



REPORT

Foundation Investigation and Design CNR Overhead Widening Site Nos. 3-301/1 (EBL) and 3-301/2 (WBL) Ottawa, Ontario *G.W.P. 4145-10-00*

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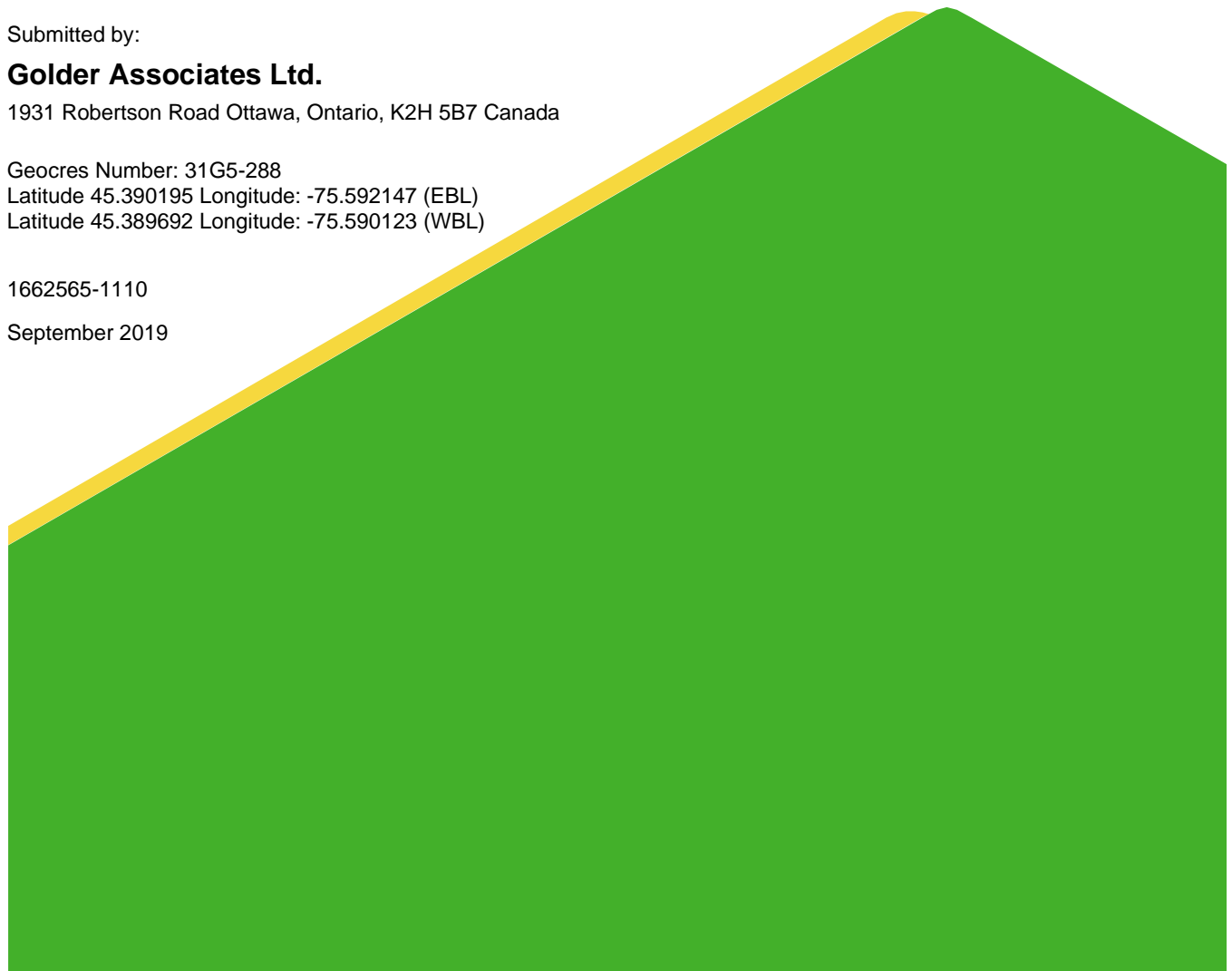
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PART A

Foundation Investigation
Proposed CNR Overhead Widening
Site Nos. 3-301/1 and 3-301/2
Highway 417
Ottawa, Ontario
G.W.P. 4145-10-00

1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by WSP Canada Group Limited (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to carry out foundation investigations associated with numerous bridge and structural culvert rehabilitations and/or replacements on Highway 417 between the Aviation Parkway and Ramsayville Road, as well as the widening of Highway 417 from Ottawa Road 174 to Hunt Club Road in Ottawa, Ontario (Assignment number 4016-E-0008).

This report presents the results of the foundation investigation carried out to collect subsurface information required for the widening, seismic retrofit and temporary protection systems for the CNR Overhead structures, Site Nos. 3-301/1 and 3-301/2 located on the Highway 417 East Bound Lanes (EBL) and West Bound Lanes (WBL), respectively, in Ottawa, Ontario (G.W.P. 4145-10-00 and W.P. 266-00-01/266-00-02). The widening of the structures are to be carried out in accordance with the current version of the Canadian Highway Bridge Design Code (CHBDC, S6-14).

The terms of reference and scope of work for the foundation investigation are outlined in the MTO's Request for Proposal (RFP), dated May 2016, and subsequent addenda. Golder's scope of work for foundation engineering services associated with the Highway 417 CNR Overhead structures is contained in Table 17.8.3 of WSP's Technical Proposal for this assignment as well as in Change Request No. 2 dated April 6, 2018. The work has been carried out in accordance with Golder's Quality Control Plan for foundation engineering services for this project, dated March 13, 2017.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 General

The CNR Overhead bridges (Sites 3-301/1 and 3-301/2) are located approximately 800 m north of the Hunt Club Road interchange in Ottawa, Ontario. The existing EBL structure is located at about Station 18+069 and the WBL structure is located at about Station 18+274. At these locations, Highway 417 is a divided highway with two travel lanes in each direction and an on-ramp in the EB direction. The highway is separated by a grassed median area with an approximate width of 145 m. The CN rail line runs east-west beneath the EBL and WBL structures.

The existing bridges were constructed in 1975 and are five-span reinforced concrete slab on pre-stressed concrete girder structures. The overall EBL structure is about 72.8 m long, with 11.2 m, 16.8 m, 16.8 m, 16.8 m, and 11.2 m spans and varies in width from about 14.3 to 17.9 m. The overall WBL structure is about 70.8 m long, with 10.2 m, 16.8 m, 16.8 m, 16.8 m, and 10.2 m spans and is about 13.0 m wide. The bridge abutments are supported on "perched" foundations on battered steel HP12 x 74 (HP310 x 110) piles end bearing on bedrock. The pier foundations are supported on vertical and battered HP12 x 74 (HP310 x 110) steel piles end bearing on bedrock.

The existing pavement grade of Highway 417 is at about Elevation 76 m; the approach embankments are approximately 10 to 11 m in height. The existing embankment side slopes were constructed at about 2 horizontal to 1 vertical (2H:1V) with approximately 3 m wide mid-slope bench. Based on visual observation at the time of the site investigation, the existing embankment slopes appear to be performing satisfactorily. Either 914 mm or 1,829 mm diameter corrugated steel pipe (CSP) culverts exist below the lower portion of embankment foreslopes between Piers 1 and 2 and Piers 3 and 4 at both the EBL and WBL structures.

Selected site photographs taken by WSP personnel showing the existing structure and surrounding area are included in Appendix D.

2.2 Regional Geology

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

The Ottawa Valley Clay Plain region is characterized by relatively thick deposits of sensitive marine clay, silt and silty clay that were deposited within the Champlain Sea basin. These deposits, known as the Champlain Sea clay or Leda clay, overlie relatively thin, commonly reworked glacial till and glaciofluvial deposits, that in turn overlie bedrock². The Russell and Prescott Sand Plains are generally characterized by a sand mantle about 3 to 5 m thick overlying an extensive deposit of sensitive marine clay deposited within the Champlain Sea basin, underlain by glacial till and bedrock.

This region is underlain by a series of sedimentary rocks, consisting of sandstones, dolostones, limestones and shales that are, in turn, underlain at depth by igneous and metamorphic bedrock of the Precambrian Shield. Regional bedrock mapping indicates that the bedrock at this site is primarily limestone and shale of the Carlsbad Formation.³

The site falls within the Western Québec (WQ) seismic zone according to the Geological Survey of Canada⁴. The WQ zone constitutes a large area which encompasses the urban areas of Montreal, Ottawa-Hull and Cornwall. Within the WQ zone recent seismic activity has been concentrated in two subzones; one along the Ottawa River and another more active subzone along the Montreal-Maniwaki axis. The two major earthquakes in the WQ zone includes the 1935 Témiscaming event which had a magnitude (i.e., a measure of the intensity of the earthquake) of 6.2, and the 1944 Cornwall-Massena event which had a magnitude of 5.6.

The topography in the area of the bridge structures ranges from about Elevation 76 m at the EBL and WBL of Highway 417, sloping down towards the CN rail line at about Elevation 67 m. The areas to the southwest of the EBL bridge, and to the northeast of the WBL bridge are forested, while the other areas are sparsely vegetated.

3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation (2017)

The subsurface investigation for the bridge widenings was carried out between June 25 and July 7, 2017 and between August 20 and September 8, 2017. During that time, 14 boreholes (17-1101 to 17-1104, 17-1106 to 17-1111, and 17-1113 to 17-1116, inclusive) were advanced at the locations shown on Drawings 1 and 2.

The boreholes were advanced as follows:

- Boreholes 17-1101, 17-1102, 17-1107, and 17-1108 were advanced through the north and south approach embankments of the EBL of Highway 417 using 108 mm inside diameter (200 mm outside diameter) continuous flight hollow stem augers on truck mounted drill rigs, supplied and operated by Forage Grenville Drilling of Grenville, Québec. Traffic control required to access the borehole locations was provided by

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

² Belanger, J.R. "Urban Geology of Canada's National Capital Area", in *Urban Geology of Canadian Cities*, Geological Association of Canada Special Paper 42, Ed. P.F. Karrow and O.L. White, 1998.

³ MacDonald, G. and Harrison, J.E. 1976 : Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Geological Survey of Canada, Map 1508A, scale 1:125,000. Geology 1967.

⁴ Natural Resources Canada (2016, February 10). Earthquake Zones in Eastern Canada, Retrieved from <http://www.seismescanada.nrcan.gc.ca/zones/eastcan-en.php#WQSZ>

Beacon Lite Ltd. of Ottawa, Ontario. The boreholes were advanced through the overburden to practical refusal to auger or casing advancement at depths between about 16.2 and 20.6 m below the existing pavement grade. Boreholes 17-1102 and 17-1107 were then cored for about 3.7 and 3.0 m, respectively, into the bedrock using NQ-sized coring equipment. A water truck was on site to supply the drill rigs with water for the coring of the bedrock.

- Boreholes 17-1109, 17-1110, 17-1115, and 17-1116 were advanced through the north and south approach embankments of the WBL of Highway 417 also using 108 mm inside diameter (200 mm outside diameter) continuous flight hollow stem augers on truck mounted drill rigs, supplied and operated by Forage Grenville Drilling of Grenville, Québec. Traffic control required to access the borehole locations was provided by Beacon Lite Ltd. of Ottawa, Ontario. The boreholes were advanced through the overburden to practical refusal to auger advancement at depths between about 16.0 and 18.2 m below the existing pavement grade. Boreholes 17-1110 and 17-1115 were then cored for about 5.7 and 3.6 m, respectively, into the bedrock using NQ or HQ-sized coring equipment. A water truck was on site to supply the drill rigs with water for the coring of the bedrock.
- Boreholes 17-1106 and 17-1111 were advanced within the median of Highway 417 adjacent to the most southern piers of the EBL and WBL structures, in the area of the proposed widening, using 108 mm inside diameter (200 mm outside diameter) continuous flight hollow stem augers on a track mounted drill rig, supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. Railway track protection required to access the borehole locations was provided by VIA Rail Canada Inc. An additional borehole (Borehole 17-1106A) was drilled adjacent to Borehole 17-1106 to retrieve a relatively undisturbed 73 millimetre diameter thin-walled Shelby tube samples of the clay using a fixed piston sampler. The boreholes were advanced through the overburden to practical refusal to auger or casing advancement at depths between about 11.2 and 12.2 m below the existing ground surface. The boreholes were then cored for about 1.5 to 4.1 m into the bedrock using NQ-sized coring equipment. A water truck was on site to supply the drill rigs with water for the coring of the bedrock.
- Boreholes 17-1103, 17-1104, 17-1113, and 17-1114 were advanced within the median of Highway 417 adjacent to the northern piers of the EBL and WBL structures, in the area of the proposed widening, using portable drilling equipment, supplied and operated by CCC Geotechnical and Environmental Drilling Ltd. of Ottawa, Ontario. Railway track protection required to access the borehole locations was provided by VIA Rail Canada Inc. The boreholes were advanced through the overburden to depths between about 7.8 and 13.6 m below the existing ground surface. The boreholes were then cored for about 3.0 to 3.5 m into the bedrock using BQ-sized coring equipment. A water truck was on site to supply the drill rigs with water for the coring of the bedrock.
- Boreholes 17-1105 and 17-1112 which were to be located within the median, adjacent to the piers south of the CNR rail line, could not be advanced due to restrictions from CN with regards to drilling in their right-of-way (i.e., no drilling was permitted south of the rail line due to buried utilities).

Samples of the overburden were obtained at vertical intervals of about 0.6 to 1.75 m, using a 50 mm outside diameter split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. In-situ vane testing was carried out within the cohesive deposits, where possible, using either an MTO “N”-sized vane or a “B”-sized vane. Rotary diamond drilling (DD) techniques were also required to advance through cobbles and boulders at one location.

Monitoring wells were installed in Boreholes 17-1108 and 17-1115 to monitor the groundwater level at the site. The monitoring wells consist of 32 mm diameter rigid PVC pipes with 3.0 m long slotted screen sections, installed within silica sand backfill and sealed by a sections of bentonite pellet backfill. The groundwater levels in the monitoring wells were measured between September 24 and 25, 2017. The monitoring wells in Boreholes 17-1108 and 17-1115 were decommissioned between September 24 and 25, 2017. The monitoring wells were decommissioned by backfilling the monitoring well with bentonite, removing the top section of the monitoring well, and the asphalt patched upon completion.

A 63.5 mm inside diameter rigid PVC casing was grouted for the full advancement depth (i.e. through the overburden and into the bedrock) at Borehole 17-1110 to allow for Vertical Seismic Profile testing.

The boreholes were backfilled with bentonite pellets, mixed with native soils in the overburden and bentonite pellets in the bedrock, except as indicated previously for the monitoring wells. The site conditions were reinstated following completion of work.

The field work was supervised by members of Golder's technical staff, who located the boreholes, supervised the drilling, sampling, and in situ testing operations, logged the boreholes, and examined and cared for the soil and bedrock samples. The soil and bedrock samples were identified in the field, placed in appropriate containers, labelled, and transported to Golder's laboratory in Ottawa for further examination. Index and classification tests consisting of water content determinations, Atterberg Limit tests, and grain size distribution analyses were carried out on selected soil samples. In addition, consolidation tests were performed on selected Shelby tube samples from Boreholes 17-1106A and 17-1116. Unconfined compressive strength tests were also carried out on selected rock core samples. The laboratory tests were carried out to MTO and/or ASTM standards, as appropriate.

Soil samples from Boreholes 17-1102, 17-1103, 17-1104, 17-1106, 17-1107, 17-1110, 17-1111, 17-1113, 17-1114, and 17-1115 were submitted to Eurofins Environment Testing for chemical analyses related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The results of the chemical analysis are presented in Appendix E.

In addition to the borehole investigation, shear wave velocity profiling was carried out at two locations on the site on May 25, 2017, using the Multichannel Analysis of Surface Waves (MASW) method. The MASW lines were located within the grassed median of Highway 417, north and south of the CNR rail line, as shown in the technical memorandum provided in Appendix F. Shear wave velocity profiling using the Vertical Seismic Profile (VSP) method was also carried out at the site on July 27, 2017, within Borehole 17-1110. The MASW and VSP testing was carried out by personnel from Golder Associates' Mississauga and Ottawa offices. For each MASW line a series of 24 low frequency (4.5 Hz) geophones were laid out at approximately 3 m intervals and a 9.9 kg sledge hammer and 45 kg weight drop were used as the seismic sources. The source locations were offset at distances of 5, 10, and 15 m off the end and collinear with the geophone array. For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in the borehole at a known depth. The active seismic source can be either a compression or shear wave. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the borehole. Traffic control required to carry out the MASW and VSP testing was provided by Beacon Lite Ltd. of Ottawa, Ontario.

The MASW and VSP test results and report are presented in Appendix F 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 borehole elevation were surveyed by Golder using a Trimble R8 GPS unit. The borehole locations, including MTM NAD83 Zone 9 northing and easting coordinates, ground surface elevations referenced to geodetic datum, and drilled depths are summarized in the following table and are shown on Drawings 1 and 2. Northing and easting grid coordinates and latitude and longitude geographic coordinates are also indicated on the Record of Borehole and Drillhole sheets.

Borehole Number	Borehole Location	MTM NAD83 Northing (m)	MTM NAD83 Easting (m)	Ground Surface Elevation (m)	Borehole Depth (m)
17-1101	EBL North Approach Embankment	5028263.4	375871.3	75.8	20.6
17-1102	EBL North Abutment	5028251.0	375875.2	75.9	23.8
17-1103	EBL North Pier 1	5028229.6	375888.5	70.5	17.1
17-1104	EBL North Pier 2	5028213.6	375894.5	66.4	11.9
17-1106	EBL South Pier 4	5028182.6	375906.8	70.3	12.8
17-1106A	EBL South Pier 4	5028182.6	375906.8	70.3	13.2
17-1107	EBL South Abutment	5028160.3	375911.2	75.8	19.2
17-1108	EBL South Approach Embankment	5028156.3	375912.9	75.7	16.2
17-1109	WBL South Approach Embankment	5028103.2	376067.6	75.9	16.0
17-1110	WBL South Abutment	5028121.3	376057.8	76.0	21.9
17-1111	WBL South Pier 4	5028135.2	376046.0	70.4	16.3
17-1113	WBL North Pier 2	5028164.4	376031.8	66.9	10.8
17-1114	WBL North Pier 1	5028179.8	376023.8	70.4	15.3
17-1115	WBL North Abutment	5028195.6	376020.9	75.9	21.7
17-1116	WBL North Approach Embankment	5028207.2	376015.1	75.9	18.2

3.2 Previous Investigation (1972)

As part of the current assignment, previously collected subsurface information pertinent to the site was reviewed and compiled. This existing subsurface information was contained in the following report:

- Report prepared by MTO (then the Department of Transportation and Communications, Ontario) titled “*Foundation Investigation Report for Proposed Structures at the Crossing of the C.N.R. and Hwy. #417 (E.B.L. and W.B.L.) Regional Municipality of Ottawa-Carleton, District No. 9 (Ottawa), W.O. 71-11124 – W.P. 10-69-03 (E.B.L.), 10-69-04 (W.B.L.)*”, dated February 28, 1972 (Geocres No. 31G5-79).

Sixteen boreholes were put down as part of the original investigation in 1972 along the then-proposed bridge alignments. The approximate borehole and ground surface elevations are shown on the Record of Borehole sheets included in Appendix C and are also shown on Drawings 1 and 2. The locations of the previous boreholes should be considered approximate since the locations were referenced to an imperial borehole location plan rather than metric MTM coordinates. The detailed subsurface soils and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are given on the borehole records from the 1972 investigation.

4.0 DESCRIPTION OF SUBSURFACE CONDITIONS

4.1 Site Stratigraphy

The Record of Borehole and Drillhole sheets from the current investigation are presented in Appendix A. Photographs of the recovered bedrock core are included on Figures A1 to A22 also 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 B1 to B10 in Appendix B. These results are also presented on the Summary of Engineering Properties, Figure B11 in Appendix B. The Record of Borehole sheets and lab testing from the previous investigation are provided in Appendix C. The results of basic chemical analysis completed on select soil samples are provided in Appendix E.

The borehole locations and the interpreted stratigraphic profiles projected along the CNR Overhead EBL and WBL structures are shown on Drawings 1 and 2, respectively. 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 MASW and VSP test results and report are presented in Appendix F 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.

In general, the subsurface conditions at the site consist of a layer of fill underlain by a deposit of sensitive silty clay overlying glacial till and shale bedrock. Shale bedrock was indicated to be present at depths ranging from about 8 to 21 m below the existing ground surface (i.e., elevations ranging from about 55 to 60 m). The groundwater level was encountered at depths of about 9 and 10 m below the existing ground surface (i.e., about Elevation 66 m).

A detailed description of the subsurface conditions encountered in the boreholes from the current investigation is as well as the 1972 investigation are provided in the following sections.

4.2 Pavement Structure, Topsoil, and Fill

Boreholes 17-1101, 17-1102, 17-1107 and 17-1108 were advanced through the pavement structure of the EBL of Highway 417 and Boreholes 17-1109, 17-1110, 17-1115, and 17-1116 were advanced through the pavement structure of the WBL of Highway 417. The asphaltic concrete was about 200 mm thick and was underlain by a granular base that ranges in thickness from about 300 to 500 mm and generally consists of gravelly sand to sandy gravel.

Topsoil fill was encountered at the ground surface at Boreholes 17-1103 and 17-1114. The topsoil has a thickness of about 200 mm.

A layer of embankment/grade fill exists at the ground surface, or below the topsoil or pavement structure, where encountered, in the boreholes advanced for the current investigation and extends to depths ranging from about 0.6 to 10.7 m below the ground surface (i.e., elevations ranging from about 64.1 to 66.0 m). The fill generally consists of granular fill with various compositions of sand and gravel (i.e., gravelly sand, sandy gravel, sand and gravel, silty sand, and sand). Sandy silty clay to clayey silt fill was encountered beneath the granular fill at Boreholes 17-1113 and 17-1114. Organic matter, cobbles, and boulders were also encountered within the fill.

SPT 'N' values measured in the fill generally range from about 7 to greater than 50 blows per 0.3 m of penetration indicating a loose to very dense compactness, but more typically compact to dense.

The results of grain size distribution testing carried out on several samples of the fill are provided on Figures B1a and B1b in Appendix B. The measured water content of the fill ranges from approximately 3 to 34 percent.

A layer of buried topsoil was encountered below the fill in Boreholes 17-1106, 17-1107, 17-1108, and 17-1109. The buried topsoil has a thickness ranging from about 0.3 to 0.8 m and extends to depths ranging from about 4.6 to 10.7 m (i.e., elevations ranging from about 65.0 to 65.7 m). The measured water content of two samples of the topsoil are about 24 and 39 percent.

An approximately 0.2 m thick layer of sandy silt containing organic matter underlies the fill at Borehole 17-1115.

4.3 Silty Clay, Clay and Clayey Silt

The fill, buried topsoil and sandy silt (where encountered) are underlain by a deposit of silty clay, clay and/or clayey silt (hereafter referred to as silty clay).

The upper portion of the silty clay deposit has been weathered to a grey brown crust as identified in the boreholes from the current investigation. The weathered crust has a thickness ranging from about 1.4 to 2.9 m and extends to depths ranging from about 3.1 to 13.1 m below the existing ground surface (i.e., Elevations ranging from 62.5 to 64.1 m). The depth of weathering was not identified on the Record of Borehole sheets from the previous investigation.

SPT 'N' values measured in the weathered silty clay crust range from 1 to 19 blows per 0.3 m of penetration, indicating that the deposit has a stiff to very stiff consistency.

The results of Atterberg limit testing on several samples of the weathered silty clay deposit from the current investigation gave plasticity index values ranging from about 31 and 59 percent and liquid limit values ranging from about 46 and 76 percent, indicating a deposit of intermediate to high plasticity. The results of the Atterberg limit testing are provided on Figure B2 in Appendix B. The measured water content of samples of the weathered silty clay deposit from the current investigation ranges from approximately 24 to 55 percent.

The results of grain size distribution testing carried out on two samples from the current investigation of the weathered silty clay crust are provided on Figure B3 in Appendix B.

The silty clay deposit below the depth of weathering is grey in colour. The silty clay has a thickness that ranges from about 1.6 to 5.0 m and extends to depths ranging from about 6.1 to 16.8 m below the existing ground surface (i.e., elevations ranging from about 59.1 to 61.3 m).

SPT 'N' values measured in the silty clay deposit range from "weight of hammer" to 5 blows per 0.3 m of penetration. In situ shear vane testing carried out within the unweathered silty clay deposit measured undrained shear strengths ranging from about 19 kPa to greater than 96 kPa. Lower values of 19 kPa and 23 kPa were measured at Boreholes 7 and 6, from the previous investigation prior to the construction of the embankment in the EBL and WBL, respectively, with the remainder of the boreholes showing values ranging from 31 kPa to greater than 96 kPa. The results of the in situ testing typically indicate a firm to very stiff consistency. Remolded strengths ranging from about 3 to 29 kPa were measured in the silty clay, indicating a medium to extra-sensitive deposit.

The results of Atterberg limit testing on several samples of the silty clay deposit from the current investigation gave plasticity index values ranging from about 10 to 44 percent and liquid limit values ranging from about 27 to 61 percent, indicating a deposit of low to high plasticity. The results of the Atterberg limit testing are provided on Figure B4 and Figure B11 in Appendix B. The measured water content on samples of the silty clay deposit from the current investigation ranges from approximately 33 to 63 percent.

The results of grain size distribution testing carried out on two samples of the clayey deposit from the current investigation are provided on Figure B5 in Appendix B.

The results of Atterberg limit testing, grain size distribution testing and measured water contents from the previous investigation are provided in Appendix C as well as on the Record of Borehole sheets also in Appendix C.

Laboratory oedometer consolidation testing was carried out on two samples of silty clay from Boreholes 17-1106A and 17-1116 from the current investigation as well as five samples from Boreholes 1, 2, 4, 8, and 9 from the 1972 investigation. The results of that testing are provided on Figures B6 and B7 in Appendix B for the current investigation and in Appendix C for the previous investigation, as well as summarized in the table below.

