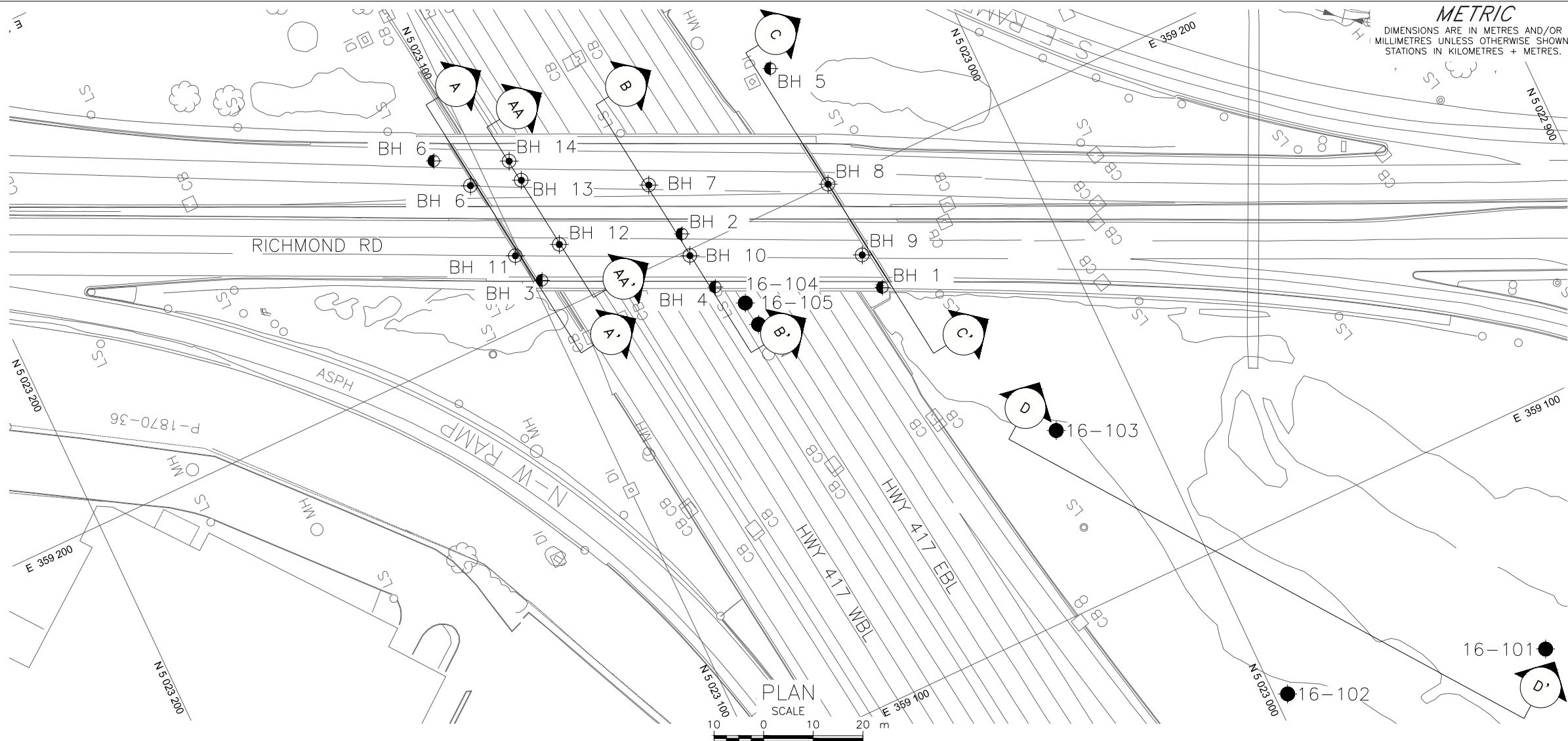
HORIZ. SCALE
10 0 10 20 m
VERT. SCALE
5 0 5 10 m

REFERENCE

Base plans provided in digital format by MMM Group, drawing file no. 03416012-XB1.dwg, received October 26, 2016.



NO.	DATE	BY	REVISION
Geocres No. 31G5-276			
HWY. 417		PROJECT NO. 1546542	DIST. EASTERN
SUBM'D. MJK	CHKD. WC	DATE: 12/21/2016	SITE: 3-039
DRAWN: JM	CHKD. WC	APPD. FJH	DWG. 2

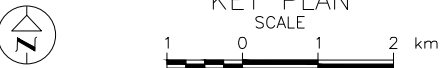
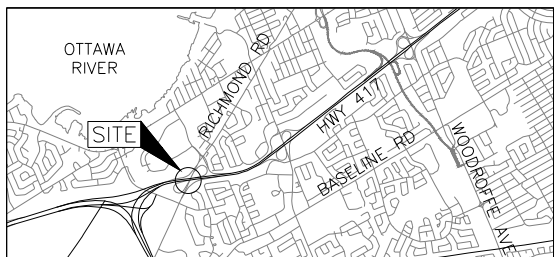


CONT No.
GWP No.4124-14-00



HIGHWAY 417 REHABILITATION
AND WIDENING
RICHMOND ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation, Geocres No. 31G05-007
- Borehole - Previous Investigation, Geocres No. 31G05-008
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- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- WL in piezometer, measured on AUG 2, 2016
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

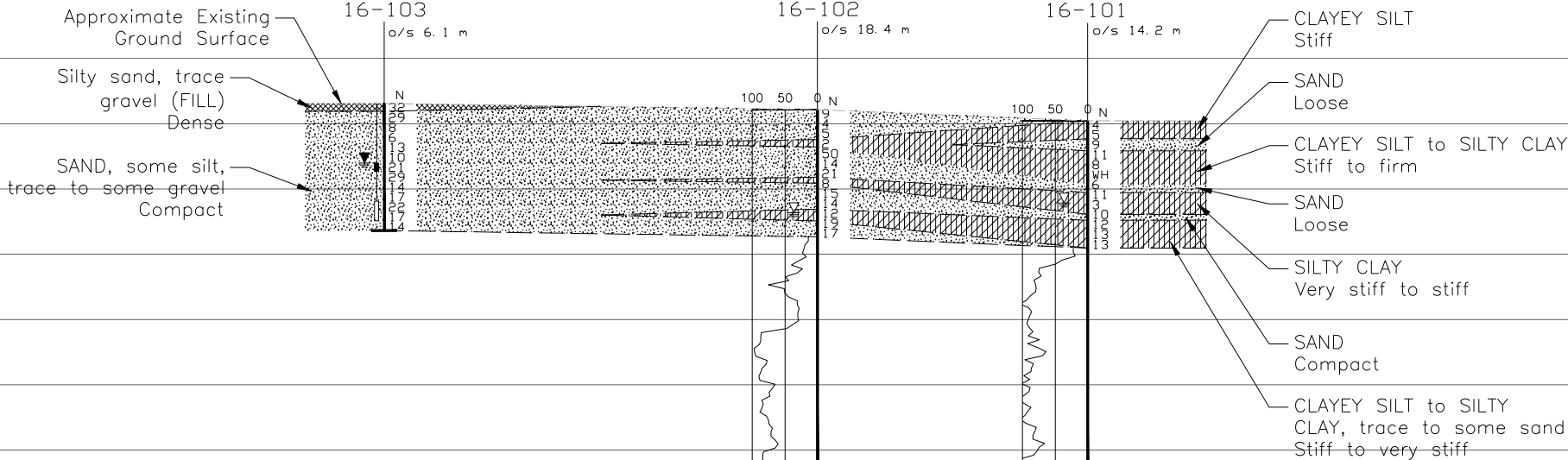
No.	ELEVATION	NORTHING	EASTING
16-101	65.2	5022946.5	359053.8
16-102	66.1	5022997.5	359067.7
16-103	66.6	5023017.2	359135.6
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16-105	65.5	5023062.5	359180.4
BH-1	65.8	5023036.7	359176.6
BH-2	65.6	5023068.8	359203.5
BH-3	65.9	5023098.3	359207.0
BH-4	65.6	5023067.2	359190.9
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BH-6	67.9	5023103.2	359230.4
BH-7	67.8	5023070.6	359215.3
BH-8	67.5	5023037.8	359200.1
BH-9	65.6	5023037.6	359184.3

NOTES

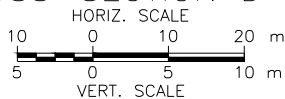
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CROSS-SECTION D-D'



REFERENCE

Base plans provided in digital format by MMM Group, drawing file no. 03416012-XB1.dwg, received October 26, 2016.



NO.	DATE	BY	REVISION
1	12/21/2016	FJH	1
Geocres No. 31G5-276			
HWY. 417		PROJECT NO. 1546542	
SUBM'D. MJK		DATE: 12/21/2016	
DRAWN: JM		SITE: 3-039	
CHKD. WC		APPD. FJH	
DWG. 3		DIST. EASTERN	



APPENDIX A

Borehole and Drillhole Records, Current Investigation

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes 16-101 to 16-105



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	

PROJECT 1546542-1010		RECORD OF BOREHOLE No 16-101		SHEET 1 OF 4		METRIC	
G.W.P. 4015-E-0017		LOCATION N 5022946.5 ; E 359053.8		ORIGINATED BY DG			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), DCPT		COMPILED BY JM			
DATUM Geodetic		DATE June 19-20, 2016		CHECKED BY KSL			

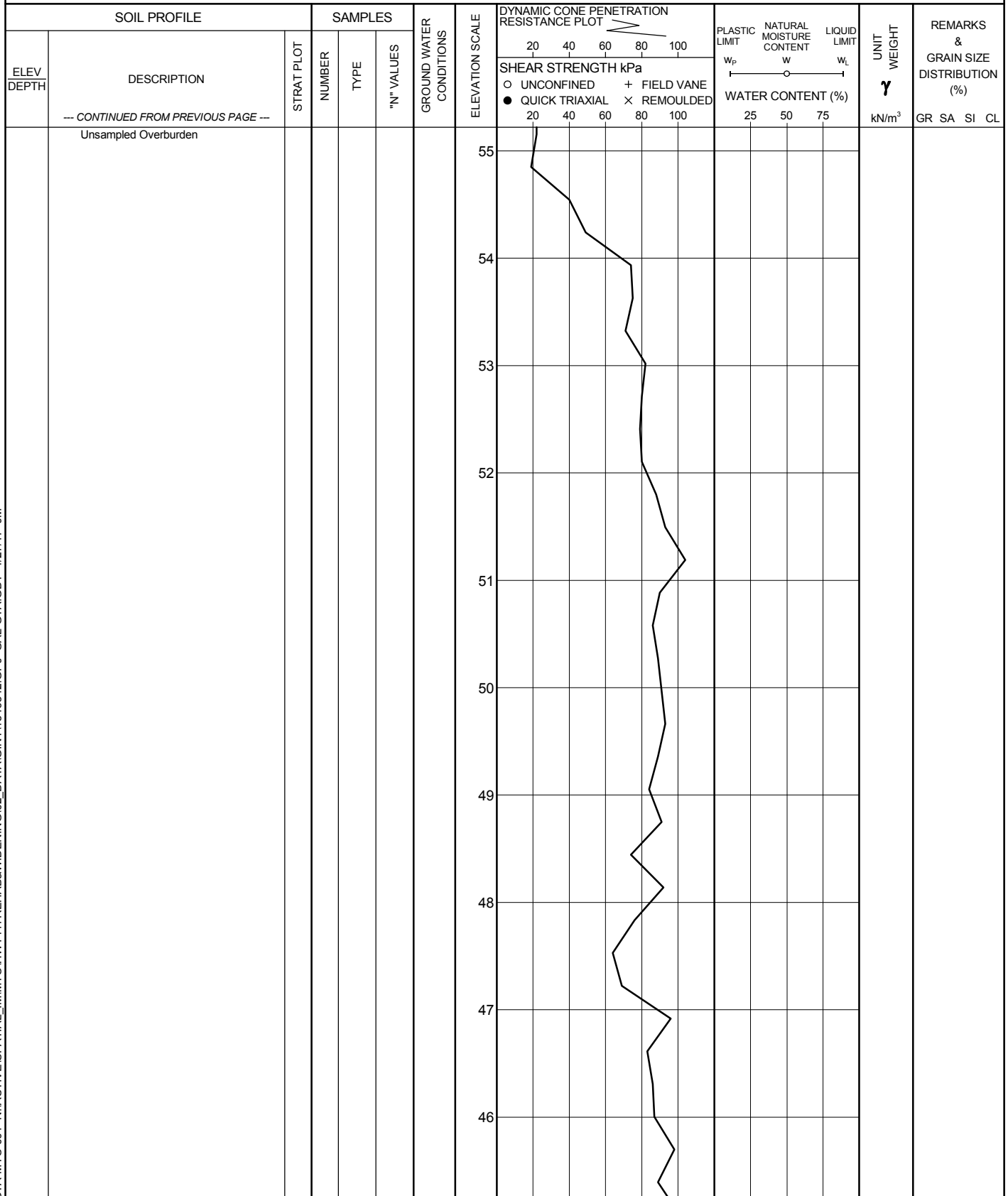
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)							
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			GR	SA	SI	CL	
								○ UNCONFINED + FIELD VANE	20	40	60	80		100	W _p	W	W _L				
								● QUICK TRIAXIAL × REMOULDED													
65.2	GROUND SURFACE																				
0.0	Silty sand (TOPSOIL)																				
0.2	Brown																				
	CLAYEY SILT		1	SS	4																
	Stiff																				
	Grey																				
	Moist																				
			2	SS	5																
63.8																					
1.4	SAND																				
	Loose																				
	Brown																				
	Moist																				
			3	SS	9																
62.9																					
2.3	CLAYEY SILT																				
	Stiff																				
	Grey																				
	Moist																				
			4	SS	11																
62.2																					
	SAND																				
	Loose																				
	Brown																				
	Moist																				
			5	SS	8																
	CLAYEY SILT, contains sand layers																				
	Firm to stiff																				
61.4																					
3.8	Grey																				
	Moist																				
	SILTY CLAY																				
	Firm																				
	Grey																				
	Moist to wet																				
			6	SS	WH																
60.6																					
4.6	CLAYEY SILT																				
	Firm																				
	Grey																				
	Wet																				
			7	SS	6																
60.3																					
4.9	SAND																				
	Loose																				
	Brown																				
	Wet																				
			8	SS	11																
59.7																					
5.5	SILTY CLAY																				
	Very stiff to stiff																				
	Grey																				
	Wet																				
																			</		

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMT\OHWY417\REHAB&WIDENING\02_DATA\GINT1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT <u>1546542-1010</u>		RECORD OF BOREHOLE No 16-101		SHEET 2 OF 4	METRIC
G.W.P. <u>4015-E-0017</u>		LOCATION <u>N 5022946.5 ;E 359053.8</u>		ORIGINATED BY <u>DG</u>	
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), DCPT</u>		COMPILED BY <u>JM</u>	
DATUM <u>Geodetic</u>		DATE <u>June 19-20, 2016</u>		CHECKED BY <u>KSL</u>	



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

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

PROJECT		RECORD OF BOREHOLE No 16-101				SHEET 4 OF 4		METRIC								
G.W.P. 1546542-1010		LOCATION N 5022946.5 ; E 359053.8				ORIGINATED BY DG										
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem), DCPT				COMPILED BY JM										
DATUM Geodetic		DATE June 19-20, 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 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa								
	--- CONTINUED FROM PREVIOUS PAGE --- END OF BOREHOLE DCPT REFUSAL NOTES: 1. Water level in open borehole at a depth of 6.3 m below ground surface (Elev. 58.9 m), measured during drilling.															

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITOHWY417REHAB&WIDENING\02_DATA\GINT1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT <u>1546542-1010</u>		RECORD OF BOREHOLE No 16-102		SHEET 1 OF 4		METRIC	
G.W.P. <u>4015-E-0017</u>		LOCATION <u>N 5022997.5 ; E 359067.7</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), DCPT</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>June 15-16, 2016</u>		CHECKED BY <u>KSL</u>			

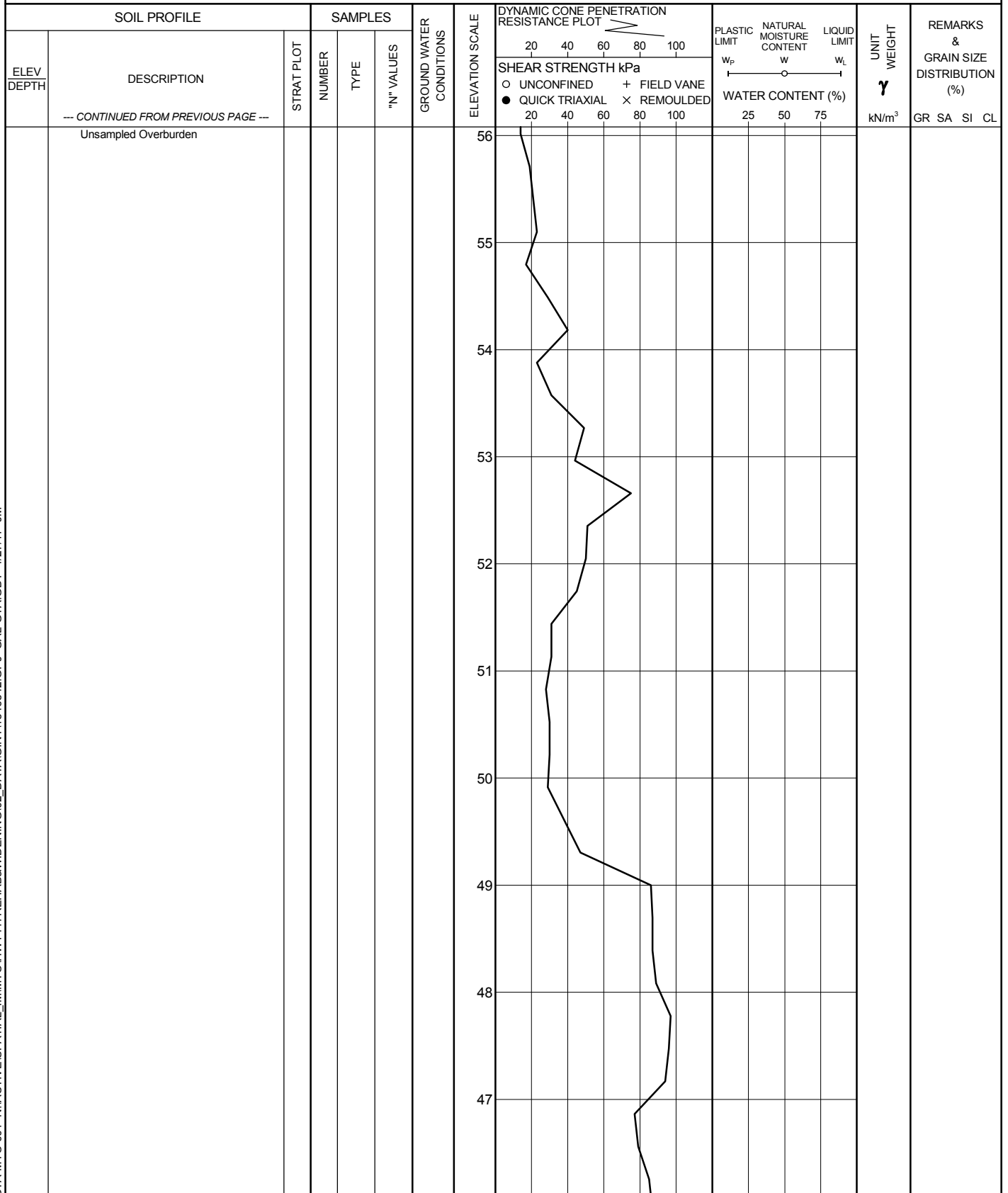
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)								
								20	40	60	80	100	W _p	W	W _L		GR	SA	SI	CL	
66.1	GROUND SURFACE						66														
65.9	Silty sand (TOPSOIL)		1	SS	9																
0.2	Dark brown																				
	Moist																				
	Silty SAND to SAND, trace clay																				
	Loose																				
	Grey-brown																				
	Moist																				
63.8																					
2.3	CLAYEY Silty SAND, some clay		4	SS	2																
	Very loose																				
	Brown																				
	Wet																				
63.2																					
2.9	SAND and GRAVEL, contains		5	SS	50																
	cobbles and boulders																				
	Compact to dense																				
	Grey-brown																				
	Moist																				
													</								

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT <u>1546542-1010</u>		RECORD OF BOREHOLE No 16-102		SHEET 2 OF 4		METRIC	
G.W.P. <u>4015-E-0017</u>		LOCATION <u>N 5022997.5 ; E 359067.7</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), DCPT</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>June 15-16, 2016</u>		CHECKED BY <u>KSL</u>			



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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTOHWY417REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT <u>1546542-1010</u>		RECORD OF BOREHOLE No 16-102		SHEET 3 OF 4		METRIC	
G.W.P. <u>4015-E-0017</u>		LOCATION <u>N 5022997.5 ; E 359067.7</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), DCPT</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>June 15-16, 2016</u>		CHECKED BY <u>KSL</u>			

SOIL PROFILE				SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT 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	WATER CONTENT (%)					GR	SA	SI	CL									
										20	40	60	80		100	25	50	75											
--- CONTINUED FROM PREVIOUS PAGE --- Unsampled Overburden										<div><div>○ UNCONFINED + FIELD VANE</div><div>● QUICK TRIAXIAL × REMOULDED</div></div>																			
								46																					
								45																					
								44																					
								43																					
								42																					
								41																					
								40																					
								39																					
								38																					
								37																					
36.9 29.2																													

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTOHWY417REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT <u>1546542-1010</u>		RECORD OF BOREHOLE No 16-102		SHEET 4 OF 4		METRIC	
G.W.P. <u>4015-E-0017</u>		LOCATION <u>N 5022997.5 ;E 359067.7</u>		ORIGINATED BY <u>DG</u>			
DIST <u>Eastern</u> HWY <u>417</u>		BOREHOLE TYPE <u>Power Auger 200 mm Diam. (Hollow Stem), DCPT</u>		COMPILED BY <u>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>June 15-16, 2016</u>		CHECKED BY <u>KSL</u>			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT 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	● QUICK TRIAXIAL	+	×	FIELD VANE	REMOULDED	WATER CONTENT (%)							
	END OF BOREHOLE DCPT REFUSAL NOTES: 1. Water level in open borehole at a depth of 7.9 m below ground surface (Elev. 58.2 m), measured during drilling.																				

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT 1546542-1010		RECORD OF BOREHOLE No 16-103		SHEET 1 OF 2		METRIC	
G.W.P. 4015-E-0017		LOCATION N 5023017.2 ; E 359135.6		ORIGINATED BY DG			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY JM			
DATUM Geodetic		DATE June 19, 2016		CHECKED BY KSL			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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
								20	40	60	80	100								
66.6	GROUND SURFACE																			
0.0	Silty sand (TOPSOIL)																			
0.2	Brown Dry		1	SS	32															
66.0	Silty sand, trace gravel (FILL)																			
0.6	Dense Dark brown Moist																			
	SAND Compact Brown Moist		2	SS	29															
65.1																				
1.5	Sandy SILT Loose Grey-brown Wet		3	SS	8															
64.5																				
2.1	SAND, some gravel to gravelly, trace to some silt and clay Loose to compact Grey Moist																			
			4	SS	6															
			5	SS	13															
			6	SS	10															
62.2																				
4.4	SAND, some silt, trace to some gravel Compact Grey Wet		7	SS	21															
			8	SS	29															
			9	SS	14															
			10	SS	17															
			11	SS	22															
			12	SS	17															
			13	SS	14															
56.9																				
9.8																				

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTD\HWY417\REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT <u>1546542-1010</u>		RECORD OF BOREHOLE No 16-103		SHEET 2 OF 2		METRIC	
G.W.P. <u>4015-E-0017</u>		LOCATION <u>N 5023017.2 ; E 359135.6</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>JM</u>			
DATUM <u>Geodetic</u>		DATE <u>June 19, 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	● QUICK TRIAXIAL	×	REMOULDED	WATER CONTENT (%)							
	--- CONTINUED FROM PREVIOUS PAGE ---																				
	END OF BOREHOLE																				
	NOTES: 1. Water level in well screen at a depth of 4.6 m below ground surface (Elev. 61.9 m), measured on August 2, 2016.																				

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT		RECORD OF BOREHOLE No 16-104				SHEET 1 OF 1		METRIC									
G.W.P. 1546542-1010		LOCATION N 5023063.1 ; E 359185.5				ORIGINATED BY KM											
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)				COMPILED BY JM											
DATUM Geodetic		DATE November 13-14, 2016				CHECKED BY MJK											
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									
65.5	GROUND SURFACE																
0.0	ASPHALTIC CONCRETE																
65.1	Gravelly sand (FILL) Grey Moist																
0.4	Gravel and sand, with non-plastic fines (FILL) Grey-brown Moist																
64.0	Sand, with gravel (FILL) Brown Moist																
1.5																	
62.6	SILTY CLAY to CLAYEY SILT Grey Moist																
3.1	END OF AUGERHOLE																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT 1546542-1010		RECORD OF BOREHOLE No 16-105		SHEET 1 OF 2		METRIC	
G.W.P. 4015-E-0017		LOCATION N 5023062.5 ; E 359180.4		ORIGINATED BY KM			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger 200 mm Diam. (Hollow Stem)/Wash Boring, HW Casing		COMPILED BY JM			
DATUM Geodetic		DATE November 13, 2016		CHECKED BY MJK			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED	20	40	60	80	100	w _p	w		w _L			
65.5	GROUND SURFACE																			
0.0	ASPHALTIC CONCRETE (0.00 - 0.16 m)																			
65.1	Gravelly sand (FILL) Very dense Grey Moist		1	SS	88															
0.4	Sand and gravel, with pieces of geo-fibre (FILL) Very dense to dense Grey-brown Moist		2	SS	49															
64.0																				
1.5	SAND, some gravel Compact Brown Moist		3	SS	23															
			4	SS	13															
62.5																				
3.1	SAND, trace gravel Loose to compact Brown Wet		5	SS	16															
			6	SS	10															
			7	SS	17															
			8	SS	4															
			9	SS	15															
58.8																				
6.7	SILTY CLAY to CLAYEY SILT, trace gravel, trace to some sand Very stiff Grey Wet		10	SS	13															
			11	SS	18															
			12	SS	9															
56.7																				
8.8	SAND, trace silt Loose to compact Grey Wet																			

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTD\HWY417\REHAB&WIDENING\02_DATA\GINT1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT		1546542-1010		RECORD OF BOREHOLE No 16-105		SHEET 2 OF 2		METRIC										
G.W.P.		4015-E-0017		LOCATION		N 5023062.5 ; E 359180.4		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)/Wash Boring, HW Casing		COMPILED BY										
DATUM		Geodetic		DATE		November 13, 2016		CHECKED BY										
JMK																		
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 (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					W _p	W			W _L	
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100							
54.2	SAND, trace silt Loose to compact Grey Wet		13	SS	12													0 97 (3)
11.3	CLAYEY SILT, trace sand Very stiff Grey Wet		14	SS	14													
51.8																		
13.7	SAND, some silt, trace gravel Loose Grey Wet		15	SS	7													0 63 27 10
51.5																		
14.0	SILT and SAND, contains silty clay pockets Loose Grey Wet																	
50.3																		
15.2	END OF BOREHOLE																	
NOTES: 1. Water level in well screen at a depth of 3.6 m below ground surface (Elev. 61.9 m), measured on November 23, 2016.																		



