



April 2017

REPORT ON

FOUNDATION INVESTIGATION 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

**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**



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.



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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.

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., a senior geotechnical engineer and Associate of Golder. Mr. Fintan Heffernan, P.Eng., the Designated MTO Foundations Contact for this project, conducted an independent quality control review of this report.

GOLDER ASSOCIATES LTD.

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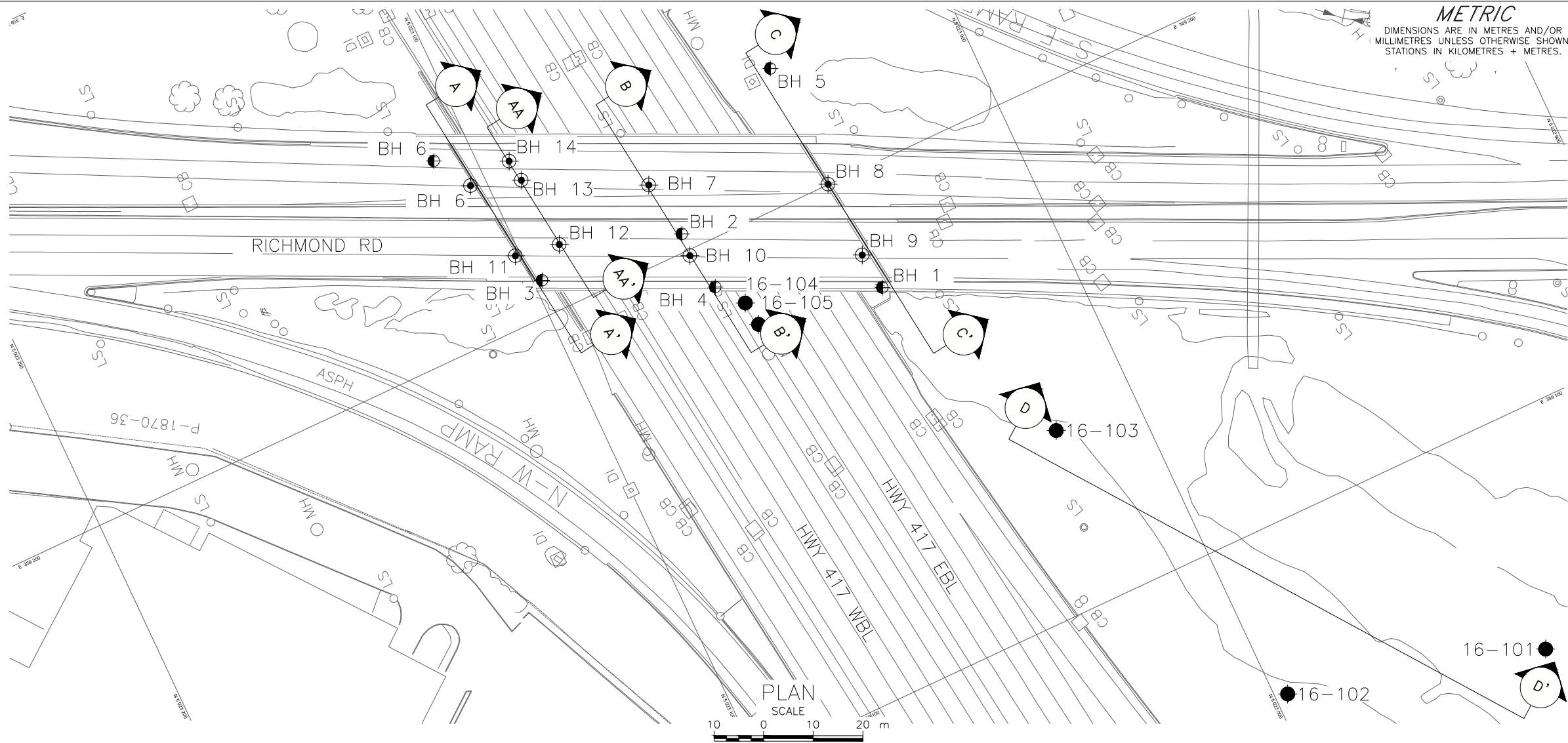


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MJK/WC/MSS/FJH/ob

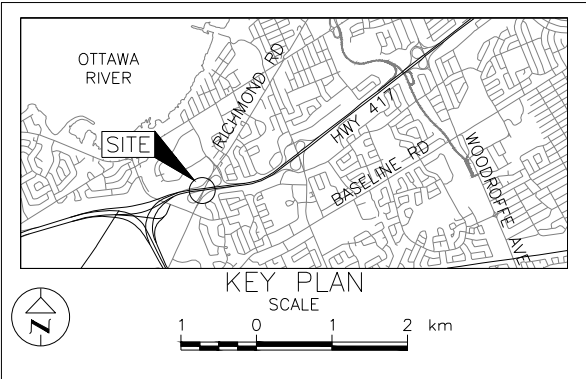
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CONT No.
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HIGHWAY 417 REHABILITATION
AND WIDENING
RICHMOND ROAD UNDERPASS
BOREHOLE LOCATIONS AND SOIL STRATA

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)

D Seal

Piezometer

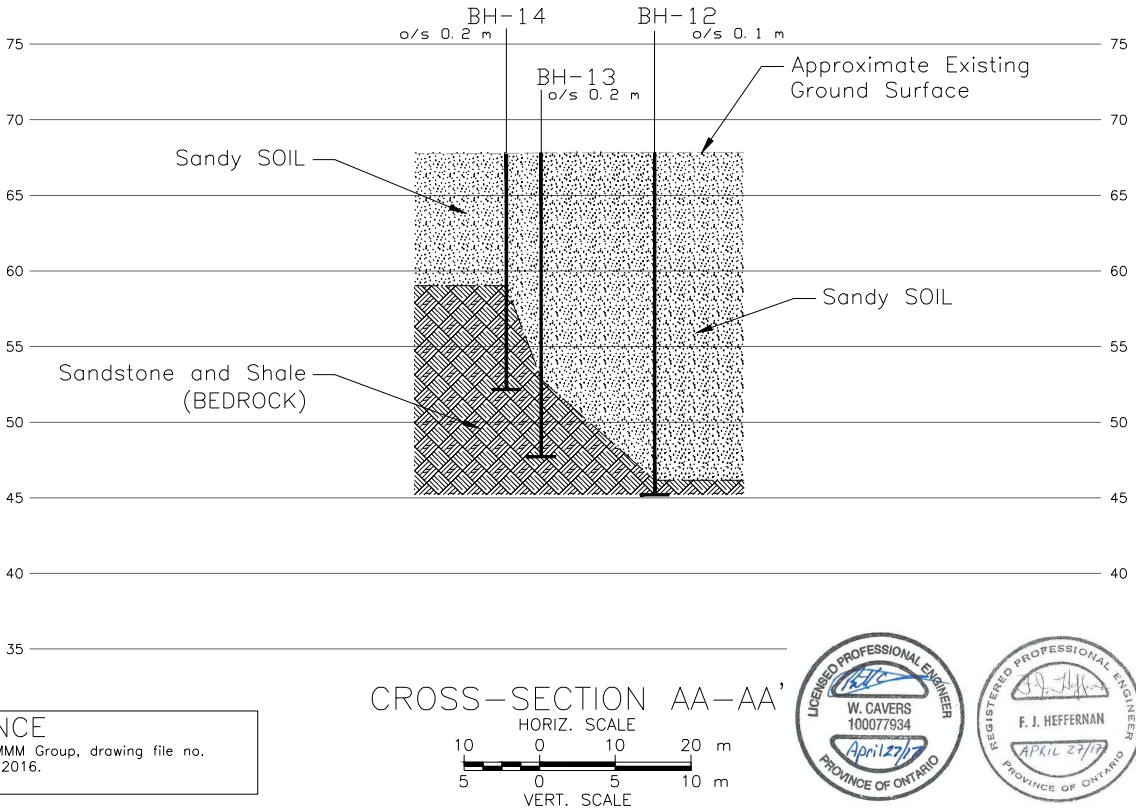
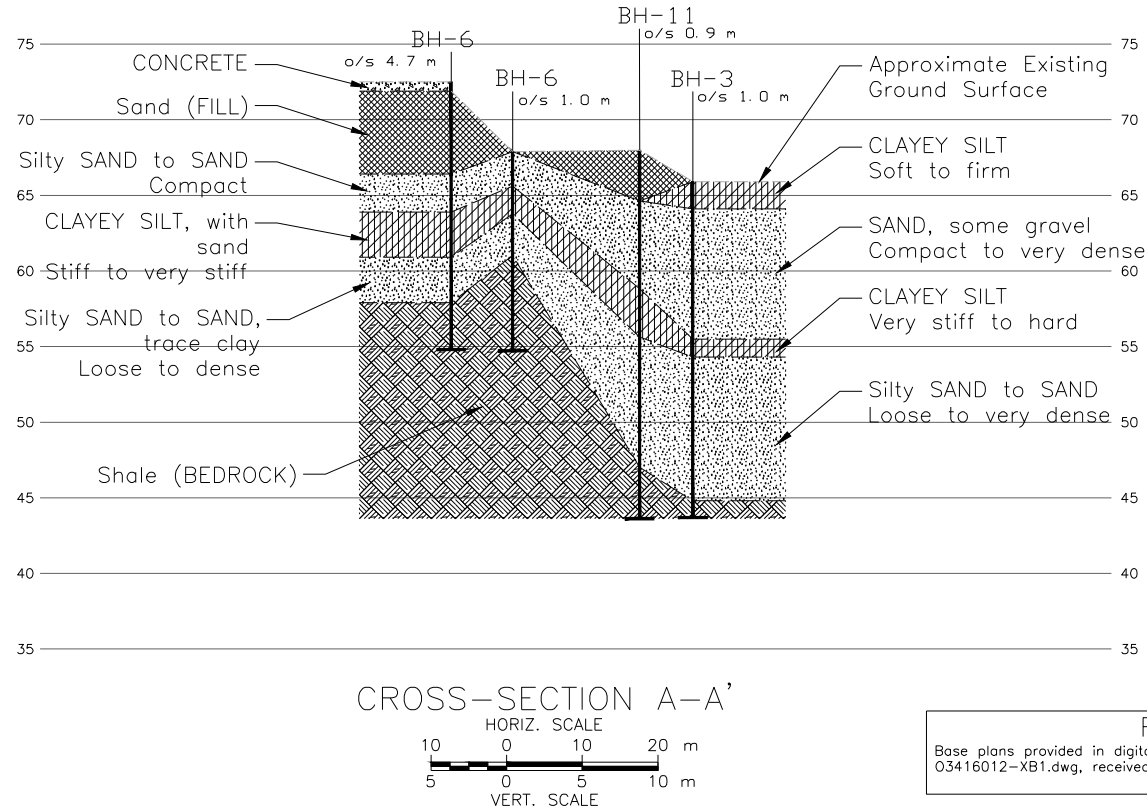
BOREHOLE CO-ORDINATES			
No.	ELEVATION	NORTHING	EASTING
16-101	65.2	5022946.5	359053.8
16-102	66.1	5022997.5	359067.7
16-103	66.6	5023017.2	359135.6
16-104	65.5	5023063.1	359185.5
16-105	65.5	5023062.5	359180.4
BH-1	65.8	5023036.7	359176.6
BH-2	65.6	5023068.8	359203.5
BH-3	65.9	5023098.3	359207.0
BH-4	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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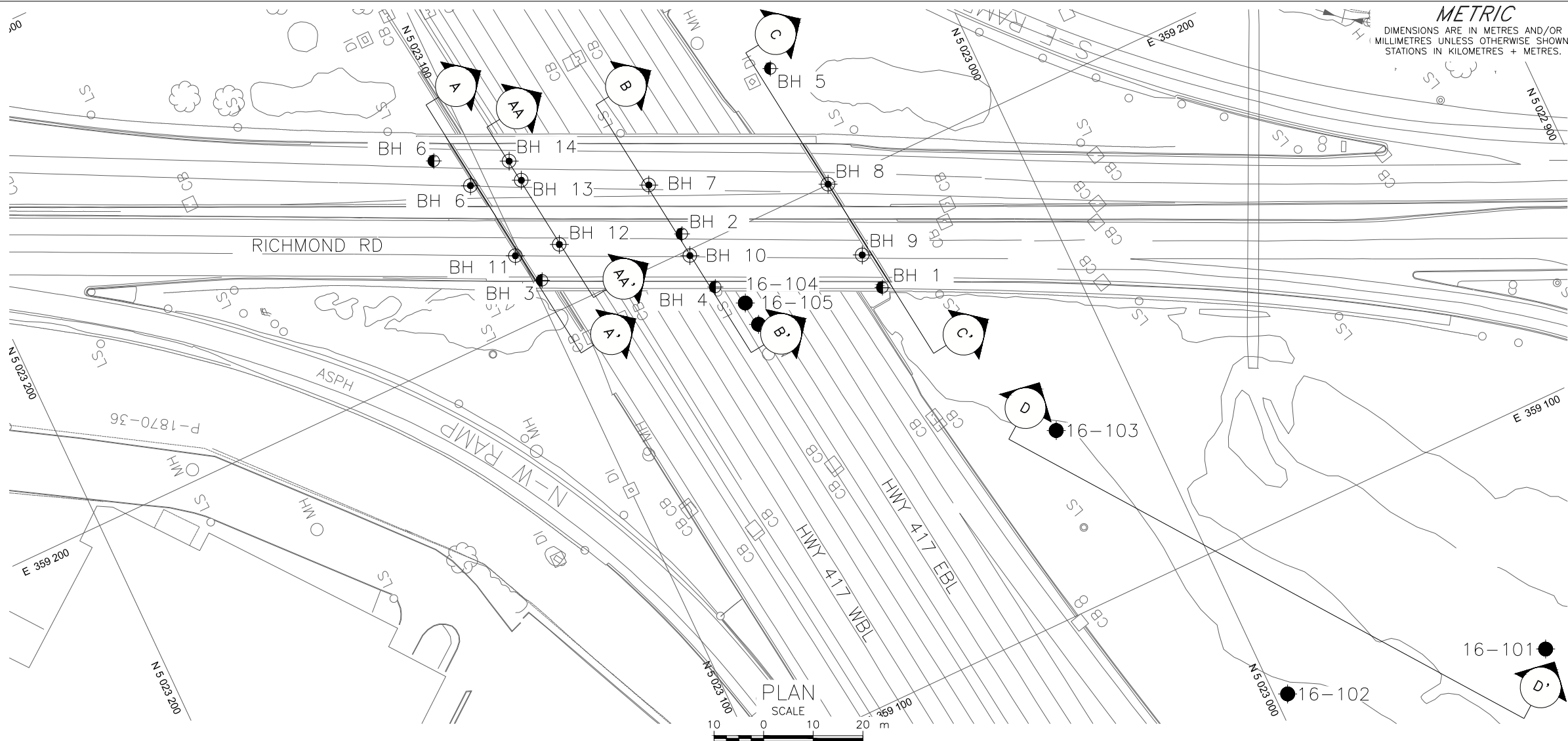


REFERENCE

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	
SUBM'D. MJK		DATE: 12/21/2016	
DRAWN: JM		SITE: 3-039	
		DWG. 1	

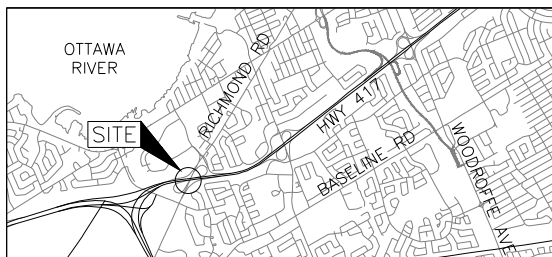


CONT No.
GWP No.4124-14-00



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
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Seal
- Piezometer
- WL in piezometer, measured on NOV 23, 2016

BOREHOLE CO-ORDINATES

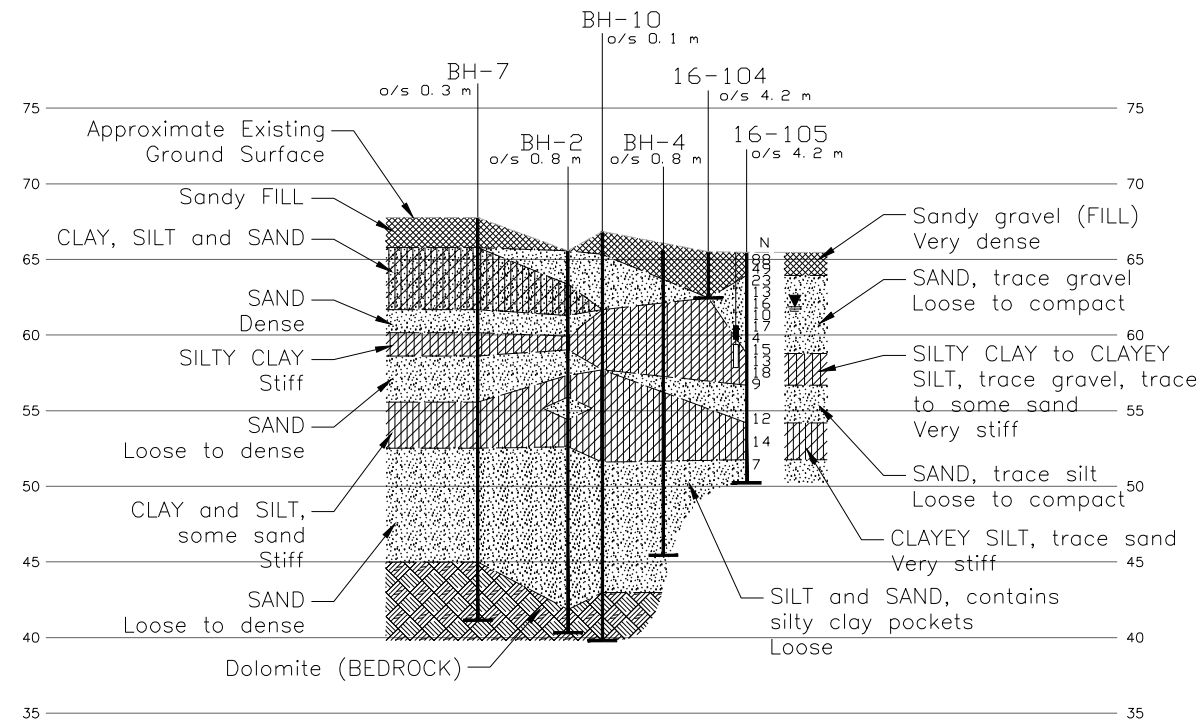
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16-102	66.1	5022997.5	359067.7
16-103	66.6	5023017.2	359135.6
16-104	65.5	5023063.1	359185.5
16-105	65.5	5023062.5	359180.4
BH-1	65.8	5023036.7	359176.6
BH-2	65.6	5023068.8	359203.5
BH-3	65.9	5023098.3	359207.0
BH-4	65.6	5023067.2	359190.9
BH-5	65.9	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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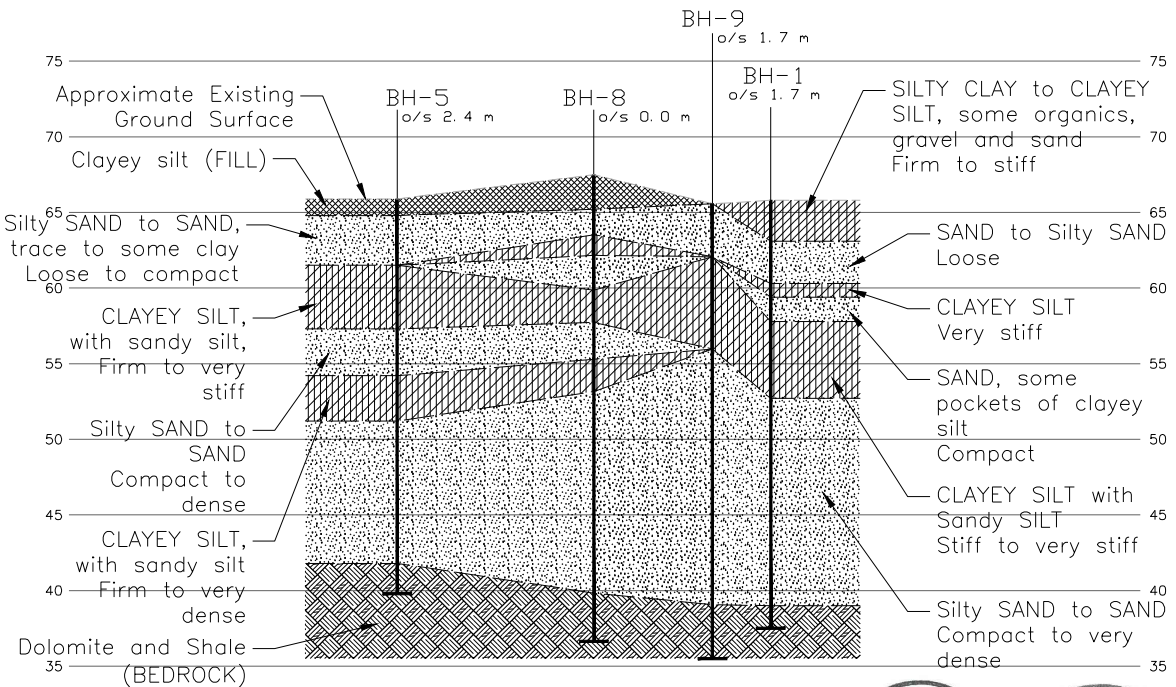
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CROSS-SECTION B-B'

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



CROSS-SECTION C-C'