Borehole/ Sample Number	Sample Depth/ Elevation (m)	Unit Weight (kN/m ³)	σ_p' (kP)	σ_{vo}' (kP)	$\sigma_p' - \sigma_{vo}'$ (kPa)	Cc	Cr	e _o	OCR
17-1106A / 1	8.1 / 62.2	16.2	165	120	45	1.33	0.013	1.75	1.4
17-1116 / 18	14.1 / 61.8	17.1	275	250	25	0.76	0.011	1.33	1.1
1 / 3	3.0 / 62.8	16.3	297	-	-	1.17	-	1.68	-
2 / 2	2.2 / 63.5	17.8	335	-	-	0.30	-	1.12	-
4 / 4	4.2 / 61.6	18.0	240	-	-	0.92	-	1.27	-
8 / 2	2.1 / 63.9	17.1	335	-	-	0.76	-	1.30	-
9 / 4	3.9 / 62.1	16.2	211	-	-	1.58	-	1.77	-

Notes: σ_p' - Apparent preconsolidation pressure
 σ_{vo}' - Computed existing vertical effective stress
Cc - Compression index
Cr - Recompression index
e_o - Initial void ratio
OCR - Overconsolidation ratio

The consolidation test results, undrained shear strengths, Atterberg limits and measured water contents are also presented on the Summary of Engineering Properties, Figure B11 in Appendix B.

4.4 Sandy Silt

A deposit of sandy silt was encountered below the silty clay in Borehole 17-1116. The sandy silt has a thickness of about 0.3 m and extends to a depth of about 15.8 m below the existing ground surface (i.e., Elevation 60.1 m).

4.5 Silt and Sand, Silty Sand, Sandy Silt and Clayey Silt Till

A deposit of glacial till was present below the silty clay and sandy silt, where encountered. The glacial till deposit generally consists of gravel, cobbles, and boulders in a matrix of sand, clay and silt in varying compositions (i.e., silt and sand, silty sand, sandy silt, and/or clayey silt) with trace to some clay. Gravelly silty sand, gravelly sandy silt as well as sand and gravel till deposits were also encountered. The glacial till extends to depths ranging from about 6.2 to 10.6 m below the existing ground surface (i.e., elevations ranging from about 55.2 to 59.9 m).

SPT 'N' values measured in the glacial till deposit range from 'weight of hammer' to greater than 50 blows per 0.3 m of penetration, but more generally ranging from 2 to 23 blows per 0.3 m of penetration, indicating a very loose to state of compact compactness. The higher blow counts could possibly reflect the presence of cobbles, boulders or the bedrock surface rather than the state of compactness of the soil matrix. Rotary diamond drilling techniques were also required to advance through the cobbles and boulders within the till at Borehole 17-1111.

The results of Atterberg limit testing on the percent passing the 425 µm sieve of 5 samples of the glacial till deposit from the current investigation gave plasticity index values ranging from about 6 to 8 percent and liquid limit values ranging from about 17 to 22 percent, indicating a silty clay to clayey silt of low plasticity. The results of the Atterberg limit testing are provided on Figure B8 in Appendix B. The measured water content on samples of the glacial till deposit ranges from about 6 to 24 percent.

The results of grain size distribution testing carried out on several samples of the glacial till from the current investigation are provided on Figures B9a and B9b in Appendix B.

The results of grain size distribution testing carried out on samples of the glacial till from the previous investigation are provided in Appendix C. The measured natural water contents within the glacial till from the previous investigation are shown on the Record of Borehole sheets also in Appendix C.

4.6 Bedrock

Refusal to auger, casing, or DCPT advancement was encountered in Boreholes 17-1101, 17-1108, 17-1109, and 17-1116 from the current investigation and Boreholes 3A, 4, 5, 5A, 6, 7, and 9 from the previous investigation. The refusal was encountered at depths ranging from about 5.6 to 20.6 m below the existing ground surface (i.e., elevations ranging from about 55.2 to 60.2 m); this has been inferred to represent the bedrock surface. However, refusal could also indicate cobbles or boulders within the glacial till.

Bedrock was proven beneath the glacial till in Boreholes 17-1102 to 17-1106, 17-1106A, 17-1107, and 17-1110 to 17-1115 from the current investigation and Boreholes 1, 1A, 2, 2A, 3, 4A, 8, and 10 from the previous investigation. The bedrock was encountered at depths ranging from about 6.2 to 20.1 m below the existing ground surface (i.e., elevations ranging from about 55.5 to 59.8 m). During the current investigation, the bedrock was cored between about 1.5 and 5.7 m depth using BQ, NQ or HQ-sized coring equipment. During the previous investigation, the bedrock was cored between about 0.6 and 3.0 m depth using BX coring equipment.

The following table summarizes the bedrock surface or refusal depths and elevations as encountered at the borehole locations for the current investigation.

Borehole Number	Borehole Location with respect to Bridge Structure	Existing Ground Surface Elevation (m)	Depth to Bedrock/ Refusal (m)	Bedrock Surface/ Refusal Elevation (m)
17-1101	EBL North Approach Embankment	75.8	20.6 ⁽¹⁾	55.2 ⁽¹⁾
17-1102	EBL North Abutment	75.9	20.1	55.8
17-1103	EBL North Pier 1	70.5	13.6	56.9
17-1104	EBL North Pier 2	66.4	8.8	57.7
17-1106	EBL South Pier 4	70.3	11.3	59.0
17-1106A	EBL South Pier 4	70.3	11.2	59.0

Borehole Number	Borehole Location with respect to Bridge Structure	Existing Ground Surface Elevation (m)	Depth to Bedrock/ Refusal (m)	Bedrock Surface/ Refusal Elevation (m)
17-1107	EBL South Abutment	75.8	16.2	59.7
17-1108	EBL South Approach Embankment	75.7	16.2 ⁽¹⁾	59.6 ⁽¹⁾
17-1109	WBL South Approach Embankment	75.9	16.0 ⁽¹⁾	59.8 ⁽¹⁾
17-1110	WBL South Abutment	76.0	16.2	59.8
17-1111	WBL South Pier 4	70.4	12.2	58.2
17-1113	WBL North Pier 2	66.9	7.8	59.1
17-1114	WBL North Pier 1	70.4	12.0	58.5
17-1115	WBL North Abutment	75.9	18.1	57.8
17-1116	WBL North Approach Embankment	75.9	18.2 ⁽¹⁾	57.7 ⁽¹⁾
1A	EBL West Approach Embankment	66.1	10.6	55.5
4A	EBL North Pier 1	66.0	9.0	57.0
5A	EBL South Pier 3	65.9	8.5 ⁽¹⁾	57.4 ⁽¹⁾
7	EBL North Pier 2	66.0	8.7 ⁽¹⁾	57.3 ⁽¹⁾
8	EBL	66.1	9.3	56.8
9	EBL South Pier 3	66.1	7.5 ⁽¹⁾	58.6 ⁽¹⁾
10	EBL	65.8	7.0	58.8
1	WBL South Pier 4	65.7	6.2	59.5
2	WBL	65.7	7.0	58.7
2A	WBL North Pier 1	65.8	6.4	59.4
3	WBL West Approach Embankment	66.0	8.1	57.9
3A	WBL West Approach Embankment	66.1	8.5 ⁽¹⁾	57.6 ⁽¹⁾
4	WBL	65.5	6.9 ⁽¹⁾	58.6 ⁽¹⁾
5	WBL	65.8	5.6 ⁽¹⁾	60.2 ⁽¹⁾
6	WBL East Approach Embankment	65.7	6.6 ⁽¹⁾	59.1 ⁽¹⁾

Note ⁽¹⁾: Depth and elevation to bedrock inferred from refusal to auger, casing or DCPT advancement.

The bedrock encountered in these boreholes consist of slightly weathered to fresh, thinly to medium bedded, black to dark grey, fine grained, porous shale. The Rock Quality Designation (RQD) values measured on recovered bedrock core samples from the current investigation typically ranged from about 50 to 100 percent indicating a generally fair to excellent quality rock. However, an RQD value of 19 percent (indicating a very poor quality rock) was encountered in the upper 0.6 m (i.e., between about Elevations 55.2 and 55.6 m) of the bedrock at Borehole 17-1102 which is located adjacent to the north abutment of the EBL structure. In addition, RQD values ranging from 0 to 33 percent (indicating a very poor to poor quality rock) were measured for the full length of bedrock cored (i.e., between about Elevations 56.6 and 59.7 m) at Boreholes 17-1106, 17-1106A, and 17-1107 which are located adjacent to Pier 4 and the south abutment, respectively, of the EBL structure. An RQD value of 22 percent (indicating a very poor quality rock) was also measured in middle portion of the bedrock cored (i.e., between about Elevations 55.6 and 57.1 m) at Borehole 17-1111 which is located adjacent to Pier 4 of the WBL structure.

Photos of the bedrock core from the current investigation are provided in Figures A1 to A22 in Appendix A.

Results of unconfined compressive strength testing carried out on two bedrock core samples from Boreholes 17-1106A and 17-1113 were about 29 and 33 MPa, as shown on Figure B10 in Appendix B. These results indicate a medium strong bedrock.

4.7 Groundwater Conditions

The groundwater conditions observed in the open boreholes during drilling at Boreholes 17-1106, 17-1109, 17-1110, and 17-1111 were between about Elevations 61.0 and 64.6 m. However, these groundwater levels are not considered representative of stabilized groundwater conditions.

The groundwater levels in the monitoring wells installed in Boreholes 17-1108 and 17-1115, measured two to three months after well installation, are summarized in the following table.

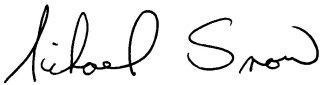
Borehole	Ground Surface Elevation (m)	Screened Interval Material	Water Level Depth (m)	Water Level Elevation (m)	Date of Reading
17-1108	75.7	Embankment Fill	9.4	66.3	September 24, 2017
17-1115	75.9	Embankment Fill/ Silty Clay	10.3	65.6	September 25, 2017


The groundwater levels measured within the open boreholes during the 1972 investigation indicated groundwater levels which ranged from about Elevation 64.6 to 66.1 m at the time of drilling. It should be noted that groundwater levels in the area are subject to fluctuations both seasonally and with precipitation events. In addition, the groundwater levels from the 1972 investigation may not be representative of the current site conditions.

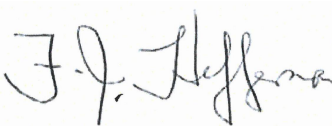
5.0 CLOSURE


This report was prepared by Mr. Alex Meacoe, P.Eng. It was reviewed by Mr. Michael Snow P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fintan Heffernan, P.Eng., a Senior Consultant with Golder and the MTO Foundations Designated Contact, conducted an independent quality control review of this report.

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WAM/MSS/FJH/mvrd

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PART B

Foundation Design
Proposed CNR Overhead Widening
Site Nos. 3-301/1 and 3-301/2
Highway 417
Ottawa, Ontario
G.W.P. 4145-10-00

6.0 FOUNDATION ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the widening and seismic retrofit of the CNR Overhead EBL and WBL bridge structures (MTO Structure Site Nos. 3-301/1 and 3-301/2) located on Highway 417 approximately 800 m north of Hunt Club Road in Ottawa, Ontario. The recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the current subsurface investigation as well as the available Geocres information for the site.

The foundation investigation report, discussion, and recommendations are intended for the use of the Ministry of Transportation, Ontario (MTO) and shall not be used or relied upon for any other purpose or by any other parties, including the construction contractor. The contractor must make their own interpretation based on the factual data in Part A (Foundation Investigation) of the report. Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project. Those requiring information on the aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

The existing bridges consist of five-span reinforced concrete slab on pre-stressed concrete girder structures. The overall EBL structure is about 72.8 m long, with 11.2 m, 16.8 m, 16.8 m, 16.8 m, and 11.2 m spans and varies in width from about 14.3 to 17.9 m. The overall WBL structure is about 70.8 m long, with 10.2 m, 16.8 m, 16.8 m, 16.8 m, and 10.2 m spans and is about 13.0 m wide. The bridge abutments are supported on steel HP12 x 74 (HP310 x 110) piles end bearing on bedrock. The abutment piles are configured in two rows, with the front row of piles battered towards the CN rail line at 1H:4V and the back row piles battered away from the CN rail line at 1H:10V, the piles for the wingwalls are vertical. The pier foundations are also supported on HP12 x 74 (HP310 x 110) steel piles end bearing on bedrock. The pier piles are configured in sets of five at each column, with a vertical pile at the centre of the column surrounded by four piles battered away from the centre of the column at 1H:5V to 1H:7V.

The existing pavement grade of Highway 417 is about Elevation 76 m at the structures and the approach embankments are approximately 10 to 11 m in height relative to CN rail line. The existing embankment side slopes are at about 2H:1V with an approximately 3 m wide mid-slope bench.

The CN rail line is located within the “valley” of the approach embankments between Piers 2 and 3 at about Elevation 67 m. The centreline of the rail line is approximately 10 m from Pier 2 and about 5 m from Pier 3 at both the EBL and WBL structures.

Based on visual observation at the time of the site investigation and subsequent discussion with WSP, no signs of foundation settlement (at the abutments or piers) were observed. However, some erosion was observed along the foreslopes of the abutments. Otherwise there is no visual evidence of embankment settlement and the existing embankment slopes appear to be performing satisfactorily.

It is understood that the existing bridges will be widened towards the median, by about 3.2 m for the EBL structure and by about 5.5 m for the WBL structure. It is also understood that the structures are being retrofit for semi-integral abutments and that Piers 2 and 3 which are adjacent to the CN rail line are to be designed for train impact loading in the event of a derailment.

The bridge widenings and seismic retrofits are to be designed in accordance with the current Canadian Highway Bridge Design Code CAN/CSA-S6-14 (CHBDC).

6.2 Seismic Design

6.2.1 Seismic Hazard and Importance Category

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

In accordance with Section 4.4.2 of the CHBDC, and as specified by the MTO, the CNR Overhead structures have been given an importance category of 'Major Route' bridges.

6.2.2 Seismic Site Classification

Multichannel Analysis of Surface Waves (MASW) geophysical testing was carried out within the Highway 417 median in the vicinity of the bridge. Vertical Seismic Profile (VSP) geophysical testing was also carried out at the site in Borehole 17-1110 (i.e., through the south approach embankment of the WBL structure). Both methods were carried out in order to evaluate the average shear wave velocity of the upper 30 m of soil/bedrock at the site. The shear wave velocities measured at the site are presented in technical memorandums in Appendix F. The results indicate average shear wave velocities in the upper 30 m of the subsurface stratigraphy of 400 m/s and 444 m/s for the two MASW lines and 393 m/s at the location of the VSP testing. Based on these values and proposed elevations of the pile caps at the abutments and piers it is considered that a Site Class C would be applicable for the design of the bridge widenings and seismic retrofits.

6.2.3 Spectral Response Values and Seismic Performance Category

In accordance with Section 4.4.3.1 of the CHBDC and based on the location of the bridges (latitude 45.39 N and longitude 75.59 W), the following are the reference Site Class C mpeak seismic hazard values based on data obtained from Earthquakes Canada (www.earthquakescanada.nrcan.gc.ca).

Site Class C Spectral Values for Subject Site

Parameter	Value at Given Probability of Exceedance in 50 Years		
	10% (475-year)	5% (975-year)	2% (2,475-year)
PGA	0.107 g	0.174 g	0.300 g
T ≤ 0.2 s	0.170 g	0.271 g	0.469 g
T = 0.5 s	0.092 g	0.145 g	0.250 g
T = 1.0 s	0.046 g	0.072 g	0.123 g
T = 2.0 s	0.021 g	0.033 g	0.058 g
T = 5.0 s	0.0048 g	0.0083 g	0.015 g
T ≥ 10.0 s	0.0019 g	0.0032 g	0.0055 g

The fundamental periods of the widened structures are unknown at this time. However, in consideration of the structures' "Major Route" importance category and the site-specific seismic hazard values given above, the bridge would fall in Seismic Performance Category 2, if the fundamental period of the structure is greater than or equal to 0.5 s, or Seismic Performance Category 3, if the fundamental period of the structure is less than 0.5 s, in accordance with Table 4.10 of the CHBDC.

Based on the *irregular* geometry of the bridges (since their skew angles are more than 20 degrees), it is understood that the structures will be designed using a “performance-based approach” as defined in the CHBDC.

6.2.4 Liquefaction Assessment

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

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

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

The liquefaction analysis was carried out using the in situ testing data collected at the borehole locations. The design groundwater level was chosen based on the measured groundwater level in the monitoring well installed in Borehole 17-1115 at Elevation 65.6 m. The CRR with depth was calculated at each borehole location using the parameter, $(N_1)_{60CS}$, that is based on the SPT “N” blow counts obtained in the field and corrected for overburden stress, rod length during sampling, assumed hammer energy efficiencies, and fines content.

The methodology used to assess liquefaction potential at the site is consistent with the “simplified” approach outlined in the CHBDC and by Idriss and Boulanger (2008). It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil. The results of the liquefaction assessment using the simplified method indicate that certain horizons of granular soils within the glacial till at the site may be considered liquefiable during the 2,475-year design earthquake. Furthermore, based on the lab results at the site, certain siltier horizons of the clay deposit (i.e., lower portions of the deposit) and more plastic horizons within the glacial till can be expected to undergo some cyclic softening during the 2,475-year design earthquake as discussed by Bray et al (2004).

However, the liquefaction methodologies outlined in Idriss and Boulanger (2008) do not account for the additional cyclic resistance provided by the aging/cementation that may be a characteristic of the glacial till at the site. Although aging of deposits is known to help resist seismic liquefaction, little research has been done in this area to quantify this. Based on Figure 9 presented in the work by Leon et al. (2006) a correction increase of about 30% in the CRR profile would appear appropriate. While such aging/cementation corrections would potentially reduce the risk of liquefaction at this site given the age of the till of about 10,000 to 15,000 years, specific testing/assessment/research on the till at this site is not available.

Work done by Harpin et al (2017) on the site response of sites in eastern Canada and the site response analyses conducted by Golder Associates at the CR31 overpass (i.e., Site No. 31-204) would suggest that a site-specific response analysis would also reduce the CSR profile, when compared to the simplified methodology outlined in Idriss and Boulanger (2008), such that an approximately 20% reduction could be expected.

In consideration of the beneficial effects of aging and the anticipated lower cyclic shear stresses in eastern Canada relative to the simplified method, higher CRR and lower CSR respectively, the plasticity present in certain till horizons, and the depth of the potentially liquefiable horizons, the extent and probability of liquefaction at the site is considered to be very small to the point of having little impact on the dynamic response of the site (i.e., Site Class) and the performance of foundation elements (i.e., no liquefaction settlement induced downdrag).

Cyclic softening of certain fine-grained layers has been accounted for in the lateral seismic response of these soils as described in Section 6.4.6. Such materials are considered to behave dominantly as “clay-like” rather than “sand-like” as discussed by Boulanger and Idriss (2006). The work outlined by Boulanger and Idriss (2007) was used to assess whether “clay-like” soils could undergo “cyclic softening”.

6.3 Bridge Foundation Options

The existing CNR Overhead bridges are five-span structures with reinforced concrete decks and non-integral abutments. The existing overhead bridges are understood to be in fair condition.

According to the available information, the existing five-span bridge structure abutments, wing walls and piers are founded on vertical and battered HP310 x 110 steel H-piles end bearing on bedrock.

Based on the subsurface conditions, only deep foundation options have been considered for the widening of the existing CNR Overhead structures, as shallow foundations would not provide sufficient bearing resistances or acceptable settlement performance for the structures.

A summary of the advantages and disadvantages associated with each deep foundation option is provided below, and a comparison of the alternative foundation options based on advantages, disadvantages, constructability and relative costs is provided in Table 1 following the text of this report.

- **Driven steel H-piles:** Steel H-piles driven to refusal on the shale bedrock could be feasible for support of the bridge widenings. This option would provide high geotechnical resistances and minimal post-construction settlements; in addition, this option would permit the use of conventional or semi-integral abutments and piers. The use of driving shoes is recommended to minimize damage while penetrating the glacial till deposit (which may contain cobbles and boulders) and seating onto the shale bedrock. Provided vibrations during pile installation are limited to the maximum peak particle velocity indicated in Section 6.7.5, vibrations during pile installation should not impact the existing bridge structure or the CN rail line. It is also considered that the piles installed using this method can be located as to not interfere with the existing battered piles. Given this foundation option is not a displacement pile, it is considered acceptable for use adjacent to the CN rail line however this foundation type may not have the required lateral stiffness to resist the train impact loading.
- **Driven steel pipe (tube) piles:** Closed-ended steel tube (pipe) piles could also be considered as a deep foundation option for support of the abutment and pier widenings. This foundation option would have similar advantages to steel H-piles in terms of high geotechnical resistances and minimal settlements. This option would permit the use of conventional or semi-integral abutments and piers. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their vertical or battered orientation if cobbles and/or boulders are encountered within the till deposit during driving. As with

H-piles, provided vibrations during pile installation are limited, these vibrations should not impact the existing bridge structure or the CN rail line. It is also considered that the piles installed using this method can be located as to not interfere with the existing battered piles. This option may pose more risk to the adjacent CN rail line since larger soil displacement is expected for this pile type when driven close-ended. As with H-piles this foundation type may not have the required lateral stiffness to resist the train impact loading.

- **Rock socketed pipe piles:** Socketed steel pipe piles could also be considered as a deep foundation option for support of the abutments. This foundation option would also have similar advantages to the options above of relatively high geotechnical resistances and minimal settlement. This option would permit the use of conventional or semi-integral abutments and piers. This foundation type would also penetrate any cobbles or boulders encountered within the glacial till deposit during installation. Socketed pipe piles may have the required stiffness to resist the expected seismic lateral loads and train impact loading as well as provide toe 'fixity' at the interface with the bedrock surface. Provided the vibrations during caisson installation are limited, vibrations during pile installation should not impact the existing bridge structure or the CN rail line. The vibrations associated with this type of pile installation would be lower than those expected for typical driven H-pile or pipe pile construction, even during penetration through the cobbles and boulders in the glacial till, thereby reducing the potential impacts on the existing structure and CN rail line. It is also considered that the piles installed using this method can be located as to not interfere with the existing battered piles. Since this type of pile installation would result in minimal soil displacement it should not impact the adjacent CN rail line.
- **Auger cast piles:** Auger-cast piles (ACPs also referred to as continuous flight auger piles) involve a continuous flight auger system drilled to a target depth. As the auger is removed, concrete or grout is injected through the hollow centre of the auger. Upon complete removal of the auger, reinforcing steel can then be inserted into the wet concrete/grout. Typical pile diameters are about 0.3 to 0.6 m and can attain depths of about 25 m in Ontario. The number of foundation contractors able to install these in Ontario is smaller relative to more common foundation types and such foundations are not commonly used on highway bridge projects in Ontario. These foundation systems are not well-suited where uplift resistance is a key consideration, where cobbles/boulders can hinder auger drilling, or where high lateral resistance is required. Based on these limitations, ACPs are not considered to be a preferred foundation alternative.
- **Rock socketed drilled concrete caissons:** Caissons deriving their support from bearing within the shale bedrock are also feasible for this site. This option would provide both high geotechnical and lateral resistances as well as minimal settlement. Caissons would require the use of temporary or permanent liners to mitigate the potential risks of ground squeezing in the clay deposits or ground loss from potential water-bearing cohesionless layers within the till soils during construction. In addition, the caissons must be socketed into the bedrock a sufficient length to provide the required bearing resistance and such sockets would have to be advanced by rock coring and/or chisel drilling into the competent shale bedrock. Provided vibrations during caisson installation are limited to the maximum peak particle velocity indicated in Section 6.7.5, vibrations during installation should not impact the existing bridge structure or the CN rail line. Due to their larger diameter it may be more difficult to locate the caissons as to avoid conflict with the existing piles. Since this type of pile installation would result in minimal soil displacement it should not impact the adjacent CN rail line. This option is not considered preferred to the driven pile options mentioned above for the bridge abutments or Piers 1 and 4, however it is considered the preferred method for Piers 2 and 3 since this foundation type would have the required stiffness to resist both the expected seismic lateral loads and train impact loading.

It is considered that the most feasible and cost-effective foundation options for the bridge widenings is driven steel H-piles founded on the bedrock at the abutments and Piers 1 and 4, and rock socketed caissons at Piers 2 and 3.

6.3.1 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the CHBDC and its Commentary, the existing bridge structures and foundation systems may be classified as having large traffic volumes and their performance as having potential impacts on other transportation corridors, hence having a “typical” consequence level associated with exceeding limits states design. Given the level of foundation investigation completed to date as presented in Sections 3.0 and 4.0, in comparison to the degree of site understanding in Section 6.5 of CHBDC, the level of confidence for design is considered to be a “typical degree of site and prediction model understanding.” Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ , of unity and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs} , from Tables 6.1 and 6.2 of the CHBDC have been used for design, as indicated in Sections 6.4 and 6.5 below. For seismic design, the consequence factor, Ψ , and resistance factor, ϕ_{gu} , should be taken as unity, as per Section 4.6.3 of the CHBDC.

6.4 Driven Steel H-Pile Foundations

6.4.1 Founding Elevation

The widenings to the existing bridge abutments and piers, may be supported on steel H-piles driven to refusal on the shale bedrock. However, this foundation type may not provide adequate lateral resistance for the foundations at Piers 2 and 3 which need to be design for train impact loading. Based on the borehole results from the investigations as well as the weathered/poorer quality rock, the following pile tip elevations are recommended for design of steel H-piles for the widened piers and abutments:

Foundation Element	Borehole Numbers	Bedrock Surface Elevation	Design Pile Tip Elevation
EBL North Abutment	17-1102	55.8	55.7
EBL Pier 1	17-1103	56.9	56.8
EBL Pier 2	17-1104	57.7	57.6
EBL Pier 3	5A	57.4 ¹	57.3
EBL Pier 4	17-1106	59.0	58.5
EBL South Abutment	17-1107	59.7	59.2
WBL North Abutment	17-1115	57.8	57.7
WBL Pier 1	17-1114	58.5	58.4
WBL Pier 2	17-1113	59.1	59.0
WBL Pier 3	4	58.6 ¹	58.5
WBL Pier 4	17-1111	58.2	58.1
WBL South Abutment	17-1110	59.8	59.7

Note ⁽¹⁾: Bedrock surface elevation interpreted from previous borehole from the 1972 foundation investigation.