APPENDIX B

Laboratory Test Results, Current Investigation

Figure 1 - Plasticity Chart – Silty Clay

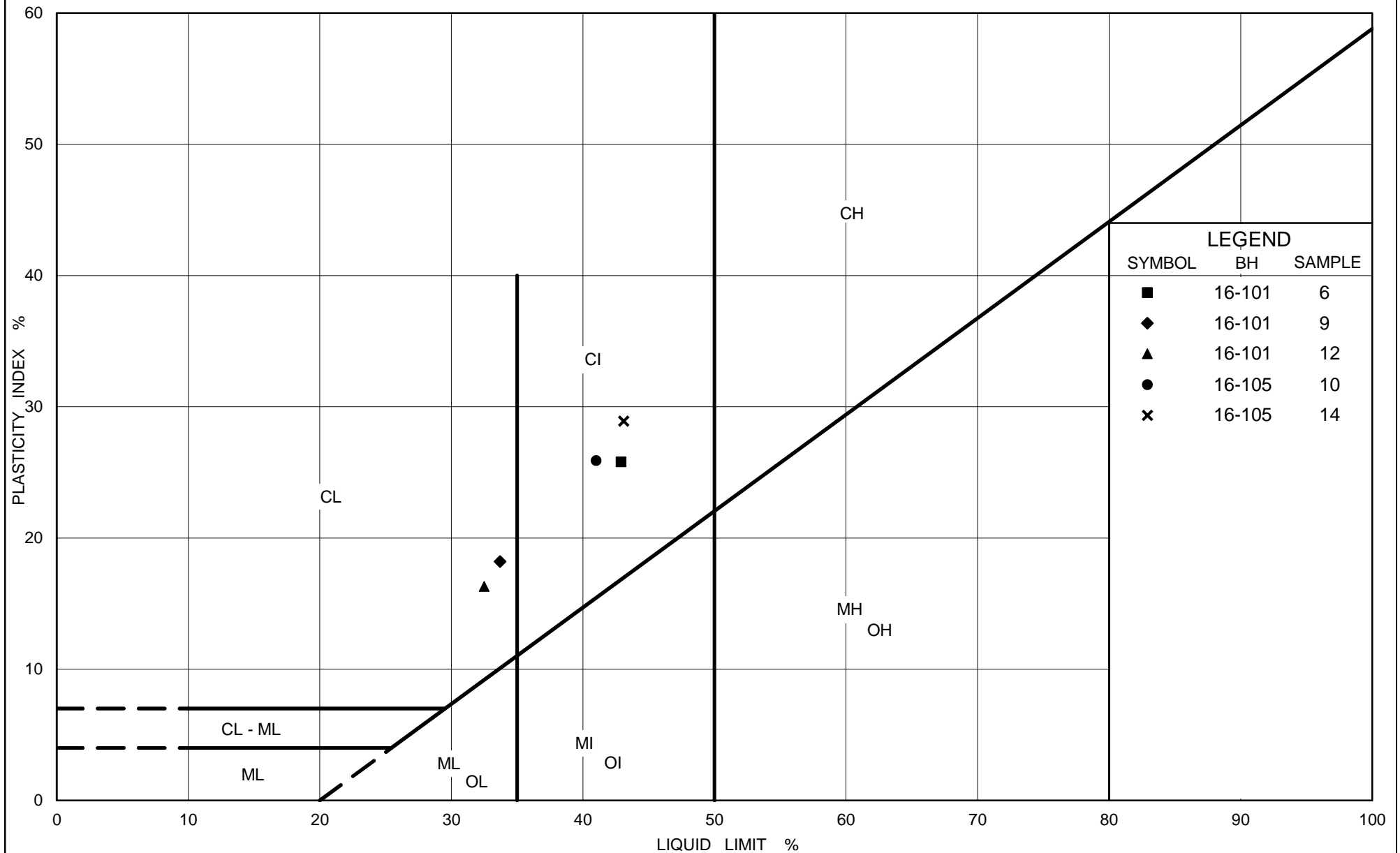
Figure 2 - Grain Size Distribution Test Results – Silty Clay

Figure 3 - Grain Size Distribution Test Results – Silty Clayey Sand

Figure 4 - Grain Size Distribution Test Results – Clean Sand

Figure 5 - Grain Size Distribution Test Results – Silty Sand

Figure 6 - Grain Size Distribution Test Results – Sand and Gravel



Ministry of Transportation

Ontario

PLASTICITY CHART SILTY CLAY

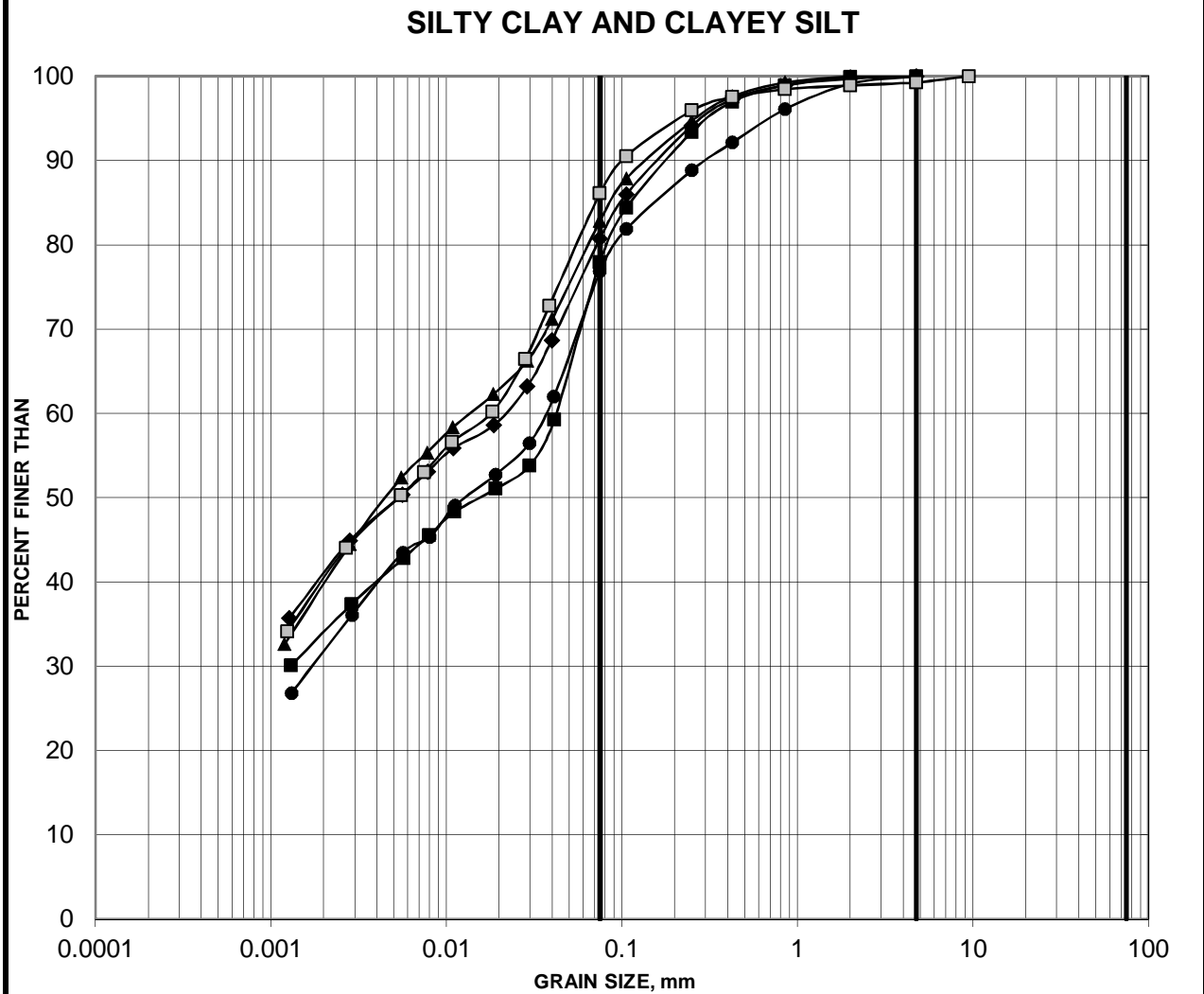
FIG No. 1

Project No. 1546542/ 1010

Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE 2



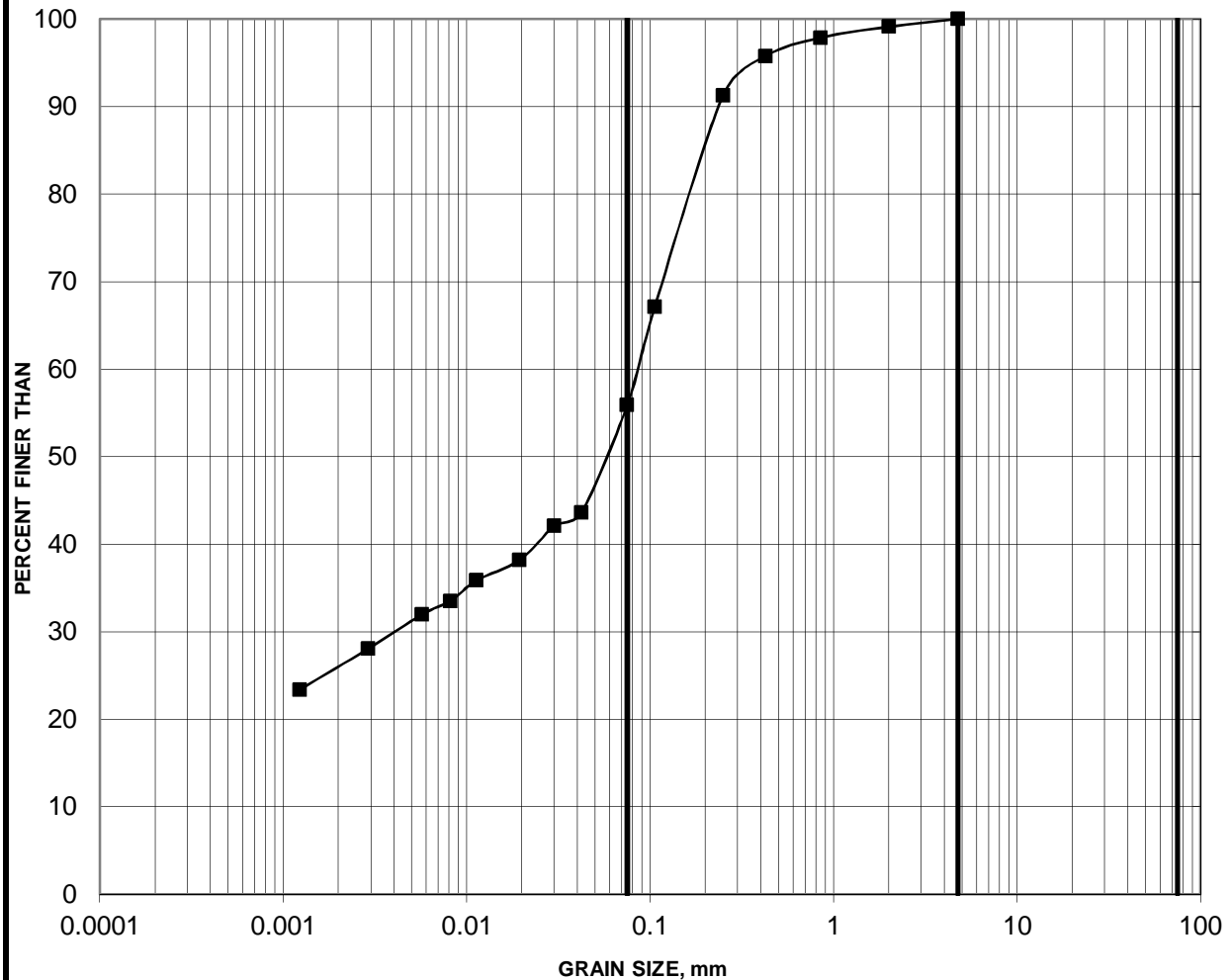
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-101	2A	0.76-1.22
16-101	9	6.10-6.71
16-101	12	8.38-8.99
16-102	11	7.62-8.23
16-105	10	6.86-7.47

GRAIN SIZE DISTRIBUTION

FIGURE 3

CLAYEY SILTY SAND

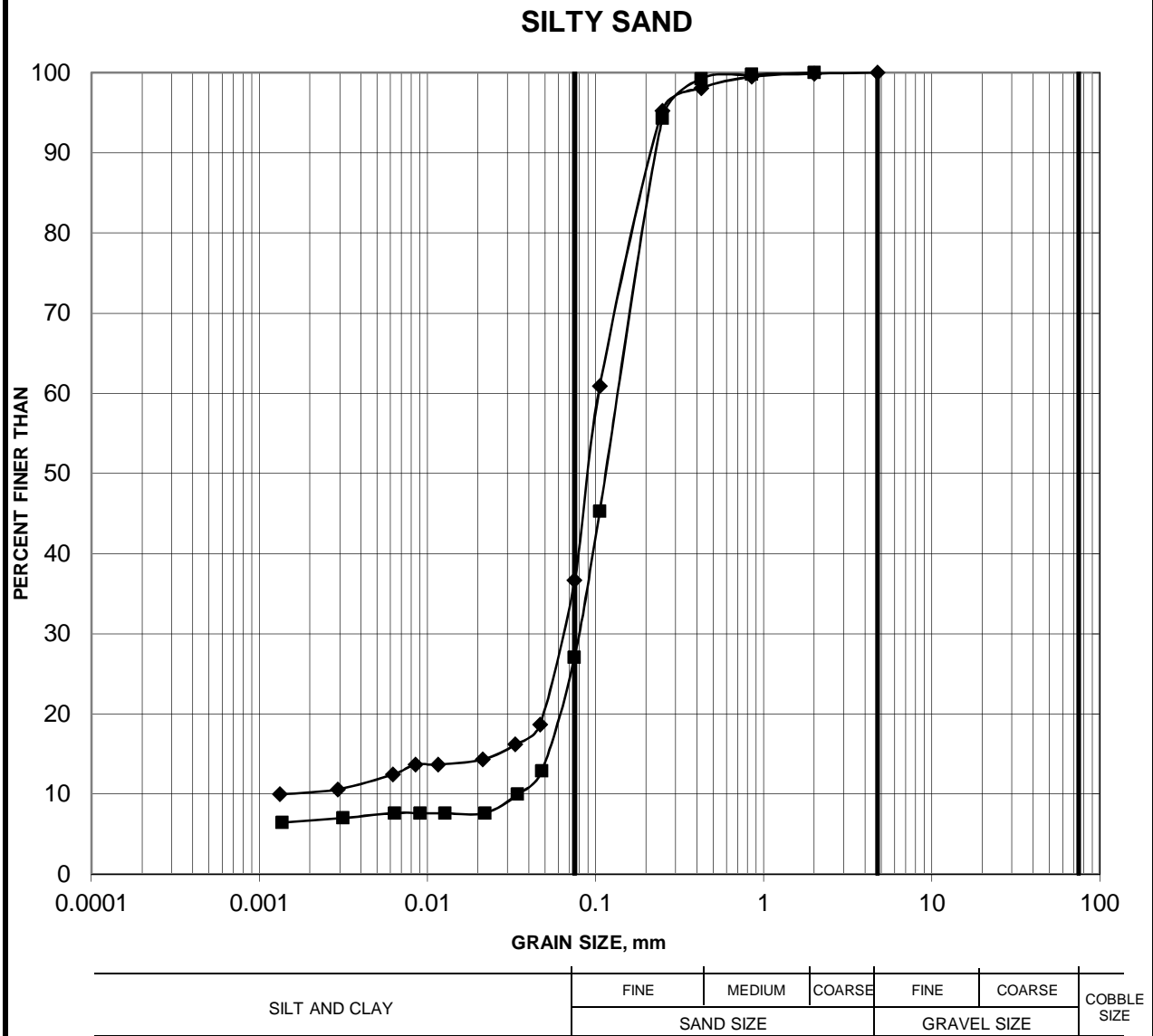


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

Borehole	Sample	Depth (m)
16-102	4	2.29-2.90

GRAIN SIZE DISTRIBUTION

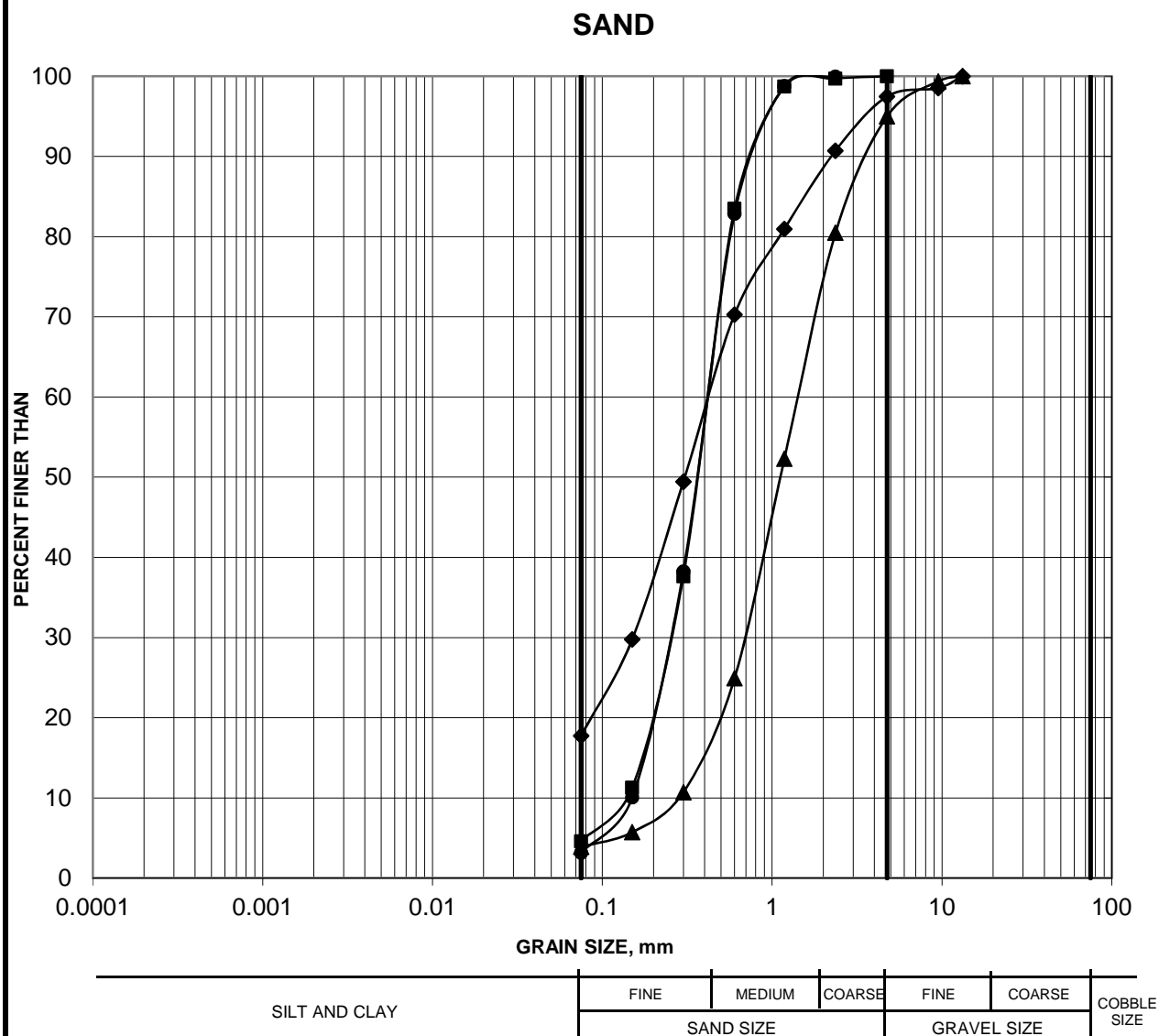
FIGURE 4



Borehole	Sample	Depth (m)
16-102	3	1.52-2.13
16-105	15A	13.72-14.02

GRAIN SIZE DISTRIBUTION

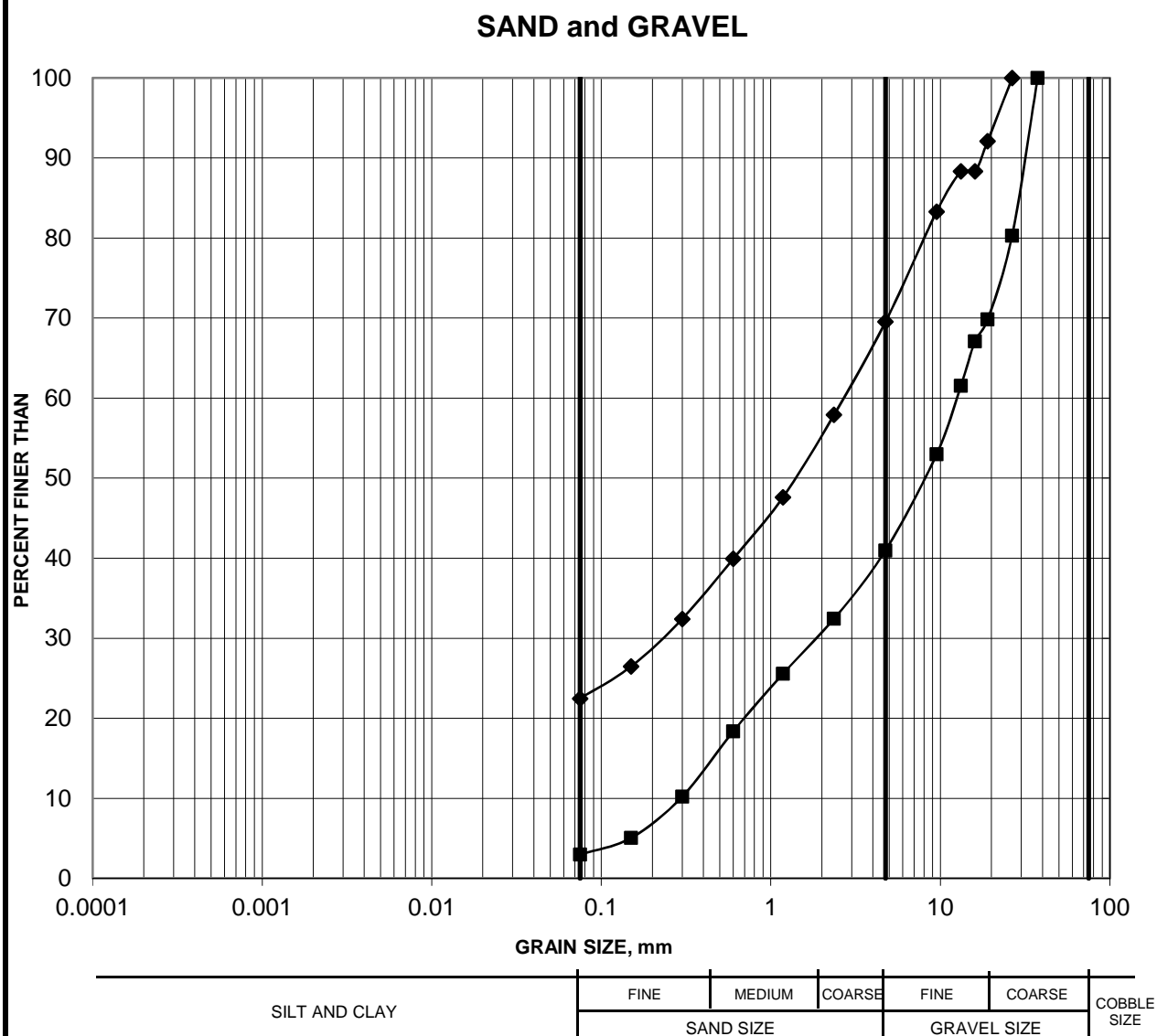
FIGURE 5



Borehole	Sample	Depth (m)
16-101	7B	4.88-5.18
16-103	13	9.15-9.76
16-105	7	4.57-5.18
16-105	13	10.67-11.28

GRAIN SIZE DISTRIBUTION

FIGURE 6



Borehole	Sample	Depth (m)
16-102	7	4.57-5.18
16-103	4	2.29-2.90



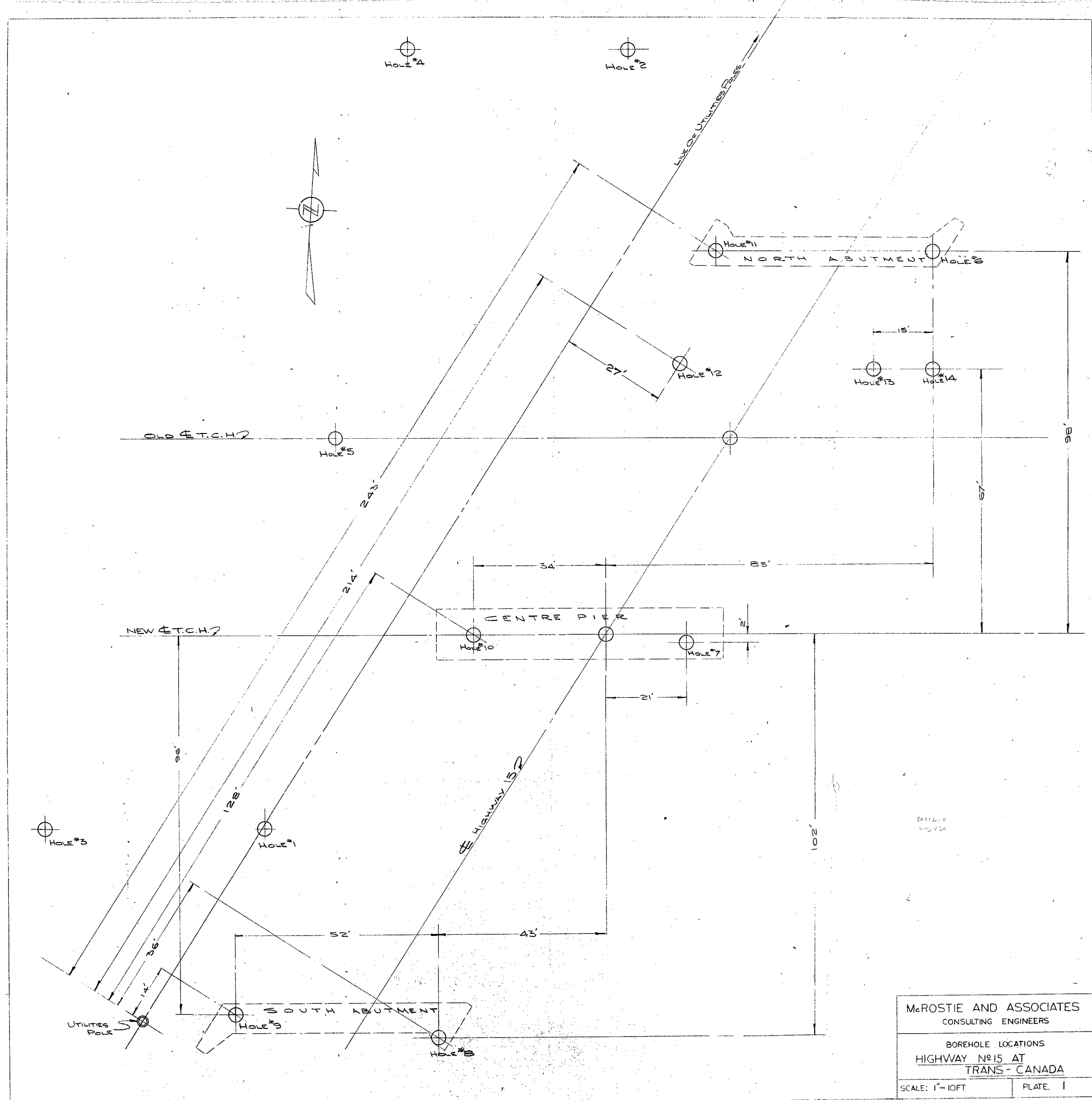
APPENDIX C

Previous Borehole Records

Records of Previous Boreholes 6 to 14 (Geocres No. 31G5-008)

Records of Previous Boreholes 1 to 4 (Geocres No. 31G5-007)

Records of Previous Boreholes 5 to 6 (Geocres No. 31G5-151)



McROSTIE AND ASSOCIATES
CONSULTING ENGINEERS

BOREHOLE LOCATIONS
HIGHWAY No. 15 AT
TRANS-CANADA

SCALE: 1"=100'

PLATE: 1

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

HIGHWAY #15 AT TRANS-CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 222.2'

DATE JULY-2-1959

HOLE NO.

REMARKS SEE: PLATE # 2

UNCOMPILED COMPRESSION STRENGTH KIPS/FT. ²		SMALL SCALE PENE. METER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PROBING OR WANE TEST	
								LB. HAMMER	NO CASING
								INCH DROP	INCH DIA. ROD
								BLOWS PER FOOT OR	SHEAR STRENGTH IN KIPS PER FT. ²
					GROUND SURFACE	0	222.2'		
			13	7-1	SANDY FILL	5'	217.2'	SAND SILT	
			8	7-2	MEDIUM DENSE CLAYEY WELL-GRADED SAND	9.5'			
			15	7-3	GRADED SAND	9.0'			
0.8, 1.0, 1.0			4	7-4	MEDIUM SOFT GRAY CLAY & SILT	10'	212.2'	OVER-NIGHT WATER LEVEL - 9.5'	
1.3	1.4, 1.4, 1.6 1.0, 1.0, 1.2 2.0, 2.0, 2.2 3.2, 3.4, 2.8 2.0, 2.0		7-5			15'	207.2'	SAND CLAY	
2.0	4.4, 4.2, 4.8		7-6		STIFF SILTY GRAY CLAY WITH SOME SAND	16.5'			
			7-7		LOOSE CLAYEY WELL-GRADED SAND	16.5'		SAND SILT	
			9	7-7	STIFF GRAY CLAY & SILT WITH SOME SAND	20'	202.2'		
			15	7-8	MEDIUM DENSE WELL-GRADED SAND	22.5'			
			14	7-9	MEDIUM DENSE WELL-GRADED SAND WITH A FEW CLAY POCKETS	22.5'	197.2'	SAND CLAY	
2.2, 2.6, 2.4			5	7-10	STIFF SILTY GRAY CLAY	25'		SILT CLAY	
			7-11		LOOSE FINE SAND WITH A FEW STONES	26'	192.2'		
			26	7-12	MEDIUM DENSE FINE SAND	35'	187.2'		
2.2, 2.0, 2.0			25	7-13	STIFF GRAY CLAY & SILT WITH SOME SAND & A FEW STONES	40'	182.2'		
						45'	177.2'		
			10	7-14	LOOSE FINE SAND	50'	172.2'		
						55'	167.2'		
			7	7-15	LOOSE FINE SAND	60'	162.2'		
						65'	157.2'		
			24	7-16	MEDIUM DENSE FINE SAND	70'	152.2'		
					CORE RECOVERY - 40% 75.5'	74.7'	147.5'		
					DOLOMITE ROCK WITH SHALE LAYERS	80.9'	142.2'		
					CORE RECOVERY - 97%	85'	137.2'		
					DOLOMITE ROCK WITH SHALE LAYERS	85'	137.2'		
					CORE RECOVERY - 97%	87.3'	134.9'		
					BOTTOM OF HOLE				

McROSTIE & ASSOCIATES
CONSULTING ENGINEERS
OTTAWA, CANADA

SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS

HIGHWAY 15 AT TRANS-CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 221.3' DATE JULY-10-1969 HOLE NO. 8
REMARKS SEE: PLATE # 2

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	PROBING OR VANE TEST		
							LB. HAMMER	NO CASING	
							INCH DROP	INCH DIA. ROD	
GROUND SURFACE 7							BLOWS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT. 2		
1-9	1.2, 1.0, 1.0 2.4, 2.0, 2.0 2.0, 2.0, 2.4 2.0-2.2		11	8-1	FILL	0	221.3		
			9	8-2		5	216.3		
			13	8-3	MEDIUM DENSE SILT WITH A LITTLE SAND	7.5			
			5	8-4	LOOSE COARSE SAND WITH SOME FINE SAND	10	211.3		
			6	8-5	LOOSE COARSE SAND WITH SOME FINE SAND	12.5			
			8-5A	MEDIUM SOFT SILTY GRAY CLAY	15	206.3			
			8-6	STIFF SILTY GRAY CLAY WITH SOME SAND	17.5				
			8-7	LOOSE COARSE SAND WITH SOME CLAY SILT	20	201.3			
			18	8-8	MEDIUM DENSE COARSE SAND WITH SOME FINE SAND	25	196.3		
			19	8-9					
2-7	1.4, 1.4, 1.0 1.0, 1.0, 1.0 2.0, 2.0, 2.0 2.0-2.2		9	8-10	MEDIUM SOFT GRAY CLAY & SILT WITH A LITTLE SAND	30	191.3		
			4	8-11	STIFF SILTY GRAY CLAY WITH SOME SAND	32.5			
			14	8-13	MEDIUM DENSE FINE SAND WITH SOME SILTY CLAY POCKETS	35	186.3		
			24	8-14	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	40	181.3		
						45	176.3		
			8	8-15	LOOSE FINE SAND	50	171.3		
						55	166.3		
			4	8-16		60	161.3		
						64.0	156.3		
						70	151.3		
			120 FORC	8-17	VERY DENSE FINE SAND	75	146.3		
						80	141.3		
						85	136.3		
			40 FORC	8-18	DOLOMITE ROCK WITH SHALE LAYERS CORE RECOVERY-77%	95	126.3		
					DOLOMITE ROCK WITH SHALE LAYERS CORE RECOVERY-95%	101.2	120.1		
					BOTTOM OF HOLE				

R. REMOULDED

% WATER CONTENT

NATURAL

LIQUID LIMIT

PLASTIC LIMIT

PLATE

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SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS

HIGHWAY #15 AT TRANS-CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 215.0

DATE JULY-23-1959

HOLE NO.