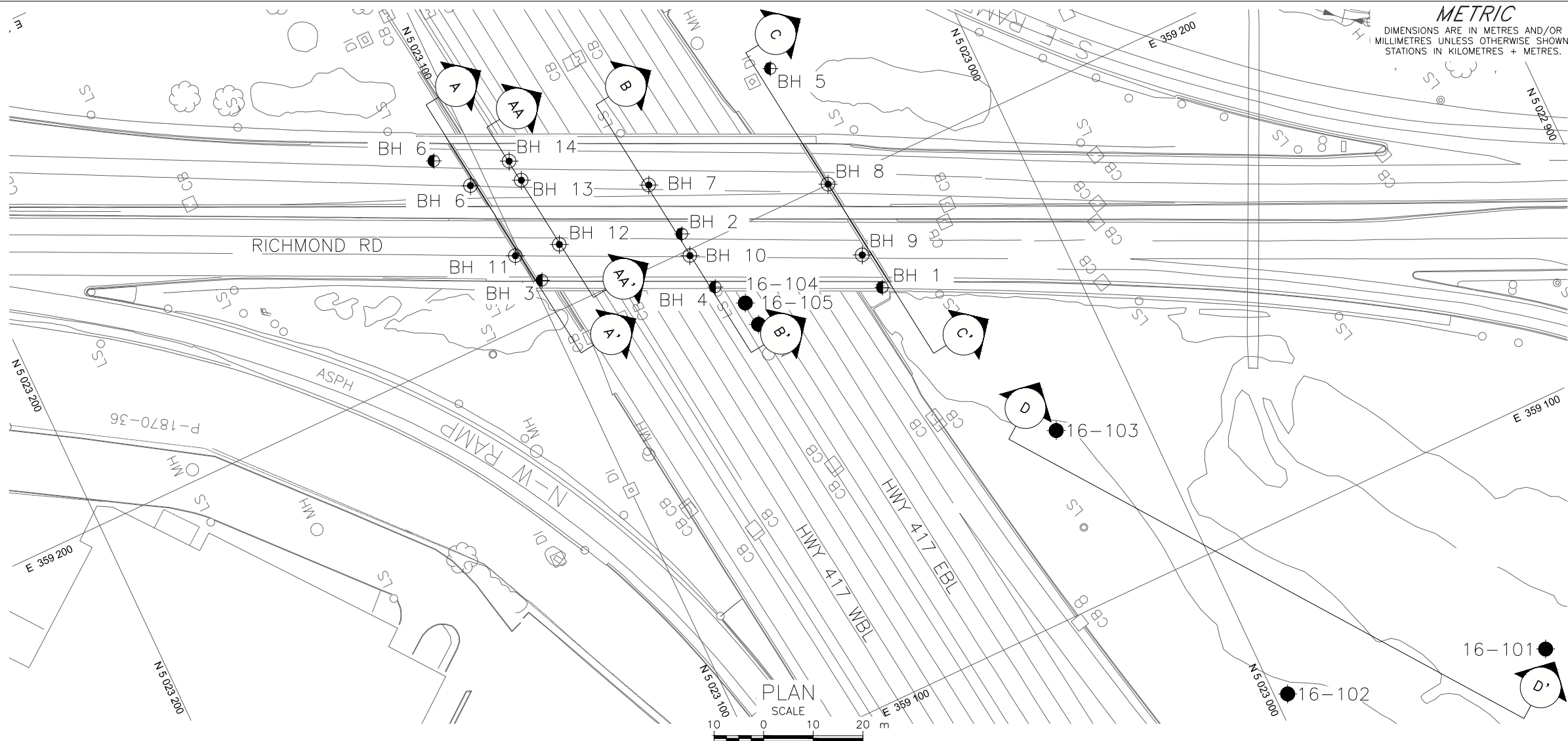
HORIZ. SCALE
10 0 10 20 m
VERT. SCALE
5 0 5 10 m

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	
SUBM'D. MJK		DATE: 12/21/2016	
DRAWN: JM		SITE: 3-039	
		DWG. 2	

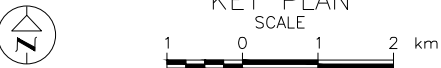
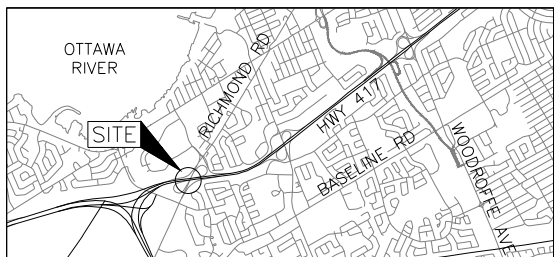


CONT No.
GWP No.4124-14-00



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
- N Standard Penetration Test Value
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- Seal
- Piezometer
- WL in piezometer, measured on AUG 2, 2016
- WL upon completion of drilling

BOREHOLE CO-ORDINATES

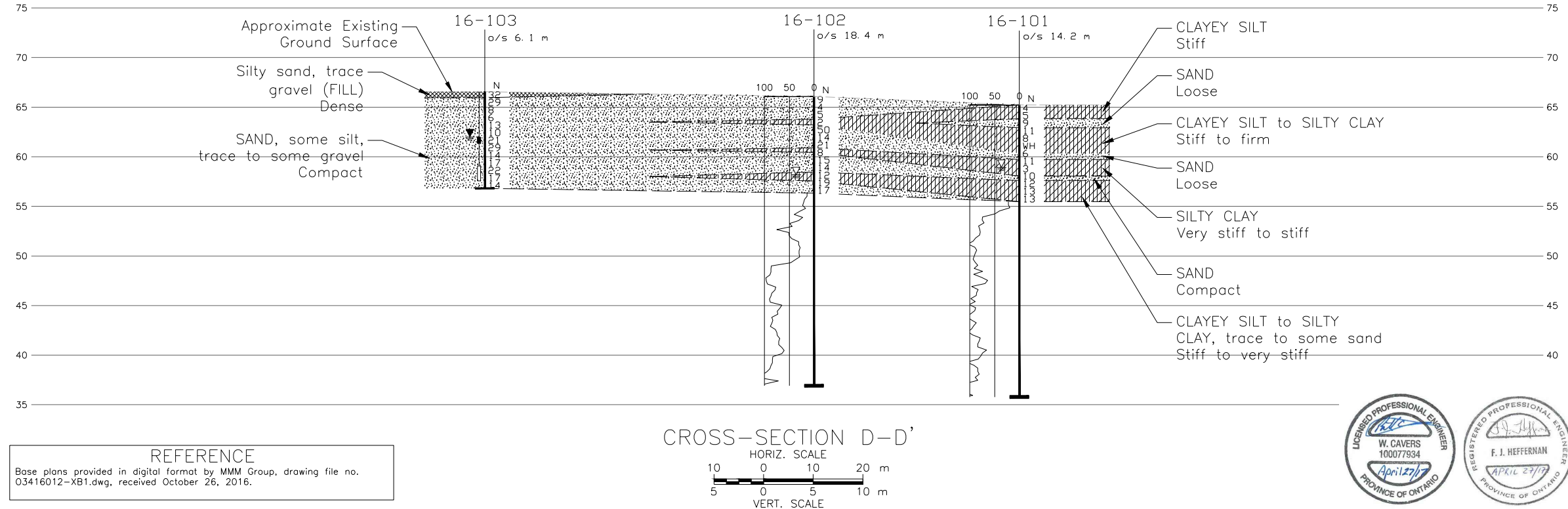
No.	ELEVATION	NORTHING	EASTING
16-101	65.2	5022946.5	359053.8
16-102	66.1	5022997.5	359067.7
16-103	66.6	5023017.2	359135.6
16-104	65.5	5023063.1	359185.5
16-105	65.5	5023062.5	359180.4
BH-1	65.8	5023036.7	359176.6
BH-2	65.6	5023068.8	359203.5
BH-3	65.9	5023098.3	359207.0
BH-4	65.6	5023067.2	359190.9
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
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BH-7	67.8	5023070.6	359215.3
BH-8	67.5	5023037.8	359200.1
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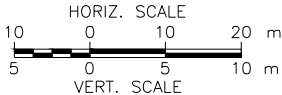
The complete Foundation Investigation and Design Report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents is specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.



REFERENCE

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CROSS-SECTION D-D'





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

	kPa	C_u, S_u	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			

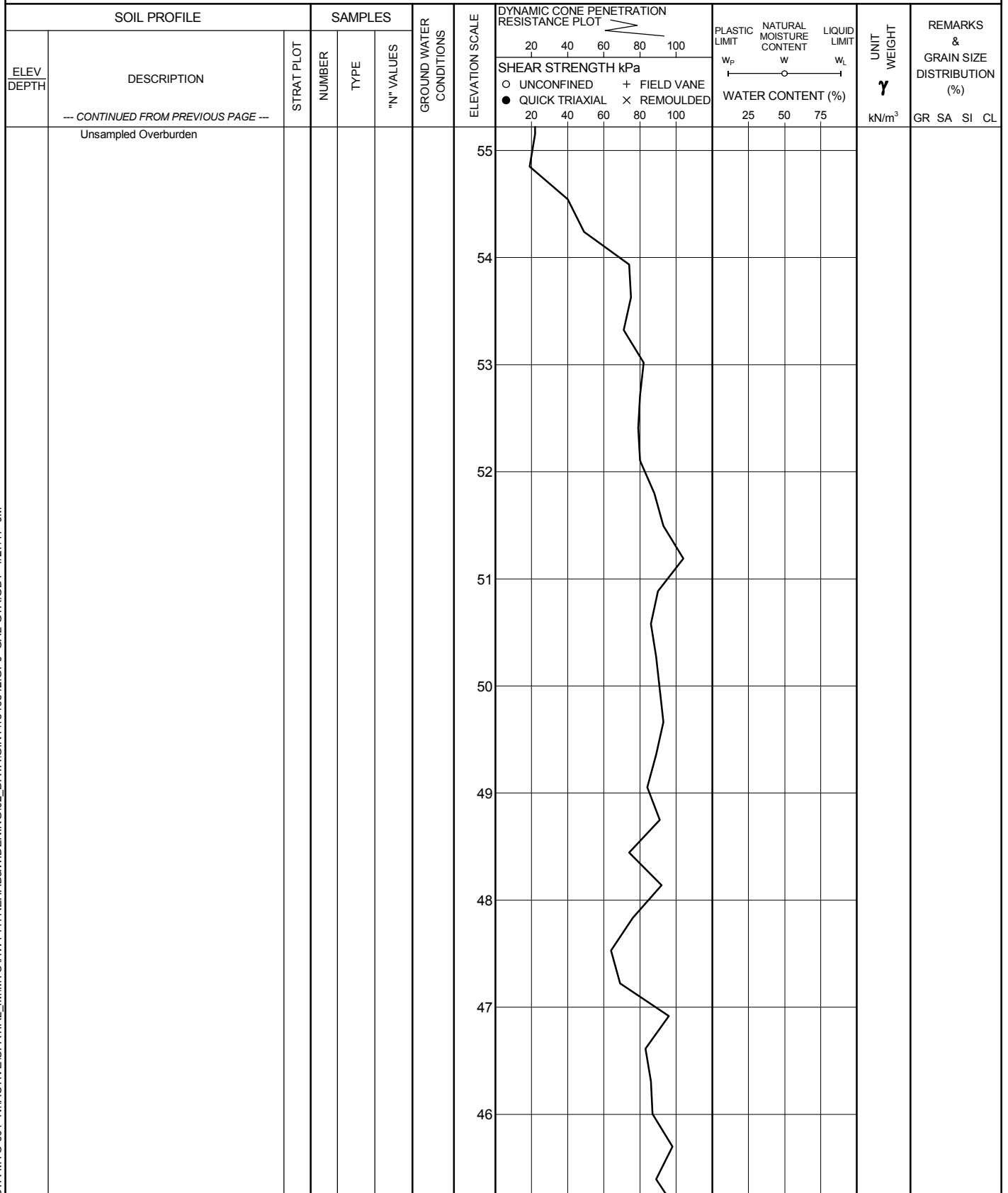
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						WATER CONTENT (%)			
								20	40	60	80	100		W _p	W	W _L	GR
65.2	GROUND SURFACE																
0.0	Silty sand (TOPSOIL)																
0.2	Brown																
	CLAYEY SILT		1	SS	4												
	Stiff																
	Grey																
	Moist																
			2	SS	5											0	22 44 34
63.8																	
1.4	SAND																
	Loose																
	Brown																
	Moist		3	SS	9												
62.9																	
2.3	CLAYEY SILT																
	Stiff																
	Grey		4	SS	11												
	Moist																
62.2																	
3.1	SAND																
	Loose																
	Brown																
	Moist		5	SS	8												
	CLAYEY SILT, contains sand																
	layers																
61.4	Firm to stiff																
3.8	Grey																
	Moist																
	SILTY CLAY		6	SS	WH												
	Firm																
	Grey																
	Moist to wet																
60.6																	
4.6	CLAYEY SILT																
	Firm																
60.3	Grey																
4.9	Wet		7	SS	6												0 95 (5)
	SAND																
	Loose																
	Brown																
	Wet																
59.7																	
5.5	SILTY CLAY		8	SS	11												
	Very stiff to stiff																
	Grey																
	Wet																
			9	SS	3											0	19 39 42
58.0																	
7.2	SAND																
	Compact																
	Grey																
	Wet																
			10	SS	10												
57.6																	
7.6	CLAYEY SILT to SILTY CLAY,																
	trace to some sand																
	Stiff to very stiff																
	Grey																
	Wet																
			11	SS	12												
			12	SS	13											0	17 43 40
			13	SS	13												
55.5																	
9.8	Unsampled Overburden																

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMT\OHVY417REHAB&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>			



Continued Next Page

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

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

PROJECT <u>1546542-1010</u>		RECORD OF BOREHOLE No 16-101		SHEET 3 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>			

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	20 40 60 80 100	20 40 60 80 100	W _p W W _L	25 50 75		
	--- CONTINUED FROM PREVIOUS PAGE --- Unsampled Overburden												
						45							
						44							
						43							
						42							
						41							
						40							
						39							
						38							
						37							
						36							
35.8 29.4													

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTOWHY417REHAB&WIDENING\02_DATA\GINT1546542.GPJ GAL-GTA.GDT 4/27/17 JM



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

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>			

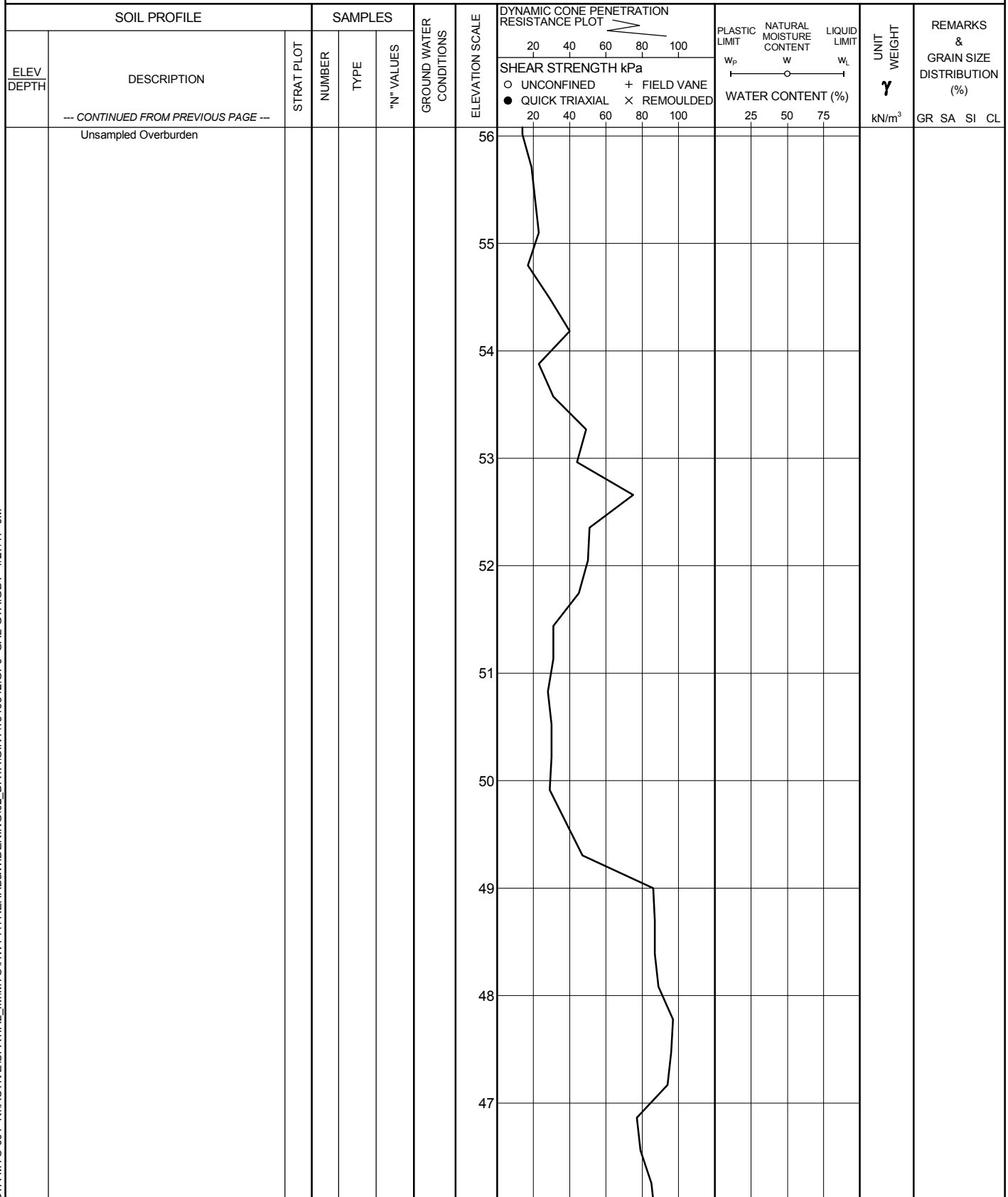
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× REMOULDED	W _p	W	W _L						
66.1	GROUND SURFACE							20	40	60	80	100								
65.9	Silty sand (TOPSOIL)																			
0.2	Dark brown Moist		1	SS	9															
	Silty SAND to SAND, trace clay																			
	Loose																			
	Grey-brown																			
	Moist																			
			2	SS	4															
			3	SS	5									○				0	73 20 7	
63.8																				
2.3	CLAYEY Silty SAND, some clay		4	SS	2									○				0	44 30 26	
63.2	Very loose																			
	Brown																			
	Wet																			
2.9	SAND and GRAVEL, contains		5	SS	50															
	cobbles and boulders																			
	Compact to dense																			
	Grey-brown																			
	Moist																			
			6	SS	14															
			7	SS	21									○				59	38 (3)	
60.9																				
5.2	CLAYEY SILT, some sand		8	SS	8															
	Stiff																			
	Grey																			
60.5	Wet																			
5.6	SAND, some gravel																			
	Compact																			
	Grey																			
	Wet																			
			9	SS	15															
			10	SS	14															
58.5																				
7.6	CLAYEY SILT, trace to some sand		11	SS	12									○				0	23 45 32	
	Stiff to very stiff																			
	Grey																			
	Wet																			
57.6																				
8.5	SAND, some gravel		12	SS	19															
	Compact																			
	Grey																			
	Wet																			
			13	SS	17															
56.4																				
9.8	Unsampled Overburden																			

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTOHWY417REHAB&WIDENING02_DATA\GINT1546542.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_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

Continued Next Page

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

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 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 (%)							
	END OF BOREHOLE DCPT REFUSAL						20	40	60	80	100				25	50	75				
	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\HWY417\REHAB&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 NATURAL LIQUID LIMIT MOISTURE 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									
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																				

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTD\HWY417REHAB&WIDENING\02_DATA\GINT1546542.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					WATER CONTENT (%)				GR	SA	SI	CL	
								○ UNCONFINED	● QUICK TRIAXIAL	+	×	FIELD VANE	REMOULDED	W _p	W		W _L				
	--- 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\HWY417REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

PROJECT		1546542-1010		RECORD OF BOREHOLE No 16-104		SHEET 1 OF 1		METRIC										
G.W.P.		4015-E-0017		LOCATION		N 5023063.1 ; E 359185.5		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger 200 mm Diam. (Hollow Stem)		COMPILED BY										
DATUM		Geodetic		DATE		November 13-14, 2016		CHECKED BY										
MJK																		
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT			REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV	DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa					WATER CONTENT (%)			γ	GR SA SI CL
								20 40 60 80 100	○ UNCONFINED + FIELD VANE	● QUICK TRIAXIAL × REMOULDED	W _p	W	W _L	25 50 75				
65.5	0.0	GROUND SURFACE ASPHALTIC CONCRETE																
65.1	0.4	Gravelly sand (FILL) Grey Moist Gravel and sand, with non-plastic fines (FILL) Grey-brown Moist						65										
64.0	1.5	Sand, with gravel (FILL) Brown Moist						64										
62.6		SILTY CLAY to CLAYEY SILT Grey Moist END OF AUGERHOLE						63										

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMT\HWHY417REHAB&WIDENING\02_DATA\GINT\1546542.GPJ GAL-GTA.GDT 4/27/17 JM

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

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								
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					WATER CONTENT (%)			
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100	W _p	W	W _L		
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED					25 50 75				
							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

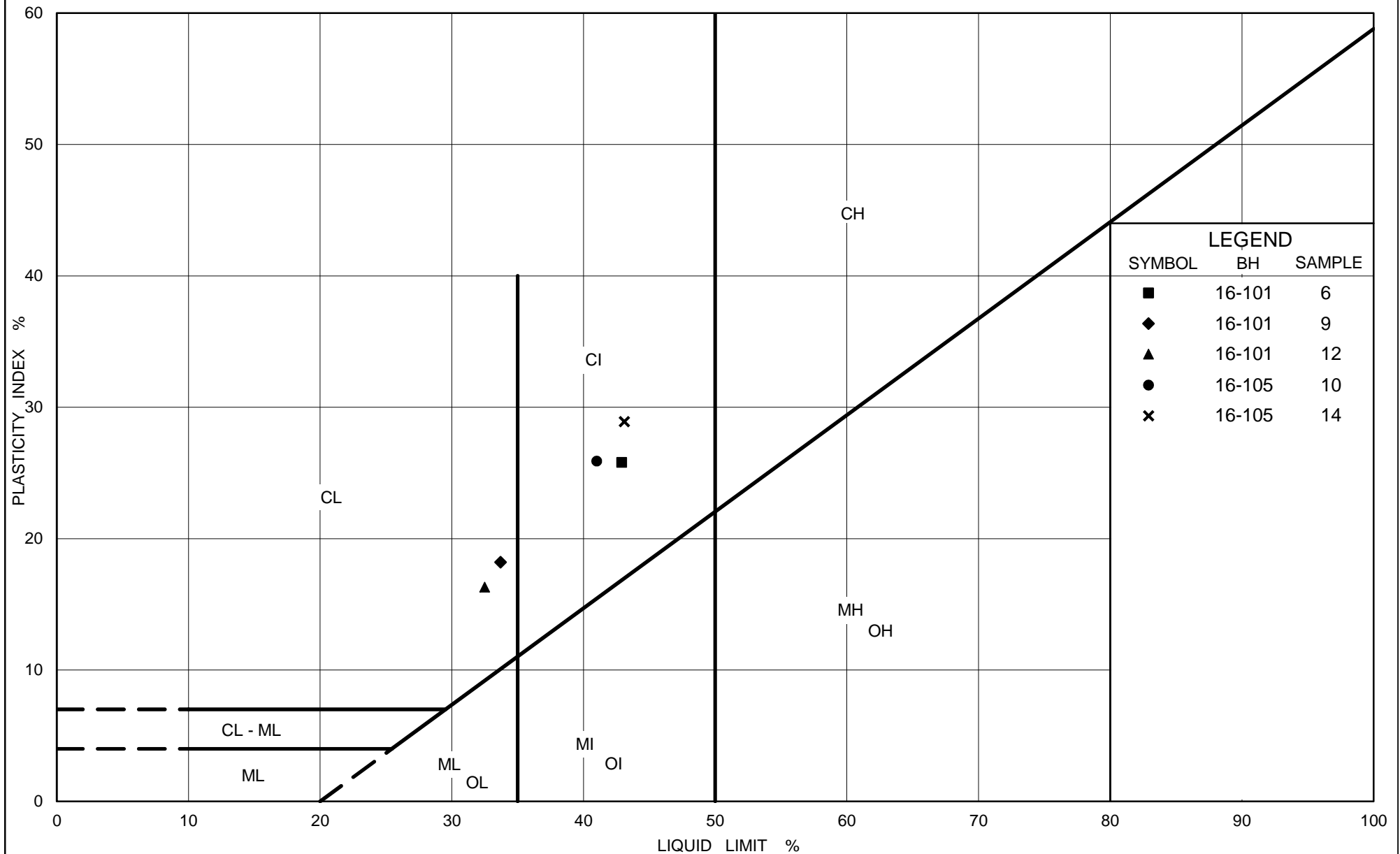
Figure 2 - Grain Size Distribution Test Results – Silty Clay