The pile caps in the area of the widening should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

For the installation of steel H-piles, consideration must be given to the potential presence of cobbles and boulders within the till deposit. A Non-Standard Special Provision (NSSP) should be included in the Contract Documents to alert the Contractor of these conditions and is provided in Appendix H of this report. The piles should be reinforced at the tip with standard bearing points (such as Titus Standard 'H' Bearing Pile Points or equivalent) to improve seating of the piles on the bedrock and to reduce the potential for damage to the piles during driving through soils that contain boulders, in accordance with OPSS.PROV 903 (*Deep Foundations*) with amendments as per Standard Special Provision (SSP) 109S18.

Vibration monitoring should be carried out during pile installation to ensure that the vibration levels at the existing structure are maintained below tolerable levels. A NSSP for vibration monitoring should be included in the contract documents and is provided in Appendix H of this report. A maximum peak particle velocity of 100 mm/s is recommended at the existing abutments and piers. The piles furthest from the existing structures should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the installation procedures for the remaining piles. Further discussion of the vibration monitoring requirements is provided in Section 6.8.5. It should be noted that CN Rail may have additional vibration monitoring requirements for the rail line.

The piles for the widened abutments and piers may need to be driven in close proximity to the battered piles supporting the existing abutments, wing walls and piers. These existing piles may be offset from their intended location or alignment and the potential exists for conflicts when driving the new piles. Current construction practice generally limits the acceptable pile offset at the surface to 75 mm and the deviation from the design inclination to 2%. Depending on the final pile configuration, the spacing between the new and existing piles may be less than this tolerance and therefore the potential exists for interference during driving of the new piles. For new piles driven within the potential zone of interference with the existing piles at the abutments, wing walls or piers (defined as a distance around the existing pile centre equal to 10% of the pile length) the driving operations should be continuously monitored by a Foundation Engineering Specialist (FES) hired by the Contract Administrator and the contractor should cease driving of the pile if the FES indicates that the driven pile may have come in contact with an existing pile. It may be necessary to extract and re-drive piles if contact between the new and existing piles is believed to exist. A NSSP for driving piles adjacent to existing vertical and battered piles should be included in the contract documents and is provided in Appendix H of this report.

6.4.2 Factored Geotechnical Axial Resistances

The factored axial compressive geotechnical resistance at Ultimate Limit States (ULS) that can be used for the existing piles is presented below and is based on the available design drawings, Geocres information, and the known quality of the steel being used for typical construction projects during that time. In accordance with the CHBDC, a geotechnical resistance factor of 0.4 was applied when calculating the axial factored compressive resistance given below, irrespective of H-pile dimension. Higher geotechnical resistance factors and consequently higher factored axial resistances could be validated through a pile load test program where dynamic and static load tests are performed on indicator piles.

Pile Size	Factored Geotechnical Axial Resistance at ULS (kN)
HP12 x 74 (HP310 x 110)	1,500 kN

The factored ULS geotechnical axial resistances for new driven steel H-piles that are successfully driven to found on the bedrock are presented below.

Pile Type	Factored Geotechnical Axial Resistance at ULS (kN)
HP 310 x 110	1,500
HP 360 x 132	1,800
HP 360 x 152	2,100

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

It should be noted that the factored axial resistances provided at ULS are limited by the geotechnical resistance of the shale bedrock. The axial resistances given above for driven piles to refusal on bedrock will not be affected by the pile spacing provided the piles are driven no closer than three pile widths centre to centre.

The factored ULS axial capacities reflect available knowledge, experience and analytical tools for developing the geotechnical capacities of these driven piles and, as such, a lower resistance factor was used. The undertaking of static pile load tests would allow for the ultimate geotechnical capacities to be validated more accurately (e.g., increased) under site-specific conditions and to support the use of higher geotechnical resistance factors allowed under the CHBDC when static load test results are available.

Pile installation should be in accordance with OPSS.PROV 903 (*Deep Foundations*) with amendments as per SSP 109F57. The drawings should incorporate the appropriate note stating that the piles should be equipped with standard bearing points and should be driven to bedrock. For piles driven to refusal on bedrock, and as described in OPSS.PROV 903, it is a generally accepted practice to reduce the hammer energy after abrupt peaking is met on the bedrock surface, and to then gradually increase the energy over a series of blows to seat the pile.

The pile termination or set criteria will be dependent on the pile driving hammer type, helmet, selected pile and length of pile; the criteria must therefore be established at the time of construction after the piling equipment is known.

Relaxation of the piles following the initial set could result from several processes, including:

- Softening of the shale bedrock into which the piles are driven
- The dissipation of negative excess pore water pressures in the overburden material above the bedrock surface
- The driving of adjacent piles

Provision should therefore be made for restriking all of the piles at least once to confirm the design set and/or the permanence of the set and to check for upward displacement due to driving adjacent piles. Piles that do not meet the design set criteria on the first restrike should receive additional restriking until the design set is met.

All restriking should be performed after 48 hours of the previous set. Since the piles will be founded on shale bedrock, it is expected that several rounds of restriking could be required.

The pile capacity should then be verified in the field by the use of the Hiley formula (MTO's Standard Drawing SS103-11, *Pile Driving Control*) and/or Pile Dynamic Analyzer (PDA) testing during pile installation on selected piles to confirm the design capacity. It is recommended PDA testing be carried out at the south abutment and Pier 4 of the EBL structure where poor quality rock was encountered. Ideally, pile installations at these locations would be carried out first, in order to verify the axial geotechnical resistance and pile termination depths at these locations of poorer rock quality.

6.4.3 Downdrag Load (Negative Skin Friction)

A compressible silty clay deposit was encountered below the embankment fill at the boreholes put down at the site. The additional loading imposed from the approximately 4 m thick and 8 m wide embankment widenings would result in consolidation settlement of the underlying compressible silty clay deposit at the abutments which could generate downdrag forces on the new and existing piles. Further discussion of the embankment settlement and proposed settlement mitigation options are provided in Section 6.7.3.

Based on the above, and assuming an underside of the pile cap level ranging from about Elevations 71.6 to 71.9 m, the unfactored downdrag load acting on a single HP 310 x 110 pile over the length of pile within the compressible soils and overlying embankment fill is estimated to be up to about 575 kN, for piles supporting the abutments. In addition, the new filling for the widening would induce sufficient settlement at the existing piles to mobilize downdrag loads.

In addition, it is understood that the existing grades at the pier locations will be maintained. Therefore, the piles at the pier locations should not experience any downdrag loading.

The structural capacity of the piles must be checked for the factored dead and downdrag loads in accordance with Sections 6.11.4.10 and C6.11.4.10 of the CHBDC.

The above downdrag loading can be discounted in the design, provided that expanded polystyrene (EPS) Geofoam light weight fill is selected as the preferred settlement mitigation option. Further discussion on this option is provided in Section 6.7.3.

6.4.4 Uplift Resistance

The following factored ULS geotechnical resistances may be used for the new and existing piles for uplift. In accordance with the CHBDC, a geotechnical resistance factor of 0.3 was applied when calculating the uplift geotechnical resistances.

Foundation Element	Factored Geotechnical Uplift Resistance at ULS (kN)
EBL North Abutment	575
EBL Pier 1	215
EBL Pier 2	115
EBL Pier 3	115
EBL Pier 4	205
EBL South Abutment	375
WBL North Abutment	395
WBL Pier 1	145
WBL Pier 2	60
WBL Pier 3	60
WBL Pier 4	145
WBL South Abutment	350

If required, additional uplift resistance could be provided by rock anchors installed into the underlying bedrock. The anchors could consist of either grouted or mechanical anchors. The rock anchors should be design, installed and tested in accordance with OPSS 942 (*Prestressed Soil and Rock Anchors*) with amendments as per SSP 109S58.

In designing grouted rock anchors, consideration should be given to four possible anchor failure modes:

- i) Failure of the steel tendon or top anchorage
- ii) Failure of the grout/tendon bond
- iii) Failure of the rock/grout bond
- iv) Failure within the rock mass, or rock cone pull-out

Potential failure modes i) and ii) are structural and are best addressed by a structural engineer.

For potential failure mode iii), the factored bond stress at the concrete/rock interface may be taken as 900 kPa for ULS design purposes at all locations with the exception of Pier 4 and the south abutment of the EBL structure where 600 kPa should be used due to the poorer quality rock at these locations. This value should be used in calculating the resistance under ULS conditions. If the response of the anchor under SLS conditions needs to be evaluated, it may conservatively be taken as the elastic elongation of the unbonded portion of the anchor under the design loading.

For potential failure mode iv), the resistance is calculated based on the unit weight (undrained) of the potential mass of rock which could be mobilized by the anchor, and resistance to shear of the rock mass. This is typically considered as the mass of rock included within a cone (or wedge for a line of closely spaced anchors) having an apex at the tip of the anchor and having an apex angle of 60 degrees. For each individual anchor, the ULS factored geotechnical resistance can be calculated based on the following equation:

$$Q_r = \varphi \frac{\pi}{3} \gamma' D^3 \tan^2(\theta)$$

- Where:
- Q_r = Factored uplift resistance of the anchor (KN)
 - φ = Geotechnical resistance factor (use 0.4)
 - γ' = Effective unit weight of rock (use 16 KN/m³ below the groundwater level)
 - D = Anchor length in metres
 - θ = one-half of the apex angle of the rock failure cone (use 30°)

Where the anchor load is applied at an angle to the vertical, the anchor capacity should be reduced as follows:

$$Q_r' = Q_r \cos(\alpha)$$

- Where:
- Q_r' = Factored uplift resistance of the anchor subject to inclined load (KN)
 - Q_r = Factored uplift resistance of the anchor (KN)
 - α = Angle between the load direction and the vertical

For a group of anchors or for a line of closely spaced anchors, the resistance must consider the potential overlap between the rock masses mobilized by individual anchors. In the case of group effects for a series of rock anchors in a rectangle with width “a” and length “b” installed to a depth “D”, the equation for the volume of the truncated trapezoid failure zone would be as follows:

$$V = \frac{4}{3} D^3 \sin^2 \varphi + aD^2 \sin \varphi + bD^2 \sin \varphi + abD$$

Where: V = Volume of the truncated trapezoid failure zone (m³)

D = Depth of anchor group (m)

a = Width of anchor group (m)

b = Length of the anchor group (m)

φ = $\frac{1}{2}$ of the apex angle of the rock failure cone, use 30°

The ULS factored geotechnical resistance for the truncated trapezoid failure formed by the group of anchors can then be calculated based on the following equation:

$$Q_r = \varphi \gamma' V$$

Where: Q_r = Factored uplift resistance of the anchor (kN)

φ = Geotechnical resistance factor, use 0.4

γ' = Effective unit weight of rock, use 16 kN/m³

V = Volume of truncated trapezoid (m³)

The method described above does not explicitly consider the tensile strength of the rock that must be overcome prior to mobilization of the weight of the rock mass. If required, the tensile strength of the rock mass can be assessed based on the unconfined compressive strength, recovery, and quality of bedrock core obtained.

It is recommended that proof load tests be carried out on the anchors to confirm their resistance. The proof load tests should be carried out in accordance with OPSS 942 (*Prestressed Soil and Rock Anchors*) with amendments as per SSP 109S58.

A Foundation Engineering Specialist (FES) hired by the Contract Administrator should be present during the installation and testing of the anchors. Care must be taken during grouting to ensure that the grouting pressure is sufficient to bond the entire length of the grouted area with minimum voids. Confirmation of sufficient embedment into the rock beneath the foundations should be carried out by the FES to make sure that the anchors are being installed in rock of adequate quality. The anchor holes must be thoroughly flushed with water to remove all debris and rock flour. It is essential that rock flour be completely removed from the holes to be grouted to promote an adequate bond between the grout and the rock. Prestressing of the anchors prior to loading will minimize anchor movement due to service loads.

6.4.5 Lateral Geotechnical Resistance

The following unfactored ULS geotechnical lateral resistances may be used for the new and existing HP 310 x 110 steel piles at the EBL and WBL structures under static and cyclic loading conditions:

Foundation Element	Unfactored Geotechnical Lateral Resistance at ULS (kN)	
	Static	Cyclic
North & South Abutments	1050	950
Piers 1 & 4	650	600
Pier 2 & 3	350	300

The values above, provide a limit on the lateral geotechnical resistances offered below the pile cap level under static and cyclic conditions using the p-y curves given in Section 6.4.6.

The ULS geotechnical lateral resistance of a pile group may be estimated as outlined in Section C6.11.3.4 of the Commentary to the CHBDC, where the lateral resistance of the pile group may be calculated using both passive earth pressure theory acting over an equivalent wall area, with a depth of seven times the pile diameter and a width equal to the width of the pile group in plan, perpendicular to the direction of the applied lateral loading and as the sum of the ULS lateral resistances of the piles in the group reduced by a group efficiency factor.

The ULS resistances provided above are unfactored values; in accordance with the CHBDC, a geotechnical resistance factor of 0.5 is to be applied when calculating the horizontal resistance.

6.4.6 Lateral Soil-Structure Interaction Springs

The foundation lateral soil-structure interaction springs required for the static and dynamic analyses of the bridge abutments and piers were computed based on the soil layers surrounding the foundations as encountered in the boreholes and the pile dimensions.

The soil-structure interaction between the bridge foundations and the surrounding soil was modeled using the load transfer method. The lateral load-displacement behaviour of the piles can be modeled using p-y curves (CFEM, 2006). P-y curves relate the lateral deflection of a single pile to the corresponding soil and bedrock reactions at any depth below ground surface.

The p-y curves were generated internally using the commercially available software programs LPILE 2016 (Version 2016.9.11), produced by ENSOFT Inc.

The p-y curves were developed considering appropriate surcharge loads imposed by the embankments fill. The curves are considered appropriate for vertical piles. For battered piles with batters of up to 1H:4V (horizontal to vertical), there is a generally recognized reduction in the p-values depending on the direction of batter relative to the direction of loading. Battered piles with a 'negative' 1H:4V batter (i.e. battered toward the direction of loading) result in a reduction in the p-values whereas piles with a 'positive' 1H:4V batter, (i.e. battered away from the direction of loading) result in an increase in the p-values. The modifications to the p-values generally offset each other for the same number of similar piles battered in opposite direction on the same foundation element.

For all loading conditions, a pinned connection was assumed between the pile head and the pile cap. Static and cyclical loading conditions were assumed for all the lateral analyses; therefore, static and cyclic p-y curves were generated for the assessment of deep pile foundations as described in the LPILE 2016 (Version 9.0) Technical Manual which is based on the studies of Wang (1982) and Long (1984).

Where "cyclic softening" of certain soil layers was identified, a 25 percent reduction in strength was considered to reflect a softer lateral response of these soils under cyclic loading conditions.

The family of static and cyclic p-y curves calculated at 0.5 m increments of depth for a single, vertical HP 310 x 110 steel H-pile at the abutments and piers are shown in tabular format and graphically in Figures G4 to G11 in Appendix G.

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

6.5 Caisson Foundations

6.5.1 Founding Elevations

Caisson foundations are considered the preferred foundation alternative for Piers 2 and 3 which need to be designed to resist train impact loading. It is understood that due to these conditions, widening of the foundations would be required on both sides of the bridge structures (i.e., not only within the median but to the west of the EBL structure and to the east of the WBL structure). However, the current foundation investigation for the widening was only carried out within the median of Highway 417 (i.e., in the area of the proposed widening), therefore the rock socket founding elevations provided below are based on a conservative assessment of the bedrock surface elevation and bedrock quality from both the current borehole and existing Geocres information.

For design purposes, it is recommended that the caissons be founded at the following elevations (i.e., assuming rock sockets of approximately 2 to 5 m in length depending on the rock quality).

Foundation Element	Borehole Numbers	Bedrock Surface Elevation	Design Rock Socket Elevation
EBL Pier 2 (Median)	17-1104	57.7	55.2
EBL Pier 2 (West)	8	56.8 ¹	54.3
EBL Pier 3 (Median)	5A	57.4 ¹	52.9
EBL Pier 3 (West)	5A	57.4 ¹	52.9
WBL Pier 2 (Median)	17-1113	59.1	56.1
WBL Pier 2 (East)	2	58.7 ¹	55.7
WBL Pier 3 (Median)	4	58.6 ¹	53.6
WBL Pier 3 (East)	17-1111	58.2	53.2

Note ⁽¹⁾: Bedrock surface elevation interpreted from previous borehole from the 1972 foundation investigation.

Due to the relatively high water table and the potential difficulty in socketing a liner into the bedrock, it may not be feasible to dewater and clean the base of the caisson and, as such, full end-bearing support may not be developed. The axial geotechnical resistance for rock-socketed caissons should be based on the side-wall (shaft) resistance of the rock socket rather than end-bearing.

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

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

If caisson caps are to be included as part of the design, they should be constructed at a minimum depth of 1.8 m for frost protection purposes, per OPSD 3090.101 (*Foundation Frost Penetration Depths for Southern Ontario*).

6.5.2 Axial Geotechnical Resistance

The *unfactored* geotechnical side wall (shaft) resistance at ULS can be taken as 1,000 kPa, provided that the caisson socket is within competent bedrock (i.e., RQD greater than 55 percent). This value assumes that the side wall of the socket will be cleaned of any smeared material. To provide full fixity, the caissons should be provided with a minimum socket length equal to 2 times the caisson diameter into the bedrock. The structural engineer should check that the shear strength of the concrete is adequate to support these loads.

For a 1.0 m diameter caisson socketed 2 to 4 m into the competent bedrock, this would equate to a factored axial geotechnical resistance at ULS of about 4,500 kN. In accordance with the CHBDC, a geotechnical resistance factor of 0.4 was applied when calculating the axial compressive resistance. SLS resistances do not apply to caissons founded within the shale bedrock, because the SLS resistance for 25 mm of settlement is greater than the factored axial geotechnical resistance at ULS.

6.5.3 Uplift Resistance

A factored ULS geotechnical resistance for uplift of 4,000 kN may be used for 1.0 m diameter caisson socketed 2 to 4 m into the bedrock. In accordance with the CHBDC, a geotechnical resistance factor of 0.3 was applied when calculating the uplift geotechnical resistance.

6.5.4 Lateral Geotechnical Resistance

The following unfactored ULS geotechnical lateral resistances may be used for the new 1.0 m diameter caissons with a 2 to 4 m rock socket at the EBL and WBL structures under static and cyclic loading conditions:

Foundation Element	Unfactored Geotechnical Lateral Resistance at ULS (kN)	
	Static	Cyclic
Pier 2 & 3	400	380

The values above, provide a limit on the lateral geotechnical resistances offered below the caisson cap level under static and cyclic conditions using the p-y curves given in Section 6.5.5.

The reductions due to group effects, may be determined as outlined in Section 6.4.5 above.

The ULS resistances provided above are unfactored values; in accordance with the CHBDC, a geotechnical resistance factor of 0.5 is to be applied when calculating the horizontal resistance.

6.5.5 Lateral Soil-Structure Interaction Springs

The foundation lateral soil-structure interaction springs required for the static and dynamic analyses of Piers 2 and 3 were computed based on the soil layers and bedrock surrounding the foundations as encountered in the boreholes as well as the caisson dimensions using the methodology and assumptions described previously in Section 6.4.6.

The family of static and cyclic p-y curves calculated at 0.5 m increments of depth for a single, vertical 1.0 m diameter caisson at Piers 2 and 3 are shown in tabular format and graphically in Figures G12 and G13 in Appendix G.

The reductions due to group effects, may be determined as outlined in Section 6.4.6 above.

6.6 Lateral Earth Pressures for Design

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

The following recommendations are made concerning the design of the walls:

- Select, free draining, non-frost susceptible granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular A or Granular B Type II, should be used as backfill behind the walls. Longitudinal drains or weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).
- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the walls, in accordance with CHBDC Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the wall. Heavy construction equipment should be maintained at a distance of at least 1 m away from the walls while the backfill soils are being placed. Hand-operated compaction equipment should be used to compact the backfill soils within a 1 m wide zone adjacent to the walls. Other surcharge loadings should be accounted for in the design, as required.
- For restrained walls, granular fill should be placed in a zone with a width equal to at least 1.8 m behind the back of the wall (Case (a) on Figure C6.20 of the Commentary to the CHBDC). For unrestrained walls, fill should be placed within the wedge-shaped zone defined by a line drawn at 1.5H:1V extending up and back from the rear face of the pile cap (Case (b) on Figure C6.20 of the Commentary to the CHBDC).

6.6.1 Static Lateral Earth Pressures for Design

The following guidelines and recommendations are provided regarding the lateral earth pressures for static (i.e., not earthquake) loading conditions. These lateral earth pressures assume that the ground above the wall will be flat, not sloping. If the inclination of the slope above the wall changes then new lateral earth pressures will need to be calculated.

- For Case (a), the pressures are based on the embankment fill and the following parameters (unfactored) may be used assuming the use of earth fill or Select Subgrade Material (SSM):

Material	Earth Fill or SSM
Soil Unit Weight:	20 kN/m ³
Coefficients of static lateral earth pressure:	
Active, K_a	0.33
At rest, K_o	0.50
Passive, K_p	3.00

- For Case (b), the pressures are based on using engineered granular fill and the following parameters (unfactored) may be used:

Material	Granular A	Granular B Type II
Soil Unit Weight:	22 kN/m ³	21 kN/m ³
Coefficients of static lateral earth pressure:		
Active, K_a	0.27	0.27
At rest, K_o	0.43	0.43
Passive, K_p	3.70	3.70

- Where the wall support does not allow lateral yielding (i.e., restrained structure where the rotational or horizontal movement is not sufficient to mobilize an active earth pressure condition), at-rest earth pressures (plus any compaction surcharge) should be assumed for geotechnical design.
- Where the wall support and superstructure allow lateral yielding, active earth pressures may be used in the geotechnical design of the structure. The movement to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the CHBDC.

6.6.2 Seismic Lateral Earth Pressures for Design

Seismic (earthquake) loading must be taken into account in the design in accordance with Section 4.6 of the CHBDC. In this regard, the following should be included in the assessment of lateral earth pressures:

- Seismic loading will result in increased lateral earth pressures acting on the wall. The wall should be designed to withstand the combined lateral loading for the appropriate static pressure conditions given in Section 6.6.1, above, plus the earthquake-induced dynamic earth pressure.
- In accordance with Sections 4.6.5 and C.4.6.5 of the CHBDC and its Commentary, for structures which do not allow lateral yielding, the horizontal seismic coefficient (k_h) used in the calculation of the seismic active pressure coefficient is taken as 1.0 times the PGA. For structures which allow lateral yielding, (k_h) is taken as 0.5 times the PGA.
- The following seismic active pressure coefficients (K_{AE}) for the two backfill cases (Case (a) and Case (b)) may be used in design. It should be noted that these seismic earth pressure coefficients assume that the back of the wall is vertical and the ground surface behind the wall is flat. Where sloping backfill is present above the top of the wall, the lateral earth pressures under seismic loading conditions should be calculated by treating the weight of the backfill located above the top of the wall as a surcharge.

■ Seismic Active Pressure Coefficients, K_{AE}

Wall Type	Design Earthquake	Site PGA	SSM	K_{AE} for Granular 'A'	K_{AE} for Granular 'B' Type II
Yielding Wall	475 Year	0.099	0.37	0.30	0.30
	975 Year	0.159	0.39	0.32	0.32
	2,475 Year	0.275	0.43	0.36	0.36
Non-Yielding Wall	475 Year	0.099	0.40	0.33	0.33
	975 Year	0.159	0.45	0.38	0.38
	2,475 Year	0.275	0.57	0.48	0.48

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

$$\sigma_h(d) = K_a \gamma d + (K_{AE} - K_a) \gamma (H-d), \text{ yielding walls}$$

$$\sigma_h(d) = K_o \gamma d + (K_{AE} - K_o) \gamma (H-d), \text{ non-yielding walls}$$

Where: $\sigma_h(d)$ is the (static plus seismic) lateral earth pressure at depth, d , (kPa);

K_a is the static active earth pressure coefficient

K_o is the static at-rest earth pressure coefficient

K_{AE} is the seismic active earth pressure coefficient

γ is the unit weight of the backfill soil (kN/m³), as given previously

d is the depth below the top of the wall (m)

H is the total height of the wall (m)

6.6.3 Considerations for EPS Light Weight Embankment Fill

As discussed in Section 6.7.3 of this report, the use of expanded polystyrene (EPS) Geofoam light weight embankment fill is proposed as one of the alternatives for mitigating settlement of the existing and widened approach embankments due to compression of the underlying silty clay deposit. Further details on the placement of the EPS backfill are provided in Section 6.7.3, however, an estimated thickness of about 4 m of EPS would be required in the areas of the widening.

In regards to the lateral earth pressures, the low unit weight and relatively high mechanical strength characteristics of the EPS blocks (in comparison to soil) will alter the design lateral earth pressures. For design purposes, the EPS could be assumed to have a unit weight of 1 kN/m³; this low unit weight could be considered in the calculation of the vertical stress level in the underlying granular backfill, and thus the horizontal lateral pressure applied to the abutment wall. Furthermore, because the EPS blocks would hold a vertical face without support, the lateral earth pressure applied by the EPS itself could be quite minor, resulting only from the resistance to lateral expansion of the material under vertical loading (i.e., from the 'poisson' effect), which is however difficult to quantify (and highly dependent on how tightly fitting the EPS blocks are placed against the

abutment). It is therefore considered that the lateral earth pressures from the EPS can be neglected. Where the backfill is relied upon to provide passive resistance to the abutment, the contribution of the EPS itself should also be neglected, but the effect of the lower unit weight and lower vertical stress level must be considered in assessing the passive resistance from the underlying backfill.

6.7 Approach Embankment Design and Construction

It is understood that the existing approach embankments will be widened towards the median of Highway 417. It is also understood that the existing approach embankment side slopes will be widened by about 6 to 8 m at the crest and by about 2 m at the toe, with grade raises of about 1 to 4 m above the existing ground surface. It is also understood that conventional 2H:1V embankment side slopes are proposed for the widenings.

Based on the borehole results, the embankment widening subgrade soils will consist of embankment/grade fill and/or native silty clay. The embankment/grade fill generally consists of varying compositions of the sand and gravel (i.e., gravelly sand, sandy gravel, sand and gravel, silty sand, and sand). The silty clay deposit is underlain by glacial till and shale bedrock.