REMARKS SEE: PLATE # 2

9

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	PROBING OR VANE TEST	
							LB. HAMMER	NO CASING
							INCH DROP	INCH DIA. ROD
							BLOWS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT.	
				GROUND SURFACE	0	215.0		
				LOGGE WELL GRADED SAND WITH A LITTLE SILT	5	210.0		
					10	205.0		
		7	9-1		11.5		0	
				VERY LOOSE CLAYEY SILT WITH A LITTLE SAND	15	200.0		
0.8, 0.6, 0.8		2	9-2		21.5	195.0	0	
				MEDIUM DENSE CLAYEY SILT WITH SOME SAND	25	190.0		
3.2, 3.4, 3.4		12	9-3		31.5	185.0	0	
					35	180.0		
		18	9-4	MEDIUM DENSE SILTY FINE SAND WITH SOME SILTY CLAY POCKETS AND A FEW PEBBLES	41.5	175.0	0 SAND	0 CLAY
					45	170.0		
		30	9-5	MEDIUM DENSE COARSE SAND & GRAVEL	51.5	165.0	0	
22 For					55	160.0		
				DENSE COARSE SAND & GRAVEL	60	155.0		
22 For					65	150.0		
		27	9-7	MEDIUM DENSE FINE SAND	70	145.0	0	
					75	140.0		
		20	9-8	MEDIUM DENSE FINE SAND WITH A LITTLE GRAVEL & A TRACE OF SILT	80	135.0	0	
					85	130.0	0 SILTY SAND	
		20	9-9	MEDIUM DENSE FINE SAND WITH CLAY & SILT LAYERS & SOME PEBBLES	86.5	128.1	0 CLAY	0 SAND
				DOLOMITE WITH SHALE LAYERS CORE RECOVERY - 51%	90	125.0		
					92.6			
				DOLOMITE WITH SHALE LAYERS CORE RECOVERY - 52%	95	120.0		
				BOTTOM OF HOLE	98.6	112.4		
							% WATER CONTENT	
							NATURAL	PLATE
							LIQUID LIMIT	S
							PLASTIC LIMIT	

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OTTAWA CANADA

SOIL PROFILE AND SUMMARY
OF FIELD AND LABORATORY TESTS

HIGHWAY #15 AT TRANS-CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 219.2'

DATE JULY-17-1959

HOLE NO.

REMARKS SEE: PLATE #2

10

CORRECTION CORRECTIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENETROMETER KIPS/FT. ²	STANDARD PENETRATION BLOWS/FT.	SAMPLE NUMBER	DESCRIPTION OF SOIL	DEPTH IN FEET	ELEVATION	PROBING OR VANE TEST	
							15. HAMMER INCH DROP	NO CASING INCH DIA. ROD
				GROUND SURFACE			BLOWS PER FOOT OR	SHEAR STRENGTH IN KIPS PER FT. ²
					0'	219.2'		
				FILL	5'	214.2'		
					10'	209.2'		
				MEDIUM DENSE COARSE SAND	15'	204.2'		
					20'	199.2'		
				VERY STIFF SILTY GRAY CLAY WITH FINE SAND LAYERS	25'	194.2'		
					30'	189.2'		
				LOOSE CLAYEY SILT	35'	184.2'		
					40'	179.2'		
				MEDIUM DENSE CLAYEY SILT	45'	174.2'		
					50'	169.2'		
				LOOSE SILTY FINE SAND	55'	164.2'		
					60'	159.2'		
				MEDIUM DENSE, FINE SAND WITH SOME SILT	65'	154.2'		
					70'	149.2'		
				MEDIUM DENSE SANDY SILT WITH CLAY LAYERS	75'	144.2'		
					78.5'	140.9'		
				DOLOMITE WITH SHALE LAYERS CORE RECOVERY - 93%	80'	139.2'		
					83.2'			
				DOLOMITE WITH SHALE LAYERS CORE RECOVERY - 37%	85'	134.2'		
				BOTTOM OF HOLE	88'	130.5'		

WATER CONTENT	PLATE
NATURAL	G
LIQUID LIMIT	
PLASTIC LIMIT	

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CONSULTING ENGINEERS

OTTAWA CANADA

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

HIGHWAY #15 AT TRANS-CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 222.4'

DATE JULY 29-59

HOLE NO.

REMARKS SEE PLATE #2

12

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	PROBING OR VANE TEST				
							LB. HAMMER INCH DROP	NO CASING INCH DIA. ROD			
				GROUND SURFACE	0	222.4'	BLOWS PER FOOT OR SHEAR STRENGTH IN KIPS PER FT. ²				
					10'	212.4'	← OVER-NIGHT WATER LEVEL - 8.1'				
					20'	202.4'					
				SANDY SOIL	30'	192.4'					
					40'	182.4'					
					50'	172.4'					
					60'	162.4'					
					71.1'						
				WEATHERED DOLOMITE WITH SHALE LAYERS CORE RECOVERY - 67%	74.2'	148.2'					
				BOTTOM OF HOLE							
							20	40	60	80	100
							% WATER CONTENT			PLATE 8	
							NATURAL <input type="checkbox"/>				
							LIQUID LIMIT <input type="checkbox"/>				
							PLASTIC LIMIT <input type="checkbox"/>				

SOIL PROFILE AND SUMMARY OF FIELD AND LABORATORY TESTS

HIGHWAY #15 AT TRANS-CANADA

ELEVATION OF GROUND SURFACE (ZERO DEPTH) 222.3'

DATE JULY-27-1959

HOLE NO.

REMARKS SEE PLATE # 2

13

[illegible]

RECORD OF BOREHOLE No 1 (Formerly WP 909-64) METRIC

W P 124-87-01 LOCATION Co-Ords N 5022 813.1; E 359 153.5 ORIGINATED BY PLW
 DIST 9 HWY 417 BOREHOLE TYPE Washboring & Diamond Drill COMPILED BY LP
 DATUM Geodetic DATE 1966 03 02 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N' VALUES			20 40 60 80 100	100	Wp	W	Wl		
65.8	Ground Surface													
0.0	Silty Clay to Clayey Silt Some Organics Some Gravel and Sand Firm to Stiff Grey		1	AS	-									
			2	TW	PM									
			3	SS	7									
			4	TW	PM									
63.1			5	SS	10									
2.7	Silty Sand to Sand Loose, Grey		6	AS	-									
			7	SS	8									
			8	SS	8									
60.3			9	TW	PM									
59.4	Clayey Silt Very Stiff		10	SS	11									
6.4	Silty Sand to Sand Clayey Silt Pockets		11	SS	11									
57.8	Compact		12	TW	PM									
8.0	Clayey Silt with Interbedded Sandy Silt Stiff to Very Stiff		13	SS	10									
			14	SS	19									
			15	SS	15									
			16	SS	20									
52.7			17	SS	40									
13.1			18	SS	57									
	Silty Sand to Sand OCC. Gravelly Sand Layers Compact to very Dense		19	SS	47									
			20	SS	62									
			21	SS	68									
			22	SS	27									
	Coarse Sand		23	SS	30									
39.0	Coarse Sand													
26.8	Bedrock Shale, Sound		24	RC AXT	REC 100%									
37.5														
28.3	END OF BOREHOLE													

+3, x5: Numbers refer to 20
Sensitivity 15 + 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 2 (formerly WP 909-64) METRIC

W P 124-87-01 LOCATION Co-Ords N 5022 845.7; E 359 182.2 ORIGINATED BY PLW
 DIST 9 HWY 417 BOREHOLE TYPE Washboring & Diamond Drilling COMPILED BY LP
 DATUM Geodetic DATE 1966 03 07 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	25 50 75 100 125					
65.6	Ground Surface													
0.0	Silty Sand to Sand Compact to Very Loose		1	SS	11	*								
63.4			2	SS	3									
2.2	Clayey Silt with Interbedded Sandy Silt		3	TW	PM									
61.3	Firm to Stiff		4	SS	10									
4.3	Silty Sand to Sand Compact		5	SS	14									
60.0			6	TW	PM									
5.6	Clayey Silt with Interbedded Sandy Silt - Very Stiff		7	SS	6									
59.0	Silty Sand to Sand Loose		8	TW	PM									
57.4			9	SS	12									
8.2	Clayey Silt with Interbedded Sandy Silt Stiff		10	SS	-									
55.9	Silty Sand to Sand Loose		11	TW	PM									
54.4			12	SS	OW									
11.2	Clayey Silt with Interbedded Sandy Silt Very Stiff		13	SS	OW									
52.6			14	SS	OW									
13.0	Silty Sand to Sand OCC. Silt Layers Very Loose to Dense		15	SS	26									
			16	SS	45									
	Coarse Sand OCC. Gravel													
41.9														
23.7	Bedrock Shale, Sound		17	RC AXT	REC 98%									
40.3														
25.3	END OF BOREHOLE													
Notes: O.W. = Own Weight * Hole caved in at Elev. 65.1m Water level not established														

RECORD OF BOREHOLE No 3 (Formerly WP 909-64) METRIC

W P 124-87-01 LOCATION Co-Orda N 5022 875.0; E 359 183.5 ORIGINATED BY PLW
 DIST 9 HWY 417 BOREHOLE TYPE Washboring & Diamond Drilling COMPILED BY LP
 DATUM Geodetic DATE 1966 03 10 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
65.9	Ground Level													
0.0	Clayey Silt with Interbedded Sandy Silt Soft to Firm		1	SS	4	*								
64.1			2	TW	PM									
1.8			3	SS	14									
			4	SS	31									
	Silty Sand to Sand Some Gravel		5	SS	18									
			6	SS	26									
	Compact to Very Dense		7	SS	12									
			8	SS	11									
			9	SS	62									
55.5	Clayey Silt with Interbedded Sandy Silt		10	SS	31									
10.4			11	TW	PM									
54.3	Very Stiff to Hard		12	SS	35									
11.6			13	SS	31									
	Silty Sand to Sand		14	SS	56									
	Dense to Very Dense		15	SS	48									
44.8			16	RC	REC									
21.1	Bedrock			AXT	100%									
43.7	Shale Sound													
22.2	END OF BOREHOLE													
	* Hole caved in at Elev. 64.8m Water Level Not Established													

DEPARTMENT OF HIGHWAYS - ONTARIO

RECORD OF BOREHOLE NO. 4

FOUNDATION SECTION

MATERIALS & TESTING DIVISION

JOB 66-F-24 LOCATION Richmond Rd. & Queensway Extension, 245/26, o/s 80' Lt. ORIGINATED BY P.L.W.
W.P. 909-64 BORING DATE March 11, 1966 COMPILED BY L.P.
DATUM _____ BOREHOLE TYPE Dynamic Cone Penetration Test CHECKED BY HL

SOIL PROFILE		SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE	LIQUID LIMIT — WL		BULK DENSITY	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS / FOOT	PLASTIC LIMIT — WP	WATER CONTENT — W		
215.0										
0.0										
					210					
					200					
					190					
					180					
					170					
					160					
149.0					150					
66.0	End of Borehole				140					

RECORD OF BOREHOLE No 5

METRIC

W P 124-87-01 LOCATION Co-Ords N 5022 814.9; E 359 202.4
 DIST 9 HWY 417 BOREHOLE TYPE H-S Auger, 'B'- Casing BX Rock Core & Cone Test
 DATUM Genderic DATE 88 08 02-03
 ORIGINATED BY MS
 COMPILED BY MS
 CHECKED BY TCK

OFFICE REPORT ON SOIL EXPLORATION

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	25 50 75 100 125					
65.9	Ground Surface													
0.0	Clayey Silt (Fill)													GR SA SI CL
64.8														
1.1	Silty Sand to Sand		1	SS	11									
	Trace to Some Clay		2	SS	6									
			3	SS	4									
			4	SS	14									
	Loose to Compact		5	SS	11									
61.5			6	SS	5									0 77 13 10
4.4	Clayey Silt with Interbedded Sandy Silt Occ. Silty Clay Layers		7	TW	PH									0 4 71 25
	Firm to Very Stiff		8	TW	PH									0 13 69 18
57.3			9	SS	25									
8.6	Silty Sand to Sand		10	SS	30									0 68 21 11
	Compact to Dense		11	SS	15									
54.2			12	SS	25									
11.7	Clayey Silt with Interbedded Sandy Silt		13	SS	4									0 23 60 17
	Firm to Very Stiff		14	SS	8									
51.2			15	SS	4									
14.7	Silty Sand to Sand Occ. Clayey Silt Layers Occ. Gravelly Sand		16	SS	90/	15cm								0 73 20 7
	Loose to Very Dense													
	Clayey Silt													0 58 32 10
41.8														
24.1	Bedrock		17	RC	REC 93%									RQD = 75%
	Silty Dolostone		18	RC	REC 90%									RQD = 56%
39.8														
26.1	END OF BOREHOLE													

RECORD OF BOREHOLE No 6

METRIC

W P 124-87-01 LOCATION Co-Ords N 5 022 884.2; E 359 214.1 ORIGINATED BY MS
 DIST 9 HWY 417 BOREHOLE TYPE B-Casing, BX Rock Core COMPILED BY MS
 DATUM Geodetic DATE 88 08 09 CHECKED BY TCK

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
72.5	Ground Surface																
0.0	Concrete Slab		1	RC			72										GR SA SI CL
71.9																	
0.6																	
	Sand Fill						70										
							68										
66.4																	
6.1	Silty Sand to Sand Compact		2	SS	10		66										
			3	SS	23												
63.9							64										0 46 36 18
8.6	Clayey Silt with interbedded Silty Sand Stiff to very Stiff		4	SS	11												
			5	SS	29		62										0 46 37 17
60.9																	
11.6	Silty Sand to Sand Trace of Clay Very Dense		6	SS	63		60										
			7	SS	52												0 78 16 6
57.9							58										
14.6	Bedrock Sandstone, Shale and Silty Sandstone		8	RC	REC 89%												RQD = 8%
			9	RC	REC 87%		56										RQD = 8%
54.8																	
17.7	END OF BOREHOLE																

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



APPENDIX D

Results of MASW Testing

DATE June 12, 2016**PROJECT No.** 1546542**TO** Kim Lesage
Golder Associates Ltd.**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.



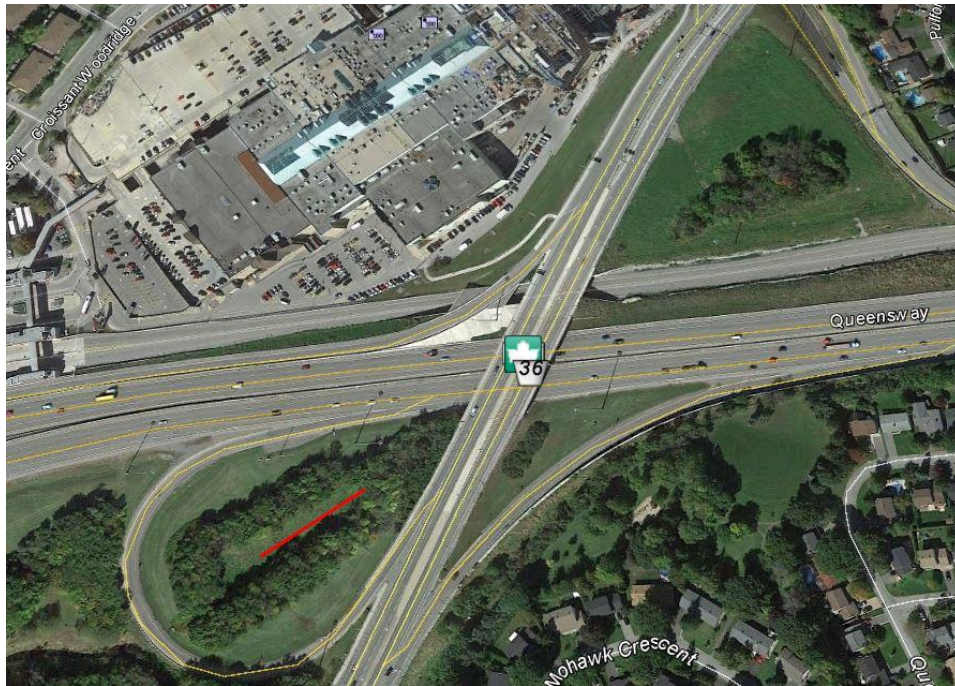


Figure 1: MASW Location Site Map at the Richmond site (MASW Line 1 in red)



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

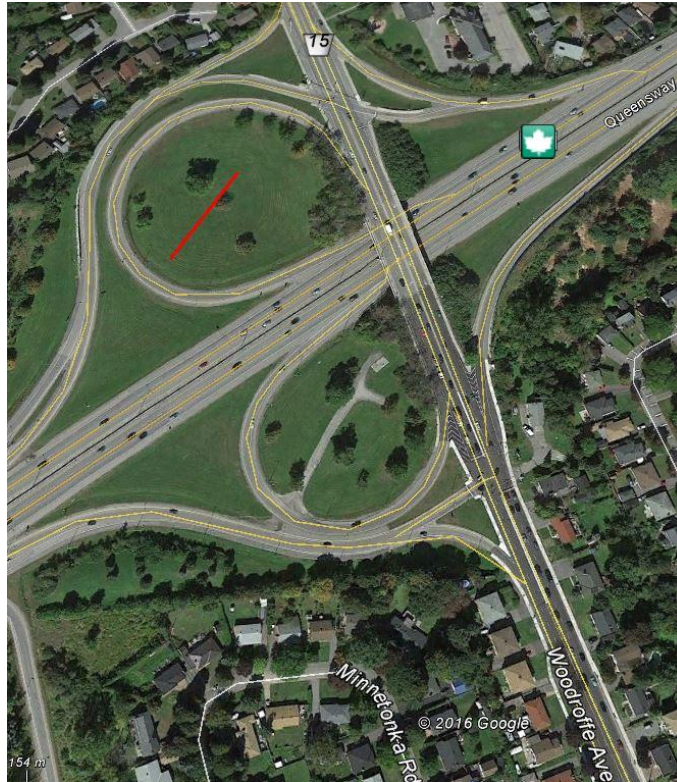


Figure 3: MASW Location Site Map at the Woodroffe site (MASW Line 3 in red)

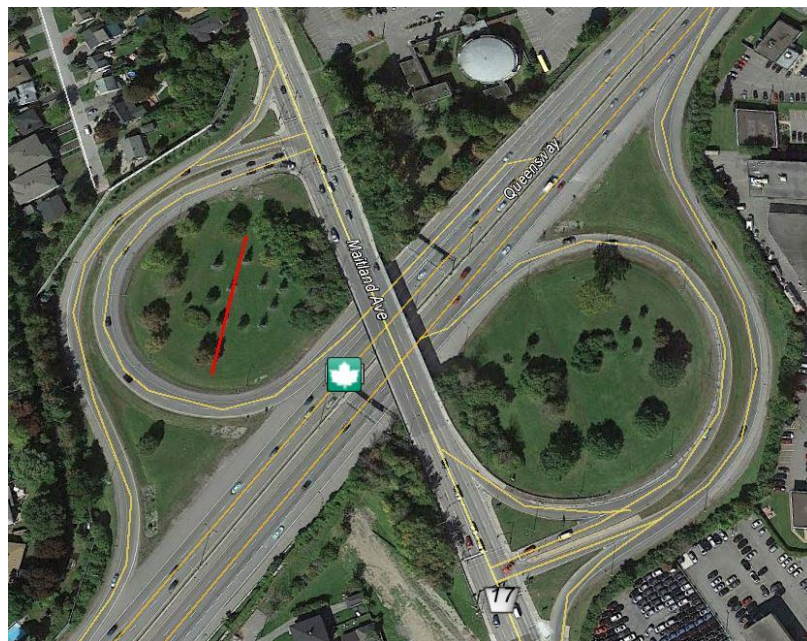


Figure 4: MASW Location Site Map at the Maitland site (MASW Line 4 in red)

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.

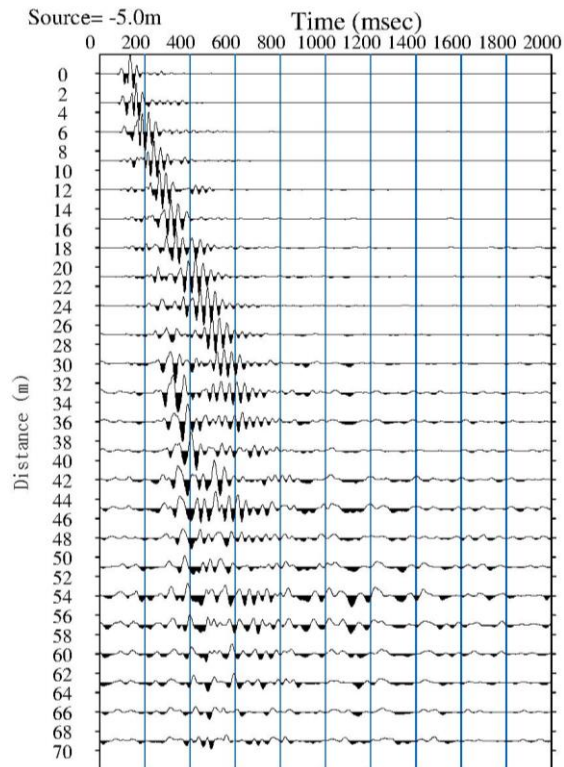


Figure 5: Typical seismic record collected along MASW Line 1

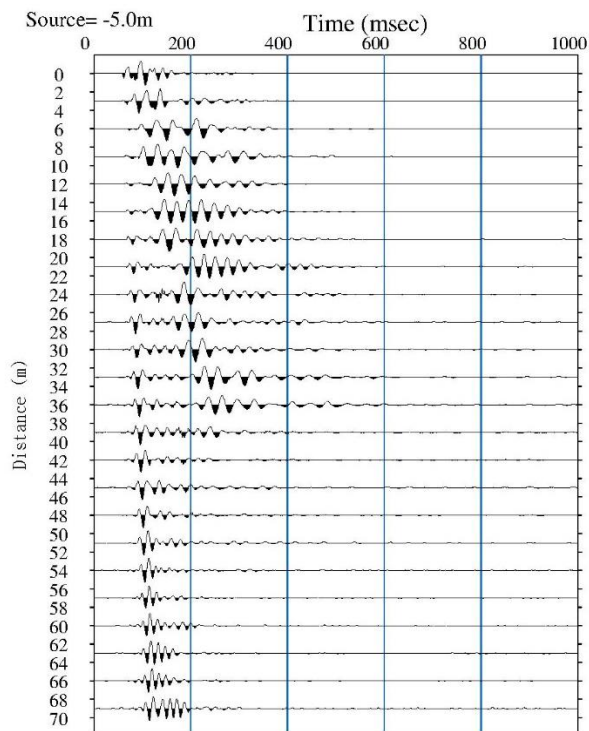


Figure 6: Typical seismic record collected along MASW Line 2

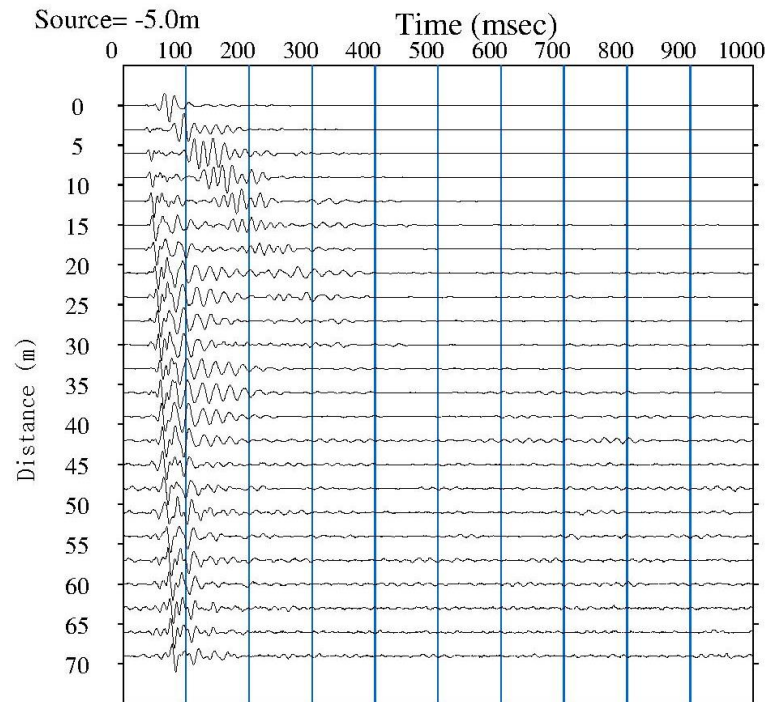


Figure 7: Typical seismic record collected along MASW Line 3

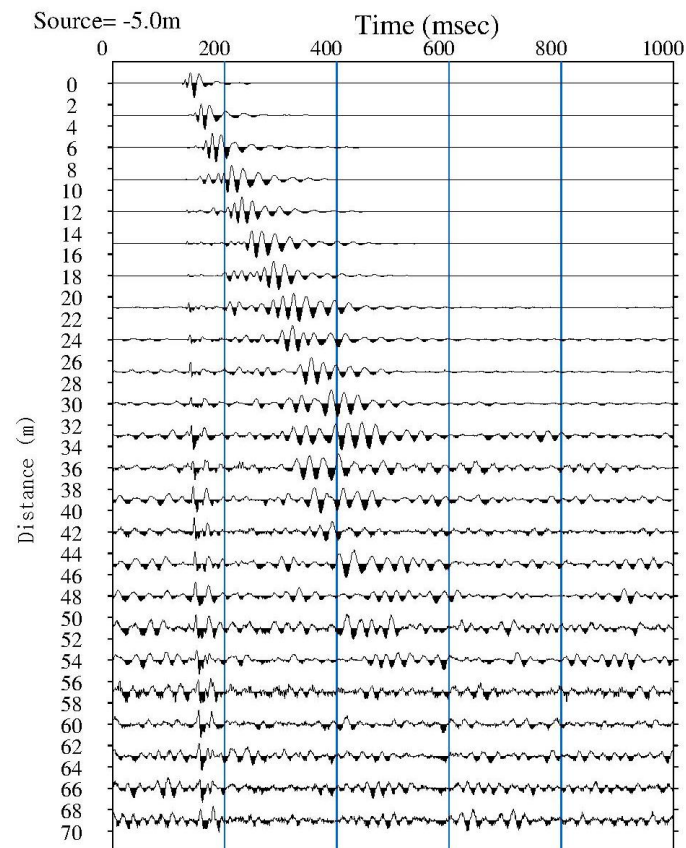


Figure 8: Typical seismic record collected along MASW Line 4

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.

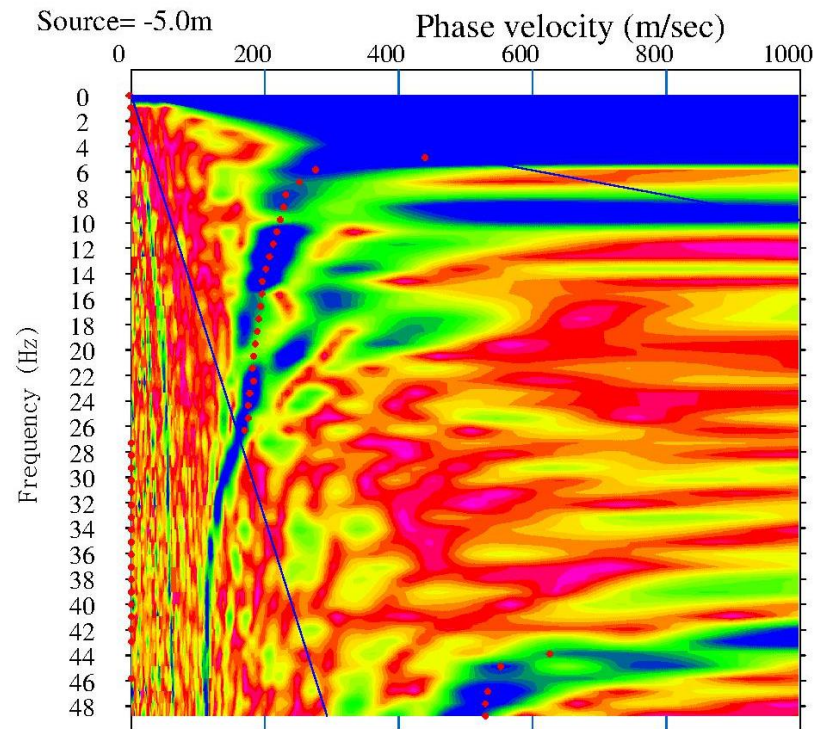


Figure 9: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1 (Richmond)

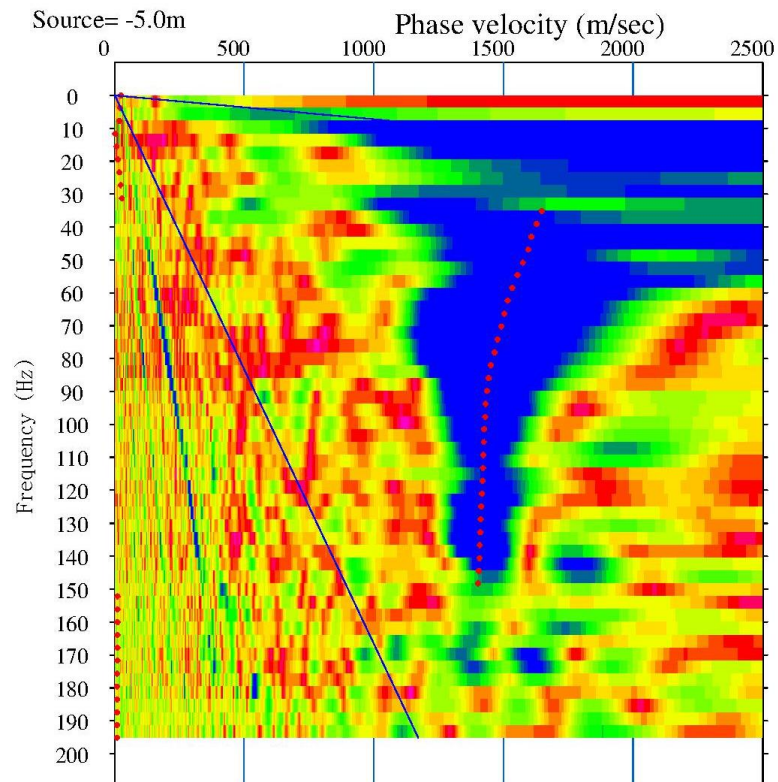


Figure 10: Active MASW Dispersion Curve Picks (red dots) along MASW Line 2 (Pinecrest).