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

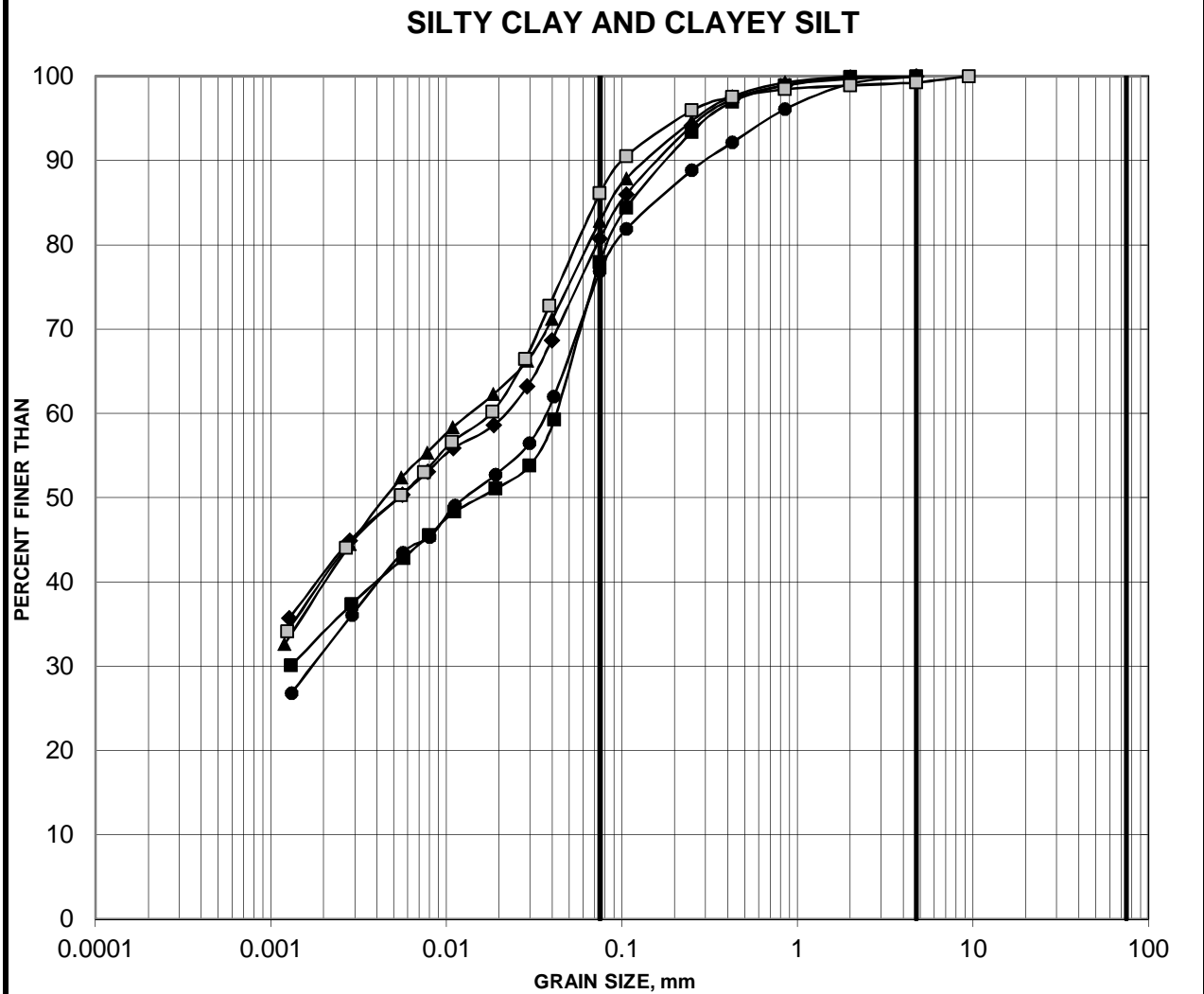
FIG No. 1

Project No. 1546542/ 1010

Compiled By : MI Checked By : CNM

GRAIN SIZE DISTRIBUTION

FIGURE 2



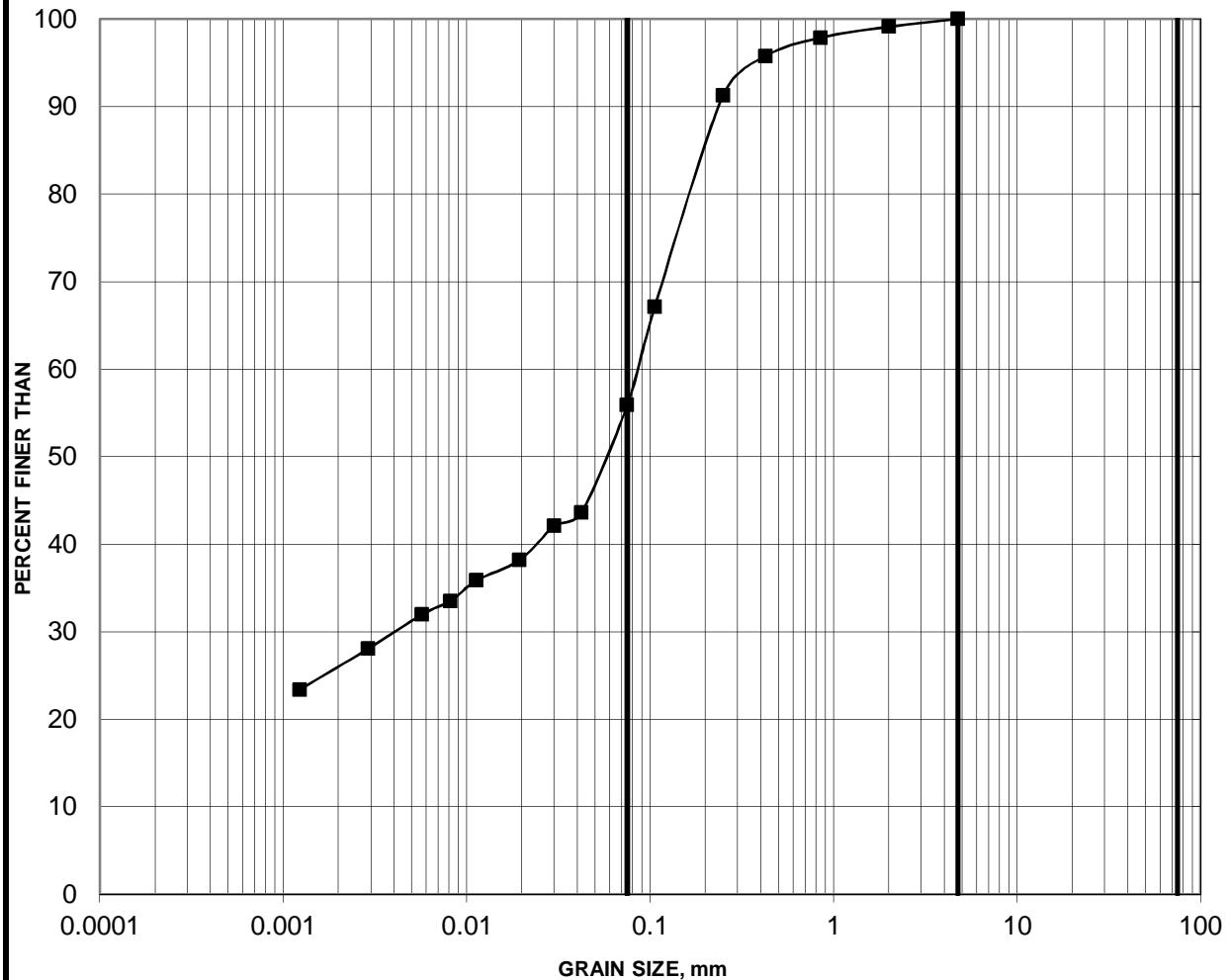
SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

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

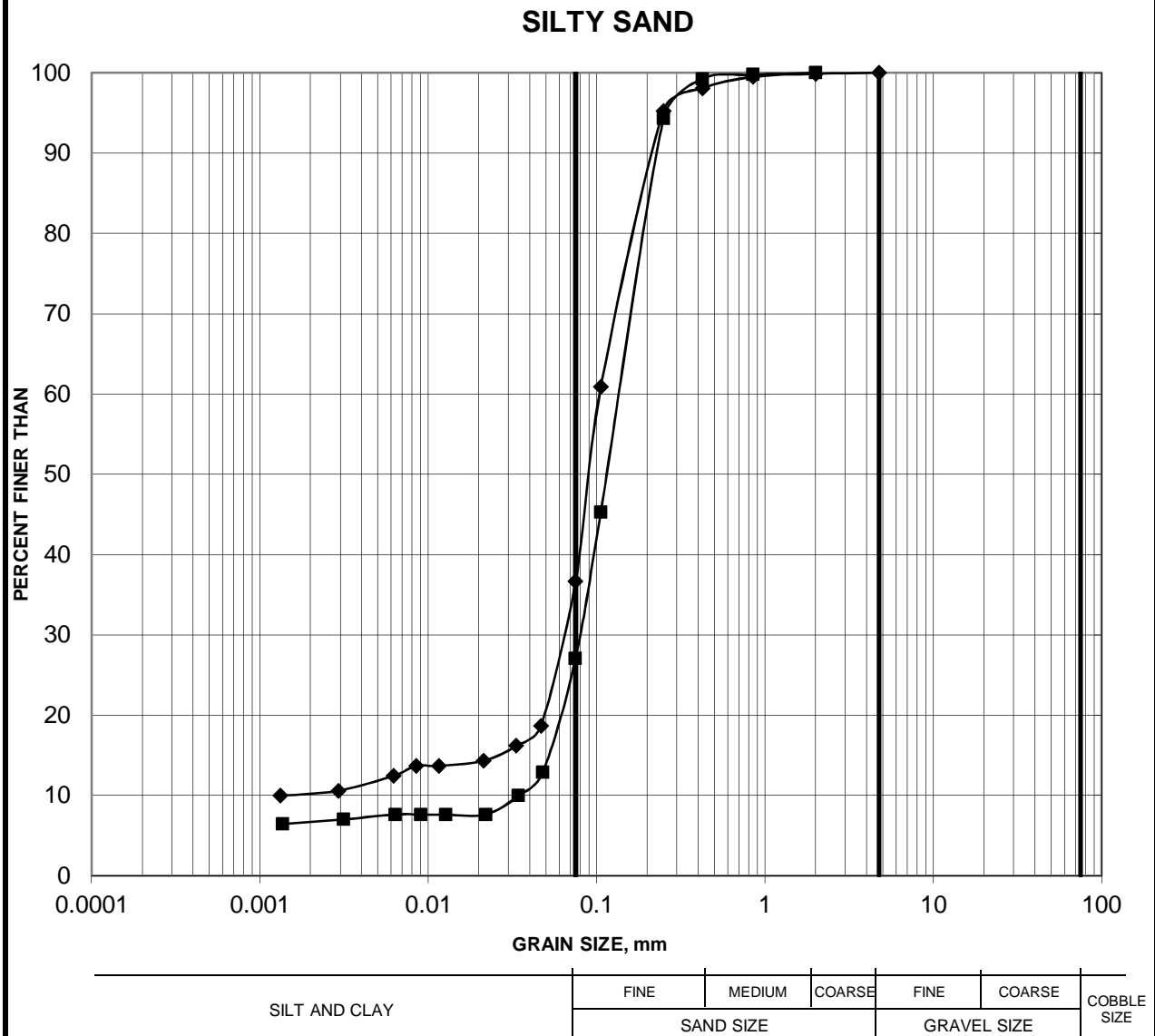


SILT AND CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
	SAND SIZE			GRAVEL SIZE		

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

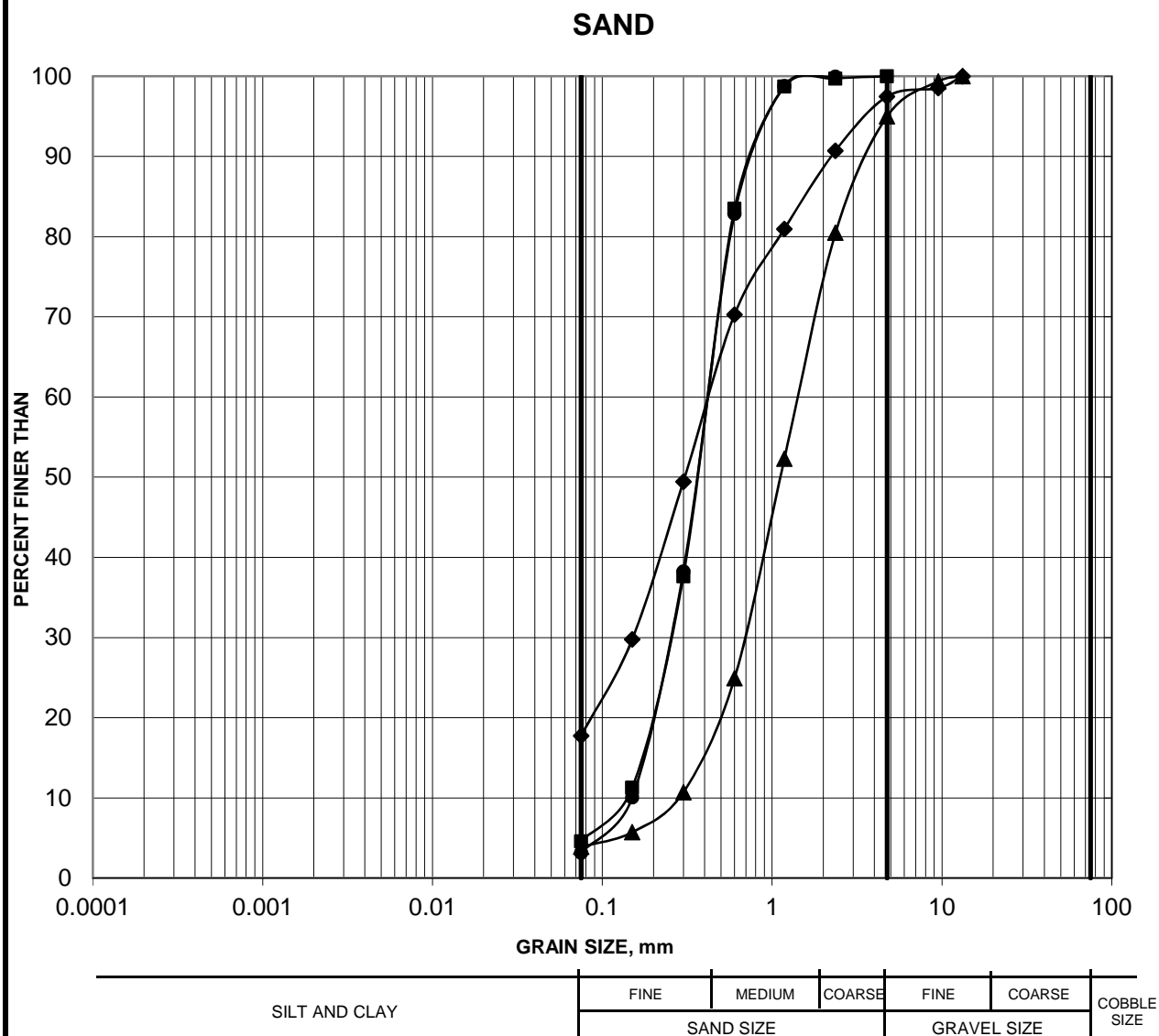
GRAIN SIZE DISTRIBUTION

FIGURE 4



GRAIN SIZE DISTRIBUTION

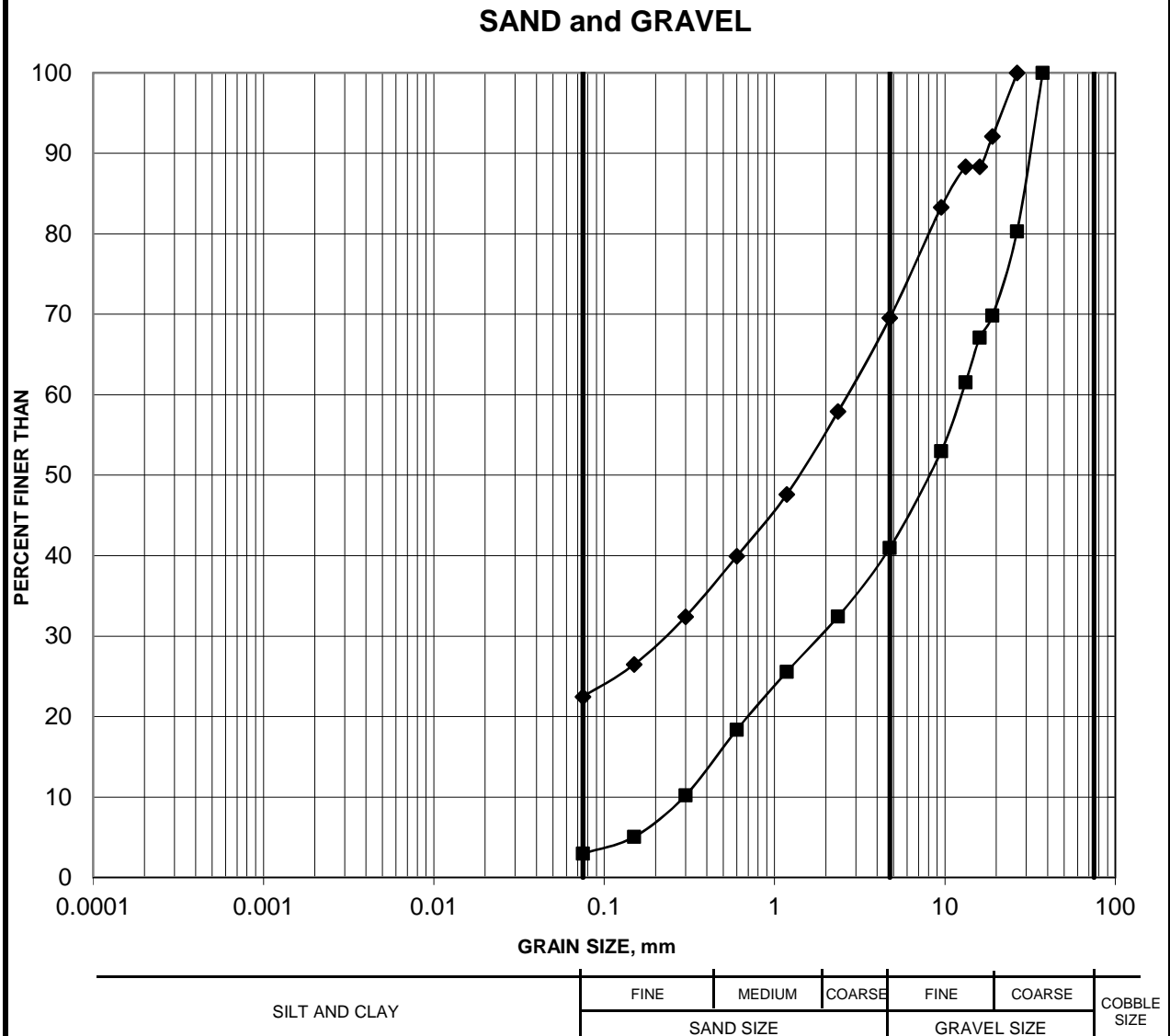
FIGURE 5



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



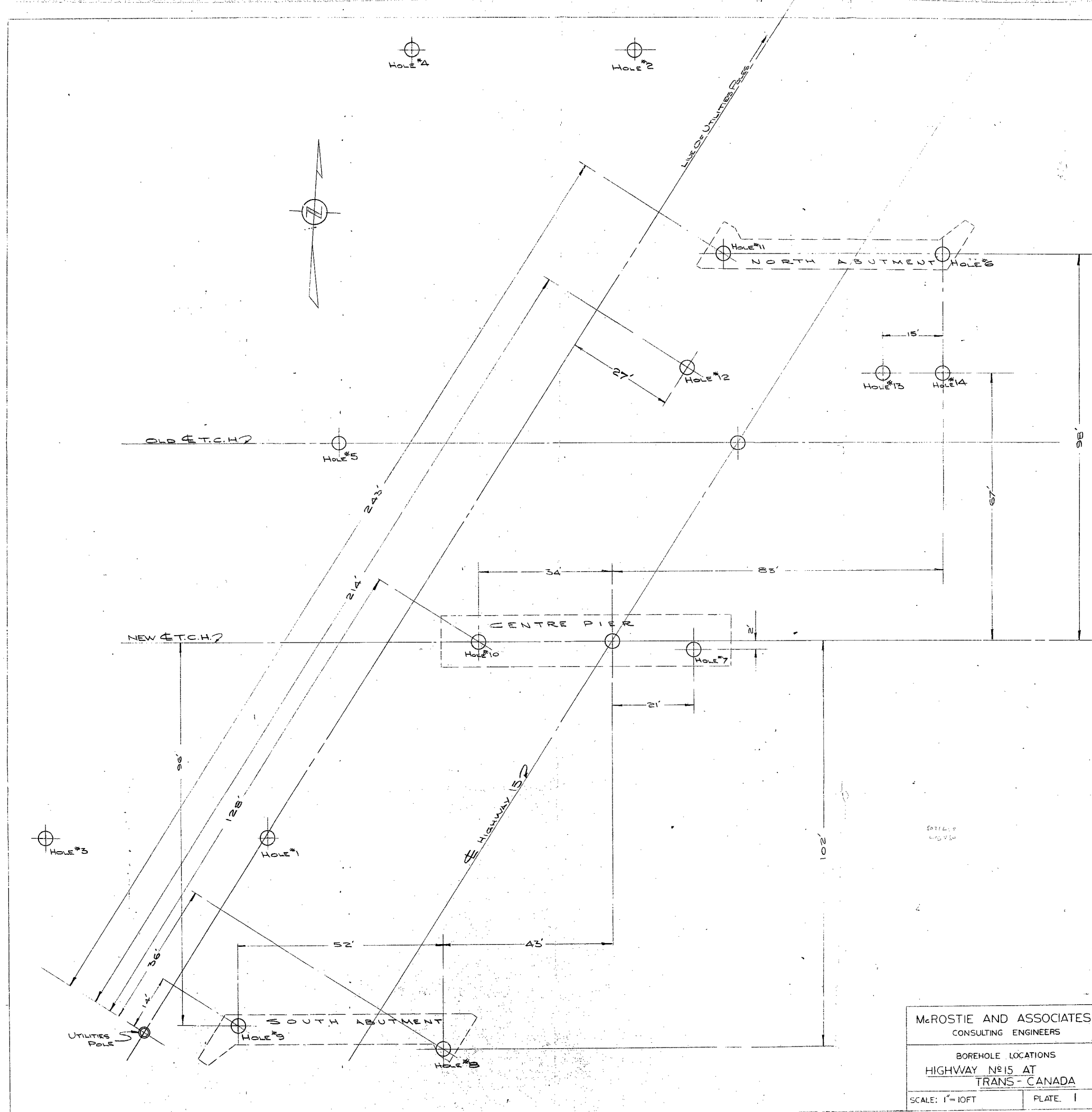
APPENDIX C

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 & 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-2-1955

HOLE NO.