6.7.1 Subgrade Preparation and Embankment Construction

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

Any new embankment fill for the approach embankments should be placed and compacted in accordance with OPSS.PROV 206 (*Grading*) and OPSS.PROV 501 (*Compacting*). Benching of the existing embankment side slopes should be carried out to “key in” the new fill materials in areas where the embankment is widened, in accordance with OPSD 208.010 (*Benching of Earth Slopes*).

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

With embankment heights greater than 8 m and slopes of 2H:1V, a higher risk of surface erosion is present. Possible measures to mitigate this would include a 2 to 3 m mid-slope bench, rip rap surfacing of the lower slope or other permanent erosion protection measures.

6.7.2 Approach Embankment Stability

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

With appropriate subgrade preparation and proper placement of earth or granular soils, the up to 11 m high approach embankments, with side slopes maintained at 2H:1V, founded on the native soils, will have a factor of safety greater than 1.3 against deep seated slope instability and a factor of safety greater than 1.1 against deep-seated global instability under seismic loading, based on an acceleration of 0.15g (which corresponds to half the PGA, as per the CHBDC). The results do however indicate that some shallow sloughing (with factors of safety less than 1.1) could occur of the embankment side slopes during seismic loading. That sloughing would not however impair the short term use of the structure and is mainly a maintenance/repair issue. The potential for sloughing could be reduced by providing well vegetated side slopes, as mentioned above in Section 6.7.1. The results of the slope stability analysis are provided in Figures G1 to G3 in Appendix G.

The following soil parameters were used in the stability analysis:

Soil Type	Bulk Unit Weight, γ (kN/m ³)	Shear Strength Parameters		
		Undrained Shear Strength, c_u (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)	Effective Cohesion, c' (kPa)
New Embankment Fill (Granular)	22.0	NA	34	0
Existing Embankment Fill	21.0	NA	32	0
Weathered Silty Clay	18.0	80	35	5
Unweathered Silty Clay	16.1 – 17.2	42 – 82	28.7	7.4
Till	23.2	NA	34	0
Bedrock	Impenetrable			

6.7.3 Approach Embankment Settlement

Settlement of the existing embankments has likely fully occurred over time since the original bridge construction in 1975. Settlement of the new approach embankment widenings, and where new fills will overlay the existing embankments, will occur as a result of compression of the new embankment fill itself, compression of the existing fill, as well as compression and consolidation of the clayey soils on which the embankments are founded.

Provided that the new embankment fill consists of granular fill, Select Subgrade Material or clean earth fill, the settlement of the new embankment fill itself (which, not including the new pavement materials, will be no more than about 4 m thick) is expected to be less than about 25 mm.

The use of granular fill for the new embankment construction would reduce the magnitude of post-construction settlement (likely to less than 15 mm), since the majority of settlement of granular fills will occur during construction.

The settlement due to compression of the existing embankment fill is considered to be comparable to or less than that of the new embankment fill materials, provided it is proof rolled prior to placement of any new fill.

The additional loading imposed by the up to about 4 m of new embankment fill would result in further consolidation settlement of the underlying compressible silty clay soils. Estimates of additional consolidation settlements were developed based on the embankment widening geometry, existing stress levels, and preconsolidation pressure profile in the silty clay soils using the software SETTLE3D.

Additional settlements induced by the new embankment fill along the existing and new piles at that abutments are expected to be more than 15 mm and as such are expected to generate downdrag loads on those piles. Such downdrag should be assessed as indicated in Section 6.4.3. Similar downdrag loads would also be generated under settlements from both the short-term preloading and surcharge loading outlined in Sections 6.7.3.2 and 6.7.3.3 below.

Settlements induced by the new embankment fill on the underlying native soils are largely a result of recompression and consolidation of the underlying silty clay soils. Up to an additional 40 to 80 mm of recompression and primary consolidation settlement within the grey silty clay deposit is expected over a period of about 5 to 10 years. That duration is based on the low hydraulic conductivity of the silty clay, which controls the rate at which pore water can be expelled and the settlement can occur. In addition, some secondary compression (i.e., creep) settlement is expected following completion of the primary consolidation. Within the typical life span of the structures, the additional secondary compression settlement is estimated to be about 10 to 15 mm.

The above settlements would be entirely differential relative to the structure widenings (which would be supported on deep foundations on bedrock). These settlement values exceed the usual values accepted by MTO for the approaches to bridges, as shown in the following table, with the exception of a distance greater than 75 m from the abutments:

Distance from Abutment (m)	Tolerable Settlement (mm)
0 to 20	< 25
20 to 50	< 50
50 to 75	< 75
> 75	< 100

Within 10 m of the abutments, it is understood that essentially negligible settlements are desired (i.e., less than 25 mm), in part because of the limited tolerance of pile supported structures to accept differential settlements.

The following settlement mitigation options have therefore been considered:

- 1) Allow the settlements to occur and periodically pad and overlay to correct the profile.
- 2) Pre-loading only.
- 3) Pre-load and surcharge the widening.
- 4) Installation of vertical wick drains.
- 5) Use of lightweight fills such as Expanded Polystyrene (EPS) or Lightweight Cellular Concrete (LCC).

Subexcavation of the silty clay is not considered a feasible option for this site given the thickness of the deposit.

Recommendations for the above options are provided in the following sections.

6.7.3.1 Allow Settlements to Occur

This option would involve allowing the roadway to settle and to accept the short-term potential impacts of the expected settlements on the roadway performance. It would be planned to pad and overlay the roadway periodically to reinstate the roadway profile. Although this option is considered to be technically feasible it is likely not appropriate considering the high volumes of traffic over these bridges. Downdrag loads on the abutment piles would also need to be taken into consideration for this option as indicated in Section 6.4.3.

6.7.3.2 Preloading

Preload site preparation treatment involves the placement of the permanent loads (typically permanent grade fills) in advance of the completion of the embankment and roadway construction to allow the subgrade soils to compress under the weight of the applied fills, thus reducing the potential post-construction settlement. Preloading treatment reduces, but does not entirely eliminate, settlement effects. Some residual, post-treatment settlement is inevitable due to the long-term secondary compression (i.e., creep) of the fine-grained subgrade soils and would be in the order of about 15 mm.

For this option, the footprint of the new embankments would be preloaded to the full widened embankment heights and allowed to settle in advance of the new lanes being paved and put in-service.

As described above, time will be required for the pore water to be expelled and the settlements to occur. Time-settlement analyses have therefore been carried out and the results of those analyses are indicated in the following table:

Residual Settlement (mm)	Preload Duration (months)
< 75 mm at 50 to 75 m	6
< 50 mm at 20 to 50 m	6 to 9
< 25 mm at 0 to 20 m	18 to 24

The results shown in the above table indicate that if these long-term post-construction settlements can be tolerated, then approximately 6 to 24 months of preload time would be required. These preload times are however only estimates. The actual duration of the preload will need to be determined based on monitoring of the ground settlements under the preload. Given the time required to achieve acceptable settlement performance of the embankments and the overall project schedule this option is considered feasible, particularly for the sections of embankment widening between 20 and 75 m from the abutments. However, if the longer estimated preload duration (i.e., of 18 to 24 months) for the sections of the embankment widening within 20 m of the abutments can't be accommodated in the project schedule then consideration can be given to adding a surcharge to the preload to accelerate the settlements as described in Section 6.7.3.3 below. Alternatively, the use of preloading in combination with wick drains or lightweight EPS fill could be considered in this area as described in Sections 6.7.3.4 and 6.7.3.5.

In the event that the preloading mitigation approach is retained within 20 m of the abutment, the existing and new abutment piles would need to be designed to accommodate downdrag loads as described in Section 6.4.3.

6.7.3.3 Preloading and Surcharging

Preload and surcharging involves the placement of a temporary fill load (in addition to the permanent fill load) to accelerate consolidation of the subgrade soils, and thus accelerate site preparation treatment. The thickness of the applied temporary surcharge varies depending on the degree to which the soil consolidation process (and thus the construction schedule) needs to be accelerated. A trade-off exists between the height or thickness of the applied preload and temporary surcharge fills, and the desired timeframe to complete site preparation treatment; a greater thickness of applied surcharge results in a shorter preload duration, but at increased cost.

For this option, the area of the embankment widenings would be preloaded/surcharged with fill and allowed to settle in advance of the new lanes being paved and put in-service.

As described above, if the longer estimated preload duration (i.e., of 12 to 24 months) for the section of the embankment widening within 20 m of the abutments can't be accommodated, a surcharge could be considered to accelerate the settlements in these areas. Additional time-settlement analyses have therefore been carried out on the basis of a 2.0 m surcharge above the design pavement level. The results of those analyses are indicated in the following table:

Surcharge Height (m)	Residual Settlement (mm)	Surcharge Duration (months)
2.0	< 25 mm at 0 to 20 m	9 to 12

The results shown in the above table indicate that if the long-term post-construction settlements can be tolerated, then approximately 9 to 12 months of preload and surcharge time would be required. This preload and surcharge time is however only an estimate. The actual duration of the preload will need to be determined based on monitoring of the ground settlements under the preload/surcharge.

In the event that the preloading/surcharging mitigation approach is retained, the existing and new abutment piles would need to be designed to accommodate downdrag loads as described in Section 6.4.3.

This option is considered the preferred mitigation approach to the embankment settlements for the area of the widening within 20 m of the abutments.

An important feature of this option is the need to place the selected surcharge for the duration of the preload time. The feasibility of that option, from a drainage and roadway geometry perspective, will need to be evaluated by the designers.

Stability analyses were carried out to evaluate the stability of the embankments with the proposed surcharge, and the resulting factors of safety were found to meet the requirements under static conditions presented in Section 6.7.2.

6.7.3.4 *Wick Drains*

Wick drains are not considered a viable option at this site given that they would need to be installed through the existing embankment fill (i.e., through the midslope bench) which consists of gravelly material that may be difficult to penetrate. In addition, given the limited depth of the clay layer that is undergoing consolidation, the relatively short preload and surcharge durations discussed above, as well as the higher cost of a wick drain installation, this option is not considered to provide advantages to a more economical preloading and surcharging mitigation approach.

6.7.3.5 *Lightweight Fills*

The use of lightweight fills such as expanded polystyrene (EPS) can be considered to reduce the stress increase on the compressible clay deposit to a level such that the embankment subgrade settlements will be within acceptable tolerances. This option could be considered in the embankment widening areas within 20 m of the abutments if the project schedule cannot accommodate the time required for preloading/surcharging or if the existing/new piles cannot tolerate the downdrag loads indicated in Section 6.4.3.

It is considered that, to achieve the needed reduction of the stress increase, either portions of or the entire embankment widenings would need to be constructed using EPS 'Geofoam' fill. To meet the previously described tolerable settlements, the following EPS thicknesses would be required:

- Abutments to 15 m back: 4.5 m thick (i.e., to the elevation of the midslope berm)
- 15 m to 25 m back: 2.5 m thick
- 25 m to 35 m back: 1.0 m thick
- Beyond 35 m back: None

The above thicknesses assume that the EPS will be placed along a vertical interface between the edge of the existing embankments and the area of the proposed widening. In addition, between 25 and 35 m from the abutments, preloading as described in Section 6.7.3.2 would be required in combination with the EPS placement in order to meet the MTO settlement criteria.

The surface of the EPS Geofoam blocks would need to be protected with a concrete slab at pavement subgrade level to distribute the traffic loads. A thickness of 125 mm is typical for the protective slab. That slab would be placed at pavement subgrade level. A sufficient pavement granular thickness is required above the EPS to limit the potential for premature icing of the roadway due to the insulating properties of the Geofoam. From that perspective, a minimum of 1 m combined thickness of granular base and subbase should be planned. The EPS blocks should also be provided with approximately 1 m of soil cover along the embankment side slopes.

The EPS is potentially soluble in hydrocarbons. To guard against dissolution of the EPS in the case of an accidental release and infiltration of fuel (such as could occur in the case of a collision), it is general practice to cover the outside surface of the EPS with polyethylene sheeting.

A suitable Geofoam type would be EPS22 in accordance with ASTM D6817-02, having a compressive strength at 5% strain of at least 115 kilopascals.

A 0.3 metre thick layer of OPSS.PROV 1010 (*Aggregates*) Granular A would be appropriate as a levelling pad beneath the EPS Geofoam, covered with up to 100 mm of mortar sand.

Special frost taper treatments would be required at the longitudinal and transverse limits of the EPS to avoid severe differential frost heaving of the pavement surface. The longitudinal treatment could consist of the sub-excavation of the subgrade soils beneath the end of the EPS to a depth of 1.8 m below pavement grade (the design frost depth), replacement with compacted Granular B Type II, and the construction of a longitudinal 20H:1V frost taper beyond the edge of the EPS, in a manner similar to OPSD 205.060.

It will not be practical to provide a similar transverse frost taper treatment, between the EPS fill under the widening and the frost susceptible subgrade of the existing lanes. It is therefore proposed that the subgrade beneath the existing lanes be insulated (for example, using a 25 mm thickness of rigid high-density insulation), such that neither the new or existing lanes will experience frost heaving. This would apply adjacent to widenings where EPS is present within 1.8 m of its finished paved surface.

An NSSP should be included in the contract documents for the supply and installation of EPS fill as provided in Appendix H of this report.

Given the relatively short preload and surcharge durations discussed above and the much higher cost of a lightweight fill mitigation, this option while feasible is not considered to provide advantages to a more economical preloading and surcharging mitigation approaches.

Other light weight fill materials could also be considered, such as lightweight cellular concrete (LCC). However, it is considered that, in this case, the unit weights of these materials are not sufficiently low to achieve the needed reductions in the final stress level.

6.7.3.6 Settlement Monitoring

Settlement monitoring of the embankments widenings during construction should be carried out by means of settlement rods. The settlement rods should be installed along Highway 417 within the midslope bench of the existing embankments (i.e., in the area of the widenings) at distances of about 5, 15, 30, 50 and 75 m from each abutment along the embankments. The settlement rods should consist of 19 mm diameter steel rods cemented in place within 100 mm diameter pre-excavated holes. The settlement rods should be surveyed at regular intervals including baseline readings before construction, daily readings during construction, and twice per week for one month following construction, and every second week up to eleven months after completion of embankment widening construction.

The installation and monitoring should be as per the NSSP for the settlement monitoring provided in Appendix H.

6.8 Construction Considerations

6.8.1 Open-Cut Excavations

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

The construction of the new pile cap extensions will require excavations up to about 2 to 4 m below the existing grade within the existing embankment fill and/or native weathered silty clay. The existing fill and weathered silty clay would be classified as Type 3 soils according to OHSA. Temporary excavations (i.e. those which are only open for a relatively short period) through these overburden soils should be made with side slopes no steeper 1H:1V.

6.8.2 Temporary Protection Systems

It is anticipated that temporary protection systems would likely be required to facilitate the excavation to pile cap level for the widening and construction of the piers adjacent to the CNR tracks as well as along EBL and WBL of Highway 417 for the widening and construction of the abutments.

The design of the shoring will be entirely the responsibility of the contractor. Where required, temporary protection systems shall be designed and constructed in accordance with OPSS.PROV 539 (*Temporary Protection Systems*) with amendments as per SSP 109S09, and the lateral movement should meet Performance Level 2. Traffic loading (either vehicle or train loading) should be included as a surcharge.

It is considered that soldier piles and lagging or an interlocking sheetpile system would be feasible at this site although the presence of cobbles or boulders within the till may impact the depth that sheetpiling can be driven and the effectiveness of the system. The presence of such obstructions is discussed further in Section 6.8.4 and a NSSP is provided in Appendix H, for inclusion in the Contract Documents to alert the Contractor to these conditions. The soldier pile and lagging or sheetpiling shall be supported against lateral movement using walers, tie backs and/or internal struts/braces.

Some settlement of the roadway surface at the edge of the temporary protection can be expected and may require some minor pavement padding at this boundary.

6.8.3 Groundwater and Surface Water Control

The groundwater level at the site is generally at the interface between the embankment fill and the native silty clay deposit (i.e. between about Elevations 64.5 and 66 m). Therefore, excavations for the construction of the abutment and pier foundations will likely involve minimal groundwater and surface water control. It should be possible to handle ground and surface water inflows by pumping from well filtered sumps established in the floor of the excavations. However, the selection and design of dewatering system is the responsibility of the contractor and should be carried out in accordance with OPSS.PROV 517 (*Dewatering*) with amendments as per SSP 517F01. Given the groundwater and soil conditions at this site, dewatering is expected to be of low complexity and it is therefore not a requirement to carry out a preconstruction survey or to require a dewatering design engineer for the dewatering system as per Table A of SP 517F01 (*Dewatering System*).

Dewatering should be carried out in accordance with NSSP FOUND003 (*Dewatering Structure Excavations*), which is an amendment to OPSS 902 (*Excavation and Backfill – Structures*).

Surface water should be directed away from the excavation areas, to prevent ponding of water that could result in disturbance and weakening of the subgrade.

6.8.4 Obstructions

It should be noted that obstructions (inferred as cobbles and boulders) were encountered within the fill and till deposit in the area of the proposed foundation widenings. The presence of such obstructions could affect excavation works, installation of temporary protection systems as well as construction of deep foundations. An NSSP is provided in Appendix H, for inclusion in the Contract Documents to alert the Contractor to these conditions.

6.8.5 Vibration Monitoring During Pile Driving

The proposed staged construction is to include construction of the widenings while the existing structures remain in service. It is therefore recommended that vibration monitoring be carried out during installation of piles or driven protection systems to assist in maintaining vibration levels within tolerable ranges for the existing portions of the bridges. A NSSP has been provided in Appendix H to address this requirement. Pre and post-construction surveys of the existing pile supported bridge structures are not considered necessary. However, CN Rail may have their own survey requirements.

A maximum peak particle velocity of 100 mm/s is recommended at the existing abutments and piers. The piles furthest from the existing structures should be driven first, in order to check the vibration level at the existing structures and, if necessary, alter the installation procedures for the remaining piles. It should be noted that CN Rail may have additional vibration monitoring requirements for the rail line.

6.9 Corrosion and Cement Type

Soil samples from Boreholes 17-1102, 17-1103, 17-1104, 17-1106, 17-1107, 17-1110, 17-1111, 17-1113, 17-1114, and 17-1115 were submitted to Eurofins Environment Testing for chemical analysis related to potential corrosion of exposed buried steel and potential sulphate attack on buried concrete elements (corrosion and sulphate attack). The results of the testing are attached in Appendix E, and are summarized in the table below.

The results indicate a low potential for concrete degradation due to the presence of sulphates, and that concrete made with Type GU Portland cement should be acceptable for substructures. However, the results also indicate a high potential for corrosion of exposed ferrous metal which should be considered in the design.

Summary of Corrosivity of Sample

Borehole / Sample No.	Sample Depth (m)	Sample Type	Chloride (%)	pH	Electrical Conductivity (mS/cm)	Resistivity (ohm-cm)	Sulphate (%)
17-1102 / 15	10.7 – 11.3	Soil	0.252	7.6	1.65	606	<0.01
17-1103 / 9	4.9 – 5.5	Soil	0.032	7.6	0.33	3030	<0.01
17-1104 / 10	7.3 – 7.9	Soil	0.006	8.8	0.20	5000	0.02
17-1106 / 8	6.1 – 6.7	Soil	0.219	7.6	1.97	508	0.02
17-1107 / 5	3.0 – 3.7	Soil	0.197	9.2	0.43	2330	<0.01
17-1110 / 13	11.4 – 12.0	Soil	0.006	8.3	0.49	2040	<0.01
17-1111 / 11	9.1 – 9.8	Soil	0.008	8.7	0.28	3570	0.04
17-1113 / 6	3.0 – 3.7	Soil	0.015	8.1	0.38	2630	<0.01
17-1114 / 17	11.0 – 11.6	Soil	0.008	8.6	0.29	3450	0.04
17-1115 / 9	6.1 – 6.7	Soil	0.030	8.3	1.00	1000	<0.01

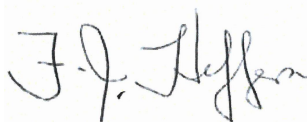
7.0 CLOSURE

This report was prepared by Mr. Alex Meacoe, P.Eng. It was reviewed by Mr. Michael Snow P.Eng., a senior geotechnical engineer and Principal with Golder. Mr. Fintan Heffernan, P.Eng., a Senior Consultant with Golder and the MTO Foundations Designated Contact, conducted an independent quality control review of this report.

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WAM/MSS/FJH/hdw

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REFERENCES

Canadian Geological Society, 2006. Canadian Foundation Engineering Manual, 4th Edition.

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.

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.

Boulanger, R.W. and Idris., 2006, "Liquefaction Susceptibility Criteria for Silts and Clays," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.132, No.11, pp. 1413-1426

Boulanger, R.W. and Idris., 2007, "Evaluation of Cyclic Softening in Silts and Clays," *Journal of Geotechnical and Geoenvironmental Engineering*, ASCE, Vol.133, No.6, pp. 641-652.

MacDonald, G. and Harrison, J.E. 1976 : Generalized Bedrock Geology, Ottawa-Hull, Ontario and Quebec, Geological Survey of Canada, Map 1508A, scale 1:125,000. Geology 1967.

Canadian Standards Association (CSA), 2014. Canadian Highway Bridge Design Code and Commentary on CAN/CSA S6-14. CSA Special Publication, S6.1-14.

Idriss, I. M., and Boulanger, R. W., 2008. "Soil Liquefaction During Earthquakes", Earthquake Engineering Research Institute.

Isenhower, W. M., Wang, S.-T., and Vasquez, L.G., 2016. Technical Manual for LPile 2016, Ensoft, Inc.

Wang, S.-T., 1982. "Development of a Laboratory Test to Identify the Scour Potential of Soils at Piles Supporting Offshore Structures," M.S. thesis, The University of Texas, Austin, Texas.

Long, J. H., 1984. "The Behaviour of Vertical Piles in Cohesive Soil Subjected to Repetitive Horizontal Loading". Ph.D. dissertation, the University of Texas, Austin, Texas.

Harpin et al., 2017. "Variation of the Stress Reduction Coefficient Between Eastern and Western Regions of North America". *Geo Ottawa 2017*

Leon et al., 2006. "Accounting for Soil Aging When Assessing Liquefaction Potential". *Journal of Geotechnical and Geoenvironmental Engineering*, 363-377

Occupational Health and Safety Act and Regulation for Construction Projects, 2017.

Natural Resources Canada (2016, February 10). Earthquake Zones in Eastern Canada, Retrieved from <http://www.seismescanada.rncan.gc.ca/zones/eastcan-en.php#WQSZ>

ASTM International:

ASTM D1586 Standard Test Method for Standard Penetration Test and Split-Barrel Sampling of Soils

Ontario Provincial Standard Specifications (OPSS)

OPSS 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS 902	Construction Specification for Excavating and Backfilling – Structures
OPSS 942	Construction Specification for Prestressed Soil and Rock Anchors

Ontario Provincial Standard Specifications (OPSS) – Provincial Oriented

OPSS.PROV 206	Construction Specification for Grading
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OPSS.PROV 501	Construction Specification for Compacting
OPSS.PROV 539	Construction Specification for Temporary Protection Systems
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes
OPSD 3101.150	Walls, Abutment, Backfill, Minimum Granular Requirement
OPSD 3090.101	Foundation Frost Penetration Depths for Southern Ontario
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement

Standard Drawings

SS103-11	Pile Driving Control
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Standard Special Provisions

SP 517F01	Dewatering System and Temporary Flow Passage System
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Non Standard Special Provisions

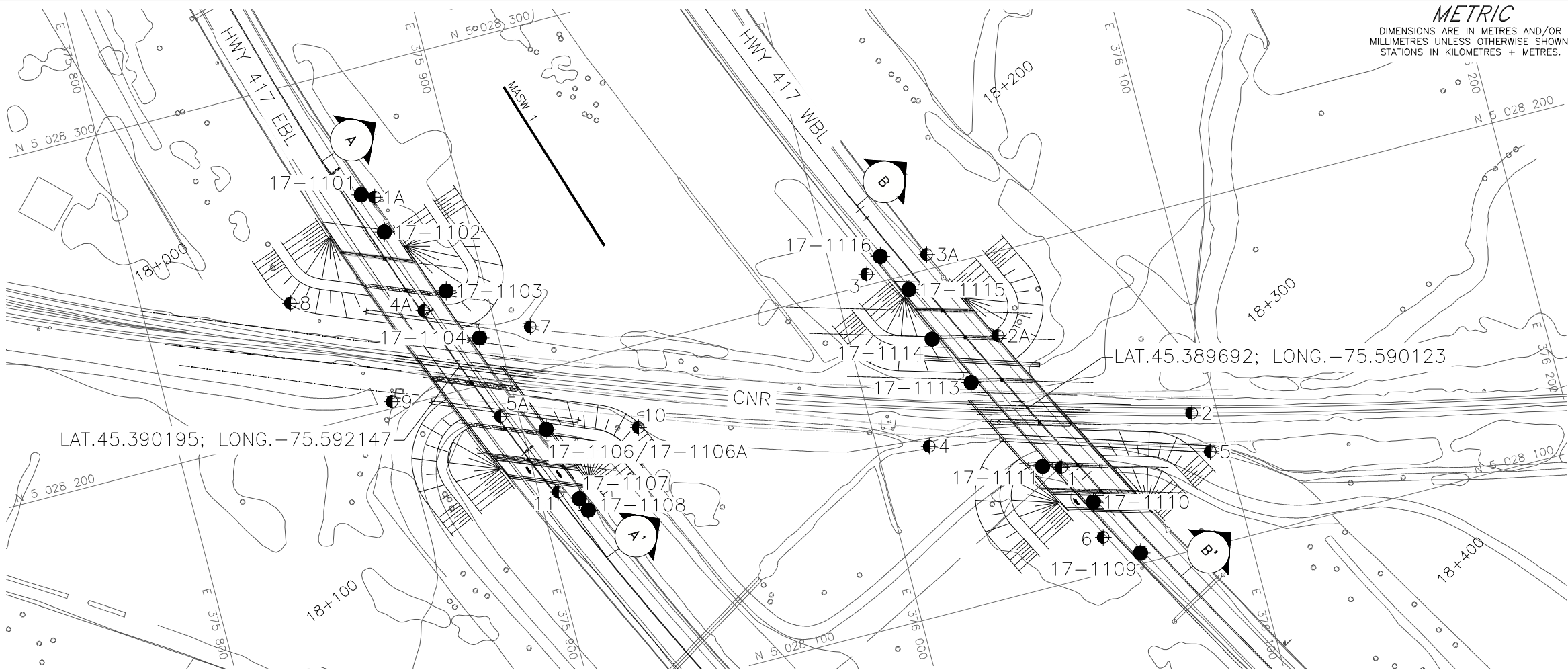
FOUND0003	Dewatering Structure Excavations
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Table 1 – Comparison of Bridge Foundation Alternatives