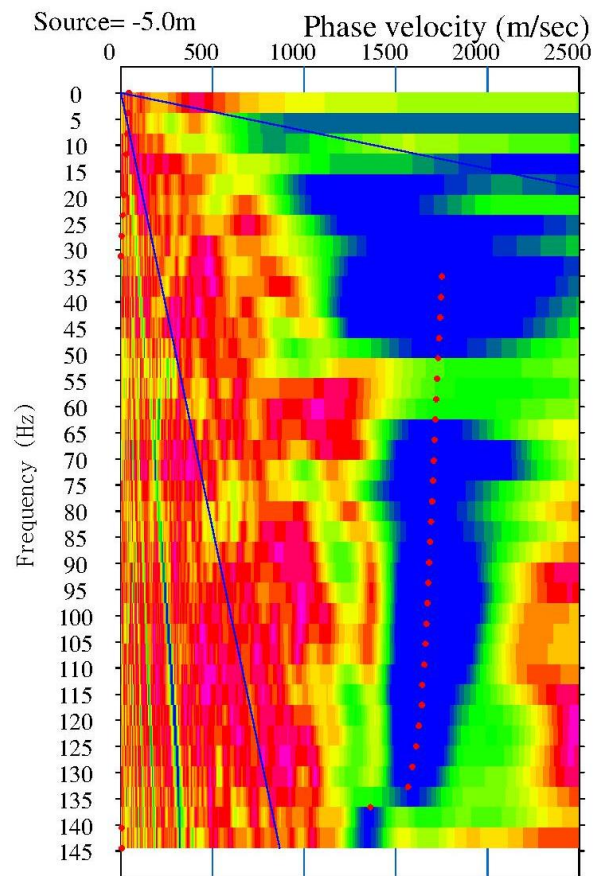


Figure 11: Active MASW Dispersion Curve Picks (red dots) along MASW Line 3 (Woodroffe).

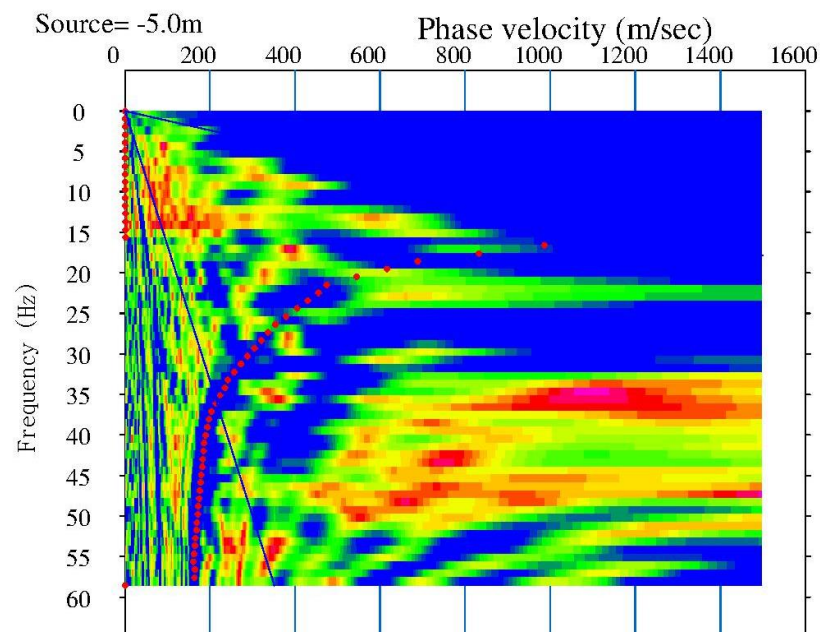


Figure 12: Active MASW Dispersion Curve Picks (red dots) along MASW Line 4 (Maitland).

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.

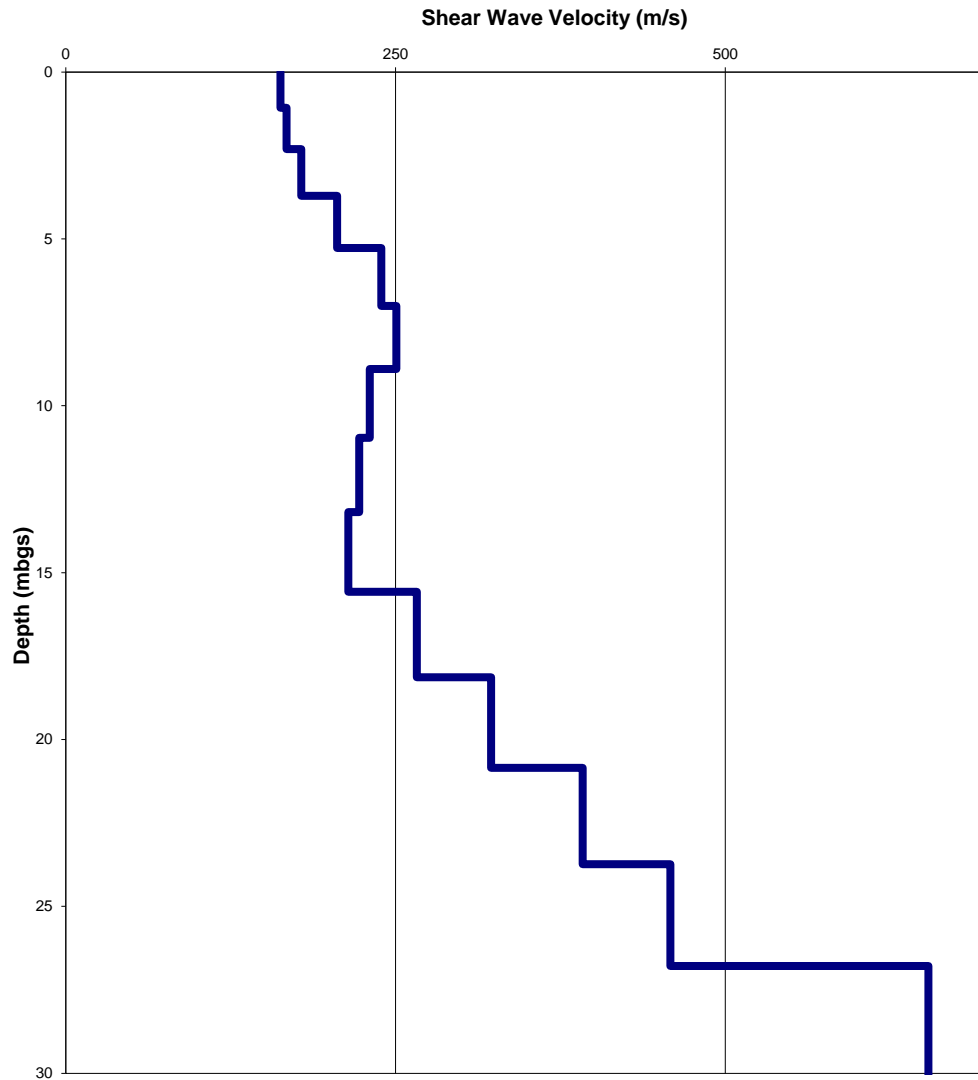


Figure 13: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

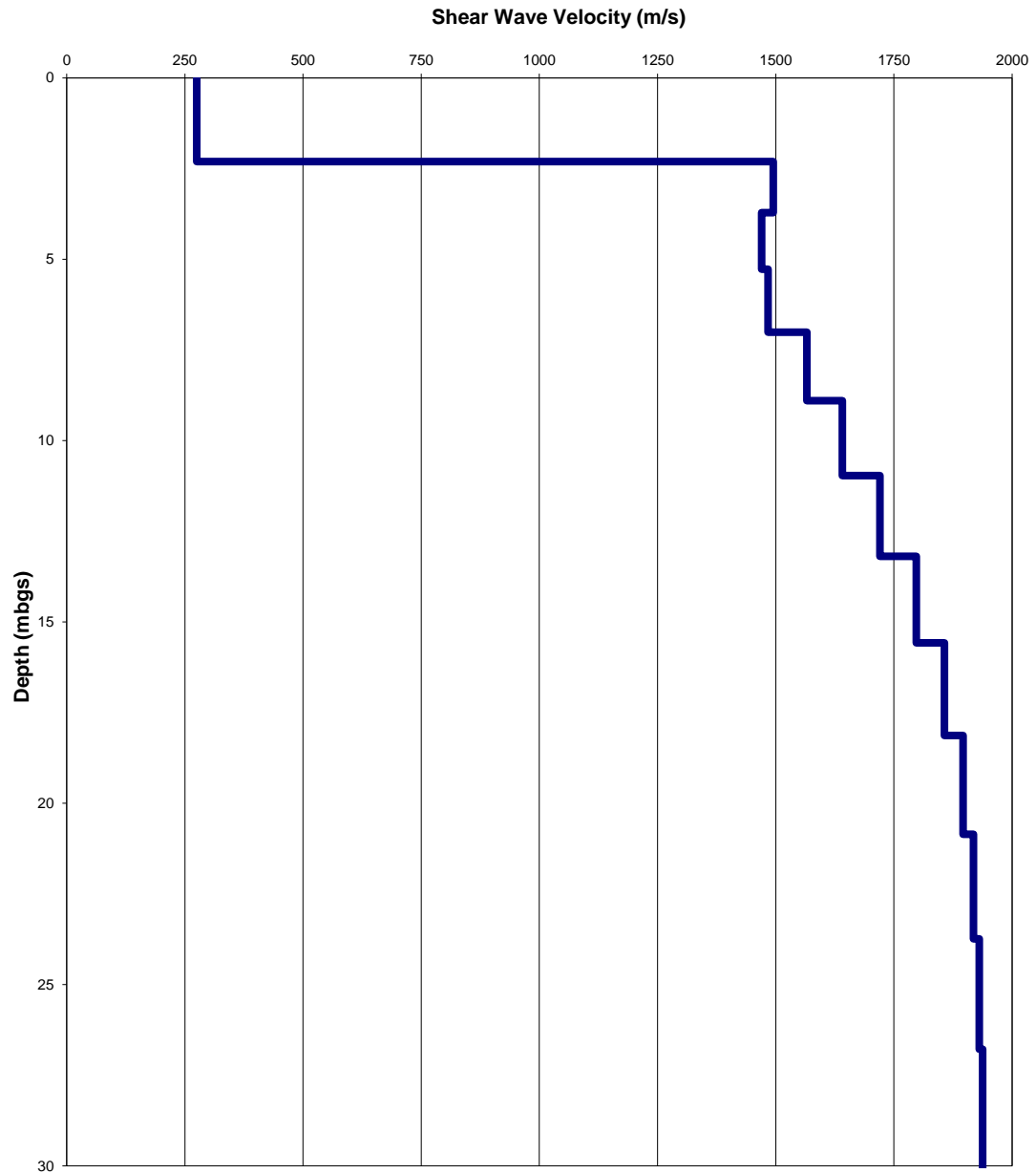


Figure 14: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 2

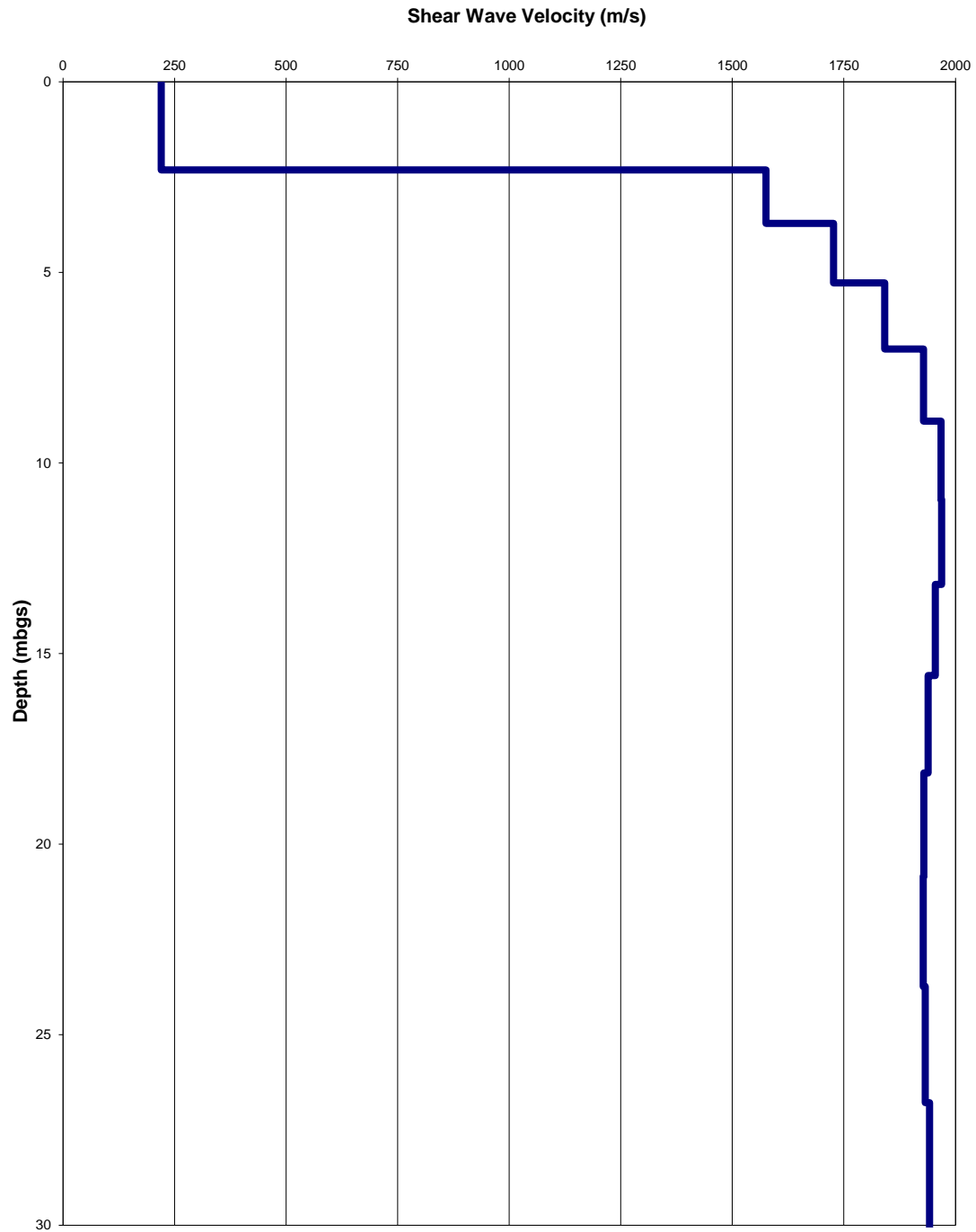


Figure 15: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 3

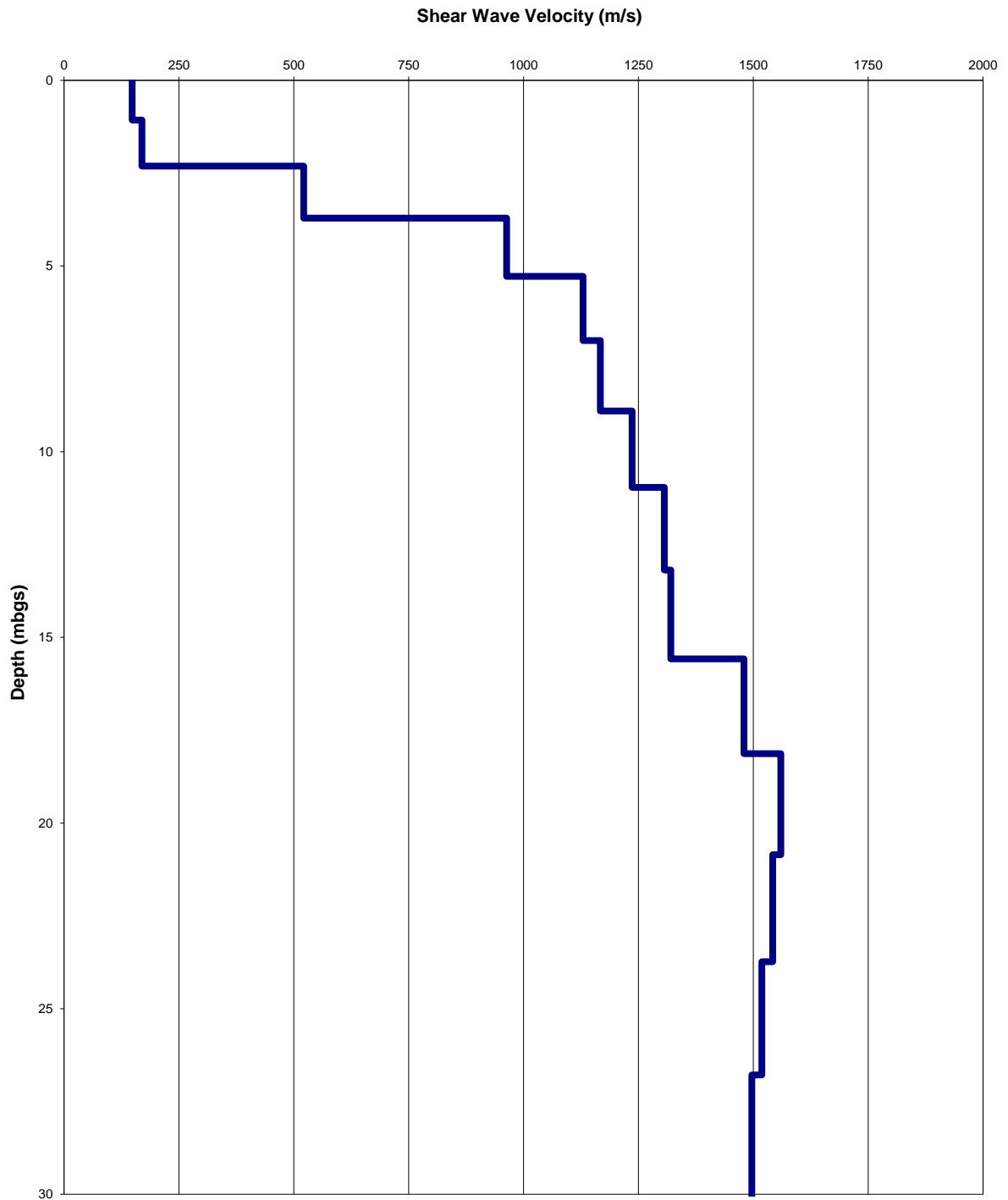


Figure 16: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 4

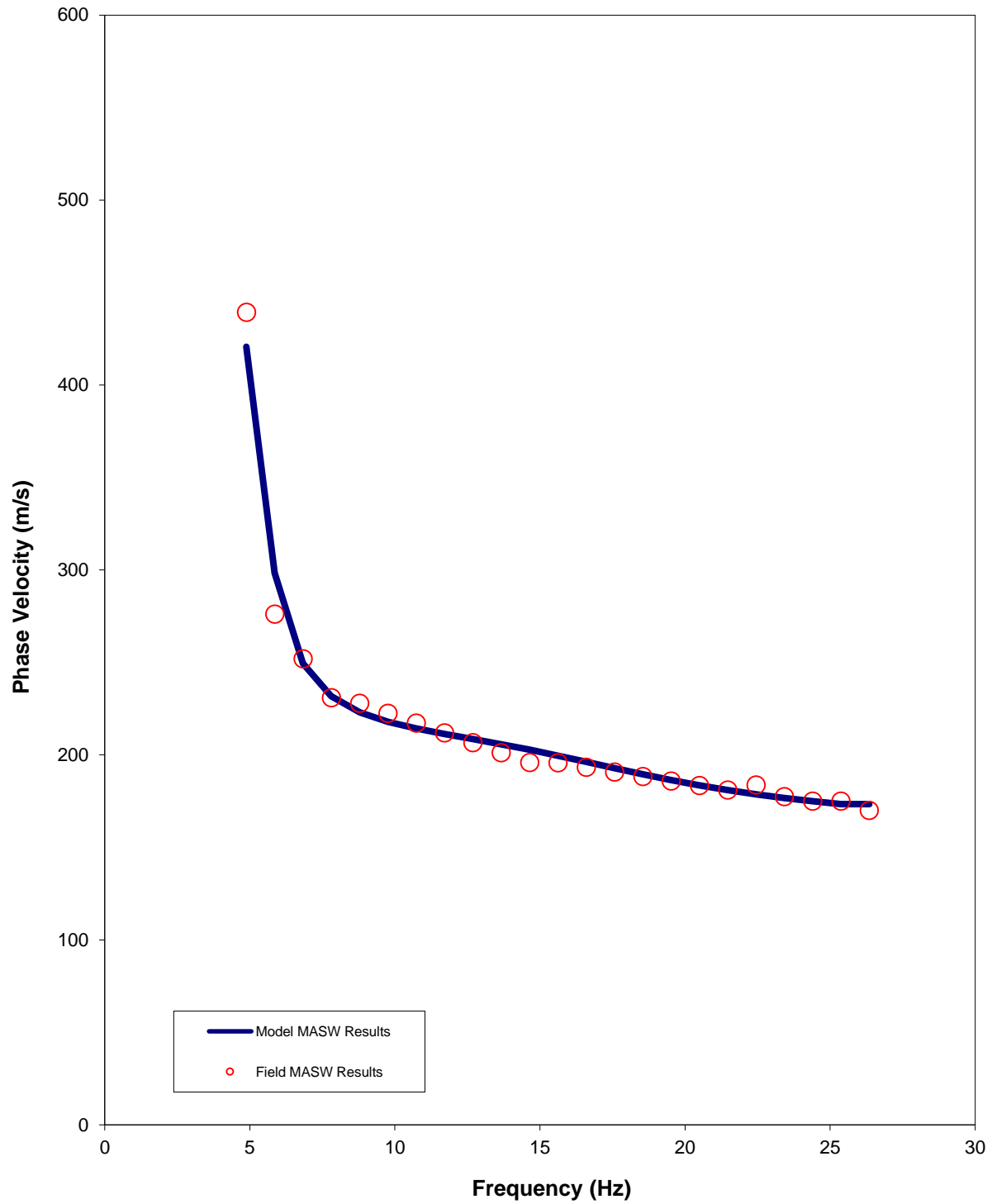


Figure 17: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

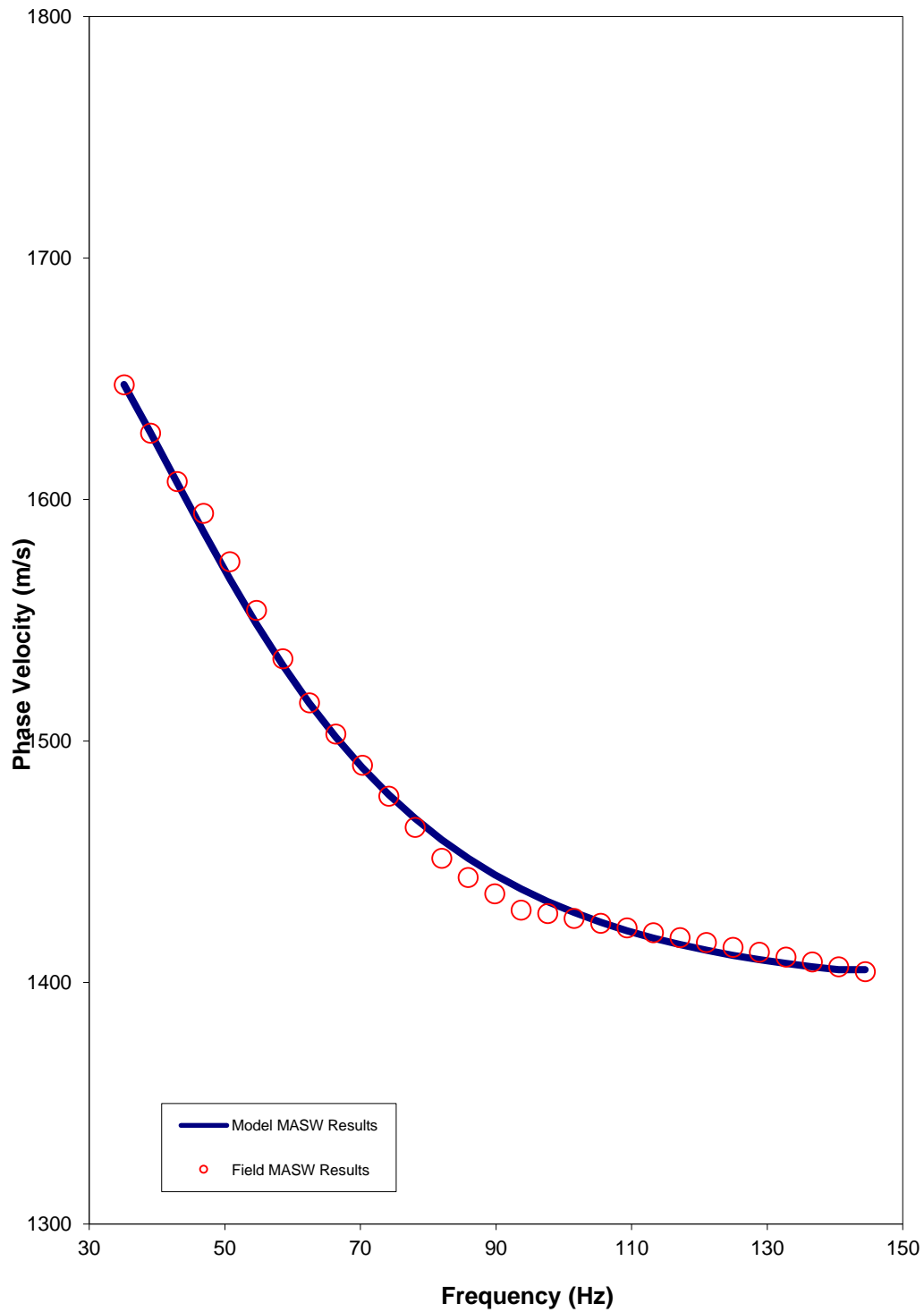


Figure 18: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 2

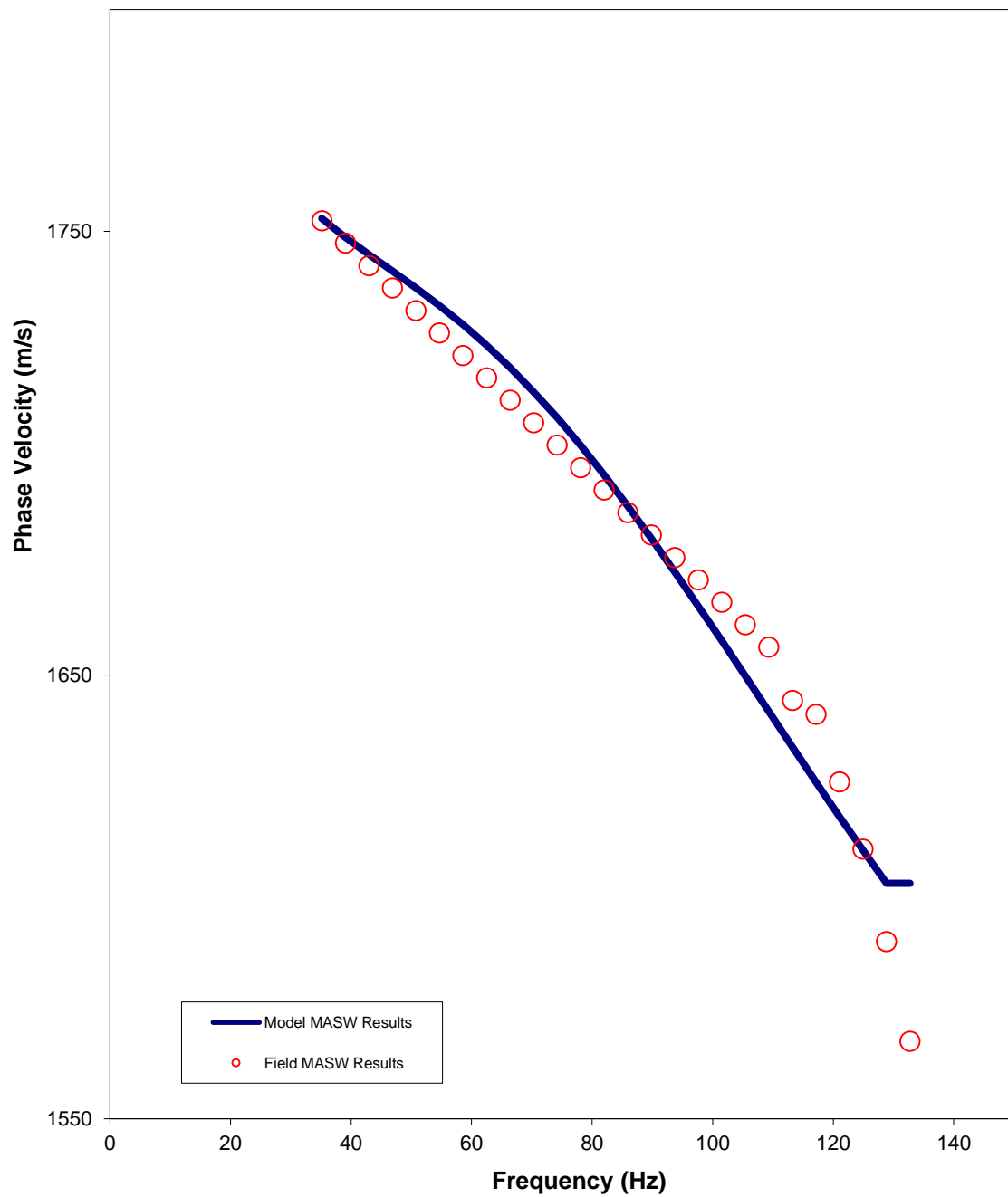


Figure 19: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

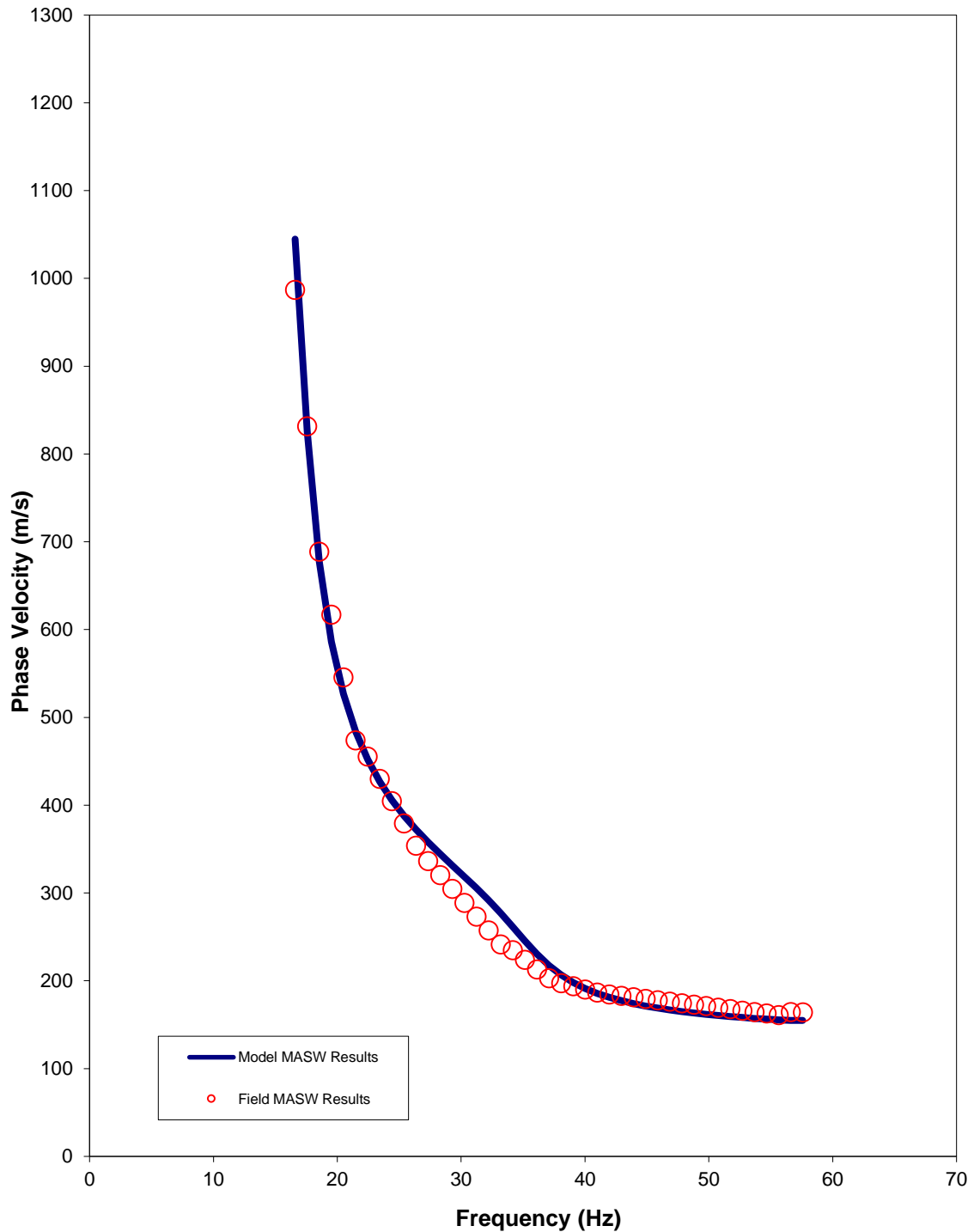


Figure 20: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

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 (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	163	0.006580
1.07	2.31	1.24	167	0.007384
2.31	3.71	1.40	178	0.007852
3.71	5.27	1.57	206	0.007613
5.27	7.01	1.73	239	0.007239
7.01	8.90	1.90	251	0.007562
8.90	10.96	2.06	230	0.008945
10.96	13.19	2.23	223	0.010001
13.19	15.58	2.39	214	0.011154
15.58	18.13	2.55	266	0.009603
18.13	20.85	2.72	322	0.008436
20.85	23.74	2.88	392	0.007361
23.74	26.79	3.05	458	0.006652
26.79	30.00	3.21	654	0.004914
Vs Average to 30 mbgs (m/s)			270	