REMARKS SEE: PLATE # 2

7

UNCONFINED COMPRESSIVE STRENGTH KIPS/FT. ²	SMALL SCALE PENE. CONE 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.
			13	7-1 SANDY FILL	0	222.2'	0	
			8	7-2 MEDIUM DENSE CLAYEY WELL-GRADED SAND	5	217.2'	0	0
			15	7-3 GRADED SAND	9.0	212.2'	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		7-5	STIFF SILTY GRAY CLAY WITH SOME SAND	15	207.2'	0	0
2.0	2.0, 2.0, 2.2 3.2, 3.4, 2.8 2-0.2		7-6	LOOSE CLAYEY WELL-GRADED SAND	15.5	207.2'	0	0
	4.0, 4.2, 4.8		9	7-7 VERY STIFF GRAY CLAY & SILT WITH SOME SAND	20	202.2'	0	0
			15	7-8 MEDIUM DENSE WELL-GRADED SAND 22.5'	20	202.2'	0	0
			14	7-9 MEDIUM DENSE WELL-GRADED SAND WITH A FEW CLAY POCKETS	22.5	197.2'	0	0
	2.2, 2.6, 2.4		5	7-10 STIFF SILTY GRAY CLAY	25	197.2'	0	0
				7-11 LOOSE FINE SAND WITH A FEW STONES	30	192.2'	0	
			26	7-12 MEDIUM DENSE FINE SAND	35	187.2'	0	
	2.2, 2.0, 2.0		25	7-13 STIFF GRAY CLAY & SILT WITH SOME SAND & A FEW STONES	40	182.2'	0	
					45	177.2'		
			10	7-14 LOOSE FINE SAND	50	172.2'	0	
					55	167.2'		
			7	7-15 LOOSE FINE SAND	60	162.2'	0	
					65	157.2'		
			24	7-16 MEDIUM DENSE FINE SAND	70	152.2'	0	
				CORE RECOVERY - 60% 75.5'	74.7	147.5'		
				DOLOMITE ROCK WITH SHALE LAYERS	80.8	142.2'		
				CORE RECOVERY - 97%	80.8	142.2'		
				DOLOMITE ROCK WITH SHALE LAYERS	85	137.2'		
				CORE RECOVERY - 97%	85	137.2'		
				BOTTOM OF HOLE	87.3	134.9'		
B - REMOVED								
							0 20 40 60 80 100	
							% WATER CONTENT	PLATE
							NATURAL 0	2
							LIQUID LIMIT 0	
							PLASTIC LIMIT A	

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							BLOWS PER FOOT OR	SHEAR STRENGTH IN KIPS PER FT.
					0	221.3	0	0
		11	0-1	FILL			0	0
		9	0-2		5	216.3	0	0
		13	0-3	MEDIUM DENSE SILT WITH A LITTLE SAND	7.5		0	0
		5	0-4	LOOSE COARSE SAND WITH SOME FINE SAND	10	211.3	0	0
1.9	1.2, 1.0, 1.0	6	0-5	LOOSE COARSE SAND WITH SOME FINE SAND	12.5		0	0
	2.0, 2.0, 2.0	8-5A	0-5A	MEDIUM SOFT SILTY GRAY CLAY	15	206.3	0	0
	2.0, 2.0, 2.4	8-6	0-6	STIFF SILTY GRAY CLAY WITH SOME SAND	17.5		0	0
	2.0, 2.0	8-7	0-7	LOOSE COARSE SAND WITH SOME CLAY SILT	20	201.3	0	0
		18	0-8	MEDIUM DENSE COARSE SAND WITH SOME FINE SAND	25	196.3	0	0
	1.4, 1.4, 1.8	9	0-9	MEDIUM SOFT GRAY CLAY & SILT WITH A LITTLE SAND	30	191.3	0	0
	1.0, 1.0, 1.6	4	0-10	STIFF SILTY GRAY CLAY WITH SOME SAND	33.5		0	0
2.7	3.0, 3.0, 2.0		0-11	MEDIUM DENSE FINE SAND WITH SOME SILTY CLAY POCKETS	35	186.3	0	0
	2.0, 2.0, 2.8		0-12	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	40	181.3	0	0
	2.0, 2.0		0-13	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	45	176.3	0	0
			0-14	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	50	171.3	0	0
			0-15	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	55	166.3	0	0
			0-16	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	60	161.3	0	0
			0-17	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	65	156.3	0	0
			0-18	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	70	151.3	0	0
			0-19	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	75	146.3	0	0
			0-20	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	80	141.3	0	0
			0-21	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	85	136.3	0	0
			0-22	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	90	131.3	0	0
			0-23	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	95	126.3	0	0
			0-24	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	100	121.3	0	0
			0-25	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	105	116.3	0	0
			0-26	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	110	111.3	0	0
			0-27	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	115	106.3	0	0
			0-28	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	120	101.3	0	0
			0-29	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	125	96.3	0	0
			0-30	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	130	91.3	0	0
			0-31	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	135	86.3	0	0
			0-32	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	140	81.3	0	0
			0-33	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	145	76.3	0	0
			0-34	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	150	71.3	0	0
			0-35	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	155	66.3	0	0
			0-36	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	160	61.3	0	0
			0-37	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	165	56.3	0	0
			0-38	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	170	51.3	0	0
			0-39	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	175	46.3	0	0
			0-40	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	180	41.3	0	0
			0-41	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	185	36.3	0	0
			0-42	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	190	31.3	0	0
			0-43	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	195	26.3	0	0
			0-44	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	200	21.3	0	0
			0-45	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	205	16.3	0	0
			0-46	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	210	11.3	0	0
			0-47	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	215	6.3	0	0
			0-48	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	220	1.3	0	0
			0-49	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	225	-3.7	0	0
			0-50	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	230	-8.7	0	0
			0-51	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	235	-13.7	0	0
			0-52	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	240	-18.7	0	0
			0-53	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	245	-23.7	0	0
			0-54	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	250	-28.7	0	0
			0-55	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	255	-33.7	0	0
			0-56	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	260	-38.7	0	0
			0-57	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	265	-43.7	0	0
			0-58	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	270	-48.7	0	0
			0-59	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	275	-53.7	0	0
			0-60	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	280	-58.7	0	0
			0-61	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	285	-63.7	0	0
			0-62	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	290	-68.7	0	0
			0-63	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	295	-73.7	0	0
			0-64	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	300	-78.7	0	0
			0-65	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	305	-83.7	0	0
			0-66	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	310	-88.7	0	0
			0-67	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	315	-93.7	0	0
			0-68	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	320	-98.7	0	0
			0-69	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	325	-103.7	0	0
			0-70	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	330	-108.7	0	0
			0-71	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	335	-113.7	0	0
			0-72	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	340	-118.7	0	0
			0-73	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	345	-123.7	0	0
			0-74	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	350	-128.7	0	0
			0-75	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	355	-133.7	0	0
			0-76	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	360	-138.7	0	0
			0-77	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	365	-143.7	0	0
			0-78	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	370	-148.7	0	0
			0-79	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	375	-153.7	0	0
			0-80	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	380	-158.7	0	0
			0-81	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	385	-163.7	0	0
			0-82	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	390	-168.7	0	0
			0-83	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	395	-173.7	0	0
			0-84	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	400	-178.7	0	0
			0-85	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	405	-183.7	0	0
			0-86	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	410	-188.7	0	0
			0-87	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	415	-193.7	0	0
			0-88	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	420	-198.7	0	0
			0-89	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	425	-203.7	0	0
			0-90	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	430	-208.7	0	0
			0-91	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	435	-213.7	0	0
			0-92	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	440	-218.7	0	0
			0-93	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	445	-223.7	0	0
			0-94	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	450	-228.7	0	0
			0-95	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	455	-233.7	0	0
			0-96	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	460	-238.7	0	0
			0-97	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	465	-243.7	0	0
			0-98	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	470	-248.7	0	0
			0-99	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	475	-253.7	0	0
			0-100	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	480	-258.7	0	0
			0-101	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	485	-263.7	0	0
			0-102	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	490	-268.7	0	0
			0-103	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	495	-273.7	0	0
			0-104	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	500	-278.7	0	0
			0-105	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	505	-283.7	0	0
			0-106	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	510	-288.7	0	0
			0-107	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	515	-293.7	0	0
			0-108	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	520	-298.7	0	0
			0-109	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	525	-303.7	0	0
			0-110	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	530	-308.7	0	0
			0-111	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	535	-313.7	0	0
			0-112	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	540	-318.7	0	0
			0-113	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	545	-323.7	0	0
			0-114	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	550	-328.7	0	0
			0-115	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	555	-333.7	0	0
			0-116	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	560	-338.7	0	0
			0-117	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	565	-343.7	0	0
			0-118	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	570	-348.7	0	0
			0-119	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	575	-353.7	0	0
			0-120	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	580	-358.7	0	0
			0-121	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	585	-363.7	0	0
			0-122	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	590	-368.7	0	0
			0-123	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	595	-373.7	0	0
			0-124	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	600	-378.7	0	0
			0-125	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	605	-383.7	0	0
			0-126	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	610	-388.7	0	0
			0-127	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	615	-393.7	0	0
			0-128	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	620	-398.7	0	0
			0-129	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	625	-403.7	0	0
			0-130	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	630	-408.7	0	0
			0-131	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	635	-413.7	0	0
			0-132	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	640	-418.7	0	0
			0-133	VERY STIFF SILTY GRAY CLAY WITH A FEW SAND LAYERS	645	-423.7	0	0
			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) 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	
				MEDIUM DENSE SILTY FINE SAND WITH SOME SILTY CLAY POCKETS AND A FEW PEBBLES	35	180.0		
		18	9-4		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				DENSE COARSE SAND & GRAVEL	55	160.0		
		22 FOR	9-6		60	155.0		
					65	150.0	0	
		27	9-7	MEDIUM DENSE FINE SAND	70	145.0		
					75	140.0	0	
		20	9-8	MEDIUM DENSE FINE SAND WITH A LITTLE GRAVEL & A TRACE OF SILT	80	135.0		
					85	130.0	0 SILTY SAND 0 CLAY 0 SAND	
		20	9-9	MEDIUM DENSE FINE SAND WITH CLAY & SILT LAYERS & SOME PEBBLES	86.5	128.1		
				DOLOMITE WITH SHALE LAYERS CORE RECOVERY - 91%	90	125.0		
					92.6			
				DOLOMITE WITH SHALE LAYERS CORE RECOVERY - 92%	95	120.0		
				BOTTOM OF HOLE	98.6	112.4		
							% WATER CONTENT	
							NATURAL	PLATE
							LIQUID LIMIT	S
							PLASTIC LIMIT	

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) 219.2'

DATE JULY-17-1959

HOLE NO.

REMARKS SEE: PLATE #2

10

CORRECTED CONFINED COMPRESSION 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	6
LIQUID LIMIT	
PLASTIC LIMIT	

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) 222.4

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				
							NATURAL	○			
							LIQUID LIMIT	□			
							PLASTIC LIMIT	△			
										PLATE	8

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.

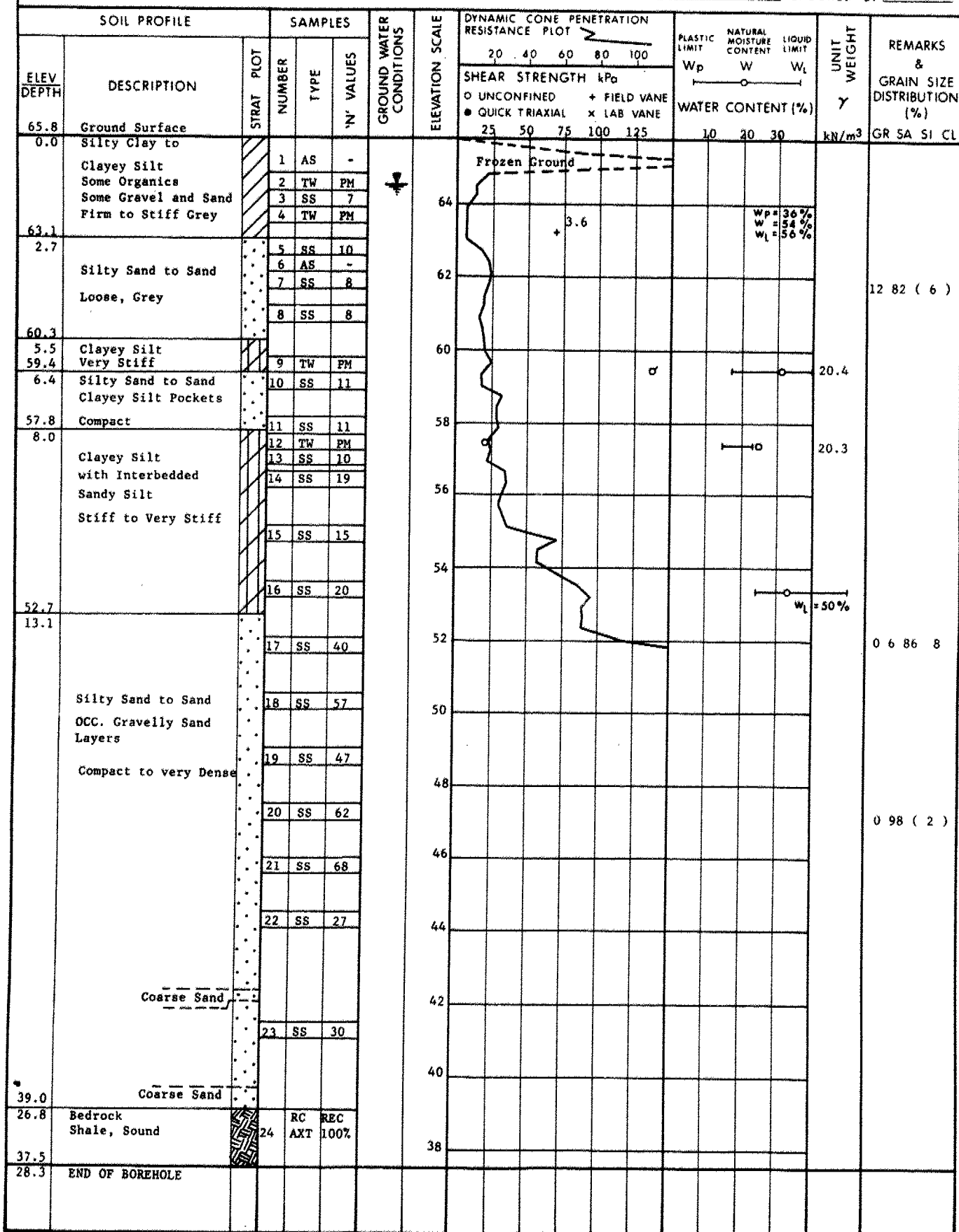
REMARKS SEE PLATE # 2

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



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						60	80	100	25	50
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																			
21.1	Bedrock		16	RC	REC														
43.7	Shale Sound			AXT	100%														
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					
					150					
149.0										
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									
	Loose to Compact		4	SS	14									
61.5			5	SS	11									
4.4	Clayey Silt with		6	SS	5									0 77 13 10
	Interbedded Sandy Silt		7	TW	PH									
	Occ. Silty Clay Layers		8	TW	PH									0 4 71 25
	Firm to Very Stiff		9	SS	25									0 13 69 18
57.3														
8.6	Silty Sand to Sand		10	SS	30									0 68 21 11
	Compact to Dense		11	SS	15									
54.2														
11.7	Clayey Silt with		12	SS	25									
	Interbedded Sandy Silt													
	Firm to Very Stiff		13	SS	4									0 23 60 17
51.2														
14.7	Silty Sand to Sand		14	SS	8									
	Occ. Clayey Silt Layers													
	Occ. Gravelly Sand													
	Loose to Very Dense		15	SS	4									0 73 20 7
	Clayey Silt		16	SS	90/	15cm								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 γ	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.



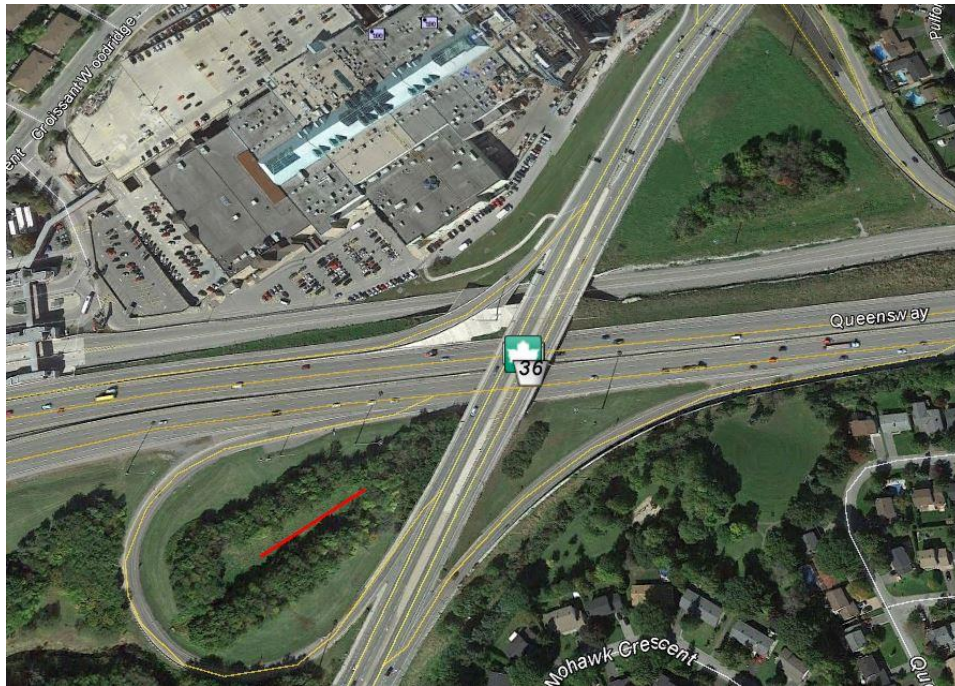


Figure 1: MASW Location Site Map at the Richmond site (MASW Line 1 in red)



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

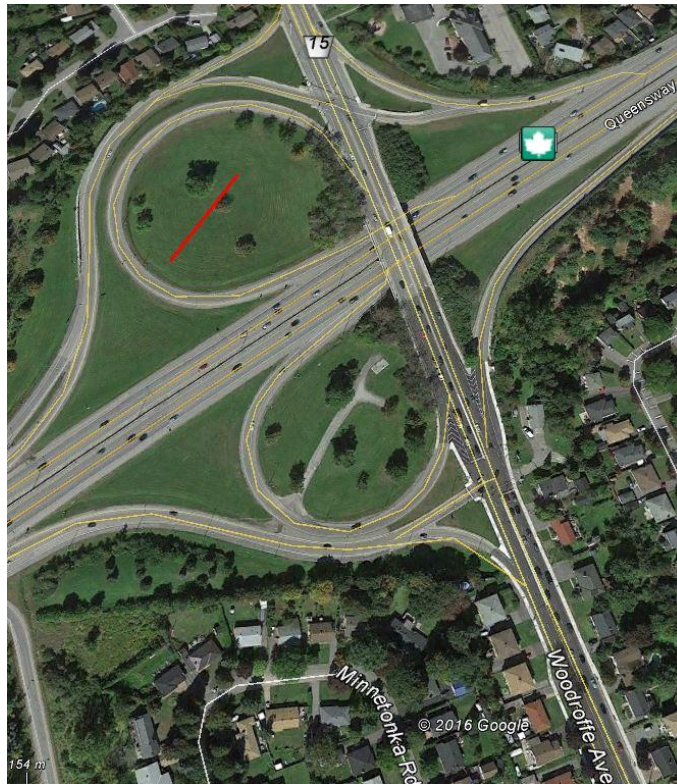


Figure 3: MASW Location Site Map at the Woodroffe site (MASW Line 3 in red)

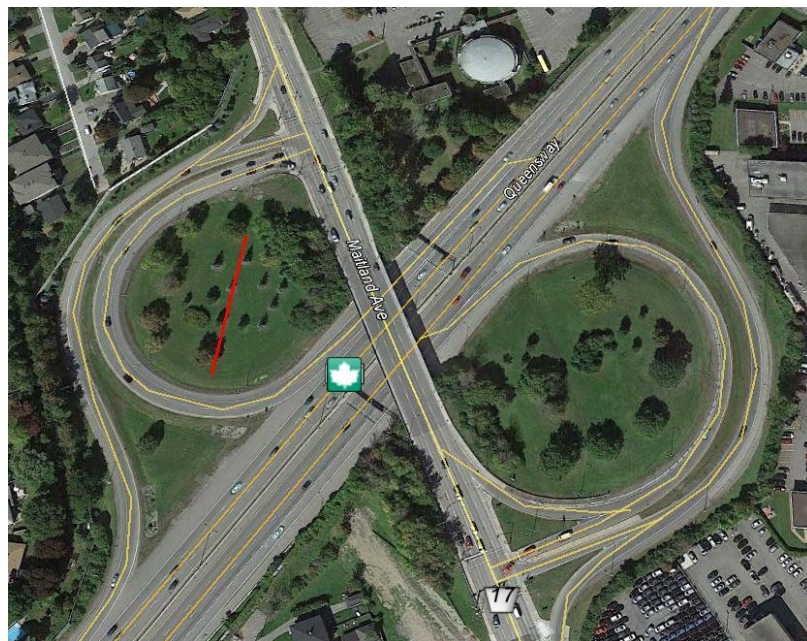


Figure 4: MASW Location Site Map at the Maitland site (MASW Line 4 in red)

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.