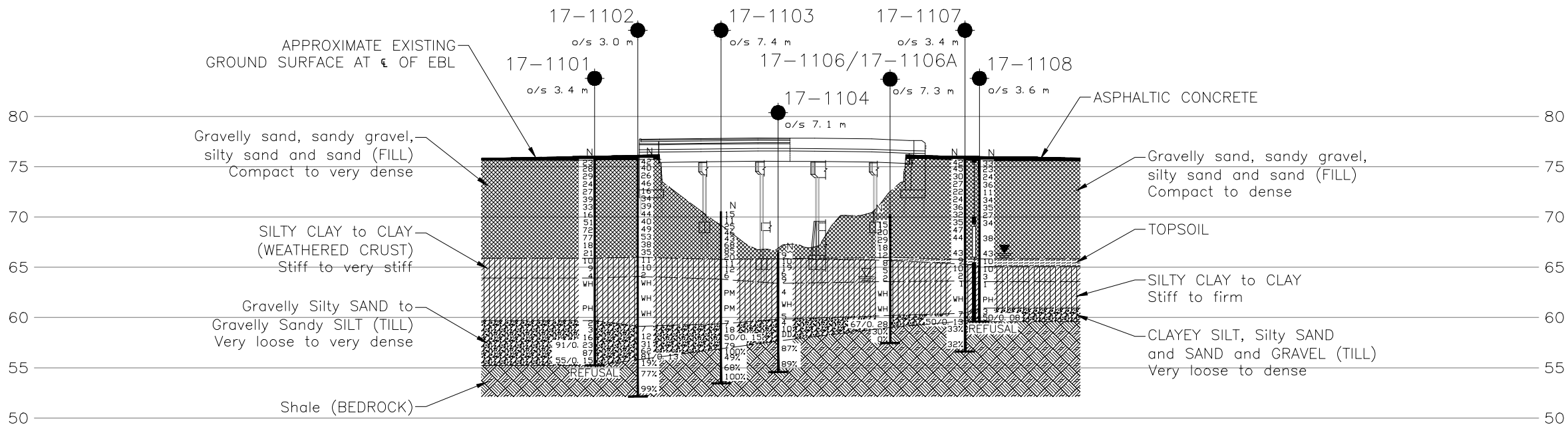
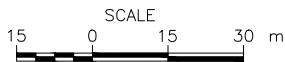
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/ Risks
Steel H-piles or pipe piles founded on the bedrock	<ul style="list-style-type: none"> Feasible Preferred option from a foundations perspective for the abutments and Piers 1 and 4. 	<ul style="list-style-type: none"> High bearing resistance Negligible settlement Compatible with existing bridge foundations Conventional construction methods for H-pile or steel pipe pile foundations Would permit the use of conventional or semi-integral abutments 	<ul style="list-style-type: none"> Possibility of encountering cobbles and boulders during installation May not provide required stiffness to resist train impact loading 	<ul style="list-style-type: none"> Less expensive than caisson option 	<ul style="list-style-type: none"> Minor potential for pile damage/deflection if cobbles and boulders are encountered during pile driving Slightly greater risk in this regard for pipe piles as compared with H-piles if boulders are encountered during pile driving
Rock socketed steel pipe piles	<ul style="list-style-type: none"> Feasible 	<ul style="list-style-type: none"> High geotechnical resistances Negligible settlement Would permit the use of conventional or semi-integral abutments May not provide required stiffness to resist train impact loading 	<ul style="list-style-type: none"> Requires specialized equipment for penetrating through the cobbles and boulders and for forming the rock socket Constrains the use of integral abutments May not provide required stiffness to resist train impact loading 	<ul style="list-style-type: none"> Rock socketing would be required 	<ul style="list-style-type: none"> Costs for rock socketed steel pipe piles higher than for steel H-piles
Auger cast piles	<ul style="list-style-type: none"> Not feasible for installation through cobbles and boulders or where uplift and high lateral resistances is required. 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A
Rock socketed drilled concrete caissons	<ul style="list-style-type: none"> Feasible Preferred option from a foundations perspective for Piers 2 and 3 	<ul style="list-style-type: none"> Higher bearing resistance Negligible settlement Higher capacity than for driven piles, so reduced number of deep foundation elements compared to piles Would provide the required stiffness to resist the expected seismic lateral loads and train impact loading resist both the expected seismic lateral loads and train impact loading 	<ul style="list-style-type: none"> Temporary liners required Possibility of encountering cobbles or boulders during installation Socketting of liner may be required to permit cleaning and inspection Coring or churn drilling will be required to form rock socket in the bedrock Due to larger diameter it may be more difficult to locate the caissons as to avoid conflict with the existing piles 	<ul style="list-style-type: none"> More expensive than steel H-pile or pipe pile option 	<ul style="list-style-type: none"> May not be able to dewater socket for cleaning and inspection
Spread/strip footings supported on native soil	<ul style="list-style-type: none"> Not feasible Inadequate bearing and excessive settlement for footings on clay subsoil. 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A 	<ul style="list-style-type: none"> N/A

Table 2 – Comparison of Settlement Mitigation Alternatives

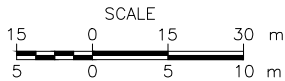
Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Constructability/ Risks
Allow embankments to settle and plan to periodically pad/overlay roadway	<ul style="list-style-type: none">Probably not feasible given high traffic volumes	<ul style="list-style-type: none">No impact on construction schedule	<ul style="list-style-type: none">Required post-construction maintenancePossible interim safety issue, between overlays, due to settlements	<ul style="list-style-type: none">Relatively low costs, but must consider post-construction maintenance costs	<ul style="list-style-type: none">Excessive roadway settlement in short term
Preloading only	<ul style="list-style-type: none">FeasiblePreferred option between 20 and 75 m from abutments	<ul style="list-style-type: none">Little post-construction maintenance required	<ul style="list-style-type: none">Delay paving for 6 to 24 months	<ul style="list-style-type: none">Slightly less expensive than surcharging, and significantly less expensive than wick drains or lightweight fill	<ul style="list-style-type: none">Some uncertainty about schedule, since cannot complete roadway construction until monitoring indicates sufficient settlement has occurred
Preloading and surcharge	<ul style="list-style-type: none">Feasible.Preferred option within 20 m of abutments	<ul style="list-style-type: none">Little post-construction maintenance required.Less delay for paving than simple preload	<ul style="list-style-type: none">Delays paving for 9 to 12 months	<ul style="list-style-type: none">Significantly less expensive than wick drains or lightweight fill	<ul style="list-style-type: none">Some uncertainty about schedule, since cannot complete roadway construction until monitoring indicates sufficient settlement has occurred
Preloading and surcharge with wick-drains	<ul style="list-style-type: none">Not feasible due to difficulties associated with installation of wick drains through the existing embankments	<ul style="list-style-type: none">N/A	<ul style="list-style-type: none">N/A	<ul style="list-style-type: none">N/A	<ul style="list-style-type: none">N/A
Lightweight fill	<ul style="list-style-type: none">Feasible	<ul style="list-style-type: none">Minimal post-construction maintenance required.Minimal impact on schedule	<ul style="list-style-type: none">Expensive	<ul style="list-style-type: none">Likely most expensive option.	<ul style="list-style-type: none">Low risk option, but contractor may successfully propose one of other options as change order



PLAN



CROSS-SECTION A-A'



NOTES

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

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

Cross-section B-B' shown on Drawing 2.

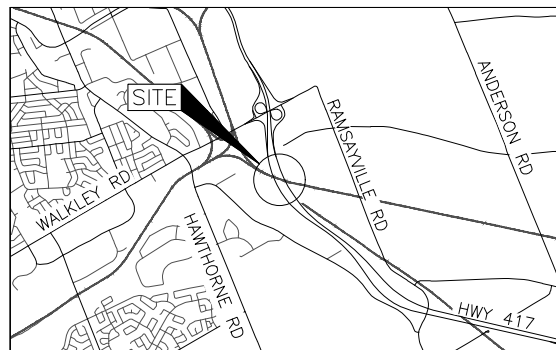
REFERENCE

Base plans provided in digital format by WSP, drawing file nos. CNR_EBL_001_GA_I_NEW_ALIGNMENT.dwg and CNR_WBL_001_GA_I.dwg, received JAN. 8, 2018.

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

CONT No.
GWP No.4145-10-00

CNR OVERHEAD WIDENING
SITE 3-301 (EBL)
HIGHWAY 417
BOREHOLE LOCATIONS AND SOIL STRATA



KEY PLAN



LEGEND

- Borehole - Current Investigation
- Borehole - Previous Investigation (Geocres No. 31G05-79)
- Seal
- Piezometer
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- DD Diamond Drilling
- 100% Rock Quality Designation (RQD)
- WL in piezometer, measured on SEPT. 24, 2017
- WL in open borehole, measured during drilling

BOREHOLE CO-ORDINATES NAD 83 (CSRS)/MTM ZONE 9			
No.	ELEVATION	NORTHING	EASTING
17-1101	75.8	5028263.4	375871.3
17-1102	75.9	5028251.0	375875.2
17-1103	70.5	5028229.6	375888.5
17-1104	66.4	5028213.6	375894.5
17-1106/17-1106A	70.3	5028182.6	375906.8
17-1107	75.8	5028160.3	375911.2
17-1108	75.7	5028156.3	375912.9
17-1109	75.9	5028103.2	376067.6
17-1110	76.0	5028121.3	376057.8
17-1111	70.4	5028135.2	376046.0
17-1113	66.9	5028164.4	376031.8
17-1114	70.4	5028179.8	376023.8
17-1115	75.9	5028195.6	376020.9
17-1116	75.9	5028133.5	376051.3
1	65.7	5028261.7	375875.0
1A	65.7	5028139.4	376092.6
2A	65.8	5028175.8	376042.9
3	66.0	5028203.1	376009.9
3A	66.1	5028204.3	376028.6
4	65.5	5028149.3	376015.1
4A	66.0	5028225.6	375880.6
5	65.8	5028127.0	376095.0
5A	65.9	5028189.7	375894.8
6	65.7	5028110.4	376058.1
7	66.0	5028213.1	375909.9
8	66.1	5028237.6	375843.1
9	66.1	5028201.9	375864.8
10	65.8	5028176.2	375933.3
11	65.9	5028163.8	375905.6







NO.	DATE	BY	REVISION
Geocres No. 31G5-288			
HWY. 417		PROJECT NO. 1662565/1110	
SUBM'D. MJK		DATE: 9/16/2019	
DRAWN: JM		SITE: .	
CHKD. MSS		APPD. FJH	
		DWG. 1	

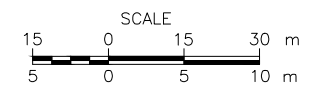




SHEET



- | | |
|---|--|
|  | Borehole — Current Investigation |
|  | Borehole — Previous Investigation
(Geocres No. 31G05–79) |
|  | Seal |
|  | Piezometer |
| N | Standard Penetration Test Value |
| 16 | Blows/0.3m unless otherwise stated
(Std. Pen. Test, 475 j/blow) |
| DD | Diamond Drilling |
| 100% | Rock Quality Designation (RQD) |
|  | WL in piezometer, measured on SEPT. 24, 2017 |
|  | WL in open borehole, measured during drilling |



Base plans provided in digital format by WSP, drawing file nos.
CNR_EBL_001_GA_I_NEW_ALIGNMENT.dwg and CNR_WBL_001_GA_I.dwg,
received JAN. 8, 2018.



APPENDIX A

Borehole and Drillhole Records, Current Investigation

Lists of Abbreviations and Symbols

Lithological and Geotechnical Rock Description Terminology

Records of Boreholes 17-1101 to 17-1104, 17-1106 to 17-1111, 17-1113 to 17-1116

Bedrock Core Photographs, Figures A1 to A22

LIST OF SYMBOLS

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

I. GENERAL		(a) Index Properties (continued)	
π	3.1416	w	water content
$\ln x$,	natural logarithm of x	w_l or LL	liquid limit
\log_{10}	x or log x, logarithm of x to base 10	w_p or PL	plastic limit
g	acceleration due to gravity	I_p or PI	plasticity index = $(w_l - w_p)$
t	time	w_s	shrinkage limit
FoS	factor of safety	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)
II. STRESS AND STRAIN		(b) Hydraulic Properties	
γ	shear strain	h	hydraulic head or potential
Δ	change in, e.g. in stress: $\Delta \sigma$	q	rate of flow
ε	linear strain	v	velocity of flow
ε_v	volumetric strain	i	hydraulic gradient
η	coefficient of viscosity	k	hydraulic conductivity (coefficient of permeability)
ν	Poisson's ratio	j	seepage force per unit volume
	total stress		
σ'	effective stress ($\sigma' = \sigma - u$)	(c) Consolidation (one-dimensional)	
σ'_{vo}	initial effective overburden stress	C	compression index (normally consolidated range)
$\sigma_1, \sigma_2, \sigma_3$	principal stress (major, minor)	C_r	recompression index (over-consolidated range)
σ_{oct}	mean stress or octahedral stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$	C_s	swelling index
τ	shear stress	C_{α}	secondary compression index
u	porewater pressure	m_v	coefficient of volume change
E	modulus of deformation	C_v	coefficient of consolidation (vertical direction)
G	shear modulus of deformation	C_h	coefficient of consolidation (horizontal direction)
K	bulk modulus of compressibility	T_v	time factor (vertical direction)
		U	degree of consolidation
III. SOIL PROPERTIES		σ'_p	pre-consolidation stress
(a) Index Properties		OCR	over-consolidation ratio = σ'_p / σ'_{vo}
$\rho(\gamma)$	bulk density (bulk unit weight)*	(d) Shear Strength	
$\rho_d(\gamma_d)$	dry density (dry unit weight)	τ_p, τ_r	peak and residual shear strength
$\rho_w(\gamma_w)$	density (unit weight) of water	ϕ'	effective angle of internal friction
$\rho_s(\gamma_s)$	density (unit weight) of solid particles	δ	angle of interface friction
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)	μ	coefficient of friction = $\tan \delta$
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)	c'	effective cohesion
e	void ratio	C_u, S_u	undrained shear strength ($\phi = 0$ analysis)
n	porosity	p	mean total stress $(\sigma_1 + \sigma_3)/2$
S	degree of saturation	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

$\tau = c' + \sigma' \tan \phi'$
shear strength = (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

Condition	N 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	Cu, Su psf
Very soft	0 to 12	0 to 250
Firm Stiff	12 to 25	250 to 500
Very stiff	25 to 50	500 to 1,000
Hard	50 to 100	1,000 to 2,000
	100 to 200	2,000 to 4,000
	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

<u>Description</u>	<u>Bedding Plane Spacing</u>
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

<u>Description</u>	<u>Spacing</u>
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

<u>Term</u>	<u>Size*</u>
Very Coarse Grained	Greater than 60 mm
Coarse Grained	2 mm to 60 mm
Medium Grained	60 microns to 2 mm
Fine Grained	2 microns to 60 microns
Very Fine Grained	Less than 2 microns
Note: * Grains greater than 60 microns diameter are visible to the naked eye.	

CORE CONDITION

Total Core Recovery (TCR)

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

Solid Core Recovery (SCR)

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

Rock Quality Designation (RQD)

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

DISCONTINUITY DATA

Fracture Index

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

Dip with Respect to Core Axis

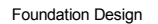
The angle of the discontinuity relative to the axis (length) of the core. In a vertical borehole a discontinuity with a 90° angle is horizontal.

Description and Notes

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

Abbreviations

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



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

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

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1101				SHEET 3 OF 3		METRIC										
G.W.P.		4145-10-00		LOCATION		N 5028263.4; E 375871.3 NAD 83 MTM ZONE 9 (LAT. 45.390703; LONG. -75.592335)				ORIGINATED BY		RI								
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NW Casing				COMPILED BY		ZS								
DATUM		Geodetic		DATE		June 25-26, 2017				CHECKED BY		WAM								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L					
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100								
			24	SS	55/0.15															
55.2																				
20.6	END OF BOREHOLE CASING REFUSAL																			

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITOHWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/18/19 ZS

PROJECT 1662565-1110		RECORD OF BOREHOLE No 17-1102		SHEET 1 OF 4		METRIC	
G.W.P. 4145-10-00		LOCATION N 5028251.0; E 375875.2 NAD 83 MTM ZONE 9 (LAT. 45.390591; LONG. -75.592288)		ORIGINATED BY RI			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		COMPILED BY ZS			
DATUM Geodetic		DATE July 5-7, 2017		CHECKED BY WAM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE LIQUID CONTENT LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)				GR	SA	SI	CL
								20	40	60	80	100	W _p	W	W _L					
75.9	GROUND SURFACE																			
75.9	ASPHALTIC CONCRETE																			
0.2	(SP) Gravelly sand (FILL) Grey Moist		1	SS	42															
75.3	(SP) Sand (FILL) Dense Brown Moist		2	SS	40															
0.6																				
74.4	(SP) Sand, some silt, trace to some gravel (FILL) Compact to dense Brown to dark brown Moist		3	SS	26															
1.5																				
			4	SS	46															
			5	SS	16															
			6	SS	34															
			7	SS	39															
			8	SS	44															
			9	SS	40															
			10	SS	49															
			11	SS	53															
			12	SS	38															
66.8	(SM) Silty sand, trace gravel (FILL) Dense Brown Wet		13	SS	35															
9.2																				
66.0																				


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\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS



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

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1102				SHEET 3 OF 4		METRIC								
G.W.P.		4145-10-00		LOCATION		N 5028251.0; E 375875.2 NAD 83 MTM ZONE 9 (LAT. 45.390591; LONG. -75.592288)		ORIGINATED BY		RI								
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		COMPILED BY		ZS								
DATUM		Geodetic		DATE		July 5-7, 2017		CHECKED BY		WAM								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100						
55.8 20.1	Shale (BEDROCK) Bedrock cored from depths 20.1 m to 23.8 m For bedrock coring details refer to Record of Drillhole 17-1102		1	RC	REC 100%												RQD = 19%	
			2	RC	REC 94%													RQD = 77%
			3	RC	REC 99%													RQD = 99%
52.1 23.8	END OF BOREHOLE																	

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITOHWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/18/19 ZS

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1102

SHEET 4 OF 4

LOCATION: N 5028251.0 ;E 375875.2

DRILLING DATE: July 5-7, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Grenville Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV.		RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																													
				DEPTH (m)	RECOVERY			R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
					TOTAL CORE %					SOLID CORE %	DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	10° 10° 10° 10°	W1 W2 W3 W4 W5 W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
		BEDROCK SURFACE		55.80																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																															</

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: RI

CHECKED: WAM







GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1103		SHEET 1 OF 3		METRIC										
G.W.P.		4145-10-00		LOCATION		N 5028229.6; E 375888.5 NAD 83 MTM ZONE 9 (LAT. 45.390397; LONG. -75.592121)		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Portable Drill, NQ/BW Casing/Rotary Drill, BW Casing		COMPILED BY										
DATUM		Geodetic		DATE		August 20-21, 2017		CHECKED BY										
								WAM										
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		
70.5	0.0	GROUND SURFACE																
0.2		(SM) Silty sand, trace gravel (FILL/TOPSOIL) Dark brown Moist		1	SS	15		70										
		(SP) Sand, some gravel, trace to some silt, trace clay (FILL) Compact to dense Brown Moist		2	SS	11												
				3	SS	25		69										20 66 10 4
				4	SS	46		68										
				5	SS	43												
67.5	3.1	(SM) Gravelly silty sand (FILL) Very dense to compact Brown Moist		6	SS	68		67										
				7	SS	85												
				8	SS	20		66										
65.8	4.7	(CI/CH) SILTY CLAY to CLAY, trace to some sand (WEATHERED CRUST) Very stiff Brown		9	SS	11		65										
				10	SS	12												
				11	SS	6		64										
63.8	6.7	(CI/CH) SILTY CLAY to CLAY, trace sand Stiff to firm Grey						63										
				12	SS	PM		62										
				13	SS	PM		61										

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT		RECORD OF BOREHOLE No 17-1103				SHEET 2 OF 3		METRIC								
G.W.P. 1662565-1110		LOCATION N 5028229.6; E 375888.5 NAD 83 MTM ZONE 9 (LAT. 45.390397; LONG. -75.592121)				ORIGINATED BY RI										
DIST Eastern HWY 417		BOREHOLE TYPE Portable Drill, NQ/BW Casing/Rotary Drill, BW Casing				COMPILED BY ZS										
DATUM Geodetic		DATE August 20-21, 2017				CHECKED BY WAM										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
--- CONTINUED FROM PREVIOUS PAGE ---																
60.2	(CL-ML) SILTY CLAY to CLAYEY SILT, some sand Firm Grey Wet															
10.3																
59.4	(SM) Gravelly Silty SAND, some clay, contains shale fragments, cobbles and boulders (TILL) Loose to compact Black Wet		14	SS	7											25 39 24 12
11.2																
58.1	(SM) Gravelly Silty SAND, contains cobbles and boulders (TILL) Very dense Dark brown Wet		15	SS	18											
12.4																
56.9	Shale (BEDROCK) Bedrock cored from depths of 13.6 m to 17.1 m For bedrock coring details refer to Record of Drillhole 17-1103		16	SS	50/0.15											RQD = 100%
13.6																
			17	RC	DD											
			18	SS	79											
53.4			19	RC	DD											RQD = 49%
			1	RC	REC 100%											
			2	RC	REC 90%											
			3	RC	REC 91%											
17.1	END OF BOREHOLE		4	RC	REC 100%											RQD = 68%
53.4																
17.1																

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1103

SHEET 3 OF 3

LOCATION: N 5028229.6 ;E 375888.5

DRILLING DATE: August 20-21, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY																FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
				ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
							TOTAL CORE %	SOLID CORE %			DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Js	10 ⁻⁶	10 ⁻⁵	10 ⁻⁴	W1	W2		W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				
		BEDROCK SURFACE		56.89																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: RI

CHECKED: WAM


GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1104		SHEET 1 OF 3		METRIC						
G.W.P.		4145-10-00		LOCATION		N 5028213.6; E 375894.5 NAD 83 MTM ZONE 9 (LAT. 45.390253; LONG. -75.592046)		ORIGINATED BY						
DIST		Eastern HWY 417		BOREHOLE TYPE		Rotary Drill, BW Casing		COMPILED BY						
DATUM		Geodetic		DATE		September 6-8, 2017		CHECKED BY						
								WAM						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
66.4	GROUND SURFACE							20 40 60 80 100	20 40 60 80 100	25 50 75				
0.0	(SP) Sand, trace silt (FILL) Loose Brown Moist		1	SS	9									
65.8							66							
0.6	(CI/CH) SILTY CLAY to CLAY (WEATHERED CRUST) Very stiff Grey-brown Moist		2	SS	10									
			3	SS	19		65							
			4	SS	6		64							
			5	SS	9									
63.4							63							
3.1	(CI/CH) SILTY CLAY to CLAY Firm Grey Wet													
			6	SS	4									
							62							
			7	SS	WH		61							
60.3														
6.1	(CL/ML) SILTY CLAY to CLAYEY SILT Stiff Grey Wet		8	SS	5		60							
59.8														
6.6	(ML) Sandy CLAYEY SILT, some gravel, contains cobbles and boulders (TILL) Loose Dark grey Wet		9	SS	4		59							
			10	SS	10									
			11	RC	DD		58							
57.6														
8.8	Shale (BEDROCK) Bedrock cored from depths of 8.8 m to 11.9 m For bedrock coring details refer to Record of Drillhole 17-1104		1	RC	REC 100%		57							

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT		RECORD OF BOREHOLE No 17-1104				SHEET 2 OF 3		METRIC									
G.W.P. 1662565-1110		LOCATION N 5028213.6; E 375894.5 NAD 83 MTM ZONE 9 (LAT. 45.390253; LONG. -75.592046)				ORIGINATED BY DG											
DIST Eastern HWY 417		BOREHOLE TYPE Rotary Drill, BW Casing				COMPILED BY ZS											
DATUM Geodetic		DATE September 6-8, 2017				CHECKED BY WAM											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100						
	Shale (BEDROCK)		1	RC	REC 100%												RQD = 87%
	Bedrock cored from depths of 8.8 m to 11.9 m For bedrock coring details refer to Record of Drillhole 17-1104		2	RC	REC 100%												RQD = 89%
54.5																	
11.9	END OF BOREHOLE																

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1104

SHEET 3 OF 3

LOCATION: N 5028213.6 ;E 375894.5

DRILLING DATE: September 6-8, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS °	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr		Ja	W1	W2	W3	W4	W5		W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
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DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: DG

CHECKED: WAM




GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS



N:\ACTIVE\SPATIAL IM\TOIHWY417REHAB&WIDENING\02 DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

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

PROJECT		RECORD OF BOREHOLE No 17-1106A				SHEET 2 OF 3		METRIC									
G.W.P. 1662565-1110		LOCATION N 5028182.6; E 375906.8 NAD 83 MTM ZONE 9 (LAT. 45.389973; LONG. -75.591894)				ORIGINATED BY DG											
DIST Eastern HWY 417		BOREHOLE TYPE Rotary Drill, NW Casing				COMPILED BY ZS											
DATUM Geodetic		DATE August 23-24, 2017				CHECKED BY WAM											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100						
60.2	(SP/GP) SAND and GRAVEL, some silt, trace to some clay, contains shale fragments, cobbles and boulders (TILL) Very dense Grey Wet																
59.1	Shale (BEDROCK)		1	RC	REC 100%												RQD = 28%
11.2	Bedrock cored from depths of 11.2 m to 13.2 m For bedrock coring details refer to Record of Drillhole 17-1106A		2	RC	REC 92%												RQD = 16%
57.1	END OF BOREHOLE																
13.2	Note(s): 1. Soil stratigraphy is inferred from Record of Borehole 17-1106.																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS




PROJECT 1662565-1110		RECORD OF BOREHOLE No 17-1106		SHEET 1 OF 3		METRIC	
G.W.P. 4145-10-00		LOCATION N 5028182.6; E 375906.8 NAD 83 MTM ZONE 9 (LAT. 45.389973; LONG. -75.591894)		ORIGINATED BY DG			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem) Rotary Drill, NW Casing		COMPILED BY ZS			
DATUM Geodetic		DATE August 22-23, 2017		CHECKED BY WAM			

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 (%)				GR	SA	SI	CL
								20	40	60	80	100	20	40	60		80	100		
70.3	GROUND SURFACE																			
0.0	(SP) Sand, trace gravel (FILL) Compact Brown Moist																			
			1	SS	15															
			2	SS	20															
			3	SS	29															
67.3	(SM) Gravelly silty sand, trace clay (FILL) Compact Brown Moist		4	SS	18															
			5	SS	12															
66.0	(SM) Silty sand, contains organic matter (TOPSOIL) Black Moist																			
4.3																				
65.7	(CI/CH) SILTY CLAY to CLAY (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist		6	SS	8															
4.6																				
			7	SS	5															
			8	SS	2															
63.6	(CL/CH) SILTY CLAY to CLAY Stiff Grey Wet																			
6.7																				
			9	SS	WH															
			10	SS	WH															

Continued Next Page

+³, X³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT		RECORD OF BOREHOLE No 17-1106				SHEET 2 OF 3		METRIC									
G.W.P. 1662565-1110		LOCATION N 5028182.6; E 375906.8 NAD 83 MTM ZONE 9 (LAT. 45.389973; LONG. -75.591894)				ORIGINATED BY DG											
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem) Rotary Drill, NW Casing				COMPILED BY ZS											
DATUM Geodetic		DATE August 22-23, 2017				CHECKED BY WAM											
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa									WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						20	40	60	80	100						
60.2	(SP/GP) SAND and GRAVEL, some silt, trace to some clay, contains shale fragments, cobbles and boulders (TILL) Very dense Grey Wet		11	SS	67/0.28												38 36 18 8
59.1	Shale (BEDROCK)		1	RC	REC 100%												RQD = 30%
57.5	END OF BOREHOLE		2	RC	REC 5%												RQD = 0%
12.8	NOTES: 1. Water level in open borehole at a depth of 6.2 m below ground surface (Elev. 64.1), measured during drilling.																