Table 2: Shear-Wave Velocity Profile along MASW Line 2 (Pinecrest)Table 3:

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

Table 4: Shear-Wave Velocity Profile along MASW Line 3 (Woodroffe)

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	220	0.004870
1.07	2.31	1.24	220	0.005619
2.31	3.71	1.40	1575	0.000889
3.71	5.27	1.57	1727	0.000907
5.27	7.01	1.73	1841	0.000940
7.01	8.90	1.90	1929	0.000983
8.90	10.96	2.06	1968	0.001047
10.96	13.19	2.23	1969	0.001130
13.19	15.58	2.39	1955	0.001223
15.58	18.13	2.55	1939	0.001318
18.13	20.85	2.72	1929	0.001410
20.85	23.74	2.88	1928	0.001496
23.74	26.79	3.05	1932	0.001578
26.79	30.00	3.21	1942	0.001655
Vs Average to 30 mbgs (m/s)			1197	

Table 5: Shear-Wave Velocity Profile along MASW Line 4 (Maitland)

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

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.

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APPENDIX E

SCPT Report (16-104)

PRESENTATION OF SITE INVESTIGATION RESULTS

Hwy 417 at Richmond Rd.

Prepared for:

Golder Associates

ConeTec Job No: 16-05039

Project Start Date: 14-Nov-2016

Project End Date: 15-Nov-2016

Report Date: 17-Nov-2016

Revised Date: 21-Nov-2016



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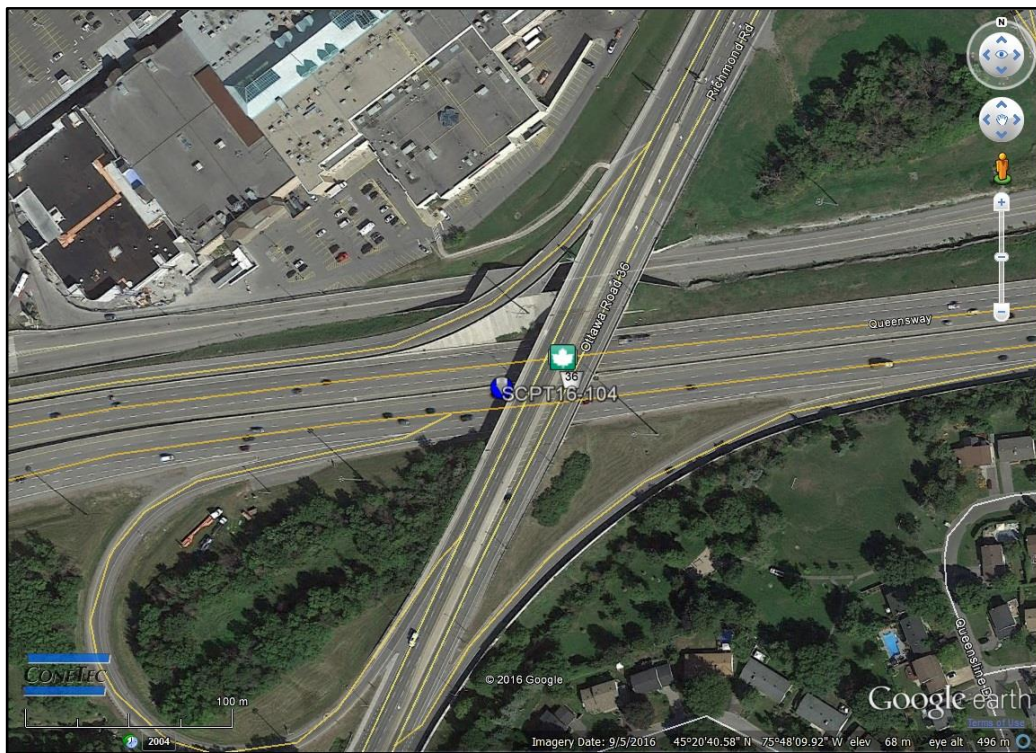
Introduction

The enclosed report presents the results of the site investigation program conducted by ConeTec Investigations Ltd. for Golder Associates at Hwy 417 and Richmond Rd., Ottawa, ON. The program consisted of one seismic cone penetration test.

Project Information

Project	
Client	Golder Associates
Project	Hwy 417 at Richmond Rd.
ConeTec project number	16-05039

A map from Google earth including the SCPT test location is presented below.



Rig Description	Deployment System	Test Type
CPT Truck (C-3)	30 ton rig cylinder	SCPT

Coordinates		
Test Type	Collection Method	EPSG Number
SCPT	Google Earth	32618

Cone Penetration Test (CPT)	
Depth reference	Depths are referenced to the existing ground surface at the time of each test.
Tip and sleeve data offset	0.1 meter This has been accounted for in the CPT data files.
Additional plots	Expanded range standard plot; advanced plot with I_c , $S_u(Nkt)$ and $N1(60)$; seismic CPT plot.

Cone Penetrometers Used for this Project						
Cone Description	Cone Number	Cross Sectional Area (cm ²)	Sleeve Area (cm ²)	Tip Capacity (bar)	Sleeve Capacity (bar)	Pore Pressure Capacity (psi)
419:T1500F15U500	419	15	225	1500	15	500
Cone 419 was used for all CPT soundings.						

Interpretation Tables	
Additional information	<p>The Soil Behaviour Type (SBT) classification chart (Robertson et al., 1986 presented by Lunne, Robertson and Powell, 1997) was used to classify the soil for this project. A detailed set of CPT interpretations were generated and are provided in Excel format files in the release folder. The CPT interpretations are based on values of corrected tip (q_t), sleeve friction (f_s) and pore pressure (u_2).</p> <p>Soils were classified as either drained or undrained based on the Soil Behaviour Type (SBT) classification chart. Calculations for both drained and undrained parameters were included for materials that classified as silt (zone 6), and for silty sand (zone 7).</p>

Limitations

This report has been prepared for the exclusive use of Golder Associates (Client) for the project titled "Hwy 417 at Richmond Rd.". The report's contents may not be relied upon by any other party without the express written permission of ConeTec Investigations Ltd. (ConeTec). ConeTec has provided site investigation services, prepared the factual data reporting, and provided geotechnical parameter calculations consistent with current best practices. No other warranty, expressed or implied, is made.

The information presented in the report document and the accompanying data set pertain to the specific project, site conditions and objectives described to ConeTec by the Client. In order to properly understand the factual data, assumptions and calculations, reference must be made to the documents provided and their accompanying data sets, in their entirety.

The cone penetration tests (CPTu) are conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada.

ConeTec's piezocone penetrometers are compression type designs in which the tip and friction sleeve load cells are independent and have separate load capacities. The piezocones use strain gauged load cells for tip and sleeve friction and a strain gauged diaphragm type transducer for recording pore pressure. The piezocones also have a platinum resistive temperature device (RTD) for monitoring the temperature of the sensors, an accelerometer type dual axis inclinometer and a geophone sensor for recording seismic signals. All signals are amplified down hole within the cone body and the analog signals are sent to the surface through a shielded cable.

ConeTec penetrometers are manufactured with various tip, friction and pore pressure capacities in both 10 cm² and 15 cm² tip base area configurations in order to maximize signal resolution for various soil conditions. The specific piezocone used for each test is described in the CPT summary table presented in the first Appendix. The 15 cm² penetrometers do not require friction reducers as they have a diameter larger than the deployment rods. The 10 cm² piezocones use a friction reducer consisting of a rod adapter extension behind the main cone body with an enlarged cross sectional area (typically 44 mm diameter over a length of 32 mm with tapered leading and trailing edges) located at a distance of 585 mm above the cone tip.

The penetrometers are designed with equal end area friction sleeves, a net end area ratio of 0.8 and cone tips with a 60 degree apex angle.

All ConeTec piezocones can record pore pressure at various locations. Unless otherwise noted, the pore pressure filter is located directly behind the cone tip in the "u₂" position (ASTM Type 2). The filter is 6 mm thick, made of porous plastic (polyethylene) having an average pore size of 125 microns (90-160 microns). The function of the filter is to allow rapid movements of extremely small volumes of water needed to activate the pressure transducer while preventing soil ingress or blockage.

The piezocone penetrometers are manufactured with dimensions, tolerances and sensor characteristics that are in general accordance with the current ASTM D5778 standard. ConeTec's calibration criteria also meets or exceeds those of the current ASTM D5778 standard. An illustration of the piezocone penetrometer is presented in Figure CPTu.

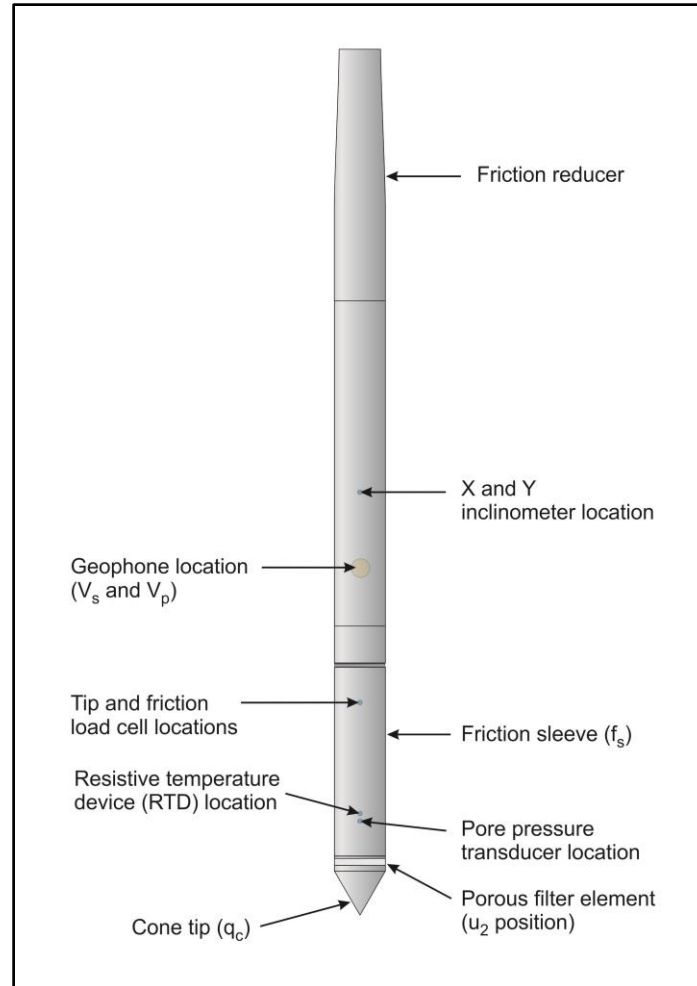


Figure CPTu. Piezocone Penetrometer (15 cm²)

The ConeTec data acquisition systems consist of a Windows based computer and a signal conditioner and power supply interface box with a 16 bit (or greater) analog to digital (A/D) converter. The data is recorded at fixed depth increments using a depth wheel attached to the push cylinders or by using a spring loaded rubber depth wheel that is held against the cone rods. The typical recording intervals are either 2.5 cm or 5.0 cm depending on project requirements; custom recording intervals are possible. The system displays the CPTu data in real time and records the following parameters to a storage media during penetration:

- Depth
- Uncorrected tip resistance (q_c)
- Sleeve friction (f_s)
- Dynamic pore pressure (u)
- Additional sensors such as resistivity, passive gamma, ultra violet induced fluorescence, if applicable

All testing is performed in accordance to ConeTec's CPT operating procedures which are in general accordance with the current ASTM D5778 standard.

Prior to the start of a CPTu sounding a suitable cone is selected, the cone and data acquisition system are powered on, the pore pressure system is saturated with either glycerine or silicone oil and the baseline readings are recorded with the cone hanging freely in a vertical position.

The CPTu is conducted at a steady rate of 2 cm/s, within acceptable tolerances. Typically one meter length rods with an outer diameter of 1.5 inches are added to advance the cone to the sounding termination depth. After cone retraction final baselines are recorded.

Additional information pertaining to ConeTec's cone penetration testing procedures:

- Each filter is saturated in silicone oil or glycerine under vacuum pressure prior to use
- Recorded baselines are checked with an independent multi-meter
- Baseline readings are compared to previous readings
- Soundings are terminated at the client's target depth or at a depth where an obstruction is encountered, excessive rod flex occurs, excessive inclination occurs, equipment damage is likely to take place, or a dangerous working environment arises
- Differences between initial and final baselines are calculated to ensure zero load offsets have not occurred and to ensure compliance with ASTM standards

The interpretation of piezocone data for this report is based on the corrected tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u). The interpretation of soil type is based on the correlations developed by Robertson (1990) and Robertson (2009). It should be noted that it is not always possible to accurately identify a soil type based on these parameters. In these situations, experience, judgment and an assessment of other parameters may be used to infer soil behaviour type.

The recorded tip resistance (q_c) is the total force acting on the piezocone tip divided by its base area. The tip resistance is corrected for pore pressure effects and termed corrected tip resistance (q_t) according to the following expression presented in Robertson et al, 1986:

$$q_t = q_c + (1-a) \cdot u_2$$

where: q_t is the corrected tip resistance

q_c is the recorded tip resistance

u_2 is the recorded dynamic pore pressure behind the tip (u_2 position)

a is the Net Area Ratio for the piezocone (0.8 for ConeTec probes)

The sleeve friction (f_s) is the frictional force on the sleeve divided by its surface area. As all ConeTec piezocones have equal end area friction sleeves, pore pressure corrections to the sleeve data are not required.

The dynamic pore pressure (u) is a measure of the pore pressures generated during cone penetration. To record equilibrium pore pressure, the penetration must be stopped to allow the dynamic pore pressures to stabilize. The rate at which this occurs is predominantly a function of the permeability of the soil and the diameter of the cone.

The friction ratio (R_f) is a calculated parameter. It is defined as the ratio of sleeve friction to the tip resistance expressed as a percentage. Generally, saturated cohesive soils have low tip resistance, high

friction ratios and generate large excess pore water pressures. Cohesionless soils have higher tip resistances, lower friction ratios and do not generate significant excess pore water pressure.

A summary of the CPTu soundings along with test details and individual plots are provided in the appendices. A set of interpretation files were generated for each sounding based on published correlations and are provided in Excel format in the data release folder. Information regarding the interpretation methods used is also included in the data release folder.

For additional information on CPTu interpretations, refer to Robertson et al. (1986), Lunne et al. (1997), Robertson (2009), Mayne (2013, 2014) and Mayne and Peuchen (2012).

Shear wave velocity testing is performed in conjunction with the piezocone penetration test (SCPTu) in order to collect interval velocities. For some projects seismic compression wave (V_p) velocity is also determined.

ConeTec's piezocone penetrometers are manufactured with a horizontally active geophone (28 hertz) that is rigidly mounted in the body of the cone penetrometer, 0.2 meters behind the cone tip.

Shear waves are typically generated by using an impact hammer horizontally striking a beam that is held in place by a normal load. In some instances an auger source or an imbedded impulsive source maybe used for both shear waves and compression waves. The hammer and beam act as a contact trigger that triggers the recording of the seismic wave traces. For impulsive devices an accelerometer trigger may be used. The traces are recorded using an up-hole integrated digital oscilloscope which is part of the SCPTu data acquisition system. An illustration of the shear wave testing configuration is presented in Figure SCPTu-1.

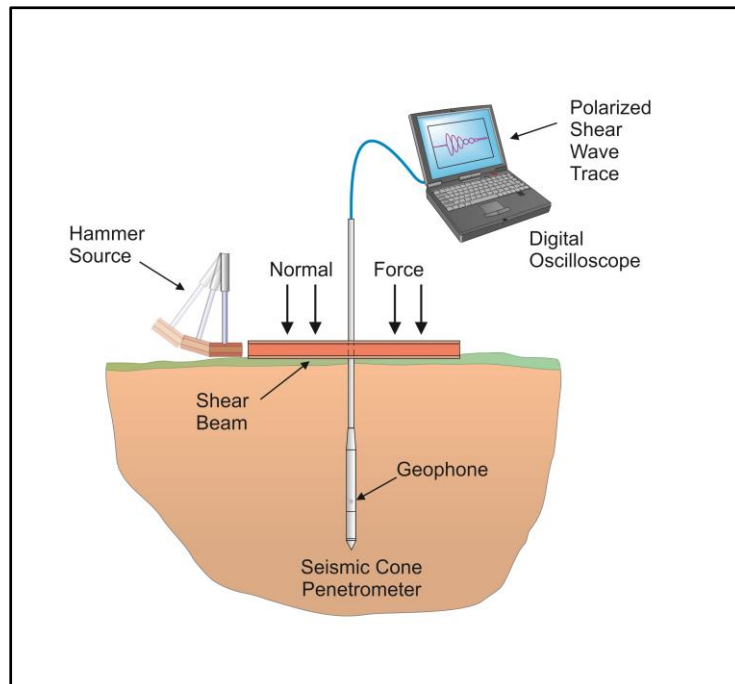


Figure SCPTu-1. Illustration of the SCPTu system

All testing is performed in accordance to ConeTec's SCPTu operating procedures.

Prior to the start of a SCPTu sounding, the procedures described in the Cone Penetration Test section are followed. In addition, the active axis of the geophone is aligned parallel to the beam (or source) and the horizontal offset between the cone and the source is measured and recorded.

Prior to recording seismic waves at each test depth, cone penetration is stopped and the rods are decoupled from the rig to avoid transmission of rig energy down the rods. Multiple wave traces are recorded for quality control purposes. After reviewing wave traces for consistency the cone is pushed to the next test depth (typically one meter intervals or as requested by the client). Figure SCPTu-2 presents an illustration of a SCPTu test.

For additional information on seismic cone penetration testing refer to Robertson et.al. (1986).

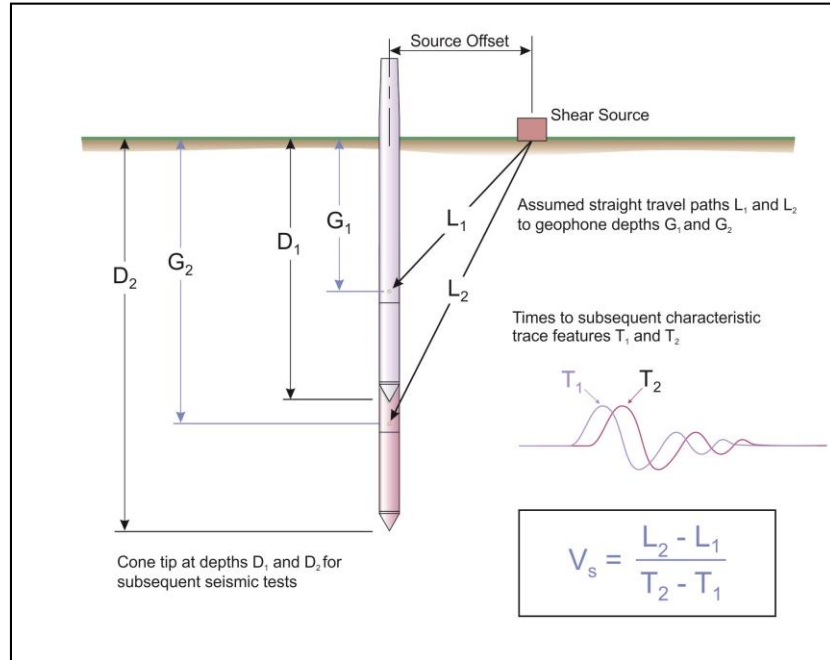


Figure SCPTu-2. Illustration of a seismic cone penetration test

Calculation of the interval velocities are performed by visually picking a common feature (e.g. the first characteristic peak, trough, or crossover) on all of the recorded wave sets and taking the difference in ray path divided by the time difference between subsequent features. Ray path is defined as the straight line distance from the seismic source to the geophone, accounting for beam offset, source depth and geophone offset from the cone tip.

The average shear wave velocity to a depth of 30 meters (V_{s30}) has been calculated and provided for all applicable soundings using an equation presented in Crow et al., 2012.

$$V_{s30} = \frac{\text{total thickness of all layers (30m)}}{\sum(\text{layer traveltimes})}$$

The layer travel times refers to the travel times propagating in the vertical direction, not the measured travel times from an offset source.

Tabular results and SCPTu plots are presented in the relevant appendix.

The cone penetration test is halted at specific depths to carry out pore pressure dissipation (PPD) tests, shown in Figure PPD-1. For each dissipation test the cone and rods are decoupled from the rig and the data acquisition system measures and records the variation of the pore pressure (u) with time (t).

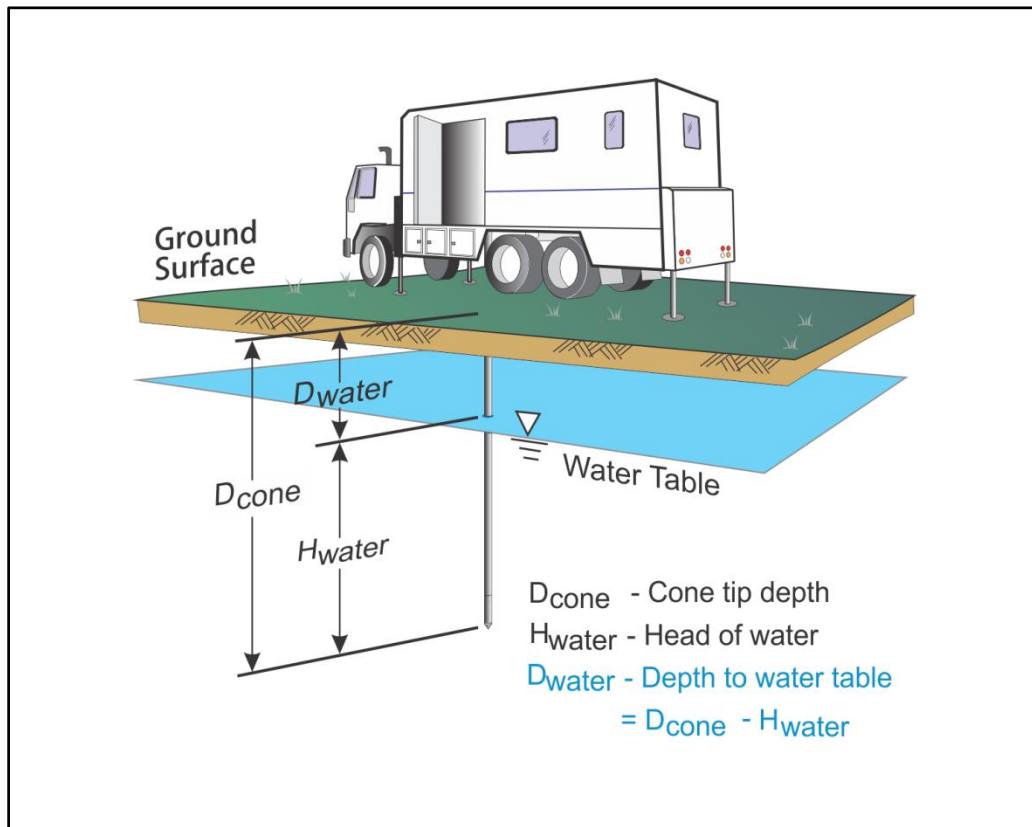


Figure PPD-1. Pore pressure dissipation test setup

Pore pressure dissipation data can be interpreted to provide estimates of ground water conditions, permeability, consolidation characteristics and soil behaviour.

The typical shapes of dissipation curves shown in Figure PPD-2 are very useful in assessing soil type, drainage, in situ pore pressure and soil properties. A flat curve that stabilizes quickly is typical of a freely draining sand. Undrained soils such as clays will typically show positive excess pore pressure and have long dissipation times. Dilative soils will often exhibit dynamic pore pressures below equilibrium that then rise over time. Overconsolidated fine-grained soils will often exhibit an initial dilatory response where there is an initial rise in pore pressure before reaching a peak and dissipating.

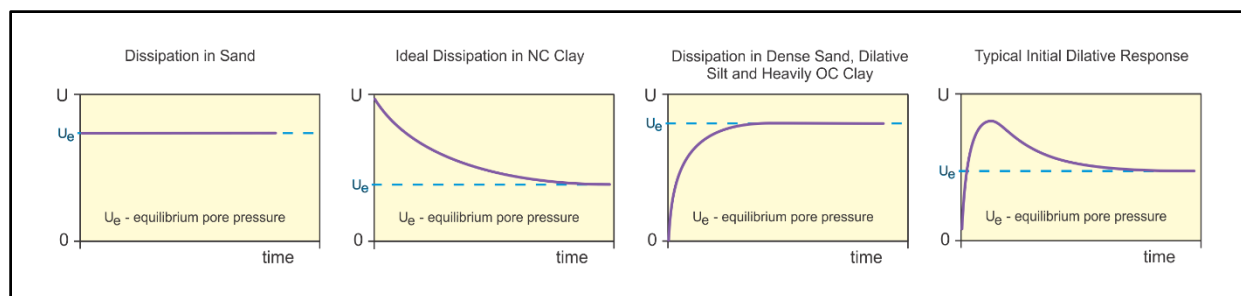


Figure PPD-2. Pore pressure dissipation curve examples

In order to interpret the equilibrium pore pressure (u_{eq}) and the apparent phreatic surface, the pore pressure should be monitored until such time as there is no variation in pore pressure with time as shown for each curve of Figure PPD-2.

In fine grained deposits the point at which 100% of the excess pore pressure has dissipated is known as t_{100} . In some cases this can take an excessive amount of time and it may be impractical to take the dissipation to t_{100} . A theoretical analysis of pore pressure dissipations by Teh and Houlsby (1991) showed that a single curve relating degree of dissipation versus theoretical time factor (T^*) may be used to calculate the coefficient of consolidation (c_h) at various degrees of dissipation resulting in the expression for c_h shown below.

$$c_h = \frac{T^* \cdot a^2 \cdot \sqrt{I_r}}{t}$$

Where:

- T^* is the dimensionless time factor (Table Time Factor)
- a is the radius of the cone
- I_r is the rigidity index
- t is the time at the degree of consolidation

Table Time Factor. T^* versus degree of dissipation (Teh and Houlsby, 1991)

Degree of Dissipation (%)	20	30	40	50	60	70	80
$T^* (u_2)$	0.038	0.078	0.142	0.245	0.439	0.804	1.60

The coefficient of consolidation is typically analyzed using the time (t_{50}) corresponding to a degree of dissipation of 50% (u_{50}). In order to determine t_{50} , dissipation tests must be taken to a pressure less than u_{50} . The u_{50} value is half way between the initial maximum pore pressure and the equilibrium pore pressure value, known as u_{100} . To estimate u_{50} , both the initial maximum pore pressure and u_{100} must be known or estimated. Other degrees of dissipations may be considered, particularly for extremely long dissipations.