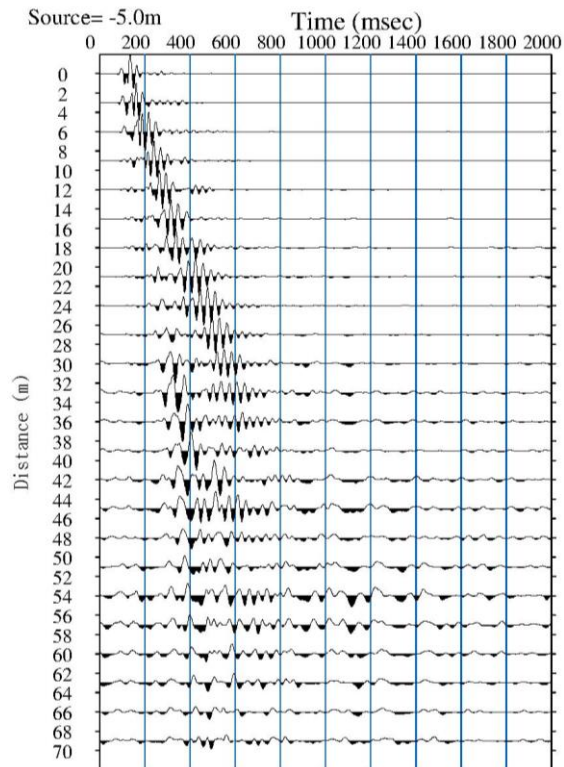


Figure 5: Typical seismic record collected along MASW Line 1

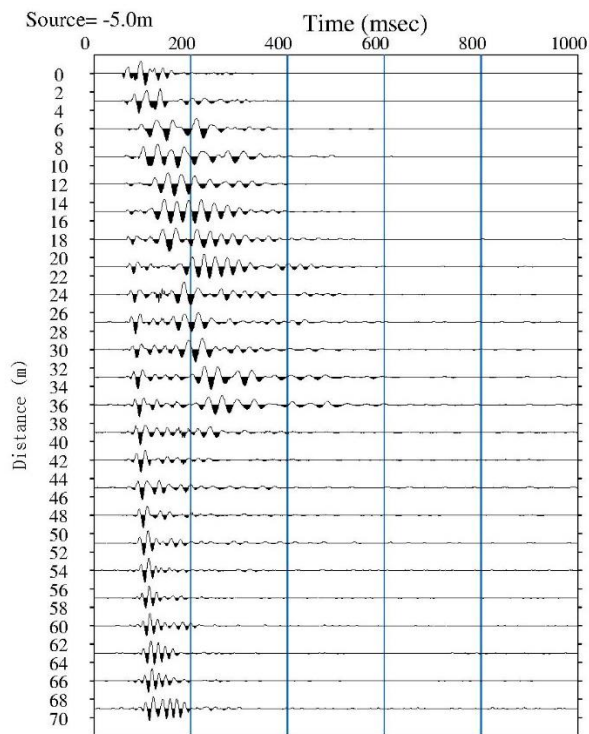


Figure 6: Typical seismic record collected along MASW Line 2

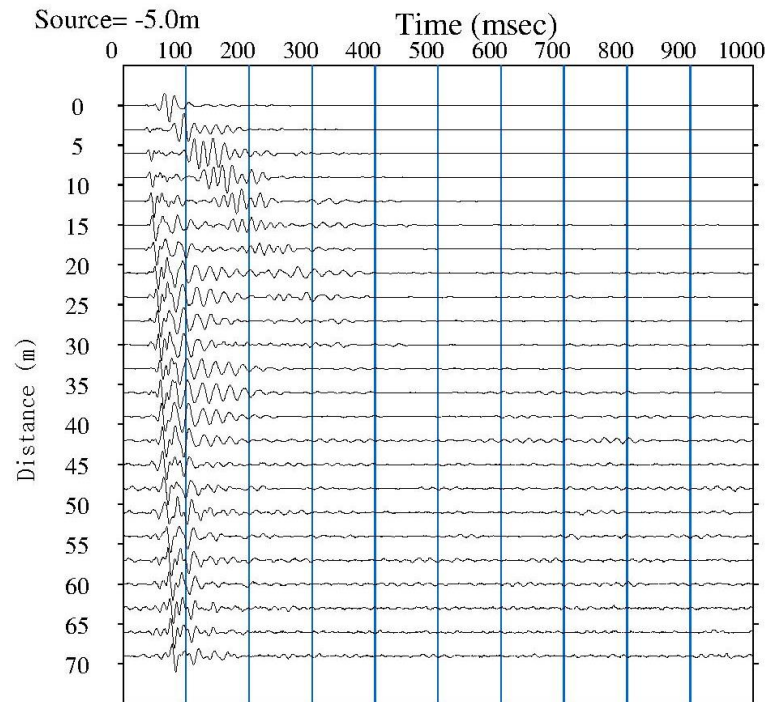


Figure 7: Typical seismic record collected along MASW Line 3

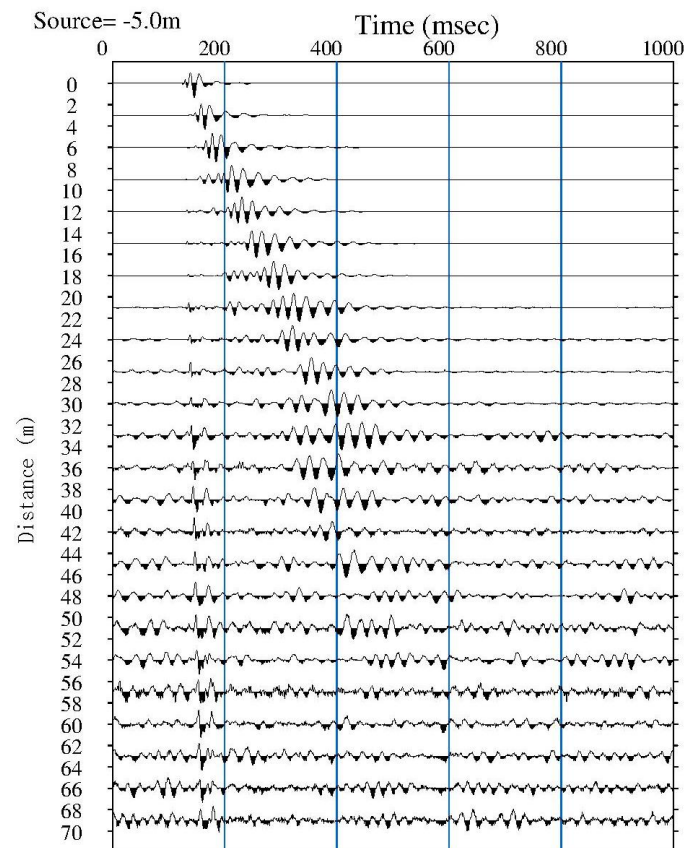


Figure 8: Typical seismic record collected along MASW Line 4

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.

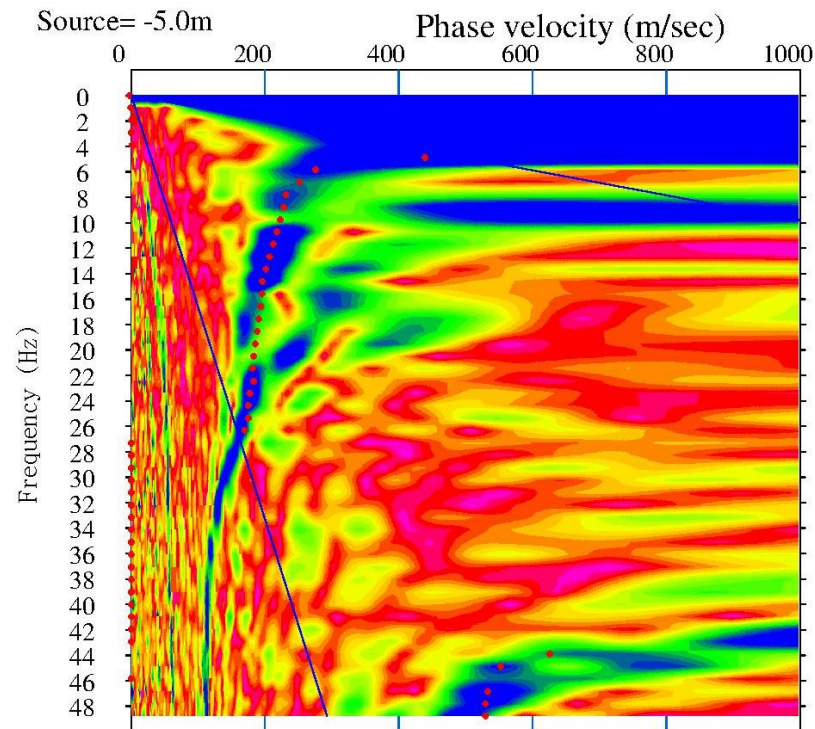


Figure 9: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1 (Richmond)

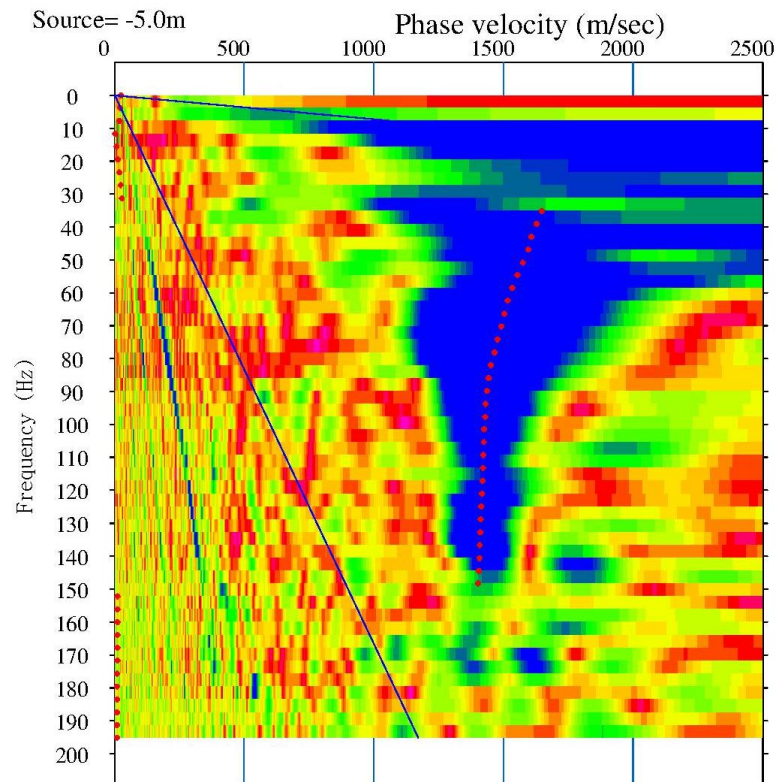


Figure 10: Active MASW Dispersion Curve Picks (red dots) along MASW Line 2 (Pinecrest).

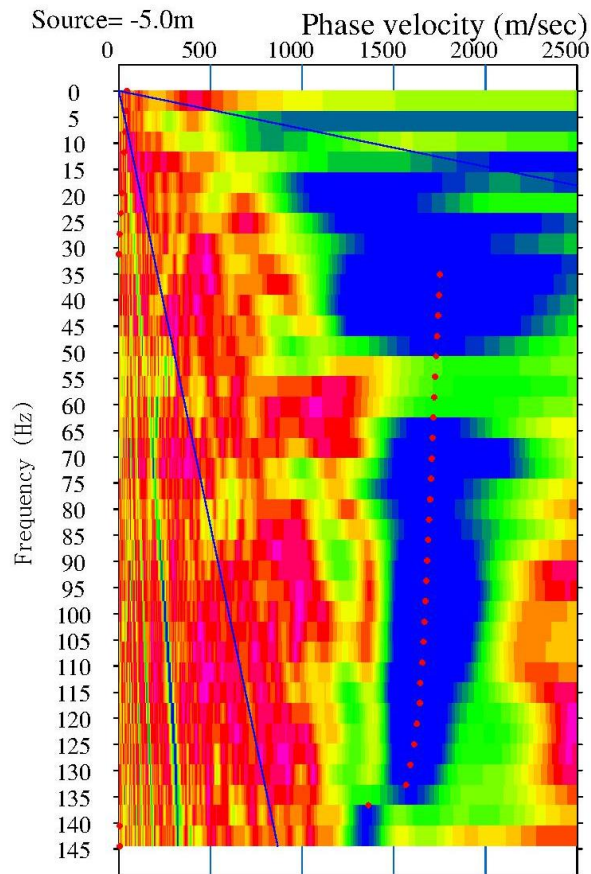


Figure 11: Active MASW Dispersion Curve Picks (red dots) along MASW Line 3 (Woodroffe).

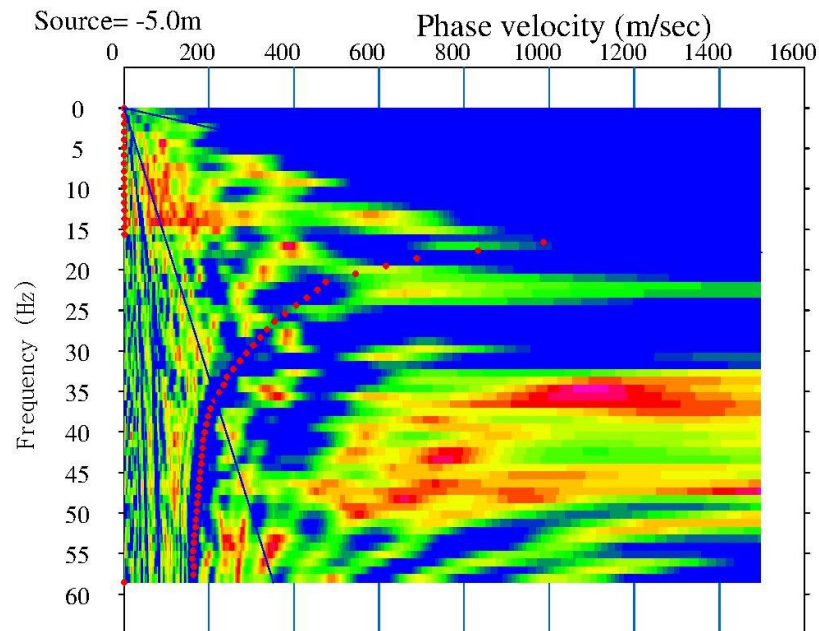


Figure 12: Active MASW Dispersion Curve Picks (red dots) along MASW Line 4 (Maitland).

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.