SHEET 3 OF 3

DATUM: Geodetic

DRILLING CONTRACTOR: CCC

[illegible]

DEPTH SCALE

1 : 50



GOLDER


LOGGED: DG

CHECKED: WAM

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS



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

PROJECT		RECORD OF BOREHOLE				No 17-1107		SHEET 2 OF 3		METRIC							
G.W.P. 1662565-1110		LOCATION		N 5028160.3; E 375911.2 NAD 83 MTM ZONE 9 (LAT. 45.389772; LONG. -75.591841)				ORIGINATED BY DG									
DIST Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core				COMPILED BY ZS									
DATUM Geodetic		DATE		June 25-26, 2017				CHECKED BY WAM									
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
--- CONTINUED FROM PREVIOUS PAGE ---																	
65.7 10.1	(CI/CH) Silty clay, contains organic matter (TOPSOIL) Dark grey Moist		13	SS	9												
65.3 10.5	(CI/CH) SILTY CLAY to CLAY (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist		14	SS	10												
			15	SS	2												
63.6 12.2	(CI/CH) SILTY CLAY to CLAY, contains silt seams Stiff to firm Grey Wet		16	SS	1												
			17	SS	WH												
60.4 15.4	(SM) Silty SAND, some gravel, contains shale fragments, cobbles and boulders (TILL) Loose to dense Dark grey Wet		18	SS	7												
59.7 16.2	Shale (BEDROCK) Bedrock cored from depths 16.2 m to 19.2 m For bedrock coring details refer to Record of Drillhole 17-1107		19	SS	50/0.13												
			1	RC	REC 100%												RQD = 33%
			2	RC	REC 100%												RQD = 32%
56.6 19.2	END OF BOREHOLE																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1107

SHEET 3 OF 3

LOCATION: N 5028160.3 ;E 375911.2

DRILLING DATE: June 25-26, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Grenville Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
						TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Js	10 ⁻⁵	10 ⁻⁴	10 ⁻³	10 ⁻²	W1	W2		W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
		BEDROCK SURFACE		59.67																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					

DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: DG

CHECKED: WAM

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1108		SHEET 1 OF 2		METRIC						
G.W.P.		4145-10-00		LOCATION		N 5028156.3; E 375912.9 NAD 83 MTM ZONE 9 (LAT. 45.389736; LONG. -75.591820)		ORIGINATED BY						
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY						
DATUM		Geodetic		DATE		June 25-26, 2017		CHECKED BY						
								WAM						
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p	W	W _L		
75.7	GROUND SURFACE													
0.0	ASPHALTIC CONCRETE													
0.3	(SP) Gravelly sand (FILL) Grey Moist		1	SS	33									
75.1	(GP) Sandy gravel (FILL) Grey Moist		2	SS	23									
0.6	(SP) Sand (FILL) Compact Brown Moist		3	SS	24									
73.9	(GP) Sandy gravel, some silt (FILL) Compact to dense Grey Moist		4	SS	36									
72.7	(SM/SP) Layered silty sand and sand (FILL) Compact Brown Moist		5	SS	11									
72.0	(GP) Sandy gravel (FILL) Compact Grey Moist		6	SS	34									
71.1	(SP) Sand (FILL) Compact Brown Moist		7	SS	35									
70.4	(GP) Sandy gravel (FILL) Compact Grey Moist		8	SS	27									
69.6	(SM) Sand, some gravel and silt (FILL) Compact Brown Moist		9	SS	34									
67.4	(SM) Silty sand (FILL) Compact Grey Wet		10	SS	38									
65.8			11	SS	43									

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMT\Hwy417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

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



GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

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

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1109		SHEET 2 OF 2		METRIC					
G.W.P.		4145-10-00		LOCATION		N 5028103.2; E 376067.6 NAD 83 MTM ZONE 9 (LAT. 45.389242; LONG. -75.589852)		ORIGINATED BY					
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)		COMPILED BY					
DATUM		Geodetic		DATE		June 29, 2017		CHECKED BY					
								WAM					
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 ---												
65.5			10	SS	33								
10.4	(Cl) Silty clay, contains organic matter (TOPSOIL)												
65.2	Dark grey												
10.7	(Cl/CH) SILTY CLAY to CLAY (WEATHERED CRUST)		11	SS	10								
	Very stiff												
	Grey-brown												
	Moist												
			12	SS	9								
			13	SS	2								
62.8	(Cl/CH) SILTY CLAY to CLAY												
13.1	Stiff to firm												
	Grey												
	Wet												
			14	SS	1								
60.5	(SM) Silty SAND, some gravel, contains cobbles and boulders		15	TP	PH								
15.4	(TILL)												
	Grey												
	Wet												
59.9	END OF BOREHOLE		16	SS	50/0.03								
16.0	AUGER REFUSAL												
NOTES: 1. Water level in open borehole at a depth of 14.9 m below ground surface (Elev. 61.0), measured during drilling.													

PROJECT 1662565-1110		RECORD OF BOREHOLE No 17-1110		SHEET 1 OF 4		METRIC	
G.W.P. 4145-10-00		LOCATION N 5028121.3; E 376057.8 NAD 83 MTM ZONE 9 (LAT. 45.389406; LONG. -75.589974)		ORIGINATED BY DG			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ Core		COMPILED BY ZS			
DATUM Geodetic		DATE July 3-5, 2017		CHECKED BY WAM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL LIMIT MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		GR	SA	SI	CL
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED						WATER CONTENT (%)						
76.0	GROUND SURFACE						20	40	60	80	100									
0.0	ASPHALTIC CONCRETE																			
0.2	(SP) Gravelly sand (FILL) Very dense Grey Moist		1	SS	58															
75.3	(SP) Sand, trace to some silt, trace gravel (FILL) Compact to very dense Brown Moist		2	SS	31															
0.7																				
			3	SS	31															
			4	SS	29															
			5	SS	20															
			6	SS	35															
			7	SS	28															
			8	SS	51															
			9	SS	39															
66.9	(SM) Silty sand (FILL) Dense to loose Brown Moist		10	SS	35															
9.1																				

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTOWHY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS



PROJECT 1662565-1110		RECORD OF BOREHOLE No 17-1110		SHEET 2 OF 4		METRIC	
G.W.P. 4145-10-00		LOCATION N 5028121.3; E 376057.8 NAD 83 MTM ZONE 9 (LAT. 45.389406; LONG. -75.589974)		ORIGINATED BY DG			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ Core		COMPILED BY ZS			
DATUM Geodetic		DATE July 3-5, 2017		CHECKED BY WAM			

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 (%)				
								20	40	60	80	100	25	50	75		
65.6 10.4 65.3 10.7	(ML) Sandy silt, contains organic matter (FILL) Dark grey Wet (Cl/CH) SILTY CLAY to CLAY (WEATHERED CRUST) Very stiff Grey-brown Moist		11	SS	9											CHEM	
			12	SS	9												
			13	SS	8												
			14	SS	1												
62.9 13.1	(Cl/CH) SILTY CLAY to CLAY Stiff Grey Wet																
			15	TP	PH												
61.1 14.9	(ML/SP) SILT and SAND, some gravel and clay (TILL) Loose Dark grey Wet																
			16	SS	5												
59.8 16.2	Shale (BEDROCK) Bedrock cored from depths 16.2 m to 21.9 m For bedrock coring details refer to Record of Drillhole 17-1110		1	RC	REC 100%												
			2	RC	REC 100%												
			3	RC	REC 100%												

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTD\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT		RECORD OF BOREHOLE No 17-1110				SHEET 3 OF 4		METRIC									
G.W.P. 4145-10-00		LOCATION N 5028121.3; E 376057.8 NAD 83 MTM ZONE 9 (LAT. 45.389406; LONG. -75.589974)				ORIGINATED BY DG											
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, HQ Core				COMPILED BY ZS											
DATUM Geodetic		DATE July 3-5, 2017				CHECKED BY WAM											
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100					
	Shale (BEDROCK) Bedrock cored from depths 16.2 m to 21.9 m For bedrock coring details refer to Record of Drillhole 17-1110		3	RC	REC 100%		55										RQD = 68%
			4	RC	REC 100%												
54.1 21.9	END OF BOREHOLE NOTES: 1. Water level in open borehole at a depth of 14.5 m below ground surface (Elev. 61.5), measured during drilling.																

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1110

SHEET 4 OF 4

LOCATION: N 5028121.3 ;E 376057.8

DRILLING DATE: July 3-5, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Grenville Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																								
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA				HYDRAULIC CONDUCTIVITY K, cm/sec		WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																												
							TOTAL CORE %	SOLID CORE %			DIP w.r.t CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	W1	W2	W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
		BEDROCK SURFACE		59.80																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									

DEPTH SCALE

1 : 50



LOGGED: DG

CHECKED: WAM

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS

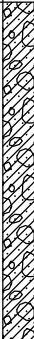

PROJECT 1662565-1110		RECORD OF BOREHOLE No 17-1111		SHEET 1 OF 3		METRIC	
G.W.P. 4145-10-00		LOCATION N 5028135.2; E 376046.0 NAD 83 MTM ZONE 9 (LAT. 45.389532; LONG. -75.590123)		ORIGINATED BY DG			
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NW Casing		COMPILED BY ZS			
DATUM Geodetic		DATE August 24-25, 2017		CHECKED BY WAM			

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
							20	40	60	80	100						
70.4	GROUND SURFACE																
0.0	(SM) Gravelly silty sand, trace clay (FILL) Compact to dense Brown Moist																
			1	SS	20												
			2	SS	32												
			3	SS	30												
67.4	(SM) Silty sand, trace gravel (FILL) Compact Brown Moist																
3.1			4	SS	15												
			5	SS	24												
65.8	(CI/CH) SILTY CLAY to CLAY (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist																
4.6			6	SS	10												
			7	SS	10												
			8	SS	6												
			9	SS	2												
62.9	(CL/CH) SILTY CLAY to CLAY Stiff to firm Grey Wet																
7.5																	
			10	SS	1												
61.3	(ML/SP) SILT and SAND, some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Dark grey Wet																
9.1			11	SS	18												

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMITO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT		RECORD OF BOREHOLE				No 17-1111		SHEET 2 OF 3		METRIC								
G.W.P. 1662565-1110		LOCATION		N 5028135.2; E 376046.0 NAD 83 MTM ZONE 9 (LAT. 45.389532; LONG. -75.590123)				ORIGINATED BY DG										
DIST Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NW Casing				COMPILED BY ZS										
DATUM Geodetic		DATE		August 24-25, 2017				CHECKED BY WAM										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
--- CONTINUED FROM PREVIOUS PAGE ---								20	40	60	80	100						
58.2 12.2	(ML/SP) SILT and SAND, some gravel and clay, contains cobbles and boulders (TILL) Compact to very loose Dark grey Wet		12	SS	WH												15 43 30 12	
			13	SS	79/0.28													
			14	RC	DD													
54.1 16.3	Shale (BEDROCK) Bedrock cored from depths of 12.2 m to 16.3 m For bedrock coring details refer to Record of Drillhole 17-1111		1	RC	REC 100%												RQD = 52%	
			2	RC	REC 95%													RQD = 22%
			3	RC	REC 99%													RQD = 55%
END OF BOREHOLE																		
NOTES: 1. Water level in open borehole at a depth of 8.8 m below ground surface (Elev. 64.6), measured during drilling.																		

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1111

SHEET 3 OF 3

LOCATION: N 5028135.2 ;E 376046.0

DRILLING DATE: August 24-25, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 55

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DISCONTINUITY DATA			HYDRAULIC CONDUCTIVITY K, cm/sec			WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
						TOTAL CORE %	SOLID CORE %			DIP w.r.t. CORE AXIS	TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁴	10 ⁻³	10 ⁻²	10 ⁻¹	W1	W2		W3	W4	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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DEPTH SCALE


1 : 50

**GOLDER**

LOGGED: DG

CHECKED: WAM


GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1113		SHEET 1 OF 3		METRIC																				
G.W.P.		4145-10-00		LOCATION		N 5028164.4; E 376031.8 NAD 83 MTM ZONE 9 (LAT. 45.389797; LONG. -75.590301)		ORIGINATED BY																				
DIST		Eastern HWY 417		BOREHOLE TYPE		Portable Drill, BW Casing/Rotary Drill, BQ Rod		COMPILED BY																				
DATUM		Geodetic		DATE		August 27-28, 2017		CHECKED BY																				
								WAM																				
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)														
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)													
66.9 0.0	GROUND SURFACE (SP) Sand, trace gravel, contains organic matter (FILL) Compact Brown to dark brown Moist to wet		1	SS	14	▽	66	65	64	63	62	61	60	59	58	57												
			2	SS	22																							
65.4 1.5	(CL/ML) Sandy silty clay to clayey silt, trace gravel (FILL) Grey Moist		3	SS	11																							
64.6 2.3	(CI/CH) SILTY CLAY to CLAY, trace to some sand (WEATHERED CRUST) Very stiff Grey-brown Moist		4	SS	7																							
			5	SS	14																							
			6	SS	6																							
63.2 3.7	(CI/CH) SILTY CLAY to CLAY, trace sand Stiff to firm Grey Wet																											
			7	SS	2																							
61.4 5.5	(ML) CLAYEY SILT, some sand Firm Grey Wet																											
60.8 6.1	(ML/SP) SILT and SAND, some gravel and clay, contains cobbles and boulders (TILL) Very loose to loose Dark brown Wet		8	SS	3																							
		9	SS	7																								
		10	SS	5																								
59.1 7.8	Shale (BEDROCK) Bedrock cored from depths of 7.8 m to 10.8 m For bedrock coring details refer to Record of Drillhole 17-1113	11	SS	50/0.15																								
		1	RC	REC 89%																								
		2	RC	REC 99%																								

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTTOHWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT 1662565-1110		RECORD OF BOREHOLE No 17-1113				SHEET 2 OF 3		METRIC												
G.W.P. 4145-10-00		LOCATION N 5028164.4; E 376031.8 NAD 83 MTM ZONE 9 (LAT. 45.389797; LONG. -75.590301)				ORIGINATED BY RI														
DIST Eastern HWY 417		BOREHOLE TYPE Portable Drill, BW Casing/Rotary Drill, BQ Rod				COMPILED BY ZS														
DATUM Geodetic		DATE August 27-28, 2017				CHECKED BY WAM														
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa												
	--- CONTINUED FROM PREVIOUS PAGE ---						<div style="display: flex; justify-content: space-between;"> 20 40 60 80 100 20 40 60 80 100 </div> <div style="display: flex; justify-content: space-between;"> ○ UNCONFINED + FIELD VANE </div> <div style="display: flex; justify-content: space-between;"> ● QUICK TRIAXIAL × REMOULDED </div>													
56.1			2	RC	REC 99%												RQD = 91%			
10.8	END OF BOREHOLE NOTES: 1. Water level in open borehole at a depth of 1.3 m below ground surface (Elev. 65.6), measured during drilling.																			

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMTO\HWY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1113

SHEET 3 OF 3

LOCATION: N 5028164.4 ;E 376031.8

DRILLING DATE: August 27-28, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																					
						RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS S.D.C.	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: RI

CHECKED: WAM

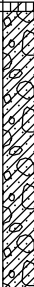

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1114		SHEET 1 OF 3		METRIC								
G.W.P.		4145-10-00		LOCATION		N 5028179.8; E 376023.8 NAD 83 MTM ZONE 9 (LAT. 45.389935; LONG. -75.590399)		ORIGINATED BY								
DIST		Eastern HWY 417		BOREHOLE TYPE		Portable Drill, NW-BW Casing/Rotary Drill, HW-BW Casing		COMPILED BY								
DATUM		Geodetic		DATE		August 29-30, 2017		CHECKED BY								
								WAM								
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
70.4	GROUND SURFACE															
0.0	(SM) Gravelly silty sand (TOPSOIL/FILL)															
0.2	Dark brown Moist		1	SS	17											
	(SP) Sand, trace to some gravel, trace silt (FILL)		2	SS	25											
	Compact to dense Brown Moist		3	SS	42											
68.6																
1.8	(SP) Gravelly sand, some silt (FILL)		4	SS	30											20 65 13 2
	Compact to dense Brown Wet		5	SS	18											
67.1																
3.4	(SP) Sand, some silt, trace clay (FILL)		6	SS	48											
	Compact to dense Grey-brown Moist		7	SS	19											
			8	SS	47											0 80 16 4
65.5																
4.9	(ML) Clayey silt, trace to some sand, contains organic matter (rootlets, wood) (FILL)		9	SS	10											
	Grey-brown Moist		10	SS	11											0 11 50 39
64.2																
6.3	(CL/CH) SILTY CLAY to CLAY, trace sand (WEATHERED CRUST)		11	SS	12											
	Very stiff Grey-brown Moist		12	SS	12											
			13	SS	9											
62.5																
7.9	(CL/CH) SILTY CLAY to CLAY															
	Stiff to firm Grey Moist to wet		14	SS	3											
60.7																
9.8																

Continued Next Page

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

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTD\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1114		SHEET 2 OF 3		METRIC										
G.W.P.		4145-10-00		LOCATION		N 5028179.8; E 376023.8 NAD 83 MTM ZONE 9 (LAT. 45.389935; LONG. -75.590399)		ORIGINATED BY										
DIST		Eastern HWY 417		BOREHOLE TYPE		Portable Drill, NW-BW Casing/Rotary Drill, HW-BW Casing		COMPILED BY										
DATUM		Geodetic		DATE		August 29-30, 2017		CHECKED BY										
								WAM										
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---							20	40	60	80	100						
66.3 10.7	(ML) CLAYEY SILT Firm Dark grey Wet		15	SS	3													
	(SM) Silty SAND, some gravel and clay, contains cobbles and boulders (TILL) Very loose to loose Dark grey Wet		16	SS	6													
			17	SS	2													
			18	SS	8													
58.4 12.0	Shale (BEDROCK) Bedrock cored from depths of 12.0 m to 15.3 m For bedrock coring details refer to Record of Drillhole 17-1114		1	RC	REC 100%													
			2	RC	REC 100%													
			3	RC	REC 100%													
55.1 15.3	END OF BOREHOLE																	

GTA-MTO 001 N:\ACTIVE\SPATIAL_IM\MTOWHY417REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1114

SHEET 3 OF 3

LOCATION: N 5028179.8 ;E 376023.8

DRILLING DATE: August 29-30, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: Portable Drill

DRILLING CONTRACTOR: CCC

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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						TOTAL CORE %	SOLID CORE %				TYPE AND SURFACE DESCRIPTION	Jr	Ja	10 ⁻⁹	10 ⁻⁸	10 ⁻⁷	10 ⁻⁶	W1	W2		W3	W5	W6																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																						
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DEPTH SCALE

1 : 50

**GOLDER**

LOGGED: DG

CHECKED: WAM

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS



GTA-MTO 001 N:\ACTIVE\SPATIAL\IMMTO\HWY417\REHAB&WIDENING\02 DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

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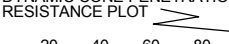

+³, ×³: Numbers refer to Sensitivity ○^{3%} STRAIN AT FAILURE

PROJECT		1662565-1110		RECORD OF BOREHOLE No 17-1115		SHEET 2 OF 4		METRIC					
G.W.P.		4145-10-00		LOCATION		N 5028195.6; E 376020.9 NAD 83 MTM ZONE 9 (LAT. 45.390079; LONG. -75.590435)		ORIGINATED BY					
DIST		Eastern HWY 417		BOREHOLE TYPE		Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		COMPILED BY					
DATUM		Geodetic		DATE		June 28 - July 4, 2017		CHECKED BY					
								WAM					
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa		W _p W W _L			
<div style="display: flex; justify-content: space-between;"> <div> <p>20 40 60 80 100</p> <p>○ UNCONFINED + FIELD VANE</p> <p>● QUICK TRIAXIAL × REMOULDED</p> <p>20 40 60 80 100</p> </div> <div> <p>25 50 75</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>--- CONTINUED FROM PREVIOUS PAGE ---</p> </div> <div> <p>65.5</p> </div> <div> <p>14 SS 20</p> </div> <div> <p>65</p> </div> <div> <p>0 5 49 46</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>10.5</p> <p>(ML) Sandy SILT, contains organic matter (rootlets) Compact Grey Moist</p> <p>(CI/CH) SILTY CLAY to CLAY, trace sand (WEATHERED CRUST) Very stiff to stiff Grey-brown Moist</p> </div> <div> <p>15 SS 14</p> </div> <div> <p>64</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>63.4</p> </div> <div> <p>16 SS 8</p> </div> <div> <p>63</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>12.5</p> <p>(CI/CH) SILTY CLAY to CLAY, trace sand Stiff Grey Moist to wet</p> </div> <div> <p>17 SS 1</p> </div> <div> <p>62</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>60.7</p> </div> <div> <p>18 SS WH</p> </div> <div> <p>61</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>15.2</p> <p>(CI/CH-ML) SILTY CLAY to CLAYEY SILT, contains silty sand seams Firm Grey Wet</p> </div> <div> <p>19 SS 1</p> </div> <div> <p>60</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>59.9</p> </div> <div> <p>20 SS WH</p> </div> <div> <p>59</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>16.0</p> <p>(ML/SP) SILT and SAND, some gravel and clay, contains cobbles and boulders (TILL) Very loose to compact Dark brown to black Wet</p> </div> <div> <p>21 SS WH</p> </div> <div> <p>58</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>57.8</p> </div> <div> <p>22 SS 18</p> </div> <div> <p>57</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>18.1</p> <p>Shale (BEDROCK)</p> <p>Bedrock cored from depths 18.1 m to 21.7 m</p> <p>For bedrock coring details refer to Record of Drillhole 17-1115</p> </div> <div> <p>1 RC REC 100%</p> </div> <div> <p>56</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>2 RC REC 100%</p> </div> <div> <p>57</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>3 RC REC 97%</p> </div> <div> <p>56</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>16 41 31 12</p> </div> <div> <p>RQD = 74%</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>RQD = 100%</p> </div> </div>													
<div style="display: flex; justify-content: space-between;"> <div> <p>RQD = 88%</p> </div> </div>													

GTA-MTO 001 N:\ACTIVE\SPATIAL_IMMT\Hwy417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-GTA.GDT 9/16/19 ZS

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

PROJECT		RECORD OF BOREHOLE No 17-1115				SHEET 3 OF 4		METRIC												
G.W.P. 1662565-1110		LOCATION N 5028195.6; E 376020.9 NAD 83 MTM ZONE 9 (LAT. 45.390079; LONG. -75.590435)		ORIGINATED BY RI																
DIST Eastern HWY 417		BOREHOLE TYPE Power Auger, 200 mm Diam. (Hollow Stem)/Rotary Drill, NQ Core		COMPILED BY ZS																
DATUM Geodetic		DATE June 28 - July 4, 2017		CHECKED BY WAM																
SOIL PROFILE			SAMPLES			DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT		REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100 WATER CONTENT (%) 25 50 75					W _p	W	W _L	γ	GR	SA	SI	CL
54.2	Shale (BEDROCK) Bedrock cored from depths 18.1 m to 21.7 m For bedrock coring details refer to Record of Drillhole 17-1115		3	RC	REC 97%		55													
21.7	END OF BOREHOLE NOTES: 1. Water level in well screen at a depth of 10.3 m below ground surface (Elev. 65.6 m), measured on Sept. 25, 2017.		4	RC	REC 96%															

PROJECT: 1662565-1110

RECORD OF DRILLHOLE: 17-1115

SHEET 4 OF 4

LOCATION: N 5028195.6 ;E 376020.9

DRILLING DATE: June 28 - July 4, 2017

DATUM: Geodetic

INCLINATION: -90° AZIMUTH: ---

DRILL RIG: CME 75

DRILLING CONTRACTOR: Grenville Drilling

DEPTH SCALE METRES	DRILLING RECORD	DESCRIPTION	SYMBOLIC LOG	ELEV. DEPTH (m)	RUN No.	FLUSH RETURN	NOTE: For abbreviations, symbols and descriptions refer to LITHOLOGICAL AND GEOTECHNICAL ROCK DESCRIPTION TERMINOLOGY														FEATURES
							RECOVERY		R.Q.D. %	FRACT. INDEX PER	DIP w.r.t CORE AXIS	DISCONTINUITY DATA		HYDRAULIC CONDUCTIVITY K, cm/sec	WEATH- ERING INDEX						
							TOTAL CORE %	SOLID CORE %				Jr	Js		W1	W2	W3	W4	W5	W6	