At any specific degree of dissipation the equilibrium pore pressure (u at t_{100}) must be estimated at the depth of interest. The equilibrium value may be determined from one or more sources such as measuring the value directly (u_{100}), estimating it from other dissipations in the same profile, estimating the phreatic surface and assuming hydrostatic conditions, from nearby soundings, from client provided information, from site observations and/or past experience, or from other site instrumentation.

For calculations of c_h (Teh and Houlsby, 1991), t_{50} values are estimated from the corresponding pore pressure dissipation curve and a rigidity index (I_r) is assumed. For curves having an initial dilatory response in which an initial rise in pore pressure occurs before reaching a peak, the relative time from the peak value is used in determining t_{50} . In cases where the time to peak is excessive, t_{50} values are not calculated.

Due to possible inherent uncertainties in estimating I_r , the equilibrium pore pressure and the effect of an initial dilatory response on calculating t_{50} , other methods should be applied to confirm the results for c_h .

Additional published methods for estimating the coefficient of consolidation from a piezocone test are described in Burns and Mayne (1998, 2002), Jones and Van Zyl (1981), Robertson et al. (1992) and Sully et al. (1999).

A summary of the pore pressure dissipation tests and dissipation plots are presented in the relevant appendix.

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REFERENCES

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The appendices listed below are included in the report:

- Cone Penetration Test Summary and Standard Cone Penetration Test Plot
- Cone Penetration Test Plot with Expanded Range
- Advanced Cone Penetration Test Plot with I_c , $S_u(Nkt)$ and $N1(60)$
- Seismic Cone Penetration Test Tabular Results
- Seismic Cone Penetration Test Plot
- Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots

Cone Penetration Test Summary and Standard Cone Penetration Test Plot



Job No: 16-05039
Client: Golder Associates
Project: Hwy 417 at Richmond Rd.
Start Date: 14-Nov-2016
End Date: 15-Nov-2016

CONE PENETRATION TEST SUMMARY

Sounding ID	File Name	Date	Cone	Assumed Phreatic Surface ¹ (m)	Final Depth (m)	Northing ² (m)	Easting (m)	Refer to Notation Number
SCPT16-104	16-05039_SP104	14-Nov-2016	419:T1500F15U500	3.6	17.800	5021640	436855	

1. The assumed phreatic surface was based on pore pressure dissipation tests unless otherwise noted. Hydrostatic conditions were assumed for the calculated parameters.
2. Coordinates were estimated from Google Earth in datum WGS84/UTM Zone 18 North.



Golder

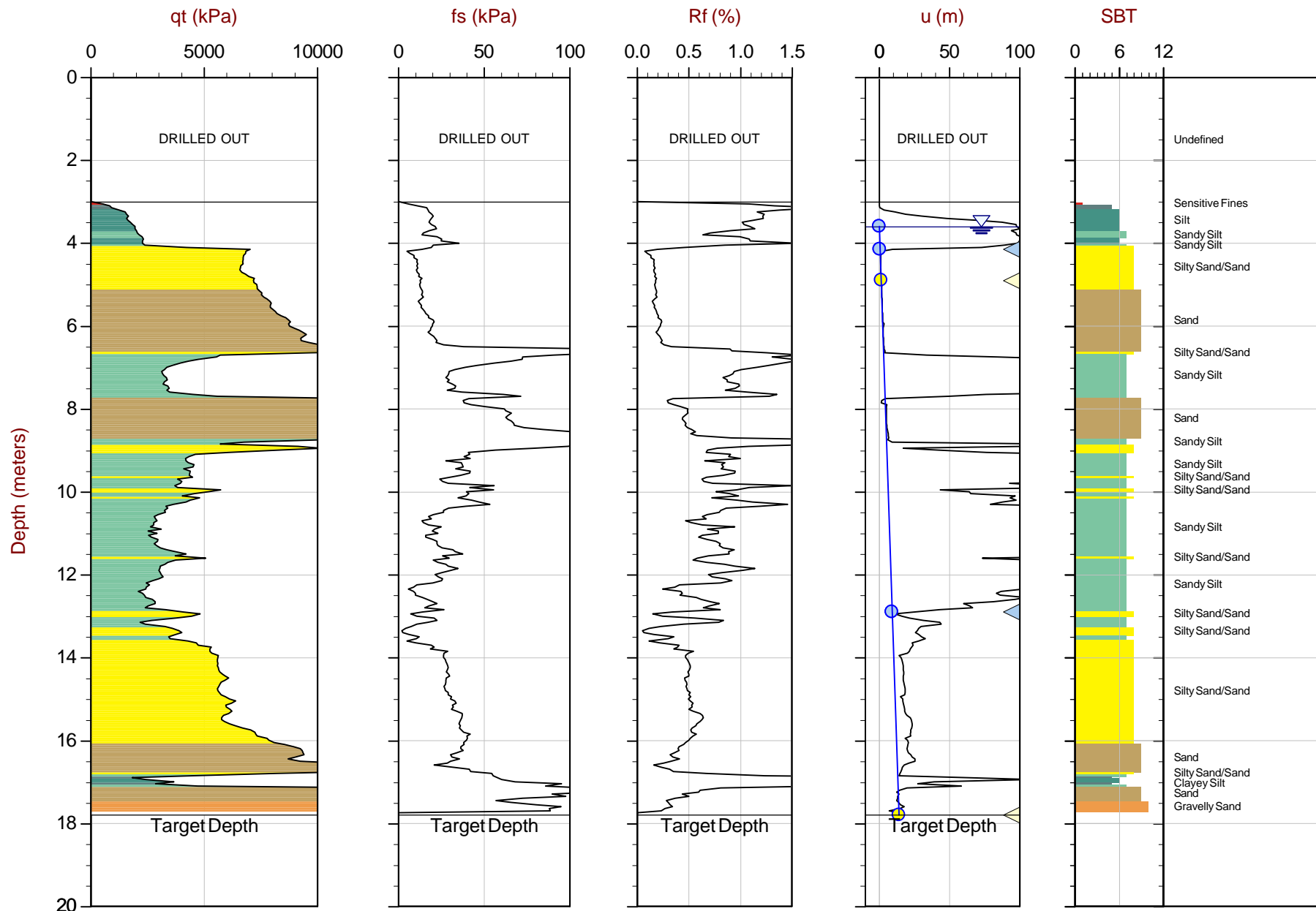
Job No: 16-05039

Date: 2016/11/14 23:13

Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104

Cone: 419:T1500F15U500



Max Depth: 17.800 m / 58.40 ft

Depth Int: 0.050 m / 0.164 ft

Avg Int: Every Point

File: 16-05039_SP104.COR

Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986

Coords: UTM18NN: 5021640m E: 436855m

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were estimated from Google Earth and are only approximate locations. The coordinates should not be used for design purposes.

Cone Penetration Test Plot with Expanded Range



Golder

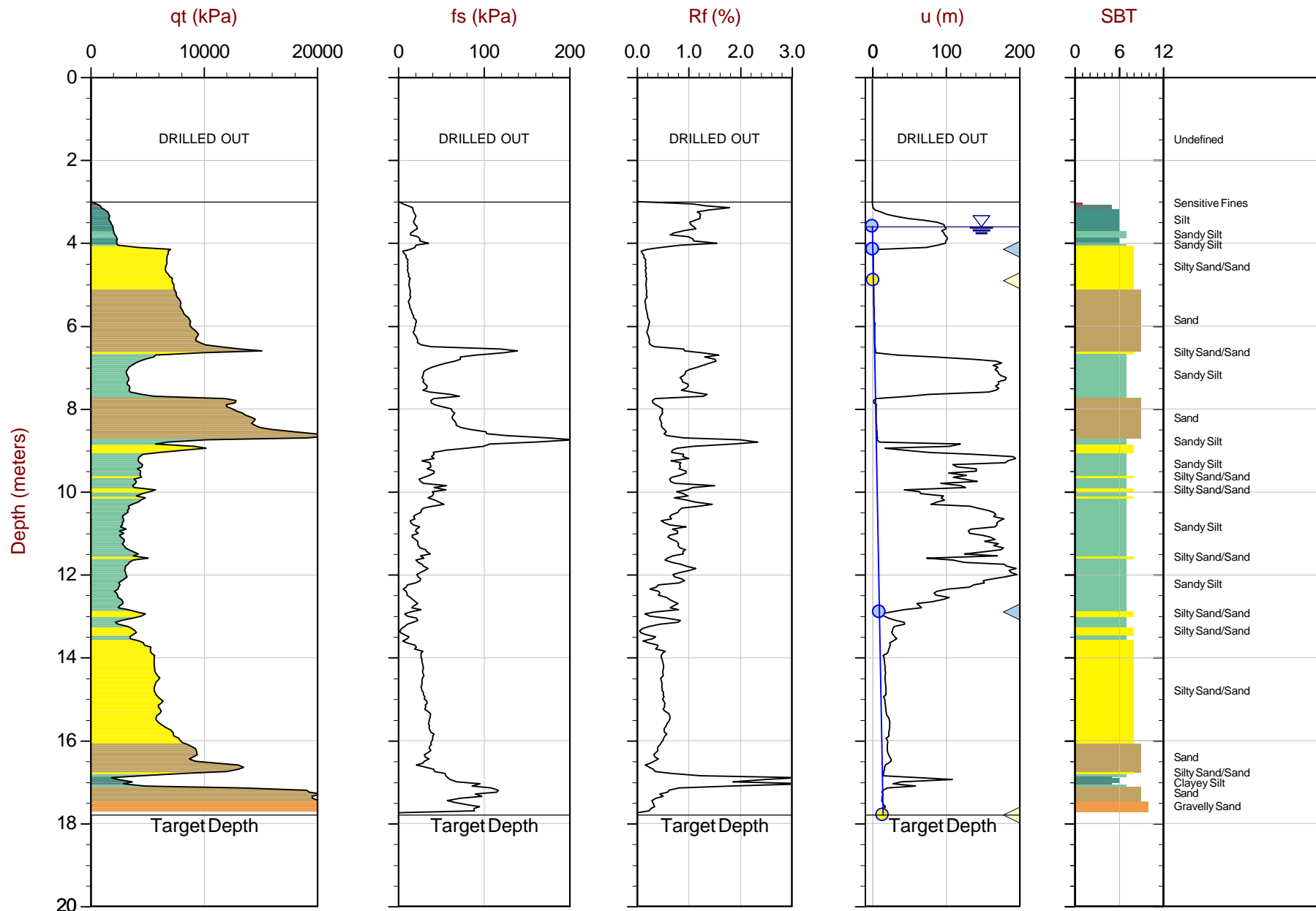
Job No: 16-05039

Date: 2016/11/14 23:13

Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104

Cone: 419:T1500F15U500



Max Depth: 17.800 m / 58.40 ft

Depth Inc: 0.050 m / 0.164 ft

Avg Int: EveryPoint

File: 16-05039_SP104.COR

Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986

Coords: UTM18NN: 5021640m E: 436855m

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were estimated from Google Earth and are only approximate locations. The coordinates should not be used for design purposes.

Advanced Cone Penetration Test Plot with I_c , $S_u(N_{kt})$ and $N1(60)$



Golder

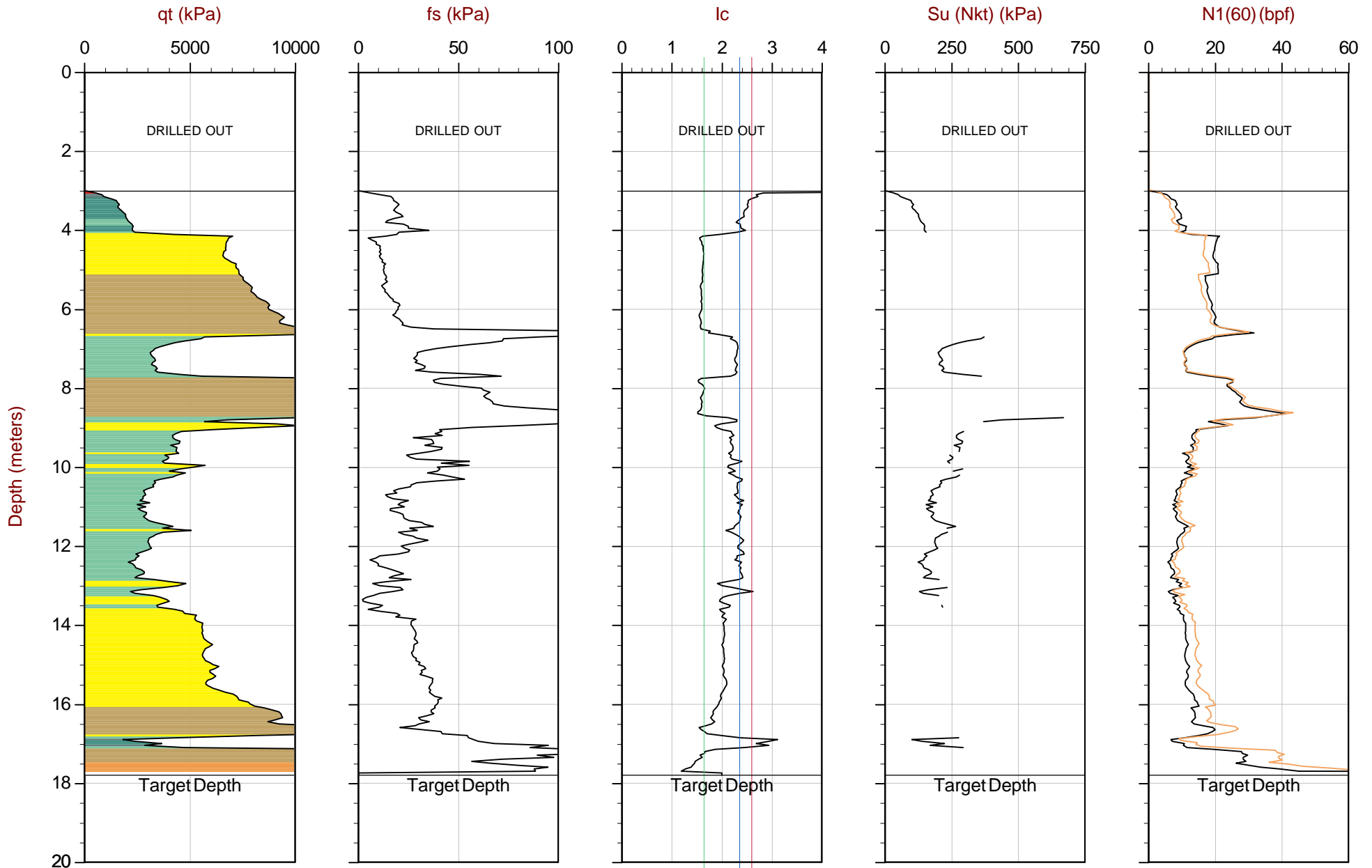
Job No: 16-05039

Date: 2016/11/14 23:13

Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104

Cone: 419:T1500F15U500



Max Depth: 17.800 m / 58.40 ft

Depth Inc: 0.050 m / 0.164 ft

Avg Int: Every Point

— N(60) (bpf)

File: 16-05039_SP104.COR

Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986

Coords: UTM18N: 5021640m E: 436855m

The reported coordinates were estimated from Google Earth and are only approximate locations. The coordinates should not be used for design purposes.

Seismic Cone Penetration Test Tabular Results



Job No: 16-05039
Client: Golder Associates
Project: Hwy 417 at Richmond Rd.
Sounding ID: SCPT16-104
Date: 14-Nov-2016

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.90	2.70	2.76			
3.90	3.70	3.74	0.99	4.62	213
4.90	4.70	4.73	0.99	4.37	227
5.90	5.70	5.73	0.99	4.25	234
6.90	6.70	6.72	1.00	3.92	254
7.90	7.70	7.72	1.00	3.83	260
8.90	8.70	8.72	1.00	3.91	255
9.90	9.70	9.72	1.00	3.65	274
10.90	10.70	10.71	1.00	3.41	293
11.90	11.70	11.71	1.00	3.18	314
12.90	12.70	12.71	1.00	3.55	281
13.90	13.70	13.71	1.00	4.55	220
14.90	14.70	14.71	1.00	4.66	214
15.90	15.70	15.71	1.00	4.33	231
16.90	16.70	16.71	1.00	3.84	260
17.80	17.60	17.61	0.90	3.10	290

Seismic Cone Penetration Test Plot



Golder

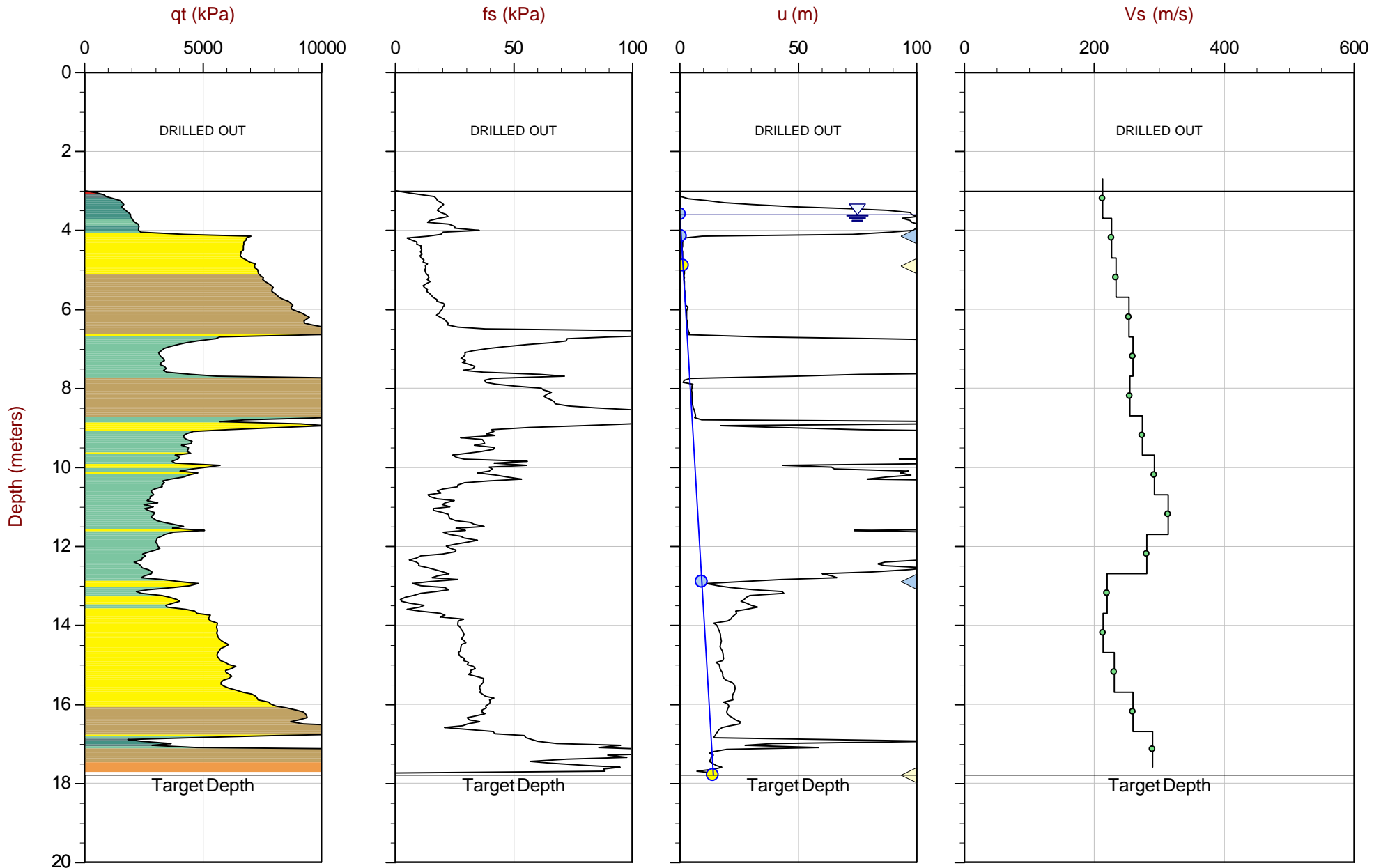
Job No: 16-05039

Date: 2016/11/14 23:13

Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104

Cone: 419:T1500F15U500



Max Depth: 17.800 m / 58.40 ft

Depth Inc: 0.050 m / 0.164 ft

Avg Int: Every Point

File: 16-05039_SP104.COR

Unit Wt: SBT Zones

SBT: Robertson and Campanella, 1986

Coords: UTM18NN: 5021640m E: 436855m

● Equilibrium Pore Pressure (Ueq)

● Assumed Ueq

◀ Dissipation, Ueq achieved

◀ Dissipation, Ueq not achieved

— Hydrostatic Line

The reported coordinates were estimated from Google Earth and are only approximate locations. The coordinates should not be used for design purposes.

Pore Pressure Dissipation Summary and Pore Pressure Dissipation Plots



Job No: 16-05039
Client: Golder Associates
Project: Hwy 417 at Richmond Rd.
Start Date: 14-Nov-2016
End Date: 15-Nov-2016

CPT_u PORE PRESSURE DISSIPATION SUMMARY

Sounding ID	File Name	Cone Area (cm ²)	Duration (s)	Test Depth (m)	Estimated Equilibrium Pore Pressure U _{eq} (m)	Calculated Phreatic Surface (m)	Assumed Phreatic Surface (m)	t ₅₀ ^a (s)	Assumed Rigidity Index (I _r)	c _h ^b (cm ² /min)
SCPT16-104	16-05039_SP104	15	160	4.150	0.6		3.6	4.6	100	152.4
SCPT16-104	16-05039_SP104	15	200	4.900	1.4	3.5				
SCPT16-104	16-05039_SP104	15	300	12.900	9.3		3.6	14.0	100	50.3
SCPT16-104	16-05039_SP104	15	200	17.800	14.1	3.7				

a. Time is relative to where umax occurred

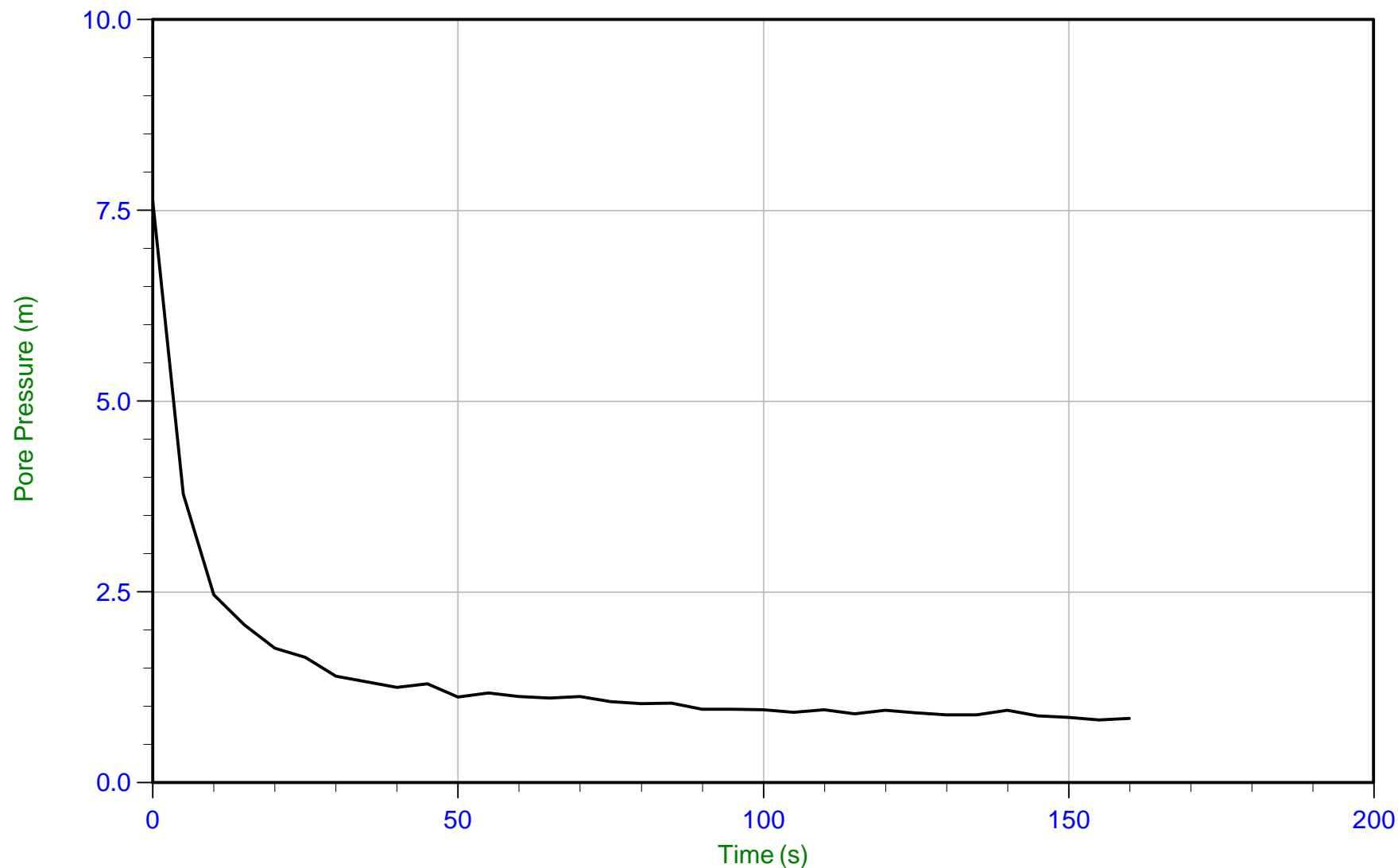
b. Houlsby and Teh, 1991



Golder

Job No: 16-05039
Date: 11/14/2016 23:13
Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104
Cone: 419:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 16-05039_SP104.PPF
Depth: 4.150 m / 13.615 ft
Duration: 160.0 s

U Min: 0.8 m
U Max: 7.6 m

WT: 3.600 m / 11.811 ft
Ueq: 0.6 m
U(50): 4.09 m

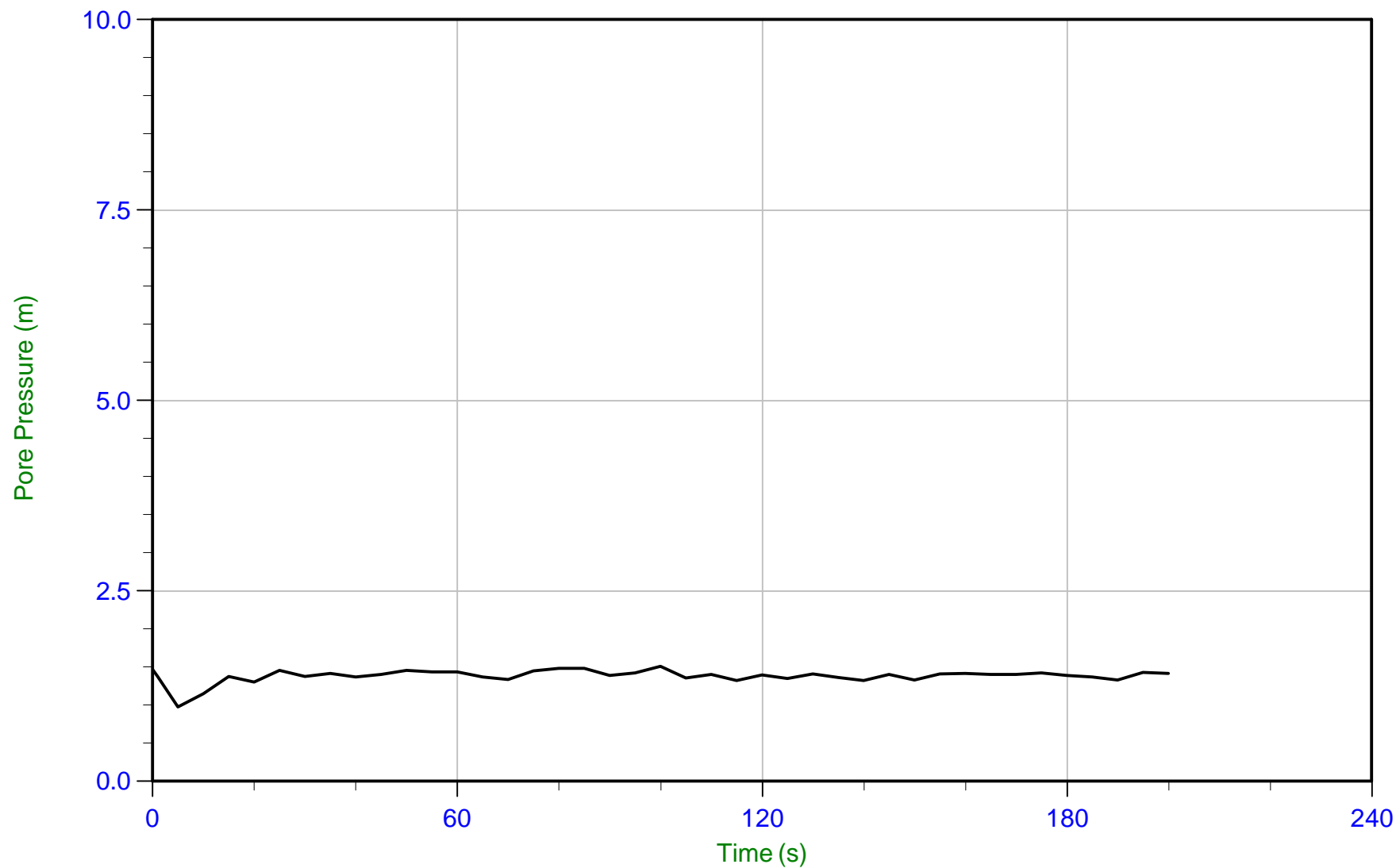
T(50): 4.6 s
Ir: 100
Ch: 152.4 sq cm/min



Golder

Job No: 16-05039
Date: 11/14/2016 23:13
Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104
Cone: 419:T1500F15U500 Area=15 cm²