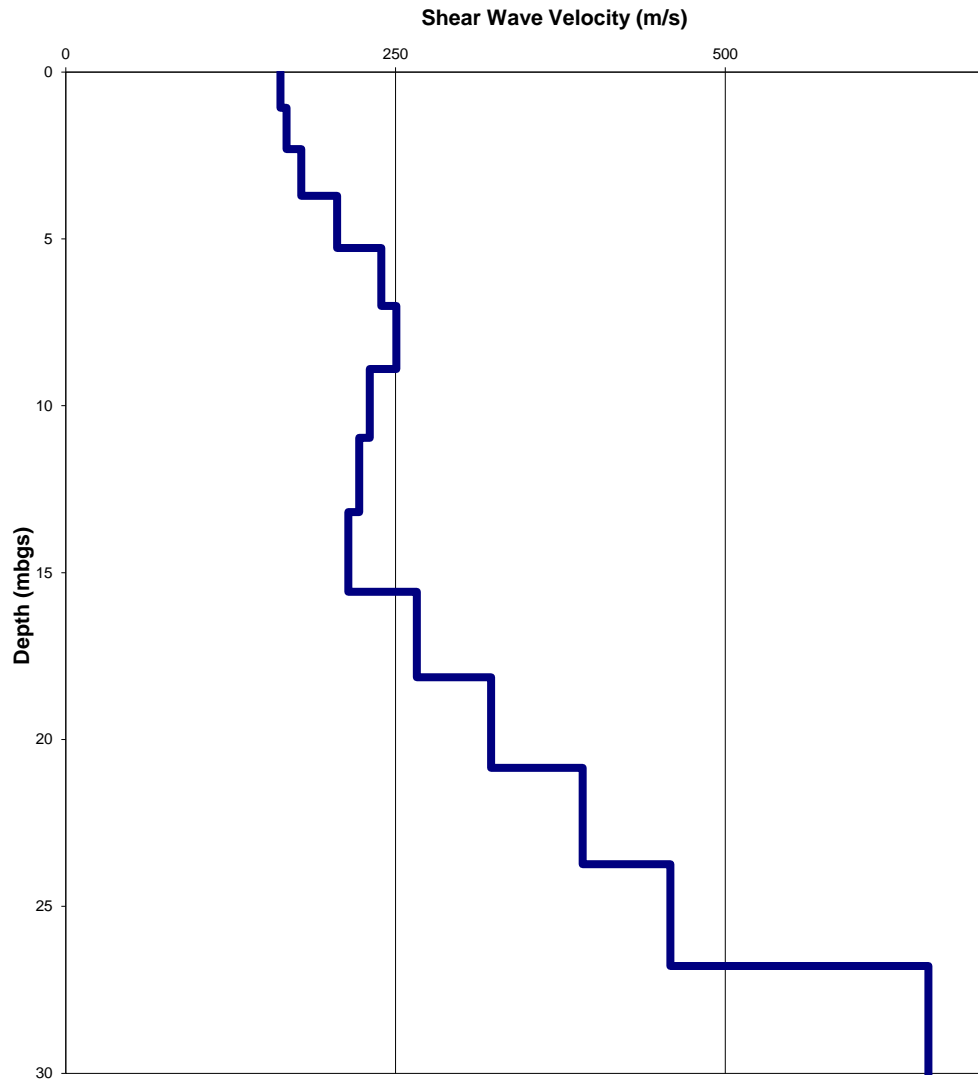


Figure 13: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

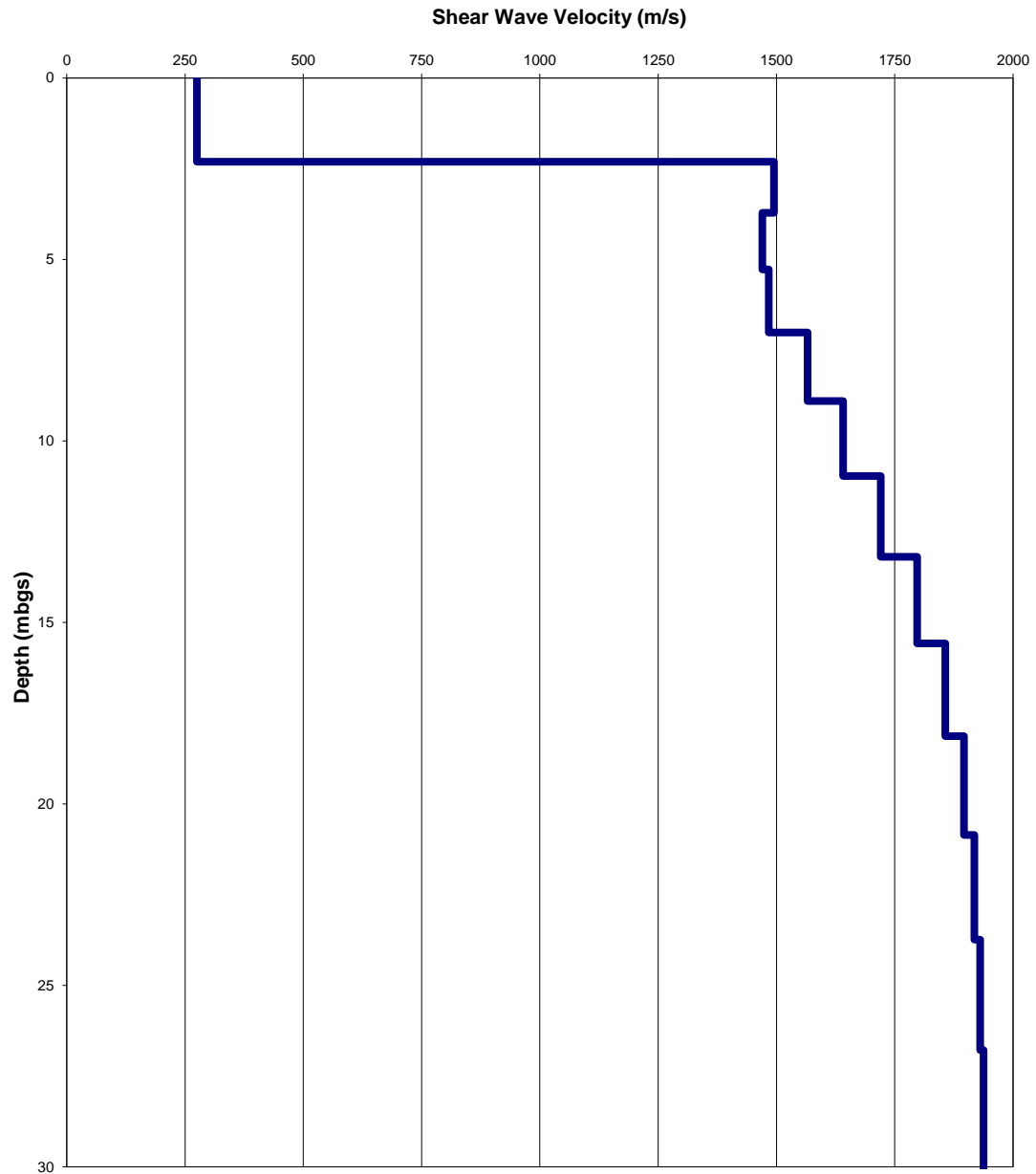


Figure 14: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 2

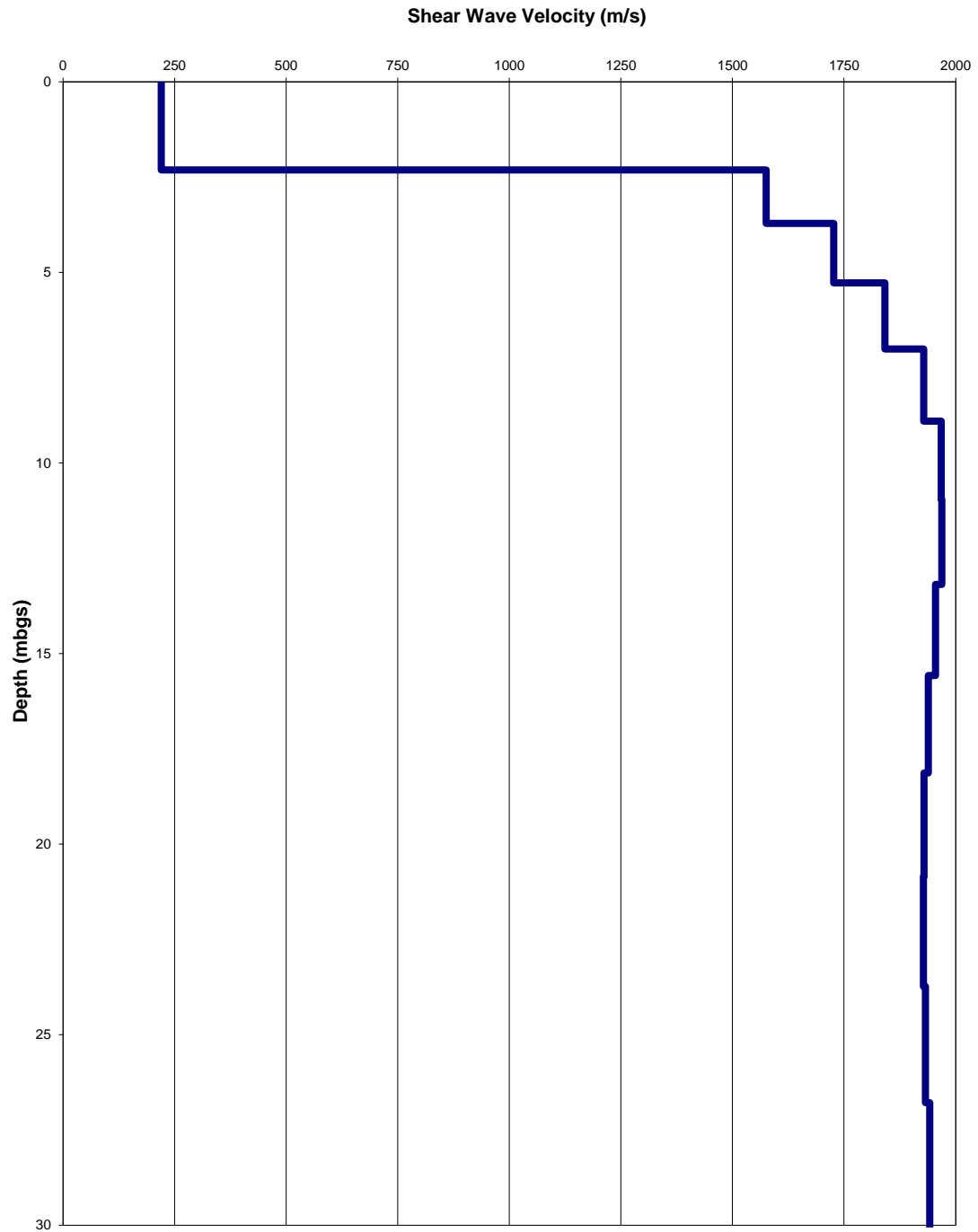


Figure 15: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 3

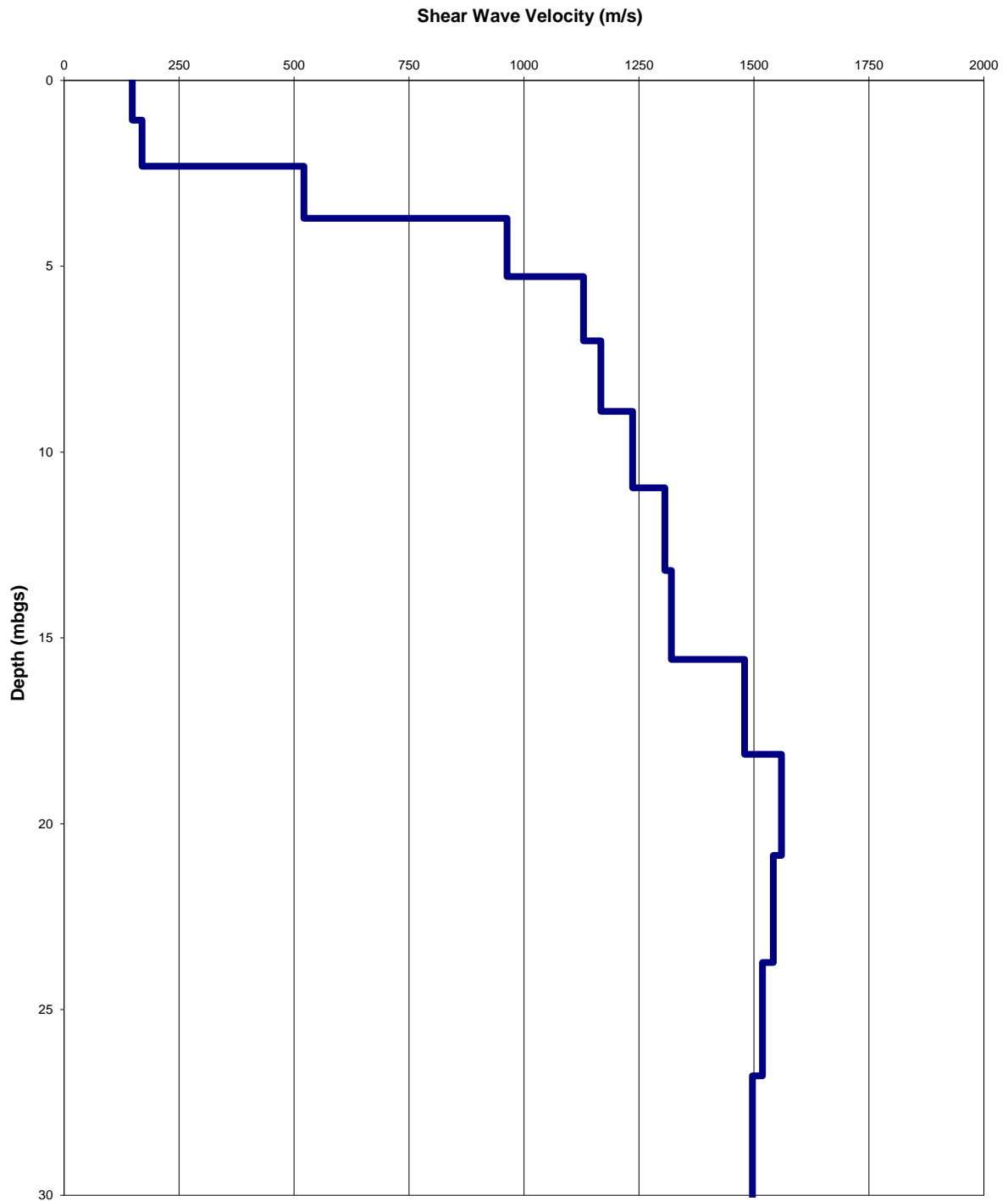


Figure 16: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 4

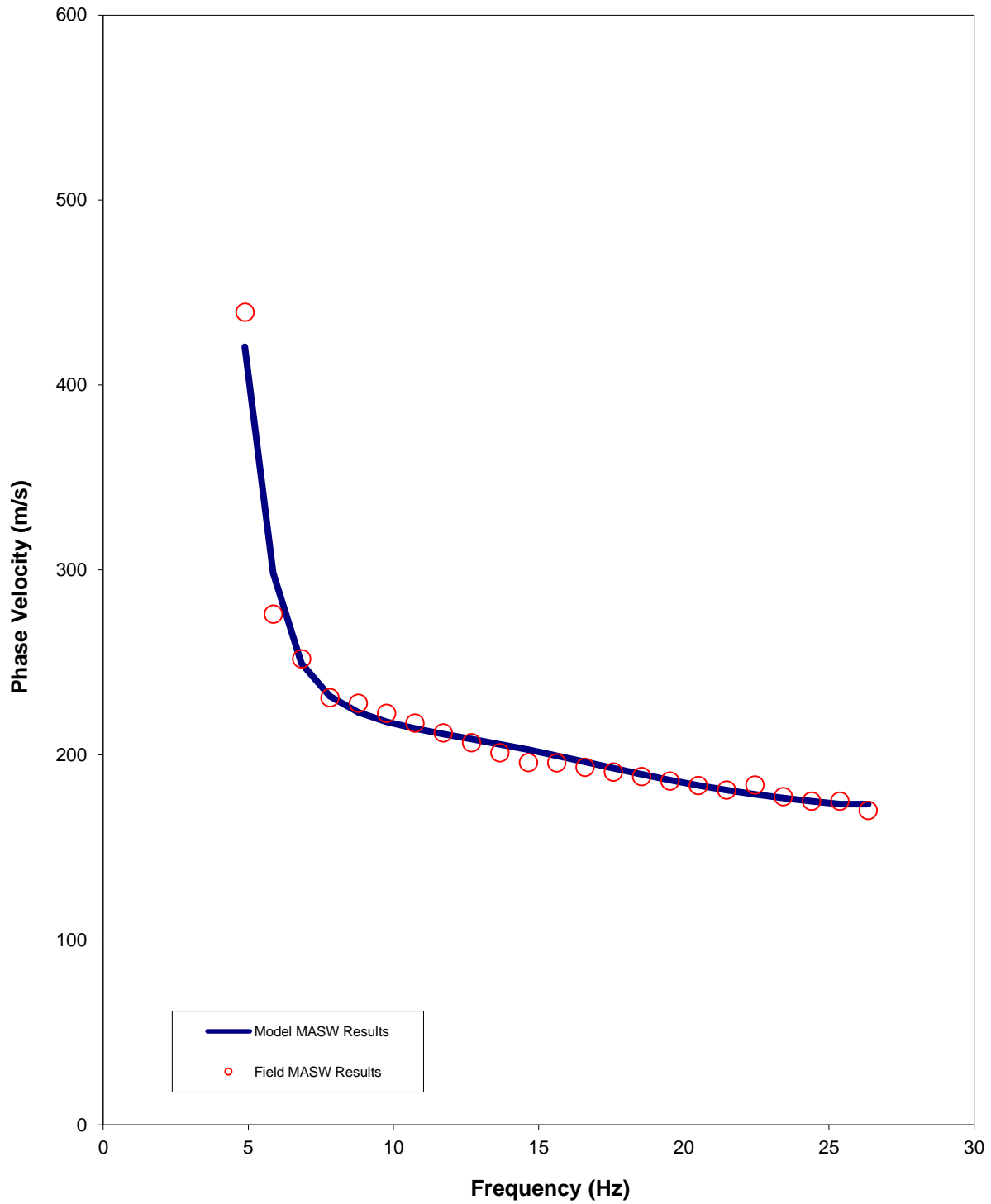


Figure 17: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

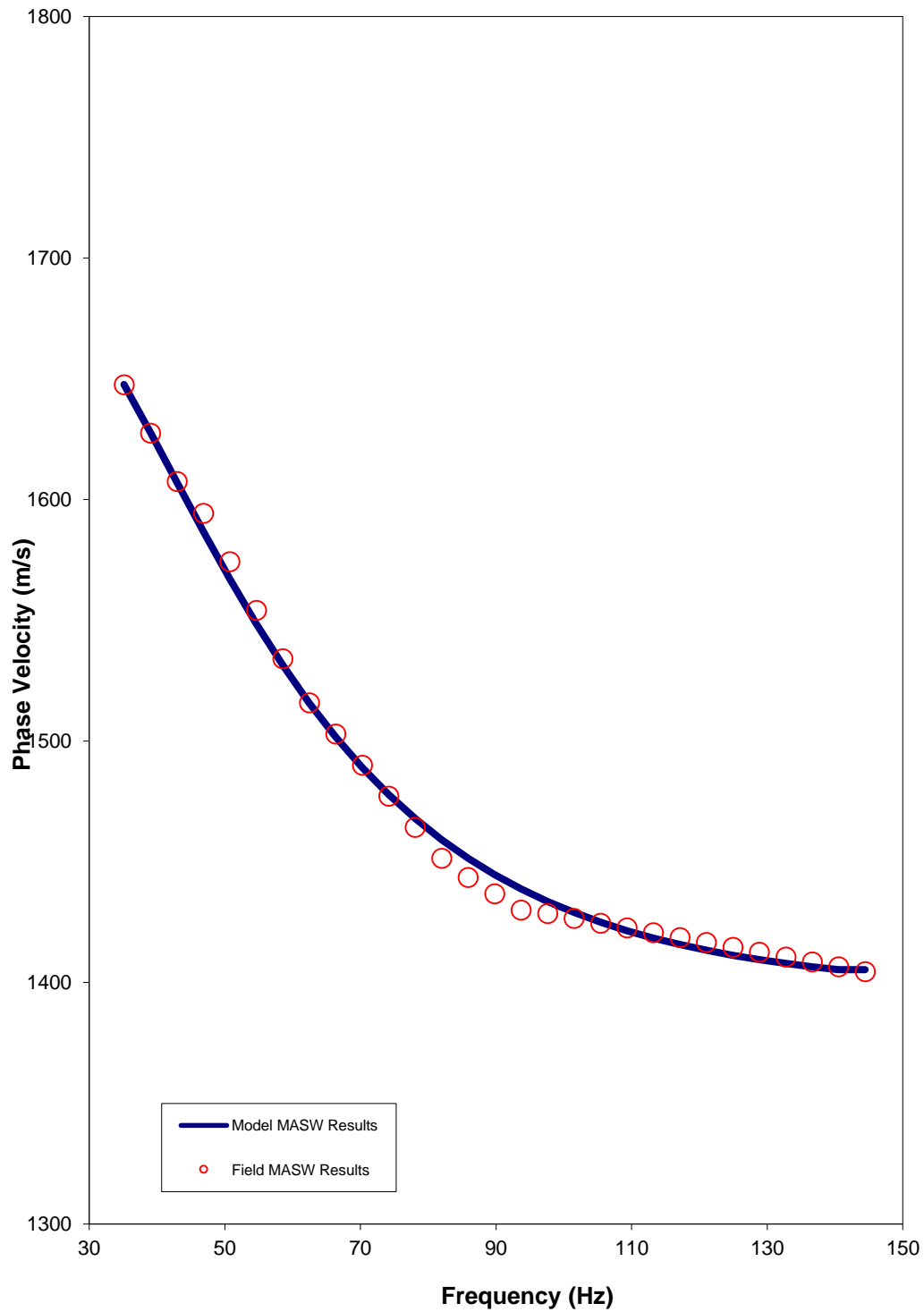


Figure 18: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 2

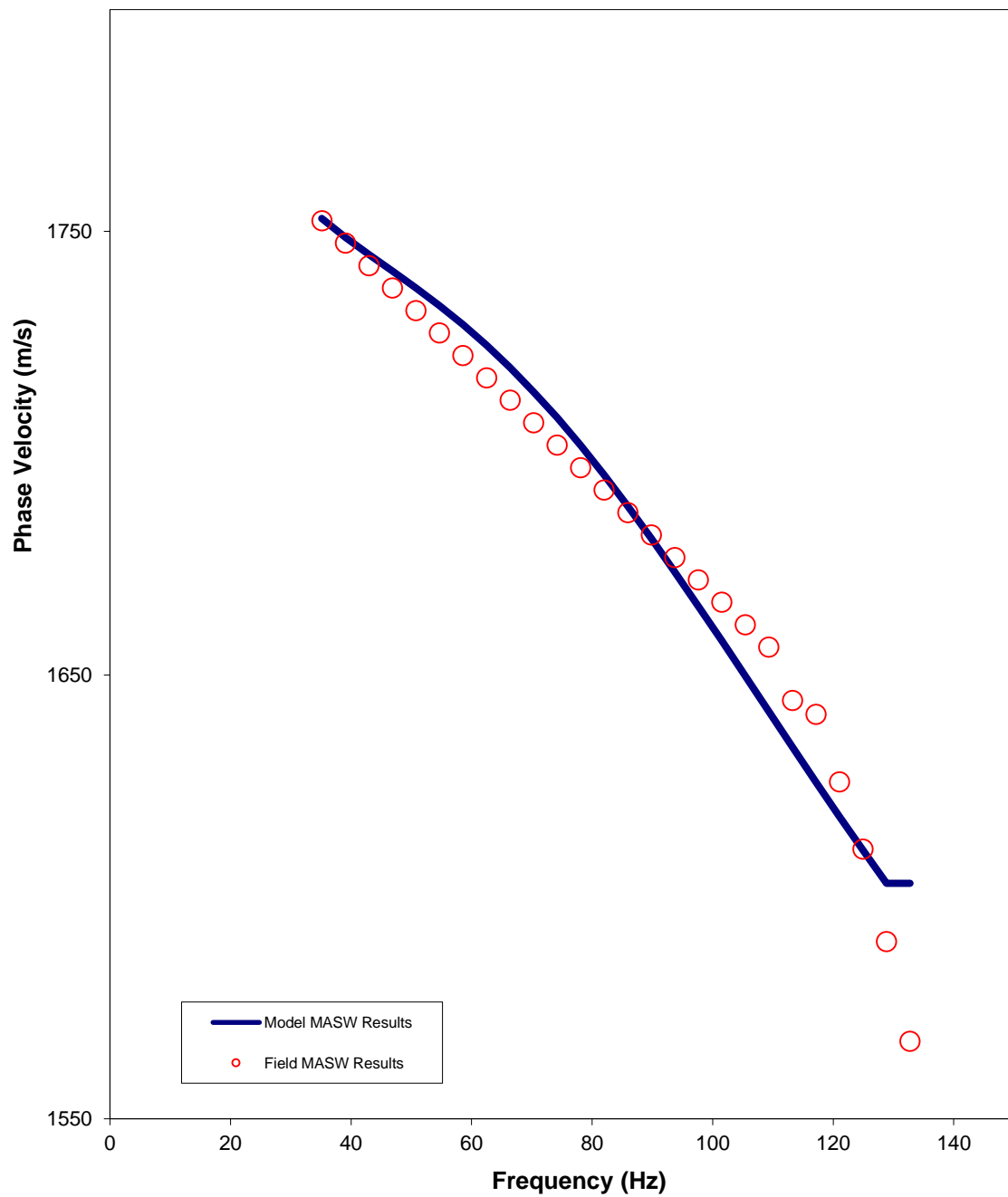


Figure 19: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

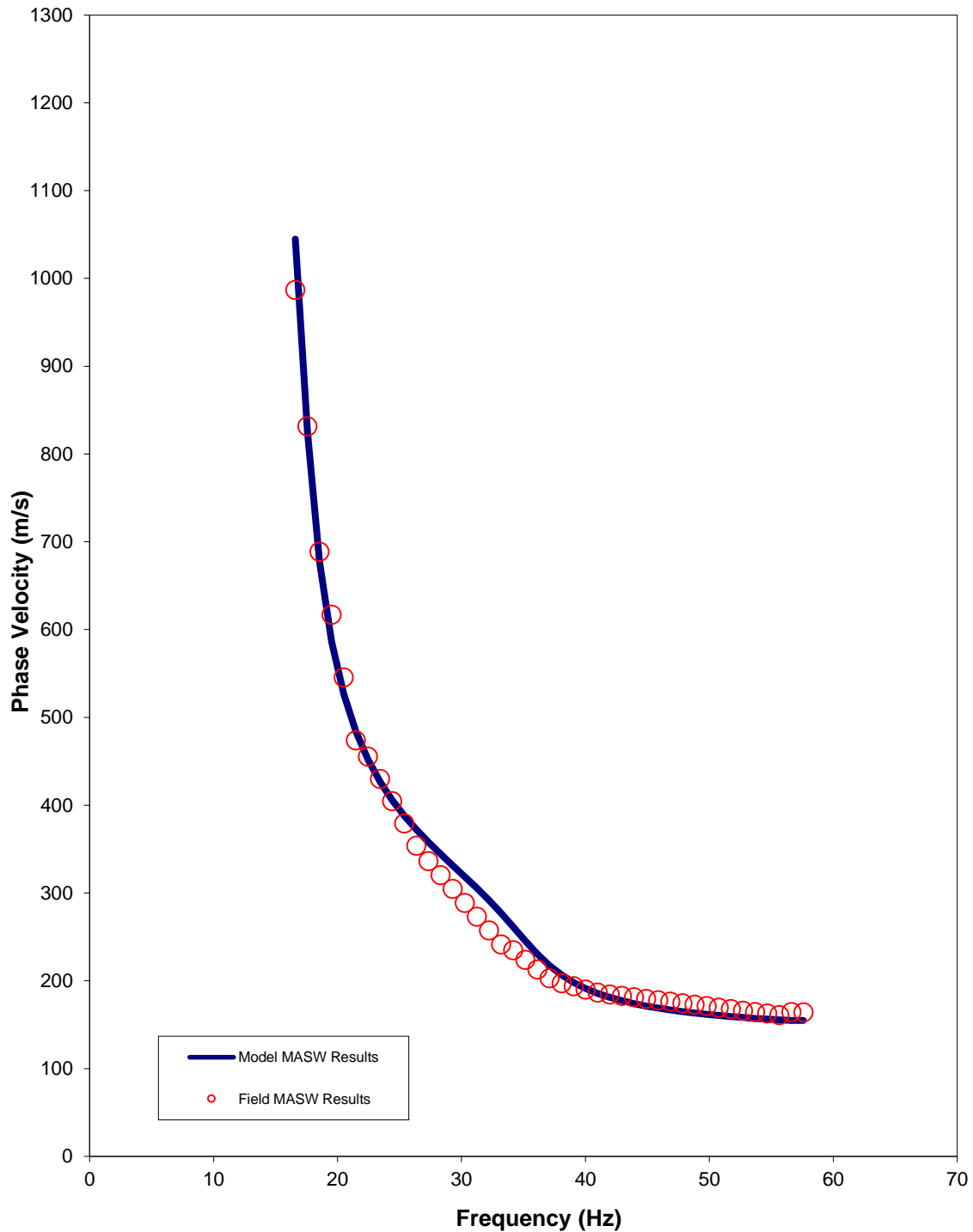


Figure 20: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 3

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.