		BEDROCK SURFACE		57.80																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																											
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DEPTH SCALE

1 : 50



LOGGED: RI

CHECKED: WAM

GTA-RCK 031 N:\ACTIVE\SPATIAL_IMMTO\HWY417\REHAB&WIDENING\02_DATA\GINT\1662565.GPJ GAL-MISS.GDT 9/16/19 ZS



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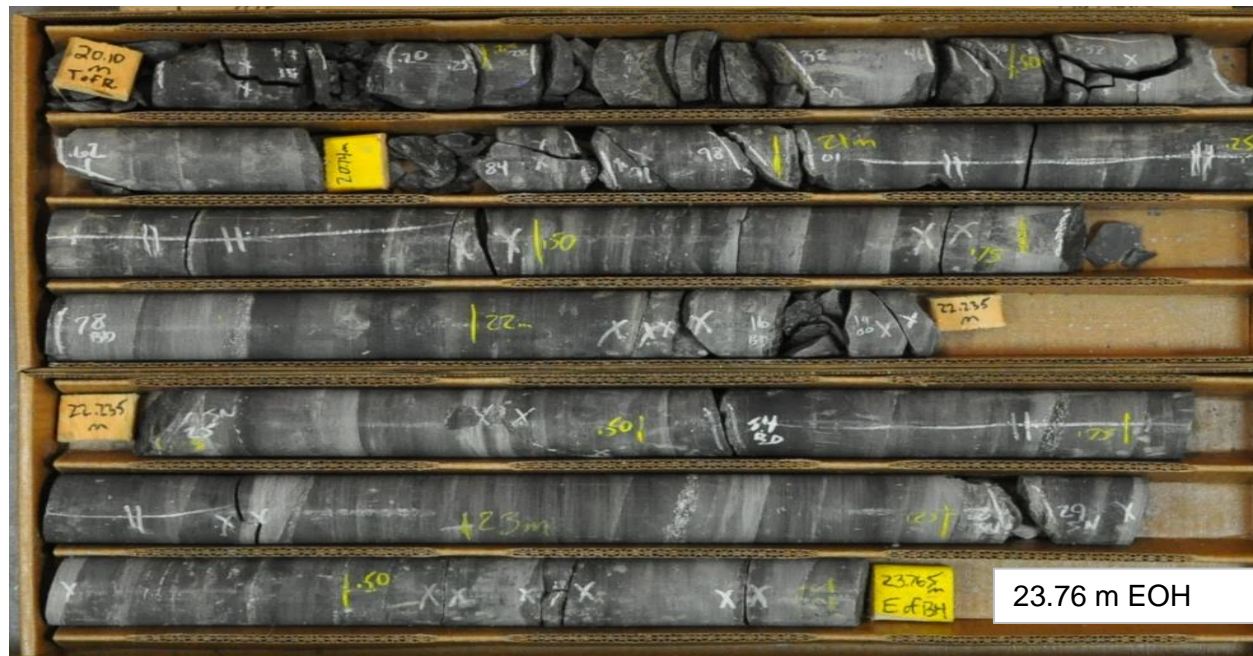
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+ 3, × 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

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

BH 17-1102 (Dry)
Cored Length of 20.10 to 23.76 metres
Core Box 1 and 2 of 2

20.10 m Top of Bedrock



23.76 m EOH



Foundation Investigation
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Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A1

BH 17-1102 (Wet)
Cored Length of 20.10 to 23.76 metres
Core Box 1 and 2 of 2

20.10 m Top of Bedrock



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Figure A2

BH 17-1103 (Dry)
Cored Length of 12.55 to 17.07 metres
Core Box 1 and 2 of 2

12.55 m

13.63 m Top of Bedrock



Note: Material in core box from 12.55 to 13.63 is gravel and cobbles in Till.



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Figure A3

BH 17-1103 (Wet)
Cored Length of 12.55 to 17.07 metres
Core Box 1 and 2 of 2

12.55 m

13.63 m Top of Bedrock



17.07 m EOH

Note: Material in core box from 12.55 to 13.63 is gravel and cobbles in Till.



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Figure A4

BH 17-1104 (Dry)
Cored Length of 8.78 to 11.87 metres
Core Box 1 and 2 of 2

8.78 m Top of Bedrock



11.87 m EOH



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Figure A5

BH 17-1104 (Wet)
Cored Length of 8.78 to 11.87 metres
Core Box 1 and 2 of 2

8.78 m Top of Bedrock



11.87 m EOH



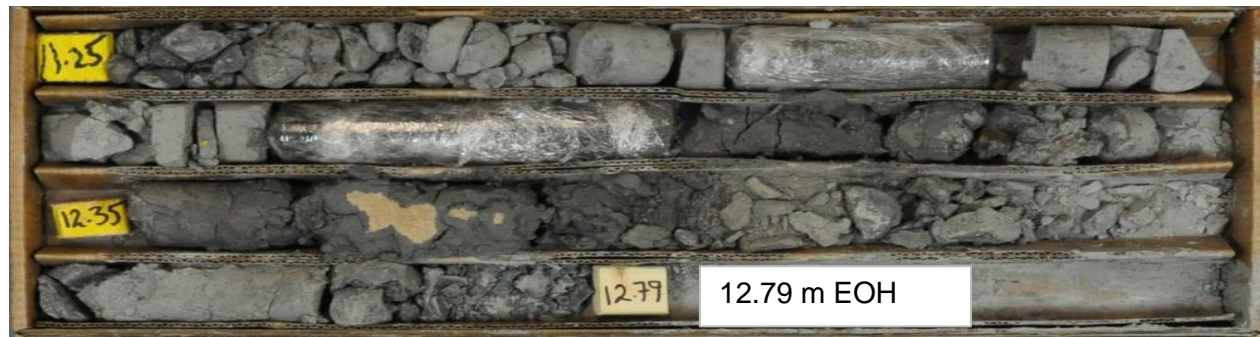
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Figure A6

BH 17-1106 (Dry)
Cored Length of 11.25 to 12.79 metres
Core Box 1 of 1

11.25 m Top of Bedrock



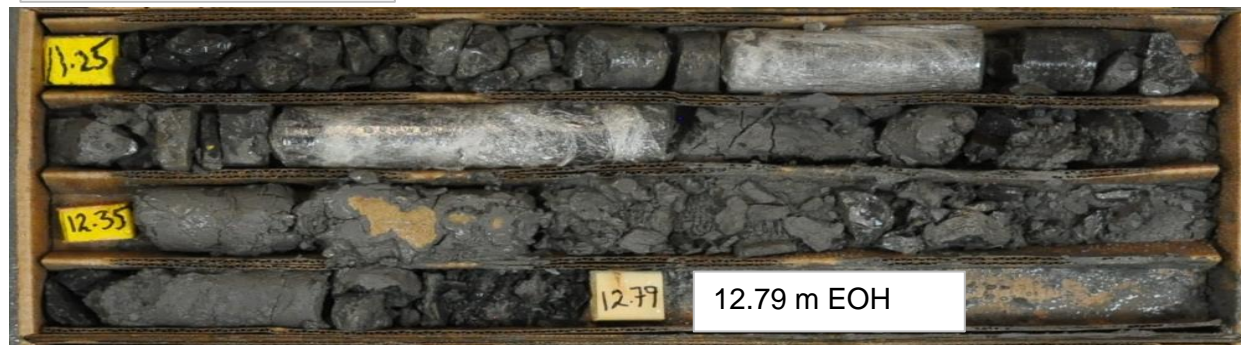
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Figure A7

BH 17-1106 (Wet)
Cored Length of 11.25 to 12.79 metres
Core Box 1 of 1

11.25 m Top of Bedrock



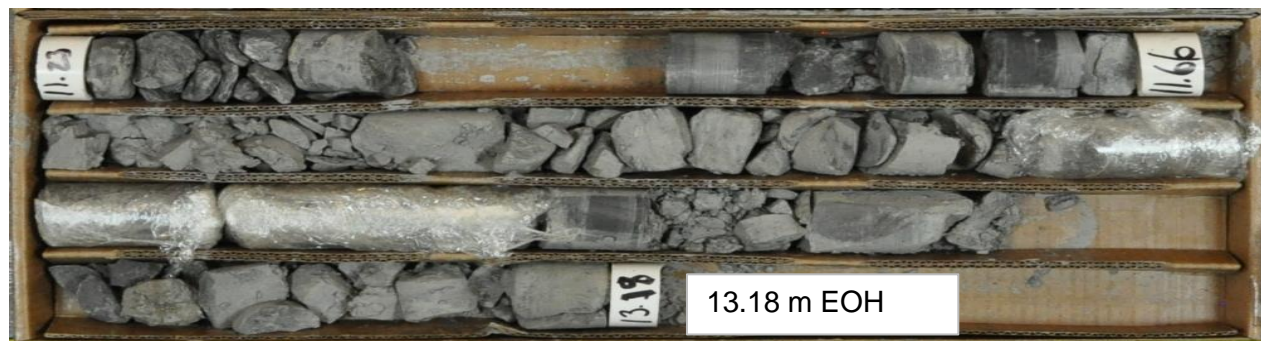
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Figure A8

BH 17-1106A (Dry)
Cored Length of 11.23 to 13.18 metres
Core Box 1 of 1

11.23 m Top of Bedrock



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Figure A9

BH 17-1106A (Wet)
Cored Length of 11.23 to 13.18 metres
Core Box 1 of 1

11.23 m Top of Bedrock



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Figure A10

BH 17-1107 (Dry)
Cored Length of 16.15 to 19.20 metres
Core Box 1 of 1

16.15 m Top of Bedrock



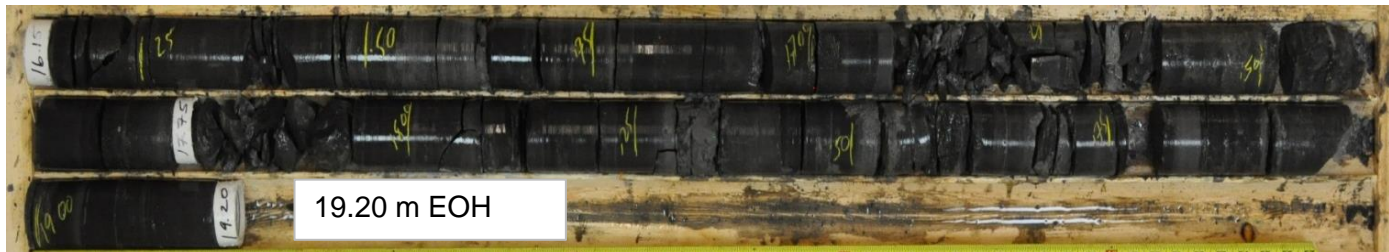
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Figure A11

BH 17-1107 (Wet)
Cored Length of 16.15 to 19.20 metres
Core Box 1 of 1

16.15 m Top of Bedrock



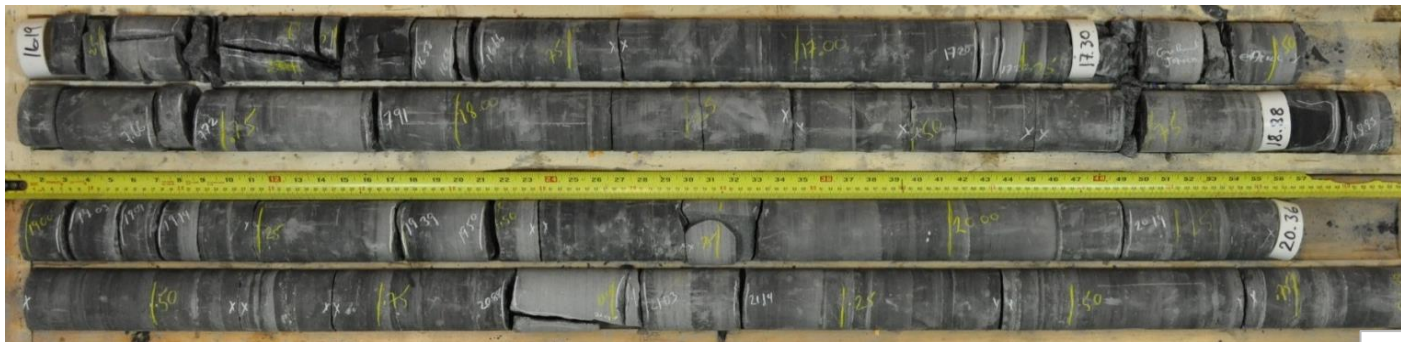
Foundation Investigation
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Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A12

BH 17-1110 (Dry)
Cored Length of 16.19 to 21.88 metres
Core Box 1 and 2 of 2

16.19 m Top of Bedrock



21.88 m EOH



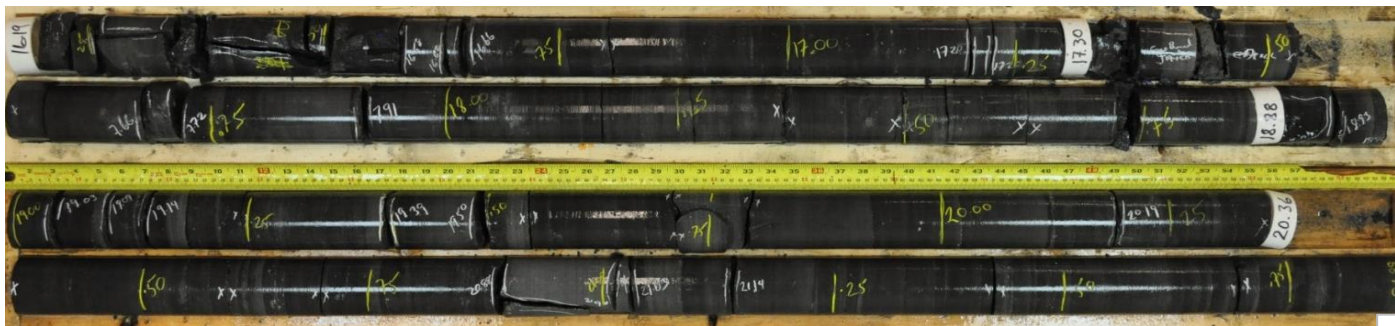
Foundation Investigation
CNR Overhead
Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A13

BH 17-1110 (Wet)
Cored Length of 16.19 to 21.88 metres
Core Box 1 and 2 of 2

16.19 m Top of Bedrock



21.88 m EOH

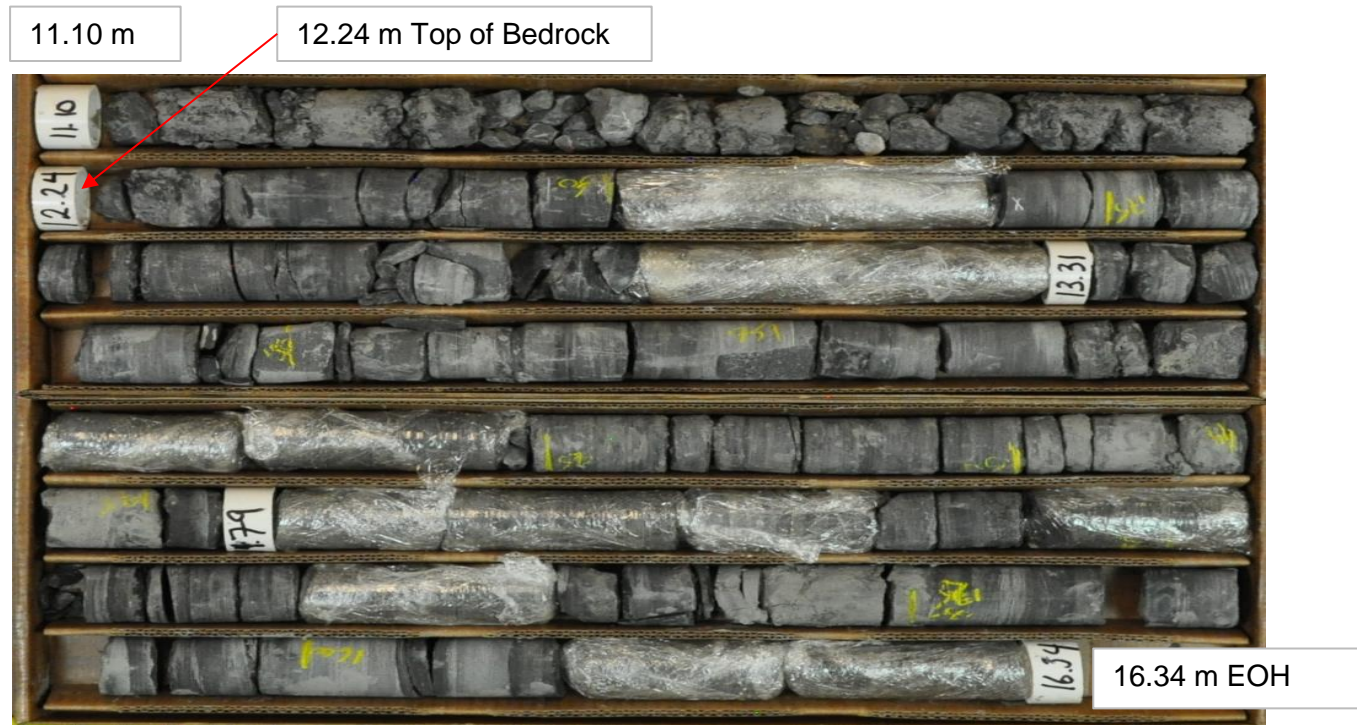


Foundation Investigation
CNR Overhead
Ottawa, Ontario

Project No.	1662565 / 1110
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Date:	2017-09-01
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Figure A14

BH 17-1111 (Dry)
Cored Length of 11.10 to 16.34 metres
Core Box 1 and 2 of 2



Note: Material in core box from 11.10 to 12.24 is gravel and cobbles in Till.



Foundation Investigation
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Figure A15

BH 17-1111 (Wet)
Cored Length of 11.10 to 16.34 metres
Core Box 1 and 2 of 2



Note: Material in core box from 11.10 to 12.24 is gravel and cobbles in Till.



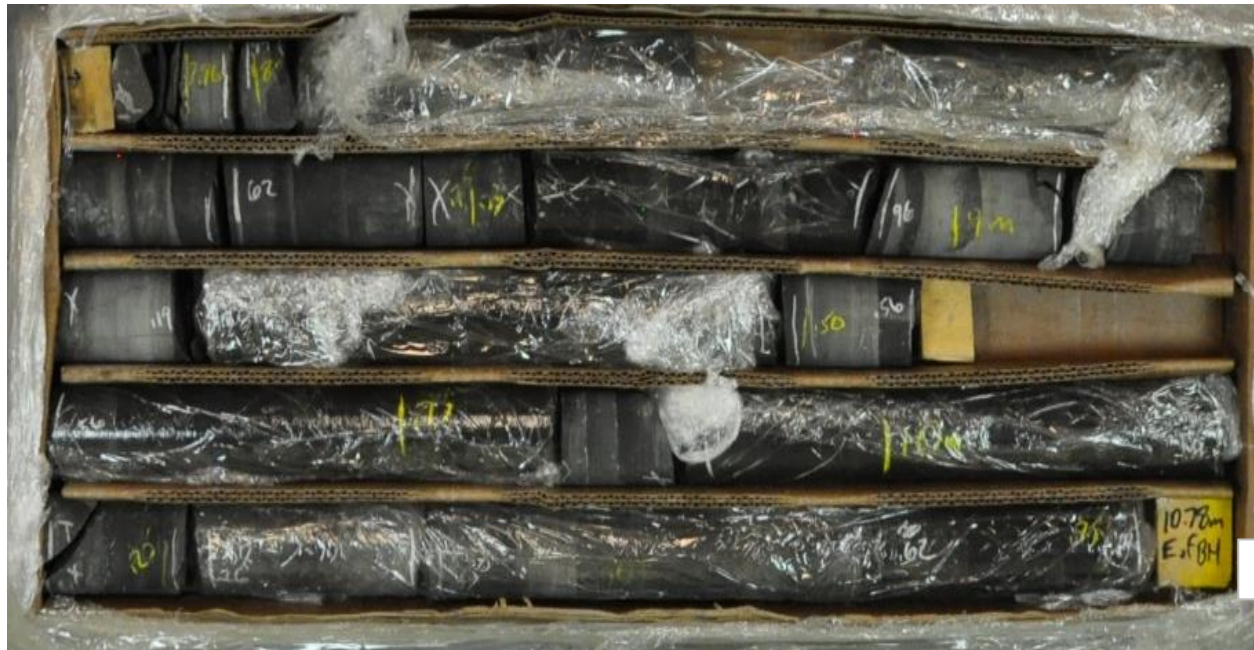
Foundation Investigation
CNR Overhead
Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A17

BH 17-1113 (Dry)
Cored Length of 7.77 to 10.78 metres
Core Box 1 of 1

7.77 m Top of Bedrock



10.78 m EOH



Foundation Investigation
CNR Overhead
Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A17

7.77 m Top of Bedrock



10.78 m EOH

BH 17-1114 (Dry)
Cored Length of 11.96 to 15.27 metres
Core Box 1 and 2 of 2

11.96 m Top of Bedrock



15.27 m EOH



Foundation Investigation
CNR Overhead
Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A19

BH 17-1114 (Wet)
Cored Length of 11.96 to 15.27 metres
Core Box 1 and 2 of 2

11.96 m Top of Bedrock



15.27 m EOH



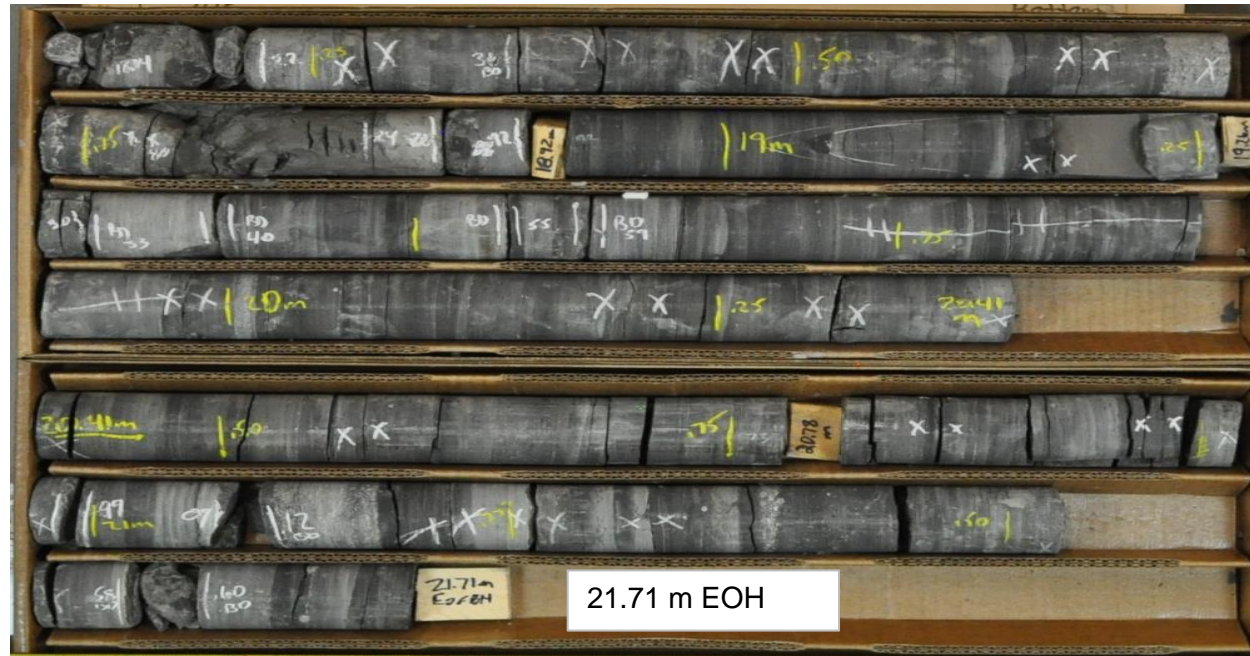
Foundation Investigation
CNR Overhead
Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A20

BH 17-1115 (Dry)
Cored Length of 18.14 to 21.71 metres
Core Box 1 and 2 of 2

18.14 m Top of Bedrock



21.71 m EOH



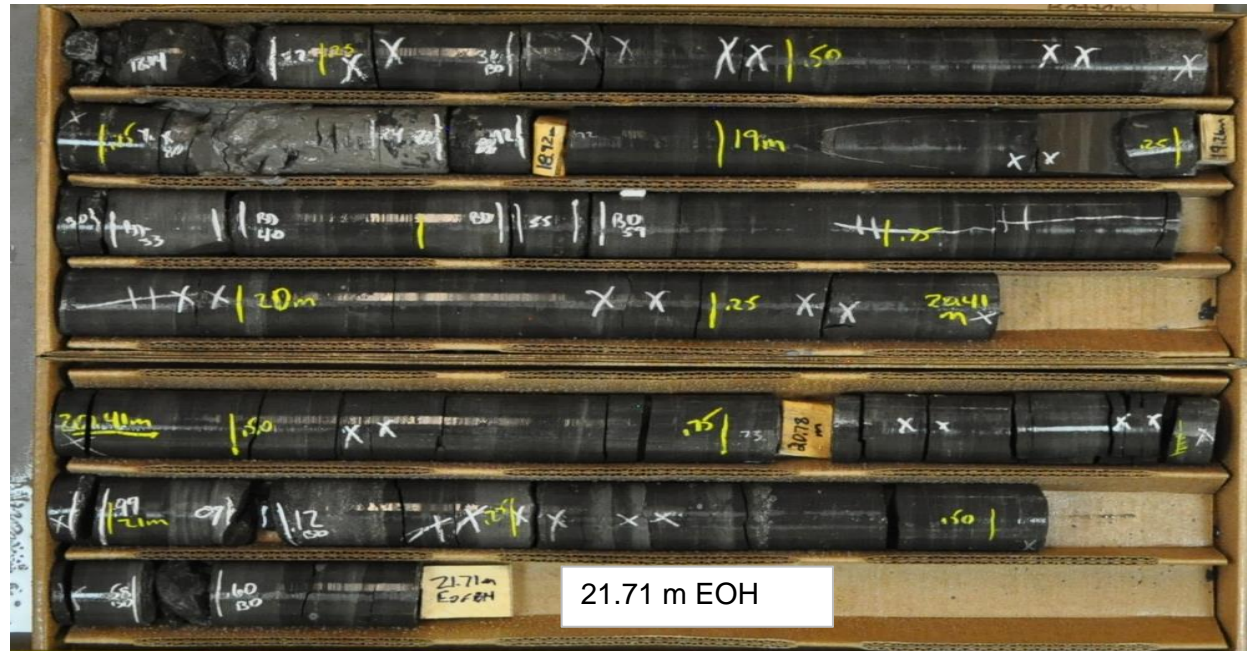
Foundation Investigation
CNR Overhead
Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A21