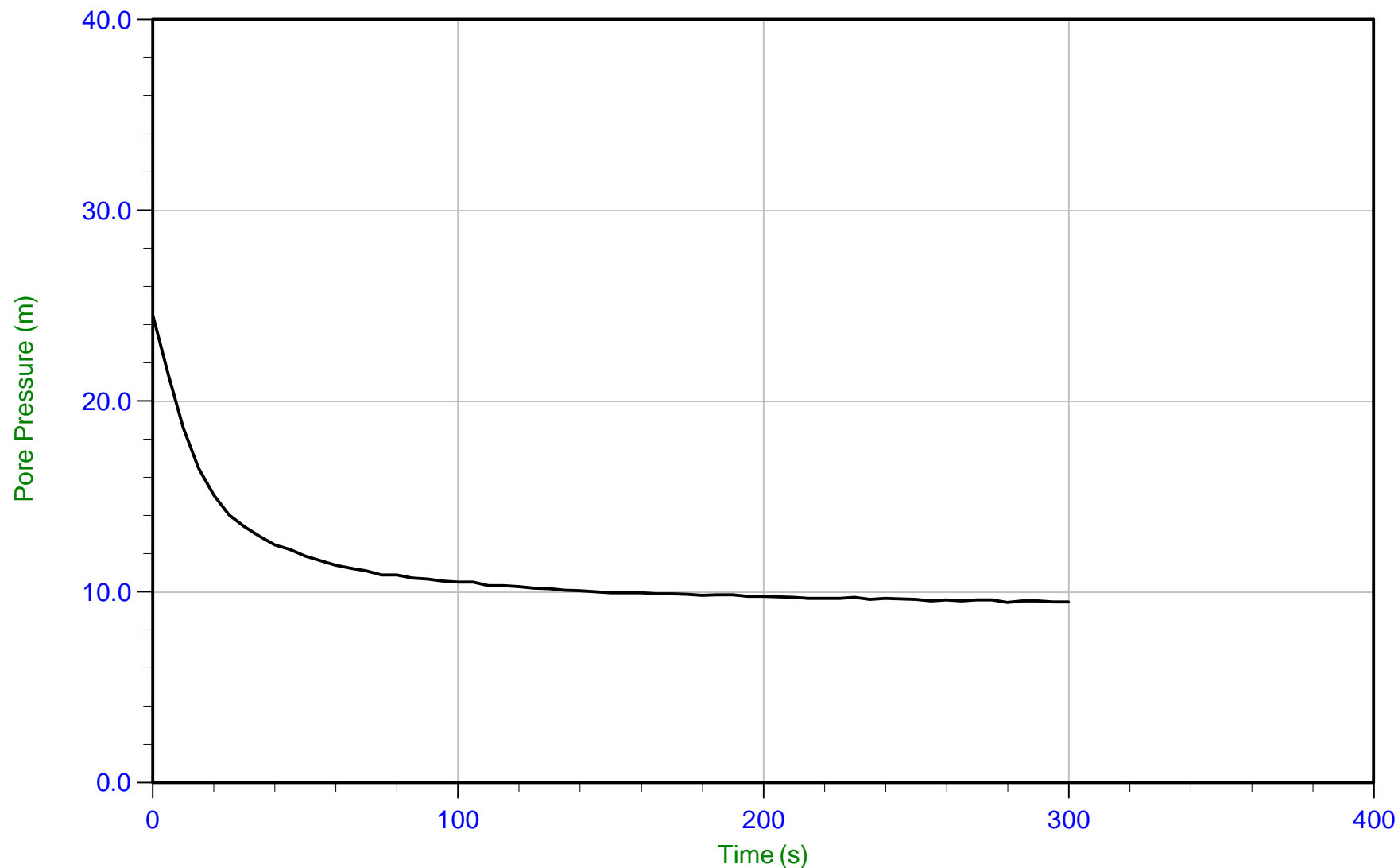
Trace Summary: Filename: 16-05039_SP104.PPF U Min: 1.0 m WT: 3.523 m / 11.558 ft
Depth: 4.900 m / 16.076 ft U Max: 1.5 m Ueq: 1.4 m
Duration: 200.0 s



Golder

Job No: 16-05039
Date: 11/14/2016 23:13
Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104
Cone: 419:T1500F15U500 Area=15 cm²



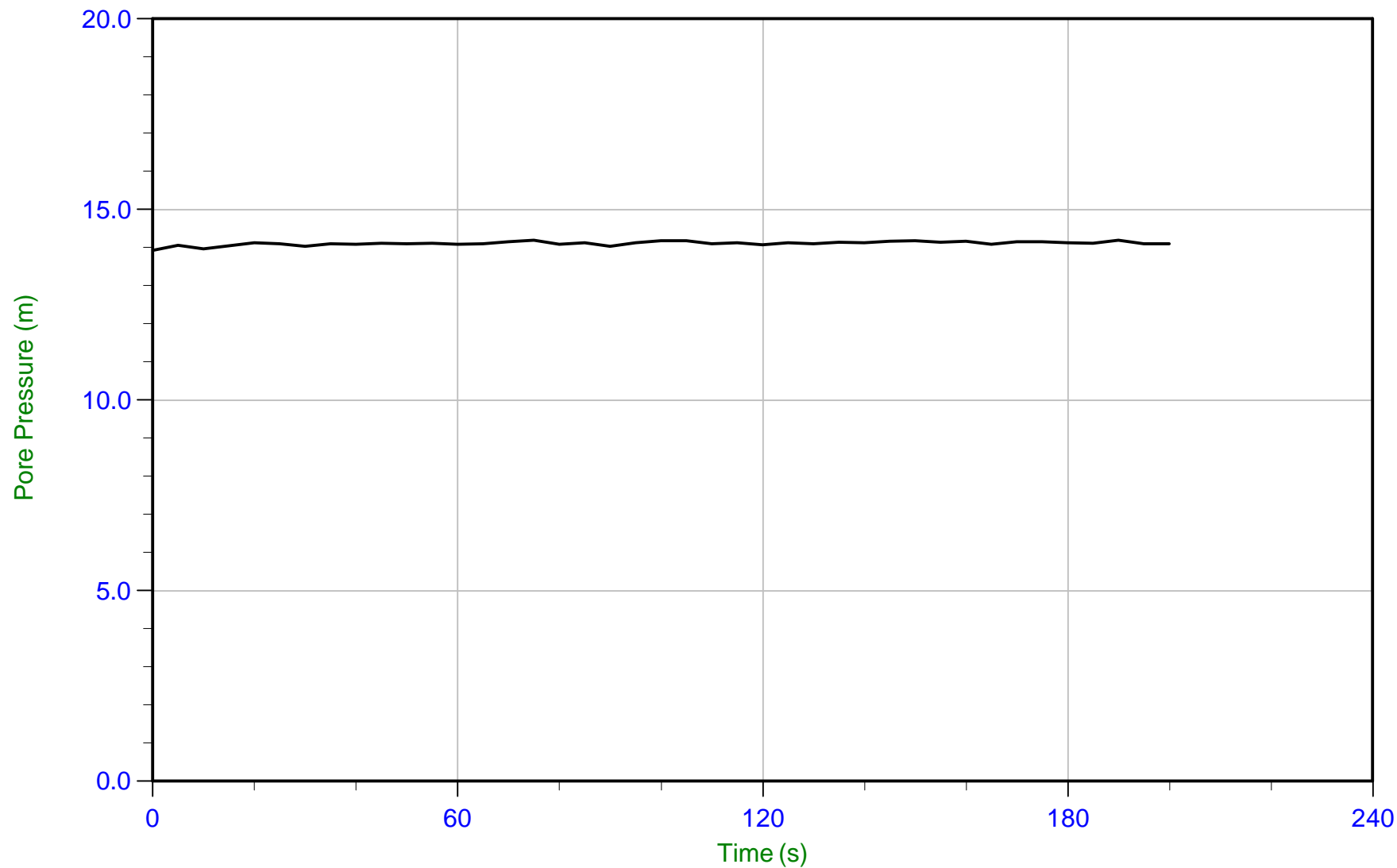
Trace Summary: Filename: 16-05039_SP104.PPF U Min: 9.5 m WT: 3.600 m / 11.811 ft T(50): 14.0 s
Depth: 12.900 m / 42.322 ft U Max: 24.5 m Ueq: 9.3 m Ir: 100
Duration: 300.0 s U(50): 16.92 m Ch: 50.3 sq cm/min



Golder

Job No: 16-05039
Date: 11/14/2016 23:13
Site: Hwy 417 at Richmond Rd.

Sounding: SCPT16-104
Cone: 419:T1500F15U500 Area=15 cm²



Trace Summary: Filename: 16-05039_SP104.PPF U Min: 13.9 m WT: 3.690 m / 12.106 ft
Depth: 17.800 m / 58.398 ft U Max: 14.2 m Ueq: 14.1 m
Duration: 200.0 s



APPENDIX F

SCPT Seismic Results (16-104)



Job No: 16-05039
Client: Golder Associates
Project: Hwy 417 at Richmond Rd.
Sounding ID: SCPT16-104
Date: 14-Nov-2016

Seismic Source: Beam
Source Offset (m): 0.55
Source Depth (m): 0.00
Geophone Offset (m): 0.20

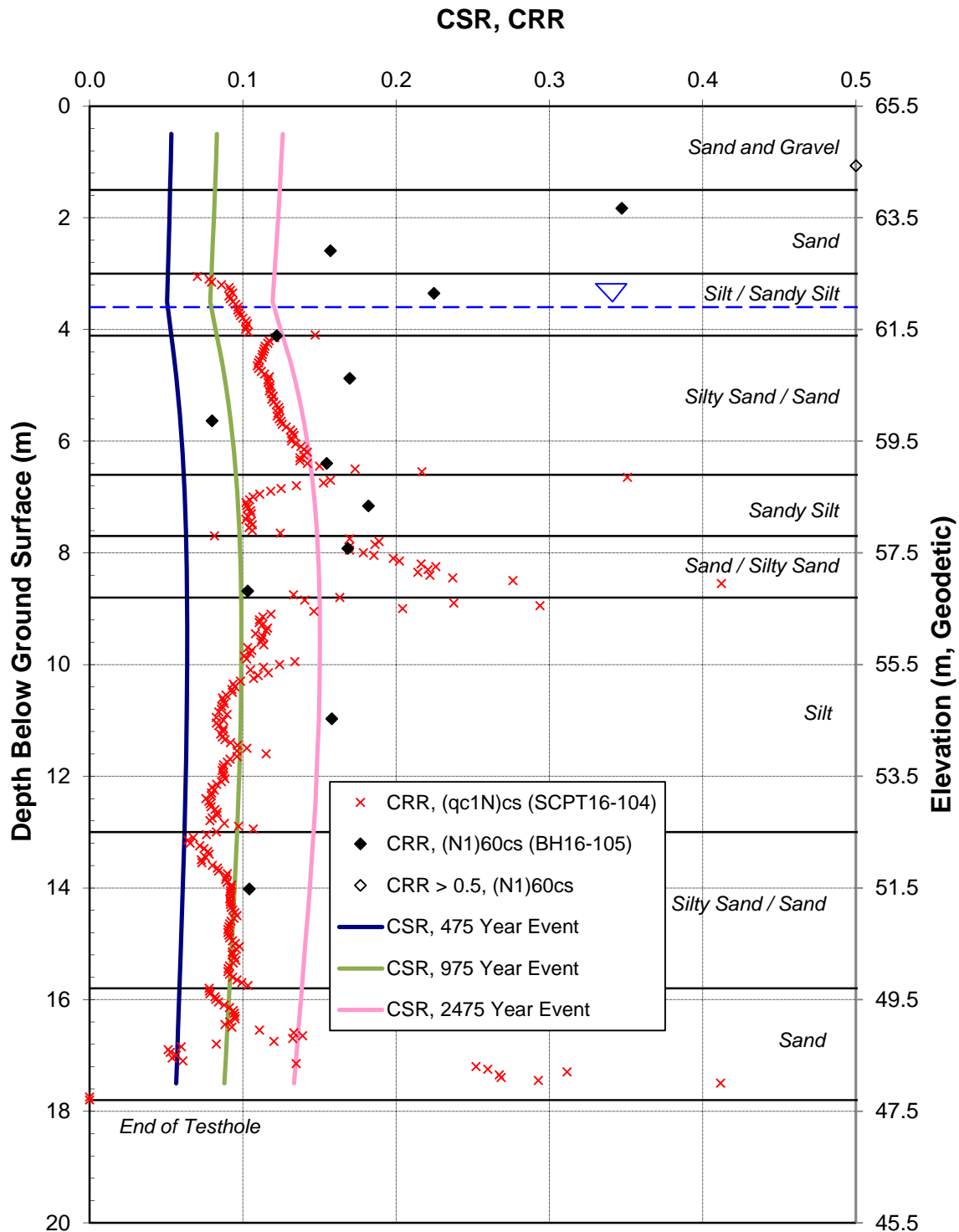
SCPT_u SHEAR WAVE VELOCITY TEST RESULTS - V_s

Tip Depth (m)	Geophone Depth (m)	Ray Path (m)	Ray Path Difference (m)	Travel Time Interval (ms)	Interval Velocity (m/s)
2.90	2.70	2.76			
3.90	3.70	3.74	0.99	4.62	213
4.90	4.70	4.73	0.99	4.37	227
5.90	5.70	5.73	0.99	4.25	234
6.90	6.70	6.72	1.00	3.92	254
7.90	7.70	7.72	1.00	3.83	260
8.90	8.70	8.72	1.00	3.91	255
9.90	9.70	9.72	1.00	3.65	274
10.90	10.70	10.71	1.00	3.41	293
11.90	11.70	11.71	1.00	3.18	314
12.90	12.70	12.71	1.00	3.55	281
13.90	13.70	13.71	1.00	4.55	220
14.90	14.70	14.71	1.00	4.66	214
15.90	15.70	15.71	1.00	4.33	231
16.90	16.70	16.71	1.00	3.84	260
17.80	17.60	17.61	0.90	3.10	290



APPENDIX G

Liquefaction Assessment Results



CLIENT
MMM Group Ltd.

CONSULTANT



YYYY-MM-DD 2017-02-24

PREPARED MJK

DESIGN MJK

REVIEW MSS

APPROVED

PROJECT
HIGHWAY 417 REHABILITATION AND WIDENING
RICHMOND ROAD UNDERPASS
SITE 3-039

TITLE
**LIQUEFACTION POTENTIAL ASSESSMENT RESULTS
CENTRAL PIER**

PROJECT No.
12-1121-0099

PHASE No.
1750

Rev.
001

FIGURE
G1

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM ANSI A



APPENDIX H

P-Y Curves for Lateral Resistance at Pier (Assuming Liquefaction)

P-y CURVES

Richmond Road Bridge - HP 360 x 109 - Piers - Liquefied Layer (Equivalent Clay Model)

SUMMARY OF P-y CURVES FOR A H-Pile 360x109 (14x73)

Description Depth (z) * Elevation P-y Curves	Loose to Compact Silty Sand																		Liquefiable Layer																		
	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m		
	Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 57.0 m		Elev. 56.0 m		Elev. 55.0 m		Elev. 54.0 m		Elev. 53.0 m		Elev. 52.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.000	1.561	0.001	4.657	0.001	9.287	0.001	15.451	0.001	23.149	0.001	32.183	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204
0.001	3.009	0.001	8.976	0.002	17.900	0.002	29.782	0.003	44.622	0.003	62.034	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408
0.001	4.262	0.002	12.712	0.003	25.351	0.003	42.178	0.004	63.193	0.004	87.852	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612
0.002	5.281	0.003	15.753	0.003	31.414	0.004	52.266	0.005	78.308	0.006	108.865	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816
0.002	6.071	0.003	18.107	0.004	36.110	0.005	60.079	0.006	90.014	0.007	125.138	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020
0.003	6.659	0.004	19.861	0.005	39.608	0.006	65.899	0.008	98.734	0.009	137.262	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224
0.003	7.084	0.004	21.131	0.006	42.140	0.007	70.111	0.009	105.045	0.010	146.035	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427
0.003	7.386	0.005	22.031	0.007	43.934	0.008	73.096	0.010	109.517	0.012	152.252	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631
0.004	7.596	0.006	22.659	0.008	45.186	0.009	75.180	0.011	112.639	0.013	156.592	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835
0.004	7.742	0.006	23.092	0.008	46.051	0.010	76.619	0.013	114.795	0.015	159.590	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039
0.005	7.842	0.007	23.390	0.009	46.645	0.011	77.606	0.014	116.274	0.016	161.646	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243	0.038	13.243
0.005	7.910	0.008	23.593	0.010	47.050	0.013	78.280	0.015	117.284	0.017	163.049	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447	0.049	14.447
0.005	7.956	0.008	23.731	0.011	47.325	0.014	78.738	0.016	117.970	0.019	164.003	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651	0.062	15.651
0.006	7.987	0.009	23.825	0.012	47.512	0.015	79.049	0.018	118.436	0.020	164.651	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855	0.078	16.855
0.006	8.009	0.009	23.888	0.013	47.638	0.016	79.259	0.019	118.751	0.022	165.090	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855	0.390	16.855
0.007	8.023	0.010	23.931	0.013	47.724	0.017	79.402	0.020	118.965	0.023	165.386	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855	0.416	16.855

Description Depth (z) * Elevation P-y Curves	Liquefiable Layer										Compact to dense Silty Sand																
	z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 21.0 m		z= 22.0 m		z= 23.0 m		z= 24.0 m		z= 24.5 m		
	Elev. 51.0 m		Elev. 50.0 m		Elev. 49.0 m		Elev. 48.0 m		Elev. 47.0 m		Elev. 46.0 m		Elev. 45.0 m		Elev. 44.0 m		Elev. 43.0 m		Elev. 42.0 m		Elev. 41.0 m		Elev. 40.0 m		Elev. 39.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		
0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.000	1.204	0.002	447.277	0.002	472.126	0.002	496.975	0.002	521.824	0.002	546.672	0.002	571.521	0.002	596.370	0.002	608.546
0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.000	2.408	0.003	862.152	0.003	910.049	0.003	957.947	0.003	1005.844	0.003	1053.741	0.003	1101.639	0.003	1149.536	0.003	1173.006
0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.001	3.612	0.005	1220.977	0.005	1288.809	0.005	1356.641	0.005	1424.473	0.005	1492.305	0.005	1560.137	0.005	1627.969	0.005	1661.206
0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.002	4.816	0.006	1513.025	0.006	1597.082	0.006	1681.138	0.006	1765.195	0.006	1849.252	0.006	1933.309	0.006	2017.366	0.006	2058.554
0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.004	6.020	0.008	1739.190	0.008	1835.811	0.008	1932.433	0.008	2029.055	0.008	2125.676	0.008	2222.298	0.008	2318.919	0.008	2366.264
0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.006	7.224	0.009	1907.680	0.009	2013.662	0.009	2119.644	0.009	2225.627	0.009	2331.609	0.009	2437.591	0.009	2543.573	0.009	2595.505
0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.010	8.427	0.011	2029.616	0.011	2142.372	0.011	2255.129	0.011	2367.885	0.011	2480.642	0.011	2593.398	0.011	2706.154	0.011	2761.405
0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.015	9.631	0.012	2116.021	0.012	2233.577	0.012	2351.134	0.012	2468.691	0.012	2586.247	0.012	2703.804	0.012	2821.361	0.012	2878.964
0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.021	10.835	0.014	2176.338	0.014	2297.245	0.014	2418.153	0.014	2539.061	0.014	2659.968	0.014	2780.876	0.014	2901.784	0.014	2961.029
0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.028	12.039	0.015	2218.005	0.015	2341.227	0.015	2464.450	0.015	2587.672	0.015	2710.895	0.015	2834.117	0.015	2957.340	0.015	3017.719
0.038	13.24																										

P-y CURVES

Richmond Road Bridge - HP 310 x 79 - Piers - Liquefied Layer (Equivalent Clay Model)

SUMMARY OF P-y CURVES FOR A H-Pile 310x79 (12x53)

Description Depth (z) * Elevation P-y Curves	Loose to Compact Silty Sand																		Liquefiable Layer																		
	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m		
	Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 57.0 m		Elev. 56.0 m		Elev. 55.0 m		Elev. 54.0 m		Elev. 53.0 m		Elev. 52.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.000	1.426	0.001	4.387	0.001	8.882	0.001	14.911	0.001	22.475	0.001	31.376	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999
0.001	2.749	0.001	8.456	0.002	17.121	0.002	28.743	0.002	43.322	0.003	60.479	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999
0.001	3.894	0.002	11.976	0.002	24.246	0.003	40.705	0.004	61.353	0.004	85.651	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998
0.002	4.825	0.002	14.840	0.003	30.046	0.004	50.442	0.005	76.028	0.006	106.138	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998
0.002	5.546	0.003	17.059	0.004	34.537	0.005	57.982	0.006	87.392	0.007	122.003	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997
0.002	6.084	0.004	18.711	0.005	37.883	0.006	63.599	0.007	95.859	0.009	133.823	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997
0.003	6.472	0.004	19.907	0.006	40.304	0.007	67.664	0.008	101.986	0.010	142.376	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996
0.003	6.748	0.005	20.755	0.006	42.020	0.008	70.545	0.010	106.328	0.011	148.437	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996
0.003	6.940	0.005	21.346	0.007	43.218	0.009	72.555	0.011	109.359	0.013	152.669	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995
0.004	7.073	0.006	21.755	0.008	44.045	0.010	73.945	0.012	111.452	0.014	155.592	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995
0.004	7.164	0.007	22.035	0.009	44.613	0.011	74.897	0.013	112.888	0.016	157.596	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994	0.031	10.994
0.005	7.227	0.007	22.227	0.010	45.000	0.012	75.547	0.015	113.868	0.017	158.964	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994	0.041	11.994
0.005	7.269	0.008	22.357	0.010	45.264	0.013	75.989	0.016	114.534	0.018	159.894	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993	0.052	12.993
0.005	7.298	0.008	22.445	0.011	45.442	0.014	76.289	0.017	114.987	0.020	160.526	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993	0.065	13.993
0.006	7.317	0.009	22.505	0.012	45.563	0.015	76.493	0.018	115.293	0.021	160.953	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993	0.323	13.993
0.006	7.330	0.009	22.545	0.013	45.645	0.016	76.630	0.019	115.500	0.023	161.243	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993	0.345	13.993
Description Depth (z) * Elevation P-y Curves	Liquefiable Layer										Compact to dense Silty Sand																										
	z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 21.0 m		z= 22.0 m		z= 23.0 m		z= 24.0 m		z= 24.5 m												
	Elev. 51.0 m		Elev. 50.0 m		Elev. 49.0 m		Elev. 48.0 m		Elev. 47.0 m		Elev. 46.0 m		Elev. 45.0 m		Elev. 44.0 m		Elev. 43.0 m		Elev. 42.0 m		Elev. 41.0 m		Elev. 40.0 m		Elev. 39.0 m												
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)											
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000												
0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.000	0.999	0.001	371.325	0.001	391.954	0.001	412.583	0.001	433.212	0.001	453.841	0.001	474.470	0.001	495.099	0.001	505.208										
0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.000	1.999	0.003	715.749	0.003	755.513	0.003	795.276	0.003	835.040	0.003	874.804	0.003	914.568	0.003	954.332	0.003	973.816										
0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.001	2.998	0.004	1013.641	0.004	1069.954	0.004	1126.268	0.004	1182.581	0.004	1238.894	0.004	1295.208	0.004	1351.521	0.004	1379.115										
0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.002	3.998	0.005	1256.096	0.005	1325.879	0.005	1395.662	0.005	1465.445	0.005	1535.228	0.005	1605.011	0.005	1674.795	0.005	1708.988										
0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.003	4.997	0.006	1443.856	0.006	1524.070	0.006	1604.284	0.006	1684.498	0.006	1764.712	0.006	1844.927	0.006	1925.141	0.006	1964.446										
0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.005	5.997	0.008	1583.734	0.008	1671.720	0.008	1759.705	0.008	1847.690	0.008	1935.675	0.008	2023.660	0.008	2111.646	0.008	2154.759										
0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.008	6.996	0.009	1684.964	0.009	1778.573	0.009	1872.182	0.009	1965.791	0.009	2059.401	0.009	2153.010	0.009	2246.619	0.009	2292.487										
0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.012	7.996	0.010	1756.696	0.010	1854.291	0.010	1951.885	0.010	2049.479	0.010	2147.073	0.010	2244.668	0.010	2342.262	0.010	2390.083										
0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.017	8.995	0.011	1806.771	0.011	1907.147	0.011	2007.523	0.011	2107.900	0.011	2208.276	0.011	2308.652	0.011	2409.028	0.011	2458.212										
0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.024	9.995	0.013	1841.363	0.013	1943.661	0.013	2045.958	0.013	2148.256	0.013	2250.554	0.013	2352.852	0.013	2455.150	0.013	2505.276										



APPENDIX I

P-Y Curves for Lateral Resistance at Pier and Abutments (No Liquefaction)

P-y CURVES
Richmond Road Bridge - HP 310 x 79 - Pier

SUMMARY OF P-y CURVES FOR A H-Pile 310x79 (12x53)

Description Depth (z) * Elevation P-y Curves	Loose to Compact Silty Sand																		Very Stiff Clayey Silt						Loose to Compact Silty Sand						Very Stiff Clayey Silt									
	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m							
	Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 57.0 m		Elev. 56.0 m		Elev. 55.0 m		Elev. 54.0 m		Elev. 53.0 m							
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)						
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000						
0.000	3.916	0.001	12.341	0.001	25.275	0.001	42.719	0.001	58.000	0.001	75.094	0.001	95.112	0.002	116.944	0.002	140.958	0.000	14.278	0.000	14.278	0.000	14.278	0.002	158.470	0.002	178.448	0.002	198.627	0.000	14.278	0.000	14.278	0.000						
0.001	7.548	0.001	23.788	0.002	48.720	0.002	82.343	0.002	111.798	0.003	144.748	0.003	183.335	0.003	225.416	0.003	271.705	0.000	28.557	0.000	28.557	0.000	28.557	0.003	305.460	0.003	343.968	0.003	382.865	0.000	28.557	0.000	28.557	0.000						
0.001	10.690	0.002	33.689	0.003	68.997	0.003	116.614	0.004	158.328	0.004	204.991	0.004	259.638	0.005	319.233	0.005	384.788	0.000	42.835	0.000	42.835	0.000	42.835	0.005	432.592	0.005	487.126	0.005	542.212	0.000	42.835	0.000	42.835	0.000						
0.002	13.247	0.003	41.747	0.004	85.500	0.005	144.507	0.005	196.199	0.005	254.023	0.006	321.741	0.006	395.591	0.007	476.826	0.000	57.113	0.000	57.113	0.000	57.113	0.006	536.065	0.006	603.643	0.006	671.904	0.000	57.113	0.000	57.113	0.000						
0.002	15.227	0.003	47.987	0.004	98.281	0.006	166.108	0.006	225.527	0.007	291.994	0.007	369.835	0.008	454.723	0.008	548.102	0.001	71.391	0.001	71.391	0.001	71.391	0.008	616.195	0.008	693.875	0.008	772.340	0.001	71.391	0.001	71.391	0.001						
0.002	16.702	0.004	52.636	0.005	107.802	0.007	182.200	0.007	247.376	0.008	320.283	0.009	405.664	0.009	498.776	0.010	601.201	0.001	85.670	0.001	85.670	0.001	85.670	0.010	675.891	0.009	761.097	0.009	847.163	0.001	85.670	0.001	85.670	0.001						
0.003	17.770	0.005	56.001	0.006	114.693	0.008	193.846	0.009	263.187	0.009	340.754	0.010	431.593	0.011	530.657	0.012	639.629	0.002	99.948	0.002	99.948	0.002	99.948	0.011	719.093	0.011	809.745	0.011	901.312	0.002	99.948	0.002	99.948	0.002						
0.003	18.526	0.005	58.385	0.007	119.575	0.009	202.098	0.010	274.392	0.011	355.261	0.011	449.967	0.012	553.248	0.013	666.859	0.003	114.226	0.003	114.226	0.003	114.226	0.013	749.706	0.013	844.217	0.012	939.683	0.003	114.226	0.003	114.226	0.003						
0.004	19.054	0.006	60.049	0.008	122.984	0.010	207.859	0.011	282.213	0.012	365.388	0.013	462.793	0.014	569.019	0.015	685.868	0.004	128.504	0.004	128.504	0.004	128.504	0.014	771.076	0.014	868.282	0.014	966.469	0.004	128.504	0.004	128.504	0.004						
0.004	19.419	0.007	61.199	0.009	125.339	0.011	211.839	0.012	287.617	0.013	372.383	0.014	471.654	0.015	579.913	0.017	698.999	0.005	142.783	0.005	142.783	0.005	142.783	0.016	785.839	0.016	884.905	0.016	984.972	0.005	142.783	0.005	142.783	0.005						
0.005	19.669	0.007	61.987	0.010	126.953	0.012	214.568	0.013	291.322	0.015	377.181	0.016	477.730	0.017	587.384	0.018	708.005	0.007	157.061	0.007	157.061	0.007	157.061	0.017	795.964	0.017	896.306	0.017	997.662	0.007	157.061	0.007	157.061	0.007						
0.005	19.840	0.008	62.525	0.011	128.055	0.014	216.430	0.015	293.851	0.016	380.455	0.017	481.877	0.019	592.483	0.020	714.150	0.009	171.339	0.009	171.339	0.009	171.339	0.019	802.873	0.019	904.086	0.019	1006.322	0.009	171.339	0.009	171.339	0.009						
0.005	19.956	0.008	62.891	0.012	128.805	0.015	217.697	0.016	295.570	0.017	382.681	0.019	484.697	0.020	595.950	0.021	718.330	0.011	185.618	0.011	185.618	0.011	185.618	0.021	807.571	0.020	909.377	0.020	1012.211	0.011	185.618	0.011	185.618	0.011						
0.006	20.035	0.009	63.139	0.012	129.313	0.016	218.556	0.017	296.737	0.019	384.192	0.020	486.610	0.022	598.303	0.023	721.165	0.014	199.896	0.014	199.896	0.014	199.896	0.022	810.759	0.022	912.967	0.022	1016.207	0.014	199.896	0.014	199.896	0.014						
0.006	20.088	0.010	63.308	0.013	129.658	0.017	219.139	0.018	297.528	0.020	385.216	0.022	487.907	0.023	599.897	0.025	723.087	0.069	199.896	0.069	199.896	0.069	199.896	0.024	812.919	0.024	915.399	0.023	1018.914	0.069	199.896	0.069	199.896	0.069						
0.007	20.124	0.010	63.421	0.014	129.891	0.018	219.533	0.020	298.063	0.021	385.908	0.023	488.784	0.025	600.975	0.026	724.387	0.074	199.896	0.074	199.896	0.074	199.896	0.025	814.381	0.025	917.045	0.025	1020.746	0.074	199.896	0.074	199.896	0.074						
Description Depth (z) * Elevation P-y Curves	Dense Silty Sand to Sand																		z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 21.0 m		z= 22.0 m	
	Elev. 52.0 m		Elev. 51.0 m		Elev. 50.0 m		Elev. 49.0 m		Elev. 48.0 m		Elev. 47.0 m		Elev. 46.0 m		Elev. 45.0 m		Elev. 44.0 m		Elev. 43.0 m		Elev. 42.0 m																			
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)																		
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000																		
	0.001	373.268	0.001	408.184	0.001	443.452	0.001	478.720	0.001	513.988	0.001	548.903	0.001	584.524	0.001	619.792	0.001	655.060	0.001	690.328	0.001	725.596																		
	0.003	719.496	0.003	786.797	0.003	854.778	0.003	922.759	0.003	990.740	0.003	1058.041	0.003	1126.702	0.003	1194.683	0.003	1262.664	0.003	1330.645	0.003	1398.627																		
	0.004	1018.947	0.004	1114.259	0.004	1210.534	0.004	1306.808	0.004	1403.083	0.004	1498.395	0.004	1595.632	0.004	1691.906	0.004	1788.181	0.004	1884.455	0.004	1980.730																		
	0.006	1262.672	0.006	1380.781	0.006	1500.084	0.006	1619.387	0.006	1738.689	0.006	1856.799	0.006	1977.295	0.006	2096.597	0.006	2215.900	0.006	2335.203	0.006	2454.505																		
	0.007	1451.414	0.007	1587.179	0.007	1724.315	0.007	1861.451	0.007	1998.586	0.007	2134.351	0.007	2272.858	0.007	2409.994	0.007	2547.130	0.007	2684.266	0.007	2821.401																		
0.008	1592.025	0.008	1740.943	0.008	1891.364	0.009	2041.785	0.009	2192.207	0.009	2341.124	0.009	2493.049	0.009	2643.471	0.009	2793.892	0.009	2944.314	0.009	3094.735																			
0.010	1693.785	0.010	1852.221	0.010	2012.257	0.010	2172.293	0.010	2332.329	0.010	2490.765	0.010	2652.401	0.010	2812.437	0.010	2972.473	0.010	3132.509	0.010	3292.546																			
0.011	1765.893	0.011	1931.074	0.011	2097.923	0.011	2264.772	0.011	2431.621	0.011	2596.802	0.012	2765.319	0.012	2932.168	0.012	3099.018	0.012	3265.867	0.012	3432.716																			
0.012	1816.230	0.013	1986.119	0.013	2157.724	0.013	2329.379	0.013	2500.934	0.013	2670.824	0.013	2844.145	0.013	3015.750	0.013	3187.355	0.013	3358.960	0.013	3530.565																			
0.014	1851.002	0.014	2024.144	0.014	2199.035	0.014	2373.925	0.014	2548.816	0.014	2721.958	0.014	2898.597	0.014	3073.485	0.015	3248.379	0.015	3423.269	0.015	3598.160																			
0.015	1874.850	0.015	2050.222	0.015	2227.366	0.016	2404.510	0.016	2581.654	0.016	2757.026	0.016	2935.942	0.016	3113.085	0.016	3290.229	0.016	3467.373	0.016	3644.517																			
0.017	1891.124	0.017	2068.018	0.017	2246.700	0.017	2425.381	0.017	2604.063	0.017	2780.957	0.017	2961.426	0.017	3140.107	0.017	3318.788	0.018	3497.470	0.018	3676.151																			
0.018	1902.191	0.018	2080.121	0.018	2259.848	0.018	2439.575	0.019	2619.303	0.019	2797.32	0.019	2978.757	0.019	3158.484	0.019	3338.211	0.019	3517.938	0.019	3697.666																			
0.019	1909.701	0.020	2088.333	0.0																																				