GOLDER ASSOCIATES LTD.



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Senior Geophysicist

SS/CRP/jl



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

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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



Prepared by:

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www.conetecdataservices.com



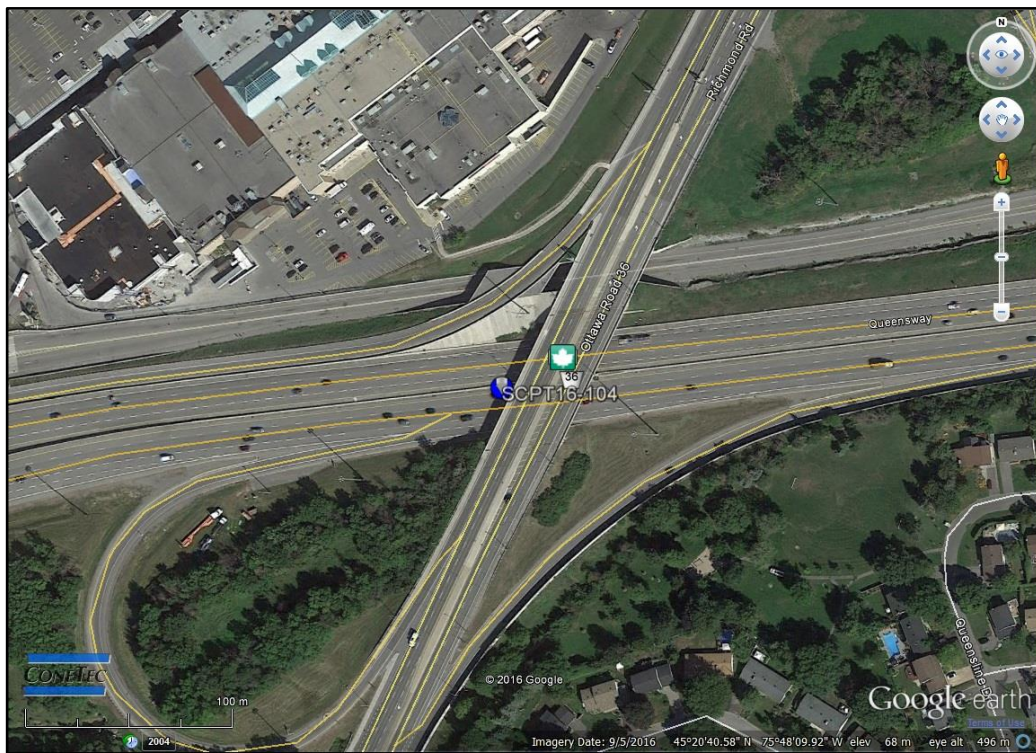
Introduction

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.

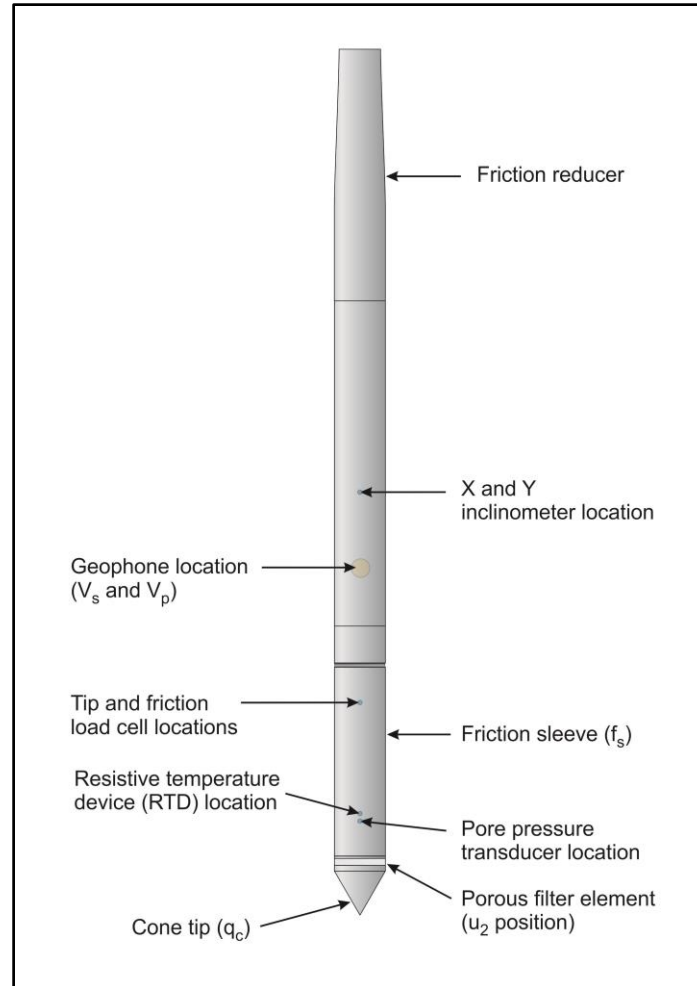


Figure CPTu. Piezocone Penetrometer (15 cm²)

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.

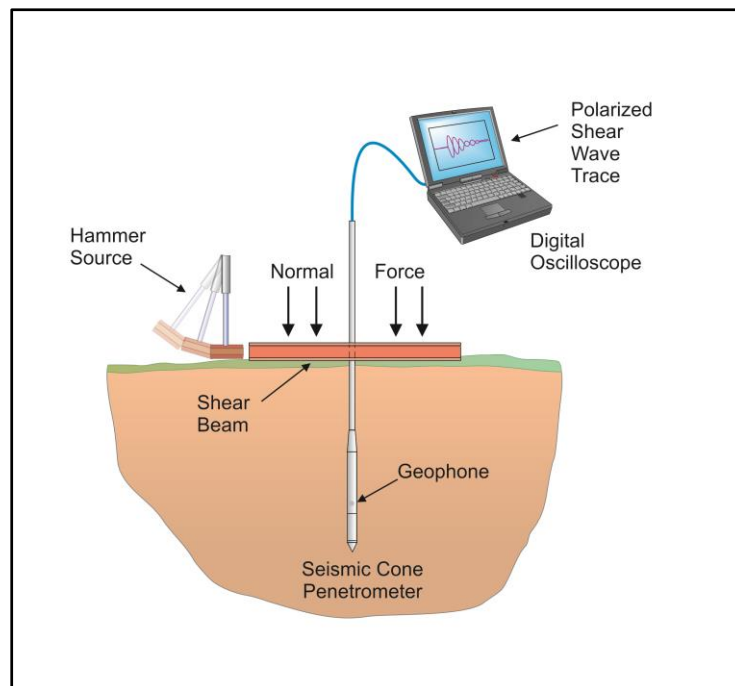


Figure SCPTu-1. Illustration of the SCPTu system

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).

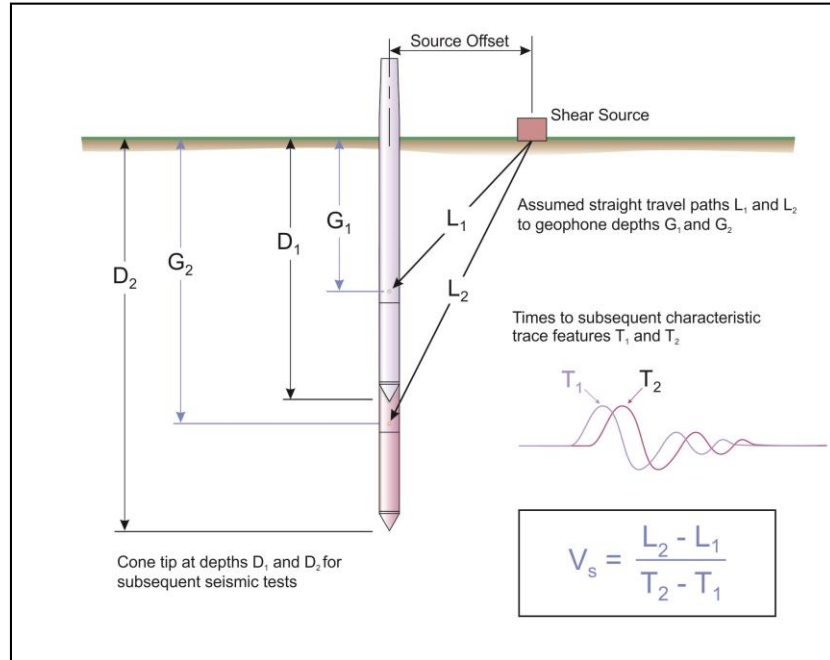


Figure SCPTu-2. Illustration of a seismic cone penetration test

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).

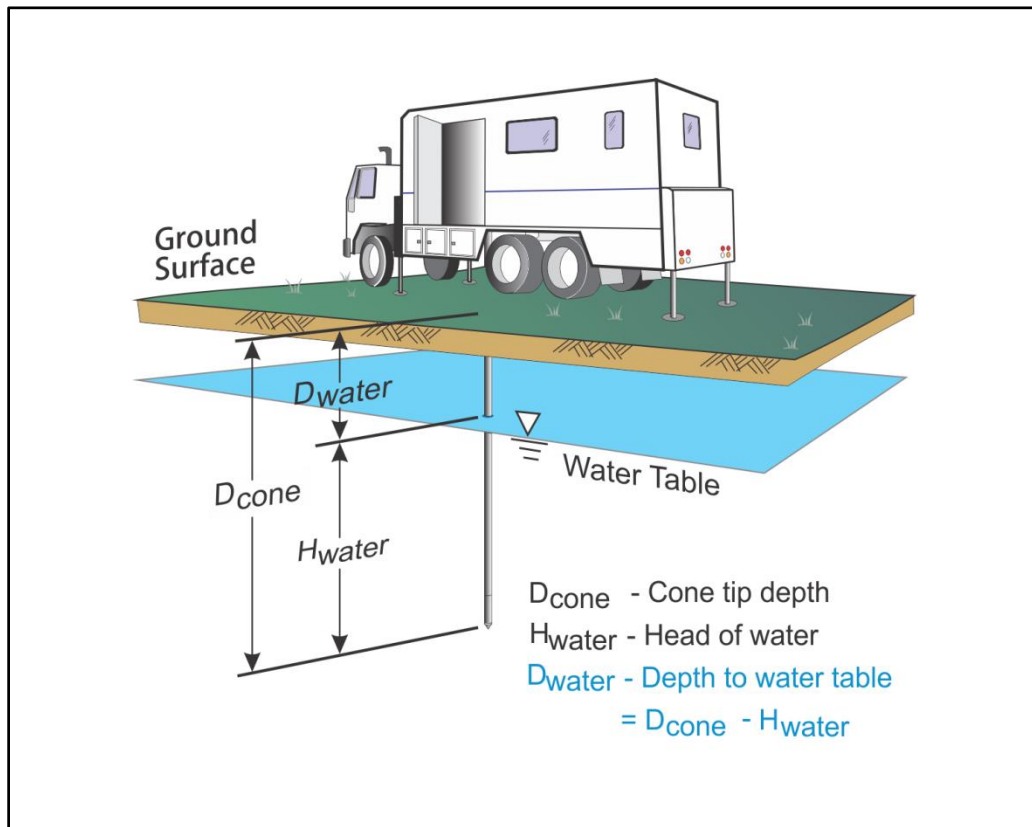


Figure PPD-1. Pore pressure dissipation test setup

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.

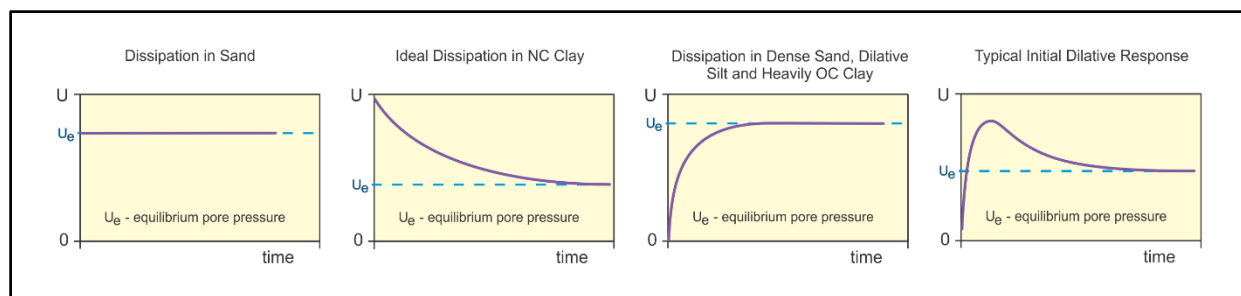


Figure PPD-2. Pore pressure dissipation curve examples

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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Teh, C.I., and Houlsby, G.T., 1991, "An analytical study of the cone penetration test in clay", *Geotechnique*, 41(1): 17-34.

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 , $S_u(N_{kt})$ 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

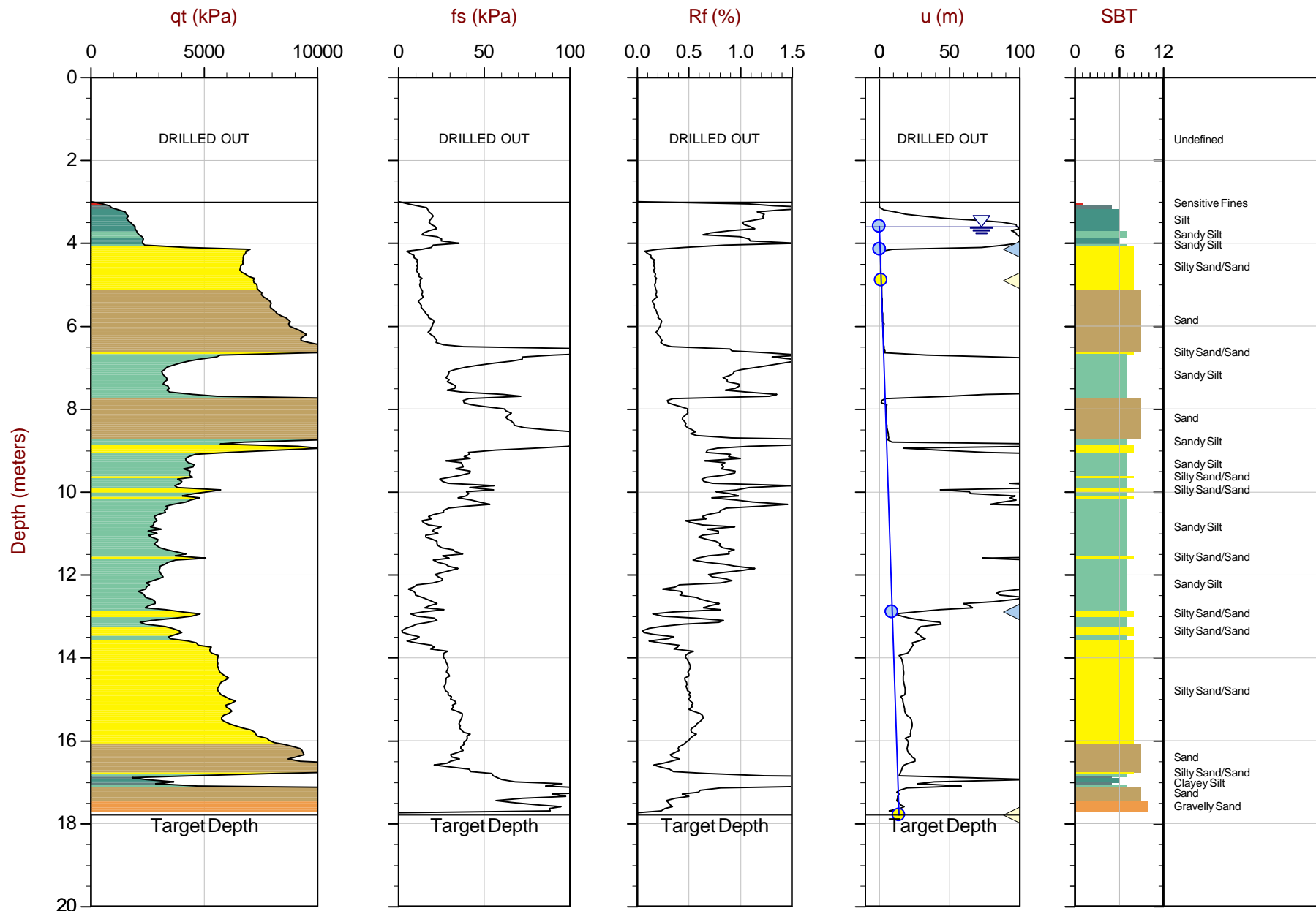
Job No: 16-05039

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 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.

Cone Penetration Test Plot with Expanded Range



Golder

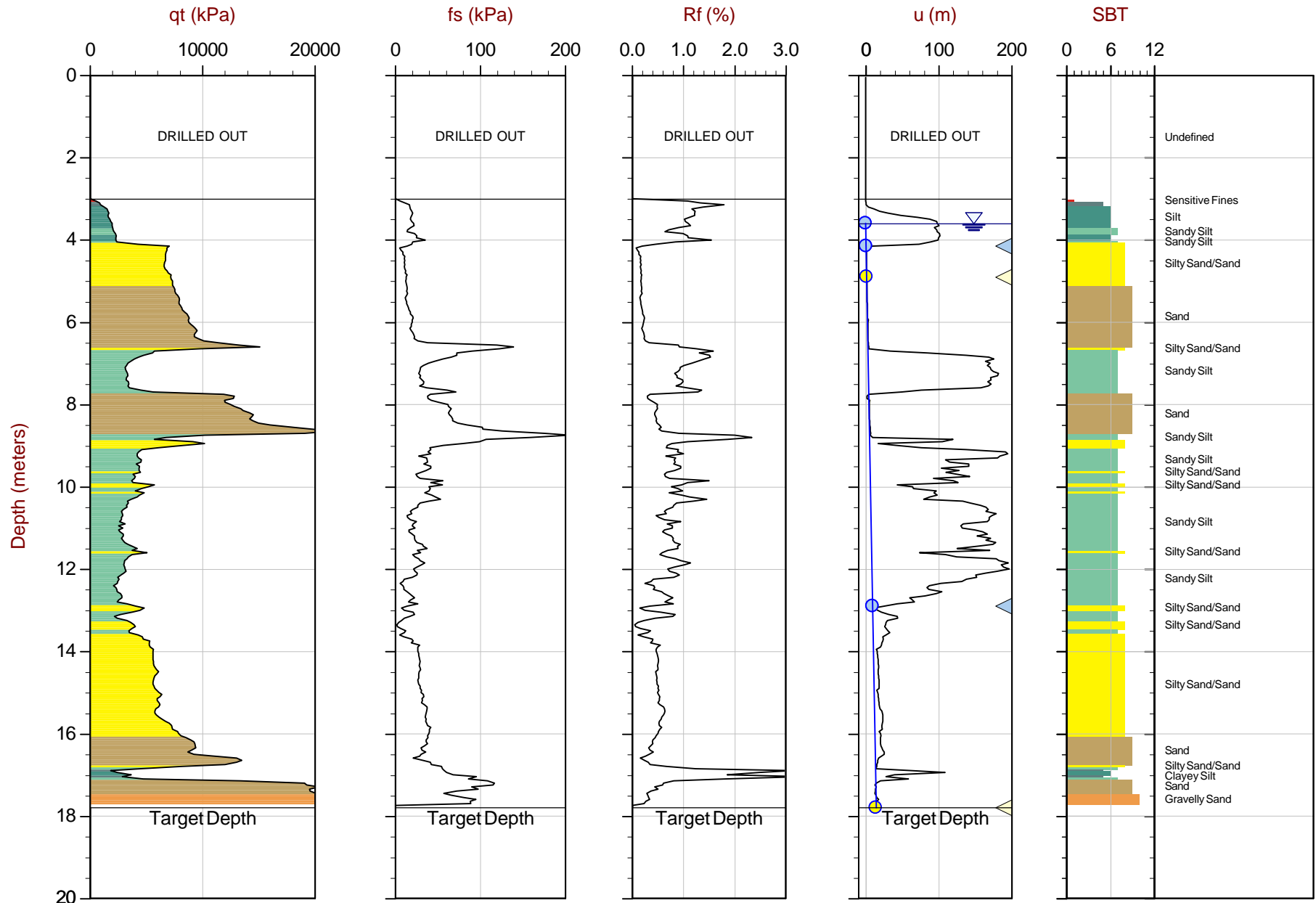
Job No: 16-05039

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: 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.