BH 17-1115 (Wet)
Cored Length of 18.14 to 21.71 metres
Core Box 1 and 2 of 2

18.14 m Top of Bedrock



21.71 m EOH



Foundation Investigation

CNR Overhead

Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	KS
Date:	2017-09-01
Checked:	SAT
Review:	FJH

Figure A22

APPENDIX B

Laboratory Test Results, Current Investigation

Figures B1a & B1b – Grain Size Distribution Test Results – Fill

Figure B2 – Plasticity Chart – Weathered Silty Clay to Clay

Figure B3 – Grain Size Distribution Test Results – Weathered Silty Clay to Clay

Figure B4 – Plasticity Chart – Unweathered Silty Clay to Clay

Figure B5 – Grain Size Distribution Test Results – Unweathered Silty Clay to Clay

Figures B6 & B7 – Consolidation Test Results

Figure B8 – Plasticity Chart – Silt and Sand, Silty Sand, and Clayey Silt Till

Figure B9a – Grain Size Distribution Test Results – Silty Sand, Sandy Silt, Sand and Gravel,
and Clayey Silt Till

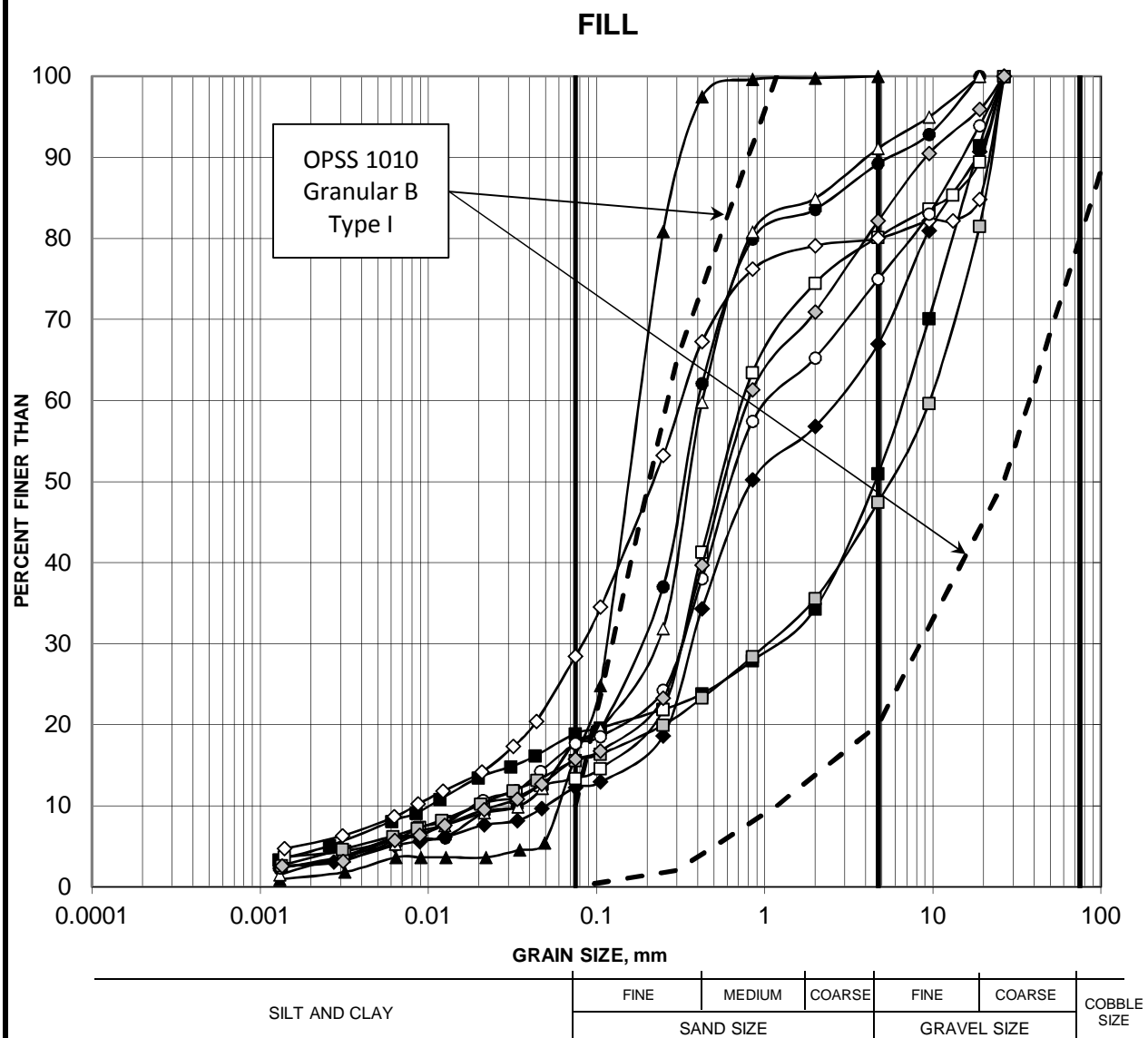
Figure B9b – Grain Size Distribution Test Results – Silt and Sand to Silty Sand Till

Figure B10 – Summary of Laboratory Compressive Strength Testing – Unconfined
Compression Tests

Figure B11 – Summary of Engineering Properties

GRAIN SIZE DISTRIBUTION

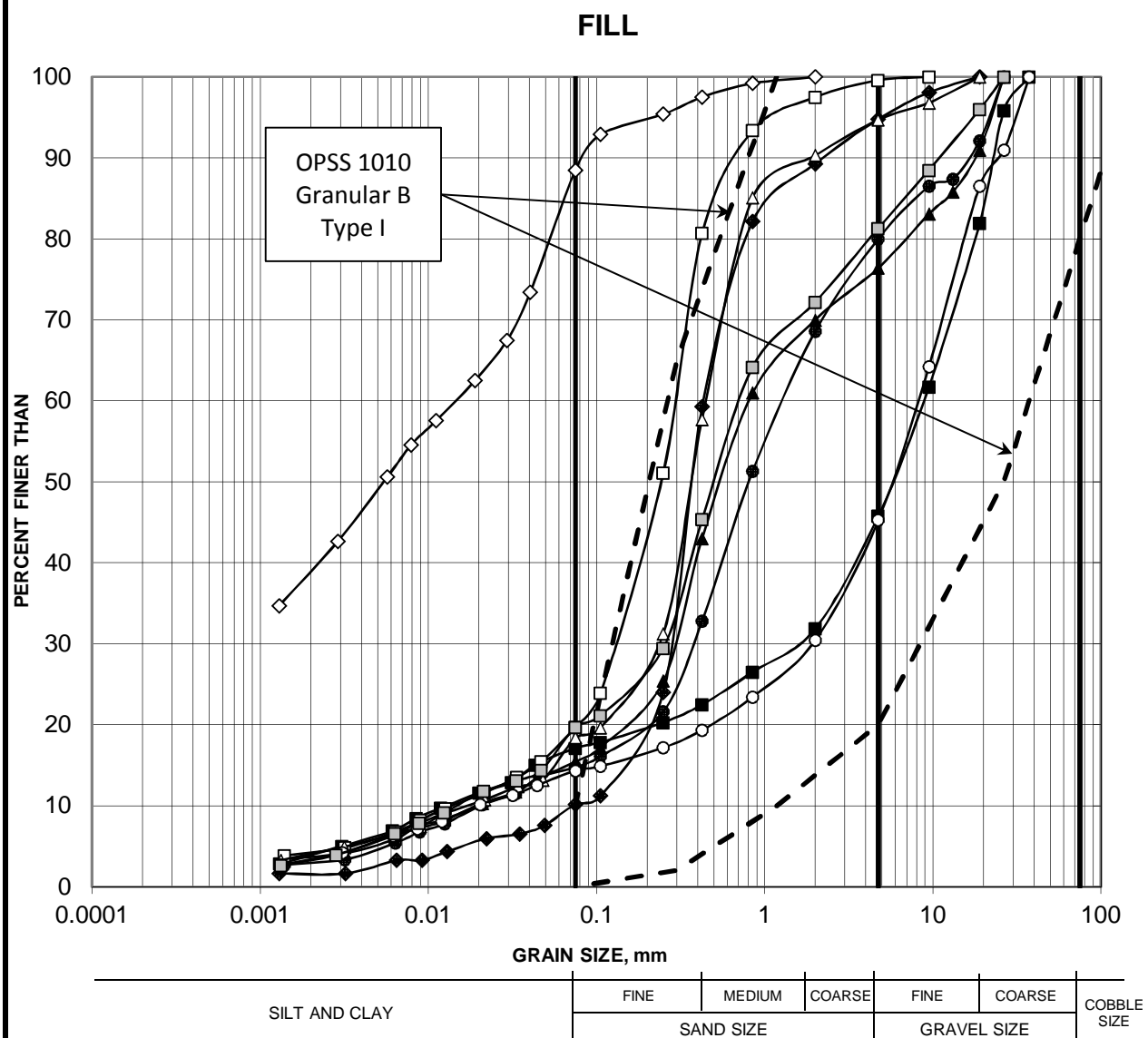
FIGURE B1a



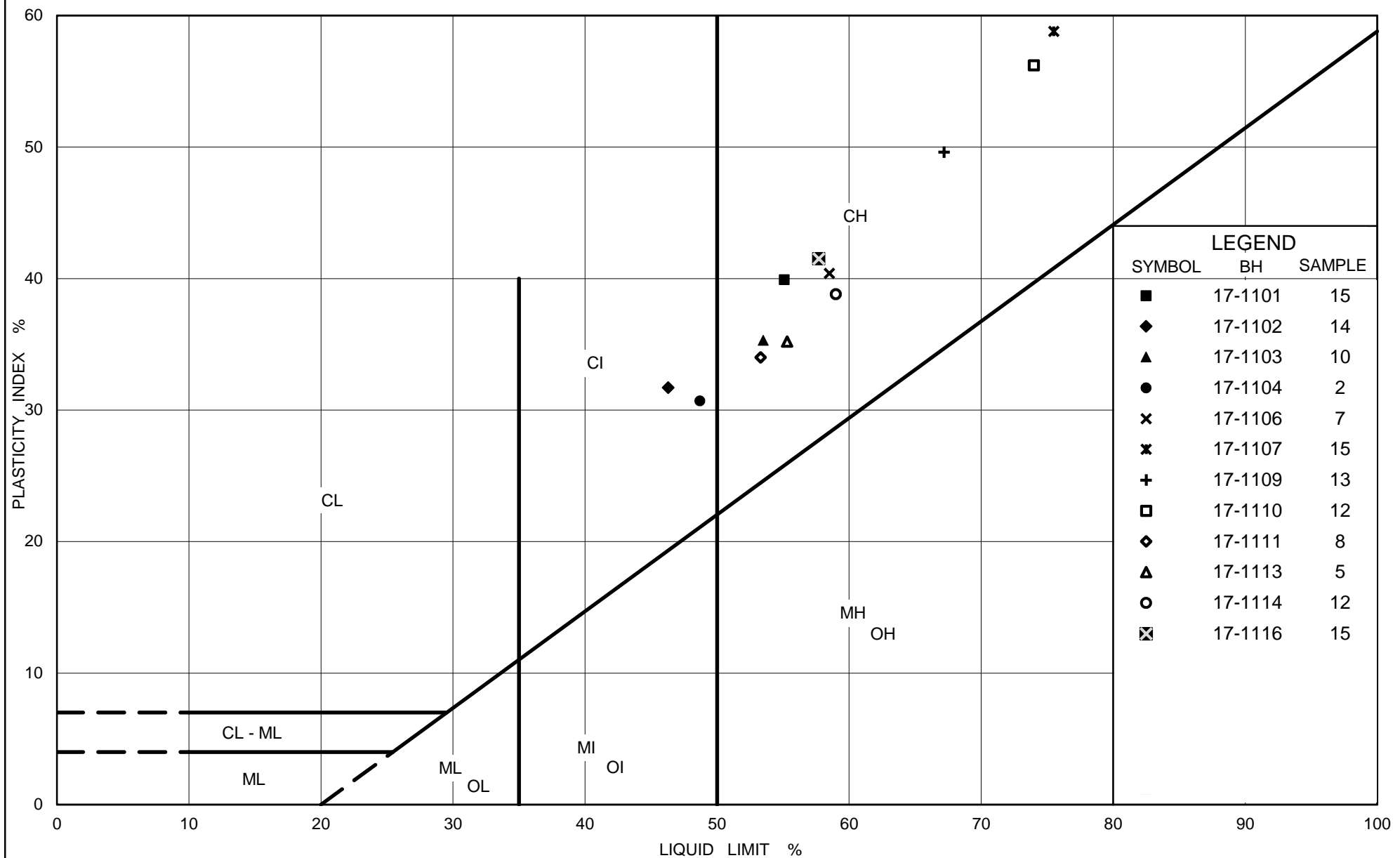
Borehole	Sample	Depth (m)
17-1101	4	2.29-2.90
17-1101	8	5.33-5.94
17-1101	12	8.38-8.99
17-1102	6	3.81-4.42
17-1103	3	1.22-1.83
17-1106	5A	3.81-4.27
17-1107	6	3.81-4.42
17-1107	11	7.62-8.23
17-1108	4	2.29-2.90
17-1108	9	6.10-6.71

GRAIN SIZE DISTRIBUTION

FIGURE B1b



Borehole	Sample	Depth (m)
17-1109	3	1.52-2.13
17-1110	7	4.57-5.18
17-1111	2	1.52-2.13
17-1114	4	1.83-2.44
17-1114	8	4.27-4.88
17-1114	10	5.49-6.10
17-1115	8	5.33-5.94
17-1116	4	2.29-2.90
17-1116	10	6.86-7.47



Ministry of Transportation

Ontario

PLASTICITY CHART WEATHERED SILTY CLAY to CLAY

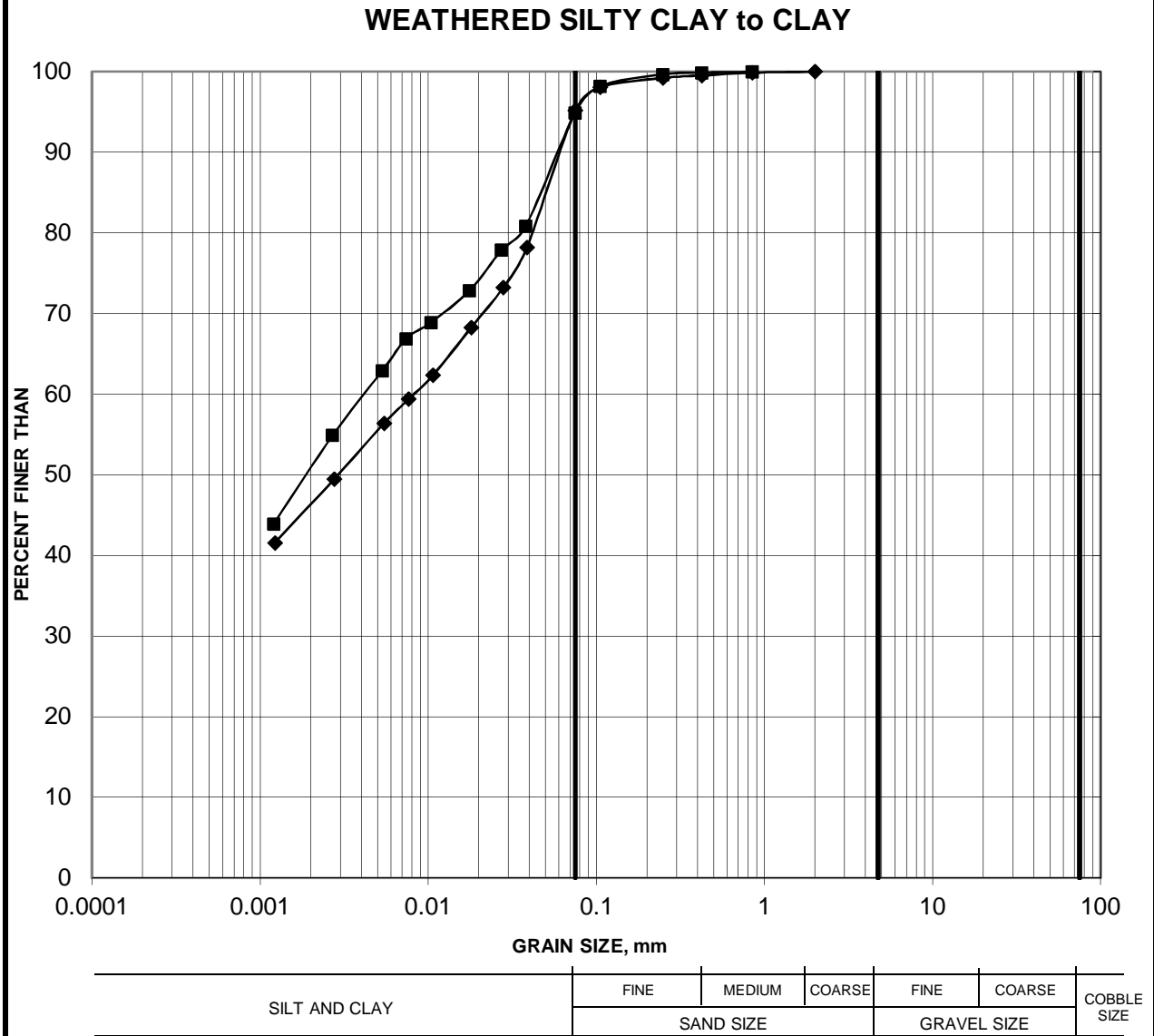
FIG No. B2

Project No. 1662565 /1110

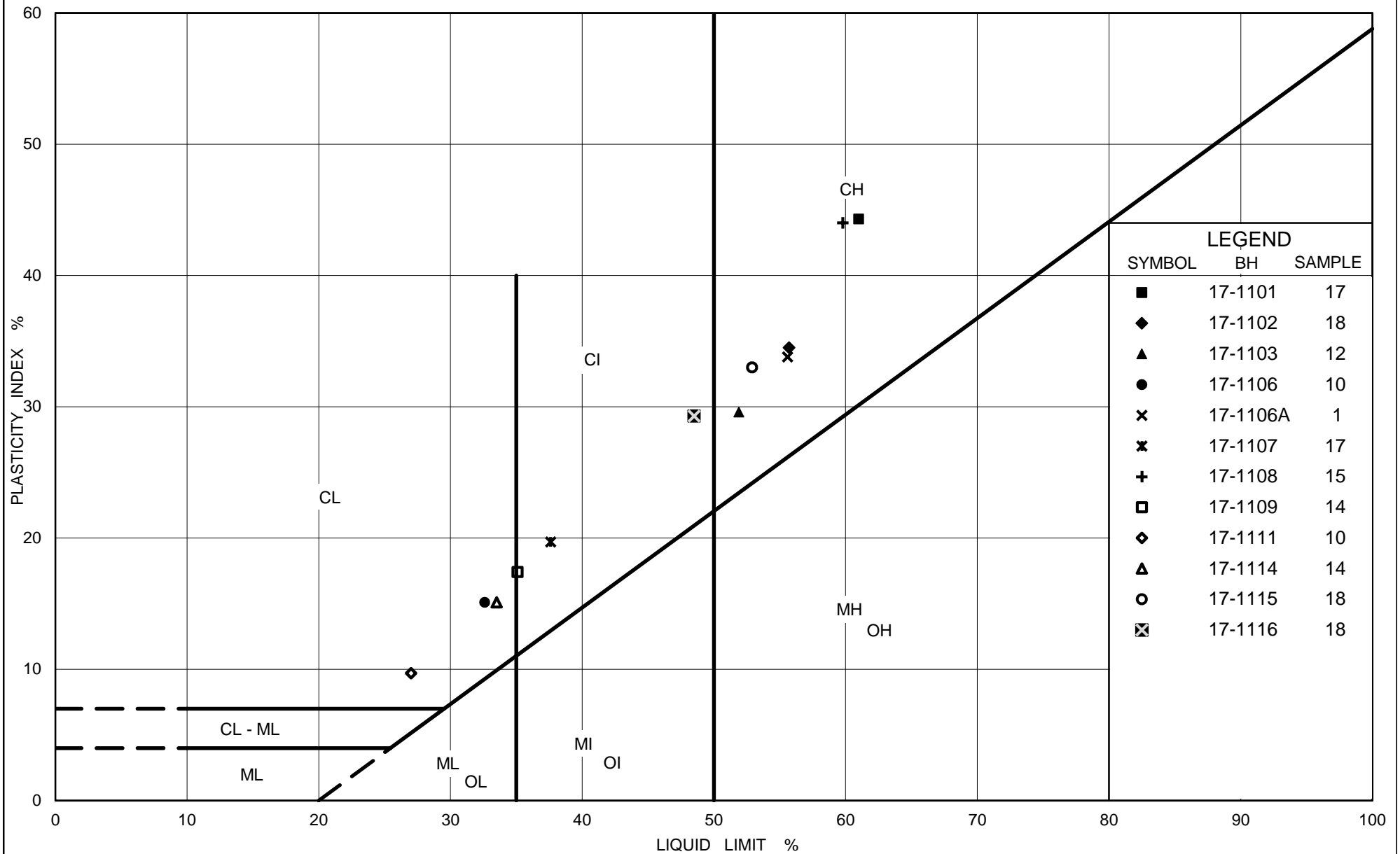
Compiled By : CNM Checked By : AC

GRAIN SIZE DISTRIBUTION

FIGURE B3



Borehole	Sample	Depth (m)
17-1108	13	10.67-11.28
17-1115	15	10.67-11.28



Ontario

Ministry of Transportation

PLASTICITY CHART UNWEATHERED SILTY CLAY to CLAY

FIG No. B4

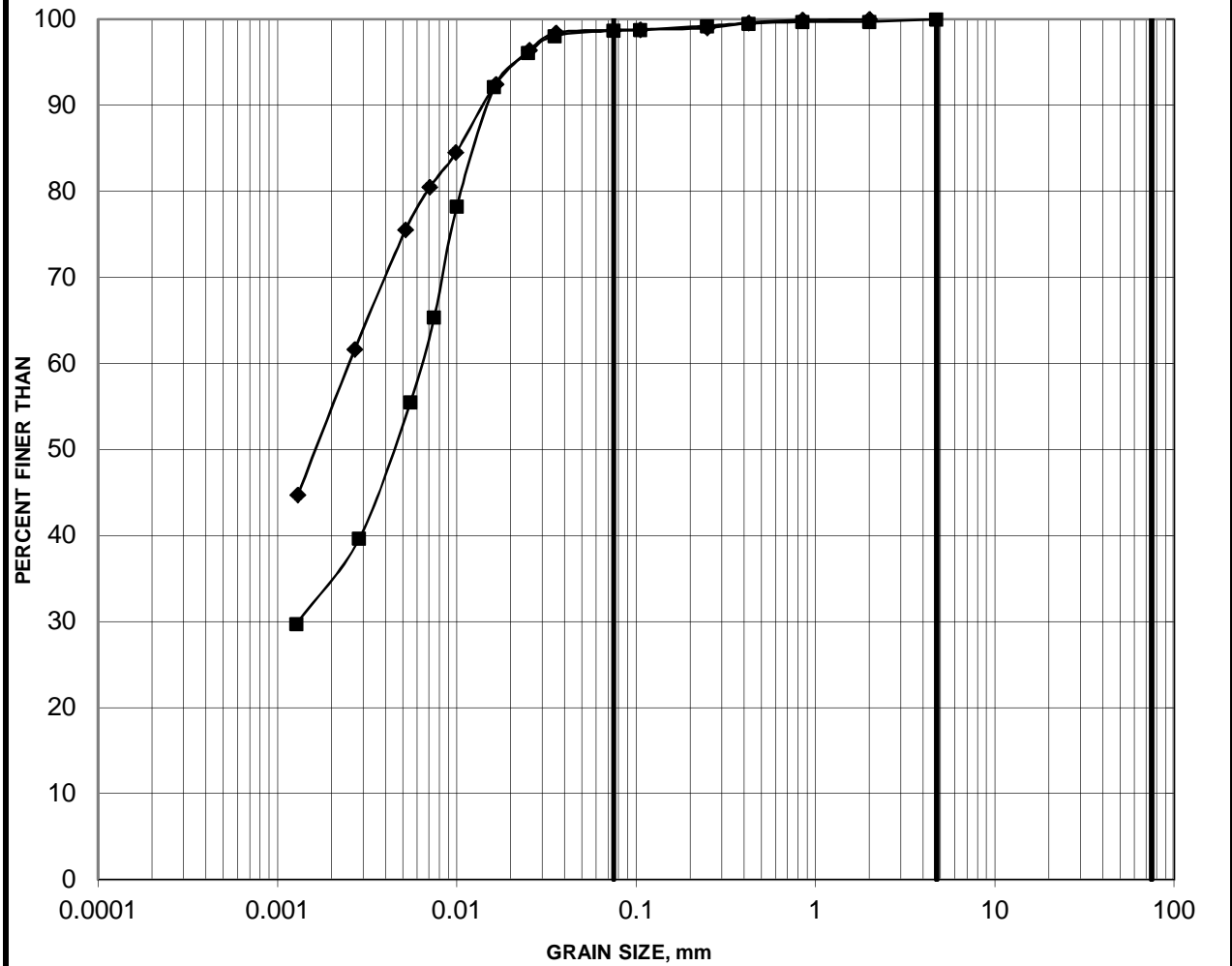
Project No. 1662565 /1110

Compiled By : CNM Checked By : AC

GRAIN SIZE DISTRIBUTION