P-y CURVES

Richmond Road Bridge - HP 360 x 108 - Pier

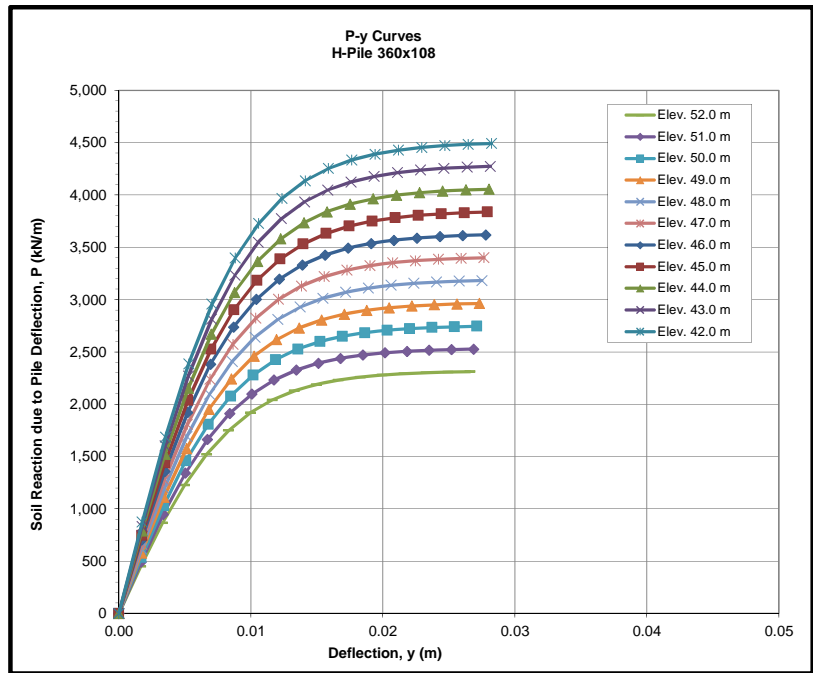
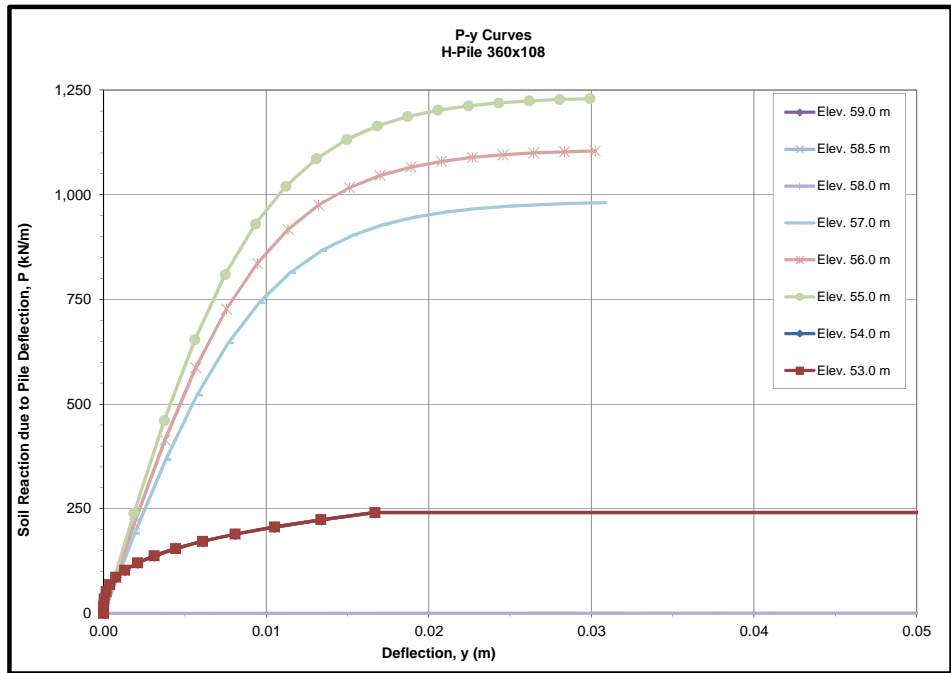
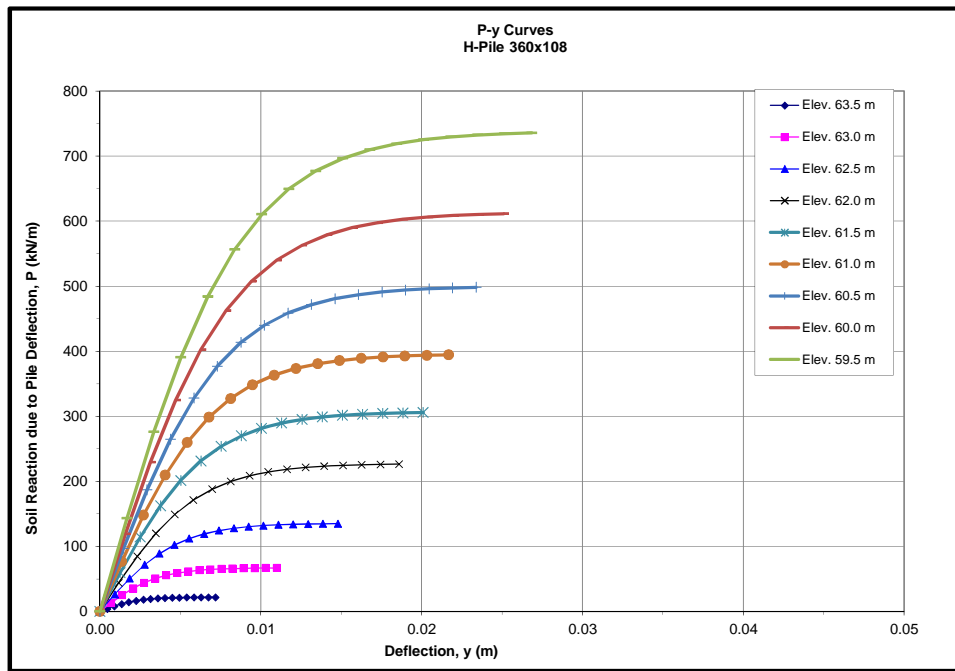
SUMMARY OF P-y CURVES FOR A H-Pile 360x108 (14x73)

Description Depth (z) * Elevation P-y Curves	Loose to Compact Silty Sand																		Very Stiff Clayey Silt						Loose to Compact Silty Sand						Very Stiff Clayey Silt			
	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0 m		z= 11.0 m	
	Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 57.0 m		Elev. 56.0 m		Elev. 55.0 m		Elev. 54.0 m		Elev. 53.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.000	4.256	0.001	13.021	0.001	26.295	0.001	44.078	0.001	59.524	0.001	76.779	0.001	96.966	0.002	118.961	0.002	143.141	0.000	17.199	0.000	17.199	0.000	17.199	0.000	0.002	190.885	0.002	214.949	0.002	239.255	0.000	17.199	0.000	17.199
0.001	8.203	0.001	25.098	0.002	50.685	0.002	84.963	0.003	114.736	0.003	147.996	0.003	186.906	0.003	229.305	0.003	275.911	0.000	34.398	0.000	34.398	0.000	34.398	0.000	0.004	367.941	0.004	414.325	0.004	461.178	0.000	34.398	0.000	34.398
0.001	11.618	0.002	35.544	0.003	71.780	0.003	120.325	0.004	162.488	0.004	209.591	0.004	264.696	0.005	324.740	0.005	390.745	0.000	51.596	0.000	51.596	0.000	51.596	0.006	0.006	521.077	0.006	586.766	0.006	653.119	0.000	51.596	0.000	51.596
0.002	14.396	0.003	44.046	0.004	88.949	0.005	149.105	0.005	201.354	0.005	259.724	0.006	328.010	0.006	402.416	0.007	484.208	0.000	68.795	0.000	68.795	0.000	68.795	0.008	0.008	645.714	0.008	727.116	0.007	809.339	0.000	68.795	0.000	68.795
0.002	16.548	0.003	50.630	0.005	102.245	0.006	171.393	0.006	231.453	0.007	298.547	0.007	377.040	0.008	462.568	0.008	556.587	0.001	85.994	0.001	85.994	0.001	85.994	0.010	0.010	742.235	0.009	835.804	0.009	930.318	0.001	85.994	0.001	85.994
0.003	18.152	0.004	55.535	0.006	112.151	0.007	187.998	0.008	253.875	0.008	327.470	0.009	413.567	0.009	507.381	0.010	610.508	0.001	103.193	0.001	103.193	0.001	103.193	0.011	0.011	814.141	0.011	916.776	0.011	1020.446	0.001	103.193	0.001	103.193
0.003	19.312	0.005	59.085	0.006	119.319	0.008	200.014	0.009	270.103	0.009	348.401	0.010	440.002	0.011	539.812	0.012	649.531	0.002	120.392	0.002	120.392	0.002	120.392	0.013	0.013	866.180	0.013	975.374	0.013	1085.672	0.002	120.392	0.002	120.392
0.004	20.134	0.005	61.600	0.007	124.399	0.009	208.529	0.010	281.601	0.011	363.234	0.012	458.733	0.013	562.793	0.013	677.182	0.003	137.591	0.003	137.591	0.003	137.591	0.015	0.015	903.055	0.015	1016.898	0.015	1131.891	0.003	137.591	0.003	137.591
0.004	20.708	0.006	63.356	0.008	127.945	0.010	214.473	0.011	289.628	0.012	373.587	0.013	471.810	0.014	578.836	0.015	696.486	0.004	154.789	0.004	154.789	0.004	154.789	0.017	0.017	928.797	0.017	1045.885	0.017	1164.156	0.004	154.789	0.004	154.789
0.004	21.104	0.007	64.569	0.009	130.394	0.012	218.580	0.013	295.174	0.014	380.740	0.015	480.843	0.016	589.918	0.017	709.820	0.006	171.988	0.006	171.988	0.006	171.988	0.019	0.019	946.579	0.019	1065.909	0.019	1186.444	0.006	171.988	0.006	171.988
0.005	21.376	0.008	65.401	0.010	132.074	0.013	221.396	0.014	298.976	0.015	385.645	0.016	487.038	0.017	597.518	0.018	718.965	0.008	189.187	0.008	189.187	0.008	189.187	0.021	0.021	958.774	0.021	1079.641	0.021	1201.729	0.008	189.187	0.008	189.187
0.005	21.562	0.008	65.969	0.011	133.221	0.014	223.317	0.015	301.572	0.016	388.993	0.018	491.265	0.019	602.705	0.020	725.206	0.011	206.386	0.011	206.386	0.011	206.386	0.023	0.023	967.096	0.023	1089.013	0.022	1212.160	0.011	206.386	0.011	206.386
0.006	21.688	0.009	66.355	0.012	134.000	0.015	224.624	0.016	303.336	0.018	391.269	0.019	494.140	0.020	606.232	0.022	729.450	0.013	223.585	0.013	223.585	0.013	223.585	0.025	0.025	972.756	0.025	1095.386	0.024	1219.254	0.013	223.585	0.013	223.585
0.006	21.774	0.010	66.617	0.013	134.529	0.016	225.511	0.018	304.534	0.019	392.814	0.020	496.091	0.022	608.625	0.023	732.330	0.017	240.784	0.017	240.784	0.017	240.784	0.027	0.027	976.596	0.026	1099.710	0.026	1224.068	0.017	240.784	0.017	240.784
0.007	21.832	0.010	66.794	0.014	134.888	0.017	226.112	0.019	305.345	0.020	393.860	0.022	497.412	0.024	610.246	0.025	734.281	0.083	240.784	0.083	240.784	0.083	240.784	0.029	0.029	979.198	0.028	1102.640	0.028	1227.329	0.083	240.784	0.083	240.784
0.007	21.871	0.011	66.914	0.015	135.130	0.019	226.519	0.020	305.894	0.022	394.568	0.023	498.307	0.025	611.344	0.027	735.601	0.089	240.784	0.089	240.784	0.089	240.784	0.031	0.031	980.959	0.030	1104.623	0.030	1229.536	0.089	240.784	0.089	240.784

Description Depth (z) * Elevation P-y Curves	Dense Silty Sand to Sand																					
	z= 12.0 m		z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 21.0 m		z= 22.0 m	
	Elev. 52.0 m		Elev. 51.0 m		Elev. 50.0 m		Elev. 49.0 m		Elev. 48.0 m		Elev. 47.0 m		Elev. 46.0 m		Elev. 45.0 m		Elev. 44.0 m		Elev. 43.0 m		Elev. 42.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.002	449.619	0.002	491.676	0.002	534.158	0.002	576.640	0.002	619.122	0.002	661.604	0.002	704.086	0.002	746.567	0.002	789.049	0.002	831.531	0.002	874.013	
0.003	866.665	0.003	947.733	0.003	1029.619	0.003	1111.505	0.003	1193.392	0.003	1275.278	0.003	1357.164	0.003	1439.050	0.004	1520.937	0.004	1602.823	0.004	1684.709	
0.005	1227.369	0.005	1342.176	0.005	1458.143	0.005	1574.110	0.005	1690.077	0.005	1806.044	0.005	1922.011	0.005	2037.978	0.005	2153.945	0.005	2269.912	0.005	2385.879	
0.007	1520.946	0.007	1663.214	0.007	1806.919	0.007	1950.625	0.007	2094.330	0.007	2238.036	0.007	2381.741	0.007	2525.447	0.007	2669.152	0.007	2812.858	0.007	2956.563	
0.008	1748.295	0.008	1911.829	0.008	2077.015	0.009	2242.202	0.009	2407.388	0.009	2572.574	0.009	2737.761	0.009	2902.947	0.009	3068.134	0.009	3233.320	0.009	3398.506	
0.010	1917.667	0.010	2097.044	0.010	2278.234	0.010	2459.423	0.010	2640.613	0.010	2821.802	0.010	3002.991	0.010	3184.181	0.011	3365.370	0.011	3546.560	0.011	3727.749	
0.012	2040.241	0.012	2231.084	0.012	2423.855	0.012	2616.626	0.012	2809.396	0.012	3002.167	0.012	3194.938	0.012	3387.709	0.012	3580.479	0.012	3773.250	0.012	3966.021	
0.013	2127.098	0.013	2326.066	0.014	2527.043	0.014	2728.021	0.014	2928.998	0.014	3129.975	0.014	3330.953	0.014	3531.930	0.014	3732.908	0.014	3933.885	0.014	4134.862	
0.015	2187.731	0.015	2392.370	0.015	2599.077	0.015	2805.783	0.015	3012.489	0.016	3219.195	0.016	3425.902	0.016	3632.608	0.016	3839.314	0.016	4046.020	0.016	4252.727	
0.017	2229.617	0.017	2438.174	0.017	2648.837	0.017	2859.501	0.017	3070.165	0.017	3280.829	0.017	3491.492	0.017	3702.156	0.018	3912.820	0.018	4123.483	0.018	4334.147	
0.018	2258.342	0.018	2469.586	0.019	2682.964	0.019	2896.342	0.019	3109.719	0.019	3323.097	0.019	3536.475	0.019	3749.853	0.019	3963.231	0.019	4176.609	0.019	4389.986	
0.020	2277.944	0.020	2491.022	0.020	2706.252	0.020	2921.482	0.021	3136.712	0.021	3351.942	0.021	3567.172	0.021	3782.402	0.021	3997.631	0.021	4212.861	0.021	4428.091	
0.022	2291.276	0.022	2505.600	0.022	2722.090	0.022	2938.579	0.022	3155.069	0.022	3371.558	0.023	3588.048	0.023	3804.538	0.023	4021.027	0.023	4237.517	0.023	4454.006	
0.023	2300.321	0.024	2515.492	0.024	2732.836	0.024	2950.180	0.024	3167.525	0.024	3384.869	0.024	3602.213	0.024	3819.557	0.025	4036.901	0.025	4254.246	0.025	4471.590	
0.025	2306.449	0.025	2522.193	0.025	2740.116	0.026	2958.040	0.026	3175.963	0.026	3393.886	0.026	3611.809	0.026	3829.732	0.026	4047.656	0.026	4265.579	0.026	4483.502	
0.027	2310.596	0.027	2526.728	0.027	2745.043	0.027	2963.358	0.028	3181.673	0.028	3399.988	0.028	3618.303	0.028	3836.618	0.028	4054.933	0.028	4273.248	0.028	4491.563	

NOTES: * Depth (z) is measured to be positive below the underside of pile cap (Elevation 64.0 m).
The P-y curves have been generated based on the following assumptions:

1. P-y curves have been generated for vertical piles (i.e. no inclination).
2. There are no pile group effects (i.e. analysis is based on a single pile).



P-y CURVES
ABUTMENTS

SUMMARY OF P-y CURVES FOR A H-Pile 310x79 (12x53)

Description Depth (z) * Elevation P-y Curves	Compact Fill												Loose to Compact Silty Sand												Stiff Clayey Silt												
	z= 0.5 m		z= 1.0 m		z= 1.5 m		z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 7.0 m		z= 8.0 m		z= 9.0 m		z= 10.0m		z= 11.0 m		z= 12.0 m		
	Elev. 67.5 m		Elev. 67.0 m		Elev. 66.5 m		Elev. 66.0 m		Elev. 65.5 m		Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.0 m		Elev. 60.0 m		Elev. 59.0 m		Elev. 58.0 m		Elev. 57.0 m		Elev. 56.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.000	3.737	0.001	11.653	0.001	23.748	0.001	40.023	0.001	60.478	0.004	88.214	0.004	98.162	0.004	110.444	0.003	117.481	0.003	124.517	0.003	131.554	0.003	138.590	0.003	152.664	0.003	166.737	0.003	180.810	0.000	30.085	0.000	30.085	0.000	30.085	0.000	30.085
0.001	7.203	0.001	22.461	0.002	45.776	0.002	77.147	0.002	116.574	0.007	170.037	0.007	189.213	0.007	212.887	0.007	226.450	0.006	240.014	0.006	253.577	0.006	267.141	0.006	294.268	0.005	321.395	0.005	348.521	0.001	37.905	0.001	37.905	0.001	37.905	0.001	37.905
0.001	10.200	0.002	31.809	0.002	64.828	0.003	109.255	0.004	165.092	0.011	240.805	0.011	267.963	0.011	301.490	0.010	320.698	0.010	339.907	0.009	359.115	0.009	378.324	0.009	416.741	0.008	455.158	0.008	493.575	0.001	47.757	0.001	47.757	0.001	47.757	0.001	47.757
0.001	12.640	0.002	39.418	0.003	80.334	0.004	135.388	0.005	204.580	0.015	298.404	0.014	332.058	0.014	373.604	0.014	397.407	0.013	421.210	0.012	445.013	0.012	468.816	0.011	516.422	0.011	564.028	0.011	611.634	0.002	54.668	0.002	54.668	0.002	54.668	0.002	54.668
0.002	14.529	0.003	45.310	0.004	92.342	0.005	155.626	0.006	235.161	0.019	343.009	0.018	381.694	0.018	429.450	0.017	456.811	0.016	484.172	0.016	511.533	0.015	538.894	0.014	593.616	0.014	648.338	0.013	703.060	0.002	60.170	0.002	60.170	0.002	60.170	0.002	60.170
0.002	15.937	0.003	49.700	0.005	101.288	0.006	170.703	0.007	257.943	0.022	376.239	0.022	418.672	0.021	471.054	0.020	501.066	0.019	531.078	0.019	561.089	0.018	591.101	0.017	651.125	0.016	711.148	0.016	771.172	0.003	64.817	0.003	64.817	0.003	64.817	0.003	64.817
0.003	16.956	0.004	52.876	0.005	107.762	0.007	181.614	0.008	274.430	0.026	400.288	0.025	445.433	0.025	501.163	0.024	533.093	0.023	565.023	0.022	596.953	0.021	628.883	0.020	692.744	0.019	756.604	0.019	820.464	0.003	68.878	0.003	68.878	0.003	68.878	0.003	68.878
0.003	17.677	0.004	55.127	0.006	112.350	0.008	189.345	0.009	286.113	0.030	417.329	0.029	464.396	0.028	522.499	0.027	555.788	0.026	589.078	0.025	622.367	0.024	655.656	0.023	722.235	0.022	788.814	0.021	855.393	0.004	75.810	0.004	75.810	0.004	75.810	0.004	75.810
0.003	18.181	0.005	56.699	0.007	115.553	0.009	194.743	0.011	294.269	0.034	429.225	0.032	477.633	0.032	537.393	0.030	571.631	0.029	605.869	0.028	640.108	0.027	674.346	0.026	742.822	0.025	811.299	0.024	879.776	0.005	81.664	0.005	81.664	0.005	81.664	0.005	81.664
0.004	18.529	0.006	57.784	0.008	117.765	0.010	198.471	0.012	299.903	0.037	437.442	0.036	486.778	0.036	547.681	0.034	582.575	0.032	617.469	0.031	652.363	0.030	687.257	0.029	757.044	0.027	826.832	0.027	896.619	0.008	93.482	0.008	93.482	0.008	93.482	0.008	93.482
0.004	18.768	0.006	58.529	0.008	119.282	0.011	201.028	0.013	303.767	0.041	443.078	0.039	493.049	0.039	554.737	0.037	590.081	0.036	625.424	0.034	660.767	0.033	696.111	0.032	766.798	0.030	837.484	0.029	908.171	0.011	102.890	0.011	102.890	0.011	102.890	0.011	102.890
0.004	18.931	0.007	59.037	0.009	120.318	0.012	202.773	0.014	306.403	0.045	446.924	0.043	497.329	0.043	559.552	0.041	595.203	0.039	630.853	0.037	666.503	0.036	702.153	0.034	773.453	0.033	844.754	0.032	916.054	0.013	110.835	0.013	110.835	0.013	110.835	0.013	110.835
0.005	19.042	0.007	59.382	0.010	121.022	0.013	203.960	0.015	308.196	0.048	449.540	0.047	500.239	0.046	562.827	0.044	598.686	0.042	634.545	0.041	670.404	0.039	706.262	0.037	777.980	0.036	849.697	0.035	921.415	0.016	117.780	0.016	117.780	0.016	117.780	0.016	117.780
0.005	19.117	0.008	59.617	0.011	121.500	0.014	204.765	0.016	309.413	0.052	451.314	0.050	502.214	0.050	565.049	0.047	601.049	0.045	637.050	0.044	673.050	0.042	709.051	0.040	781.051	0.038	853.052	0.037	925.053	0.048	117.780	0.048	117.780	0.048	117.780	0.048	117.780
0.005	19.168	0.008	59.776	0.011	121.823	0.014	205.310	0.018	310.237	0.056	452.517	0.054	503.552	0.053	566.554	0.051	602.651	0.049	638.747	0.047	674.843	0.045	710.939	0.043	783.132	0.041	855.324	0.040	927.517	0.080	117.780	0.080	117.780	0.080	117.780	0.080	117.780
0.006	19.202	0.009	59.883	0.012	122.042	0.015	205.680	0.019	310.795	0.060	453.330	0.057	504.457	0.057	567.573	0.054	603.734	0.052	639.895	0.050	676.056	0.048	712.218	0.046	784.540	0.044	856.862	0.042	929.185	0.085	117.780	0.085	117.780	0.085	117.780	0.085	117.780
Description Depth (z) * Elevation P-y Curves	Stiff Clayey Silt												Compact to dense Silty Sand																								
	z= 13.0 m		z= 14.0 m		z= 15.0 m		z= 16.0 m		z= 17.0 m		z= 18.0 m		z= 19.0 m		z= 20.0 m		z= 21.0 m		z= 22.0 m		z= 23.0 m		z= 24.0 m		z= 25.0 m		z= 26.0 m		z= 27.0 m		z= 28.0 m		z= 29.0 m				
	Elev. 55.0 m		Elev. 54.0 m		Elev. 53.0 m		Elev. 52.0 m		Elev. 51.0 m		Elev. 50.0 m		Elev. 49.0 m		Elev. 48.0 m		Elev. 47.0 m		Elev. 46.0 m		Elev. 45.0 m		Elev. 44.0 m		Elev. 43.0 m		Elev. 42.0 m		Elev. 41.0 m		Elev. 40.0 m		Elev. 39.0 m				
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)			
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000			
0.000	30.085	0.000	30.085	0.000	30.085	0.000	30.085	0.002	295.949	0.003	373.260	0.003	446.312	0.003	466.740	0.003	487.168	0.002	507.597	0.002	528.025	0.002	548.453	0.002	568.881	0.002	589.309	0.002	609.738	0.002	630.166	0.002	650.594	0.002	671.022	0.002	691.450
0.001	37.905	0.001	37.905	0.001	37.905	0.005	570.459	0.005	719.480	0.006	860.291	0.005	899.668	0.005	939.044	0.005	978.421	0.005	1017.797	0.005	1057.174	0.004	1096.550	0.004	1135.927	0.004	1175.303	0.004	1214.680	0.004	1254.056	0.004	1293.433	0.004	1332.810		
0.001	47.757	0.001	47.757	0.001	47.757	0.007	807.882	0.008	1018.925	0.009	1218.341	0.008	1274.106	0.008	1329.871	0.007	1385.636	0.007	1441.401	0.007	1497.166	0.007	1552.931	0.007	1608.695	0.006	1664.460	0.006	1720.225	0.006	1775.990	0.006	1831.755	0.006	1887.520		
0.002	54.668	0.002	54.668	0.002	54.668	0.010	1001.121	0.011	1262.644	0.011	1509.759	0.011	1578.862	0.010	1647.966	0.010	1717.069	0.010	1786.173	0.009	1855.276	0.009	1924.379	0.009	1993.483	0.009	2062.586	0.008	2131.690	0.008	2200.793	0.008	2269.896	0.008	2338.999		
0.002	60.170	0.002	60.170	0.002	60.170	0.012	1150.767	0.013	1451.383	0.014	1735.436	0.014	1814.869	0.013	1894.302	0.012	1973.035	0.012	2053.167	0.012	2132.600	0.011	2212.033	0.011	2291.466	0.011	2370.899	0.010	2450.332	0.010	2529.765	0.010	2609.198	0.010	2688.631		
0.003	64.817	0.003	64.817	0.003	64.817	0.014	1262.252	0.016	1591.991	0.017	1903.653	0.016	1990.691	0.016	2077.819	0.015	2164.947	0.014	2252.076	0.014	2339.204	0.013	2426.332	0.013	2513.460	0.013	2600.588	0.013	2687.717	0.012	2774.845	0.012	2861.973	0.012	2949.101		
0.003	68.878	0.003	68.878</																																		

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