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



Golder

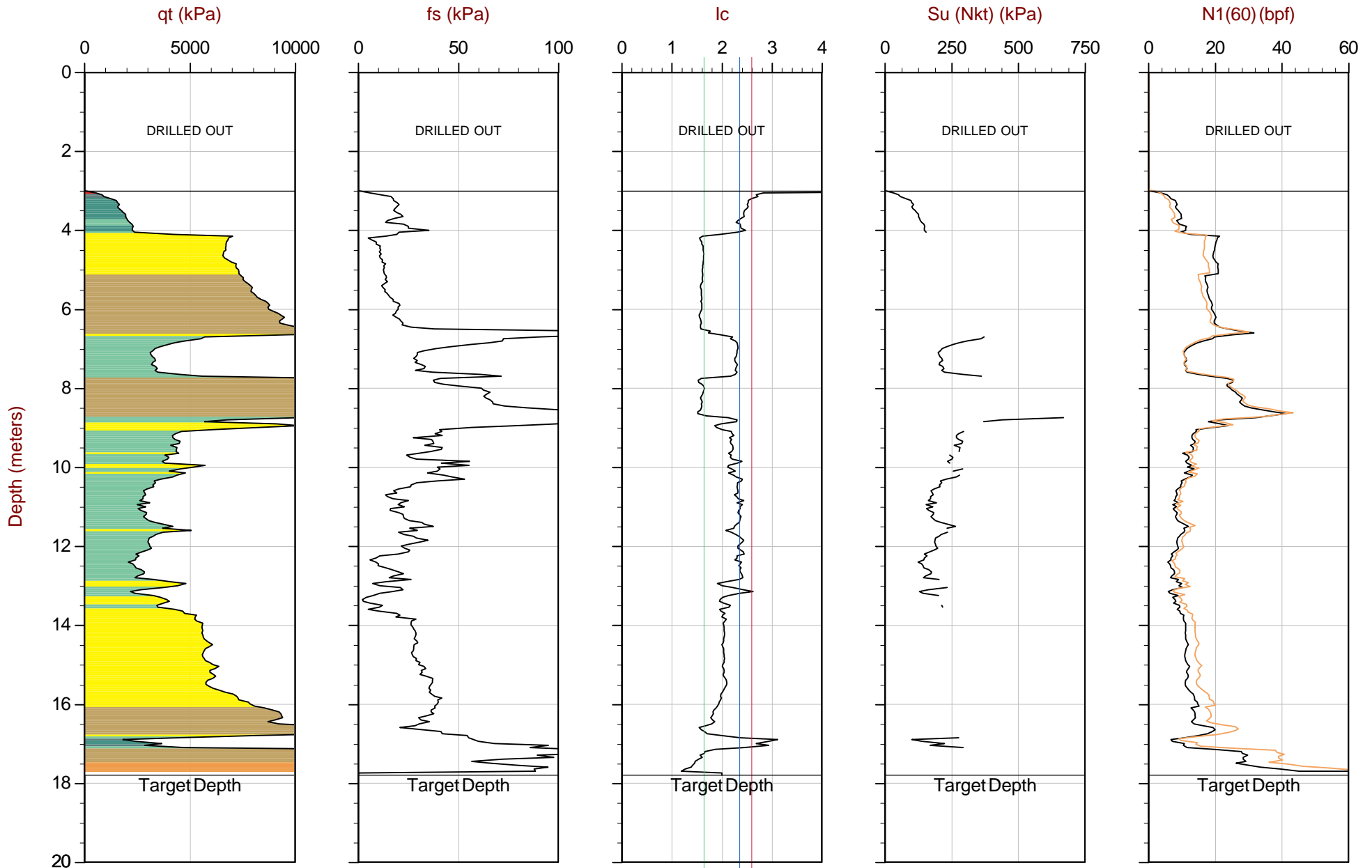
Job No: 16-05039

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

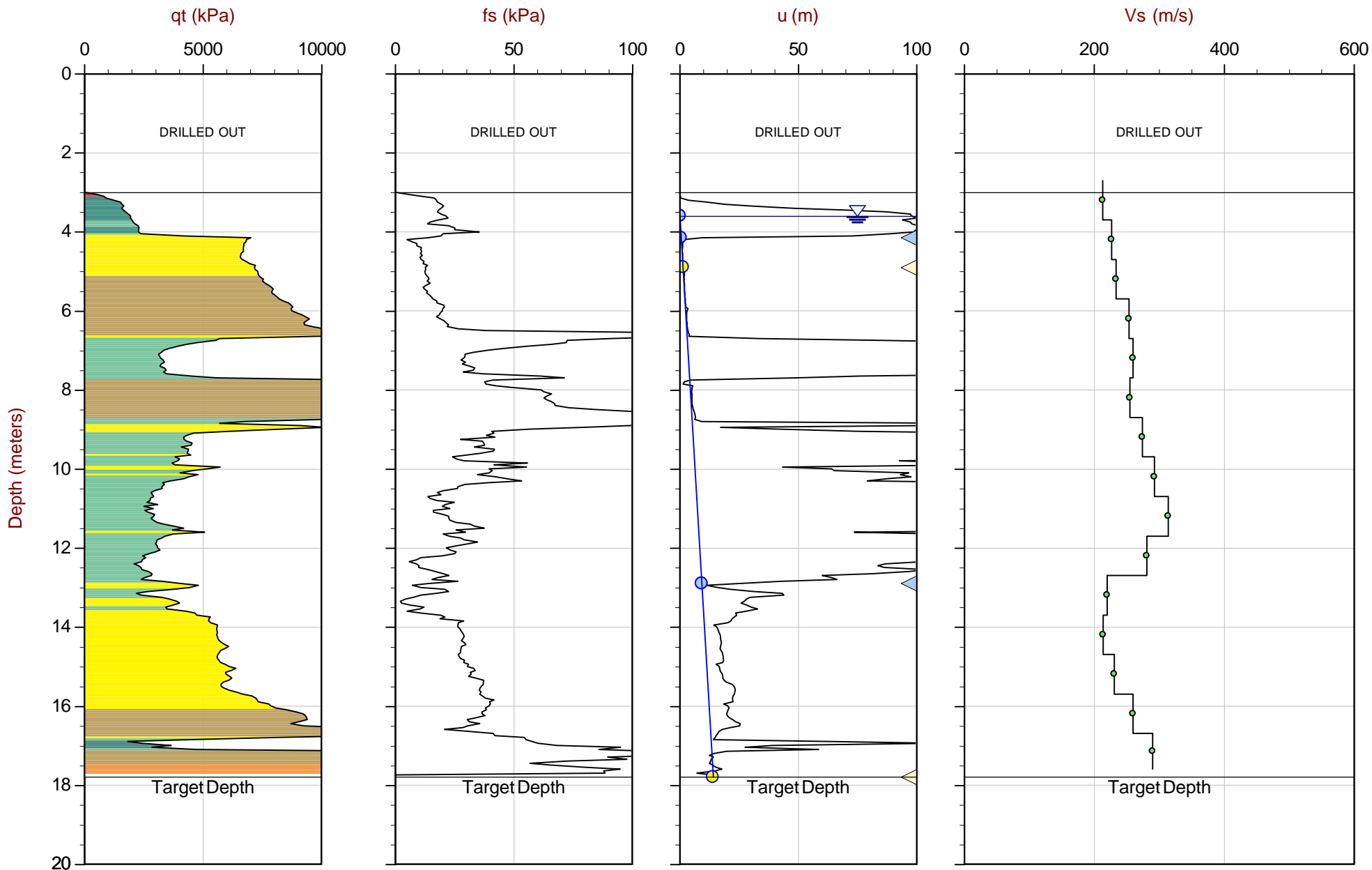
Job No: 16-05039

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

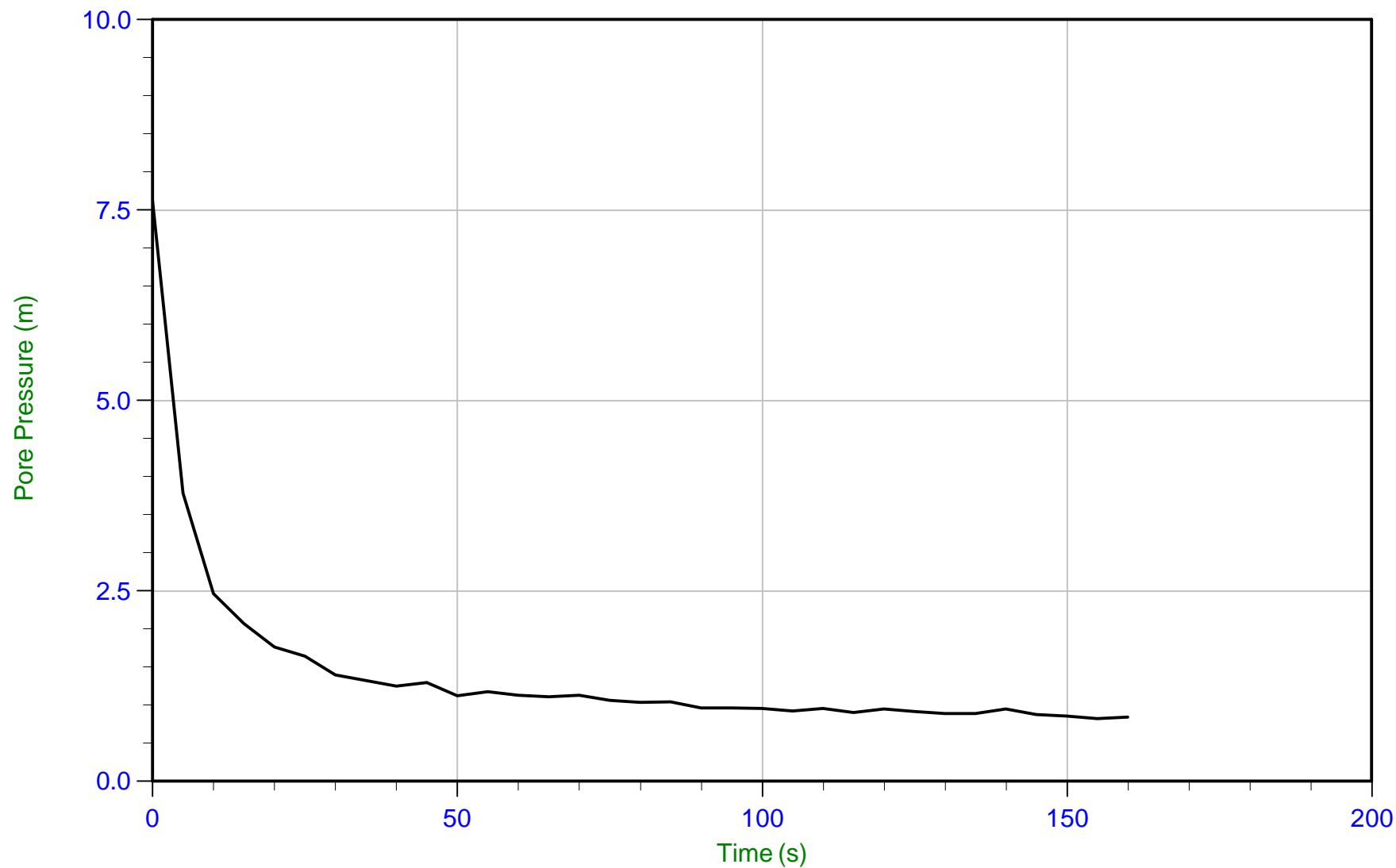
Job No: 16-05039

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

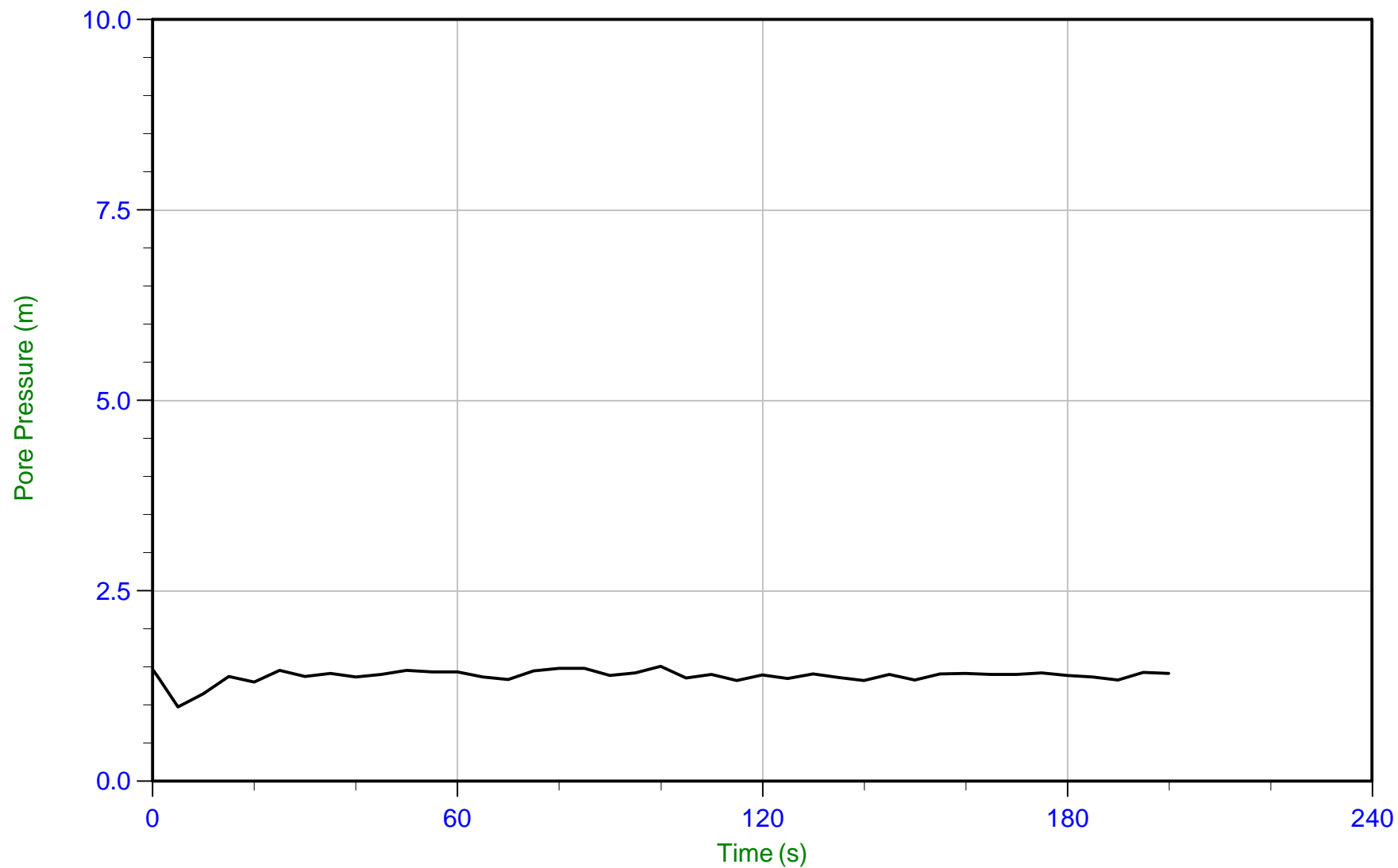
Job No: 16-05039

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
Depth: 4.900 m / 16.076 ft
Duration: 200.0 s

U Min: 1.0 m
U Max: 1.5 m

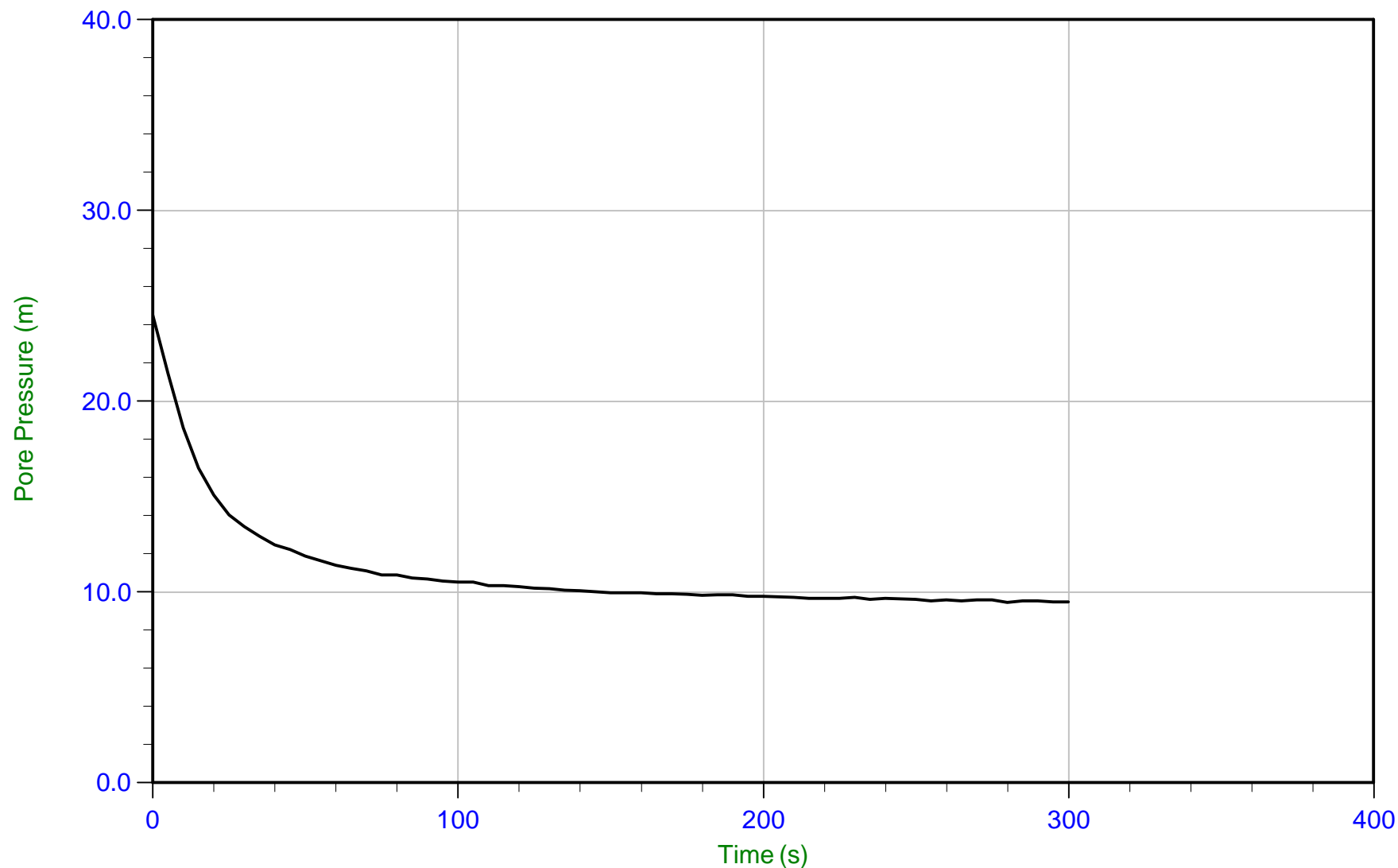
WT: 3.523 m / 11.558 ft
Ueq: 1.4 m



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



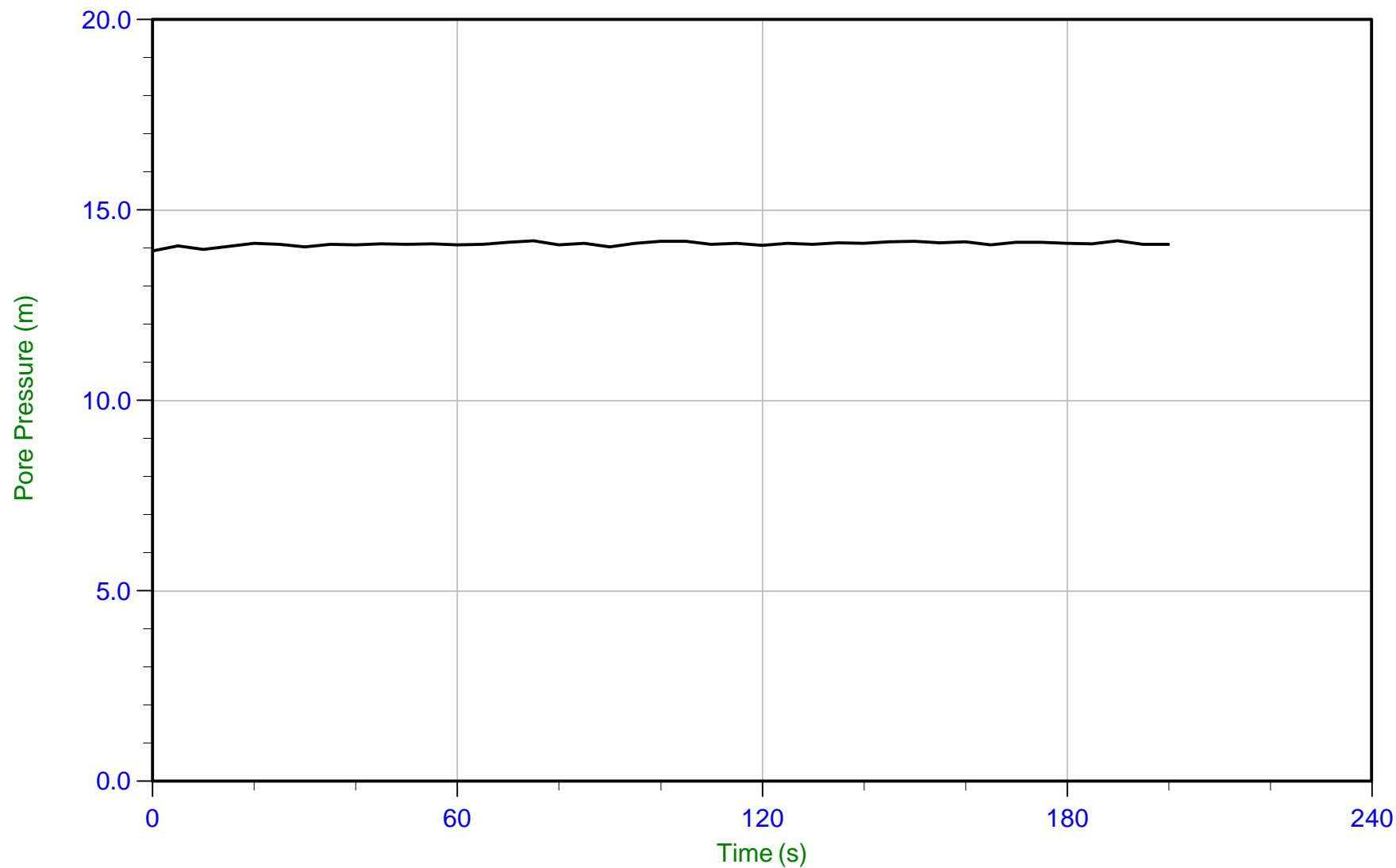
Trace Summary: Filename: 16-05039_SP104.PPF U Min: 9.5 m WT: 3.600 m / 11.811 ft T(50): 14.0 s
Depth: 12.900 m / 42.322 ft U Max: 24.5 m Ueq: 9.3 m Ir: 100
Duration: 300.0 s U(50): 16.92 m Ch: 50.3 sq cm/min



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

As a global, employee-owned organisation with over 50 years of experience, Golder Associates is driven by our purpose to engineer earth's development while preserving earth's integrity. We deliver solutions that help our clients achieve their sustainable development goals by providing a wide range of independent consulting, design and construction services in our specialist areas of earth, environment and energy.

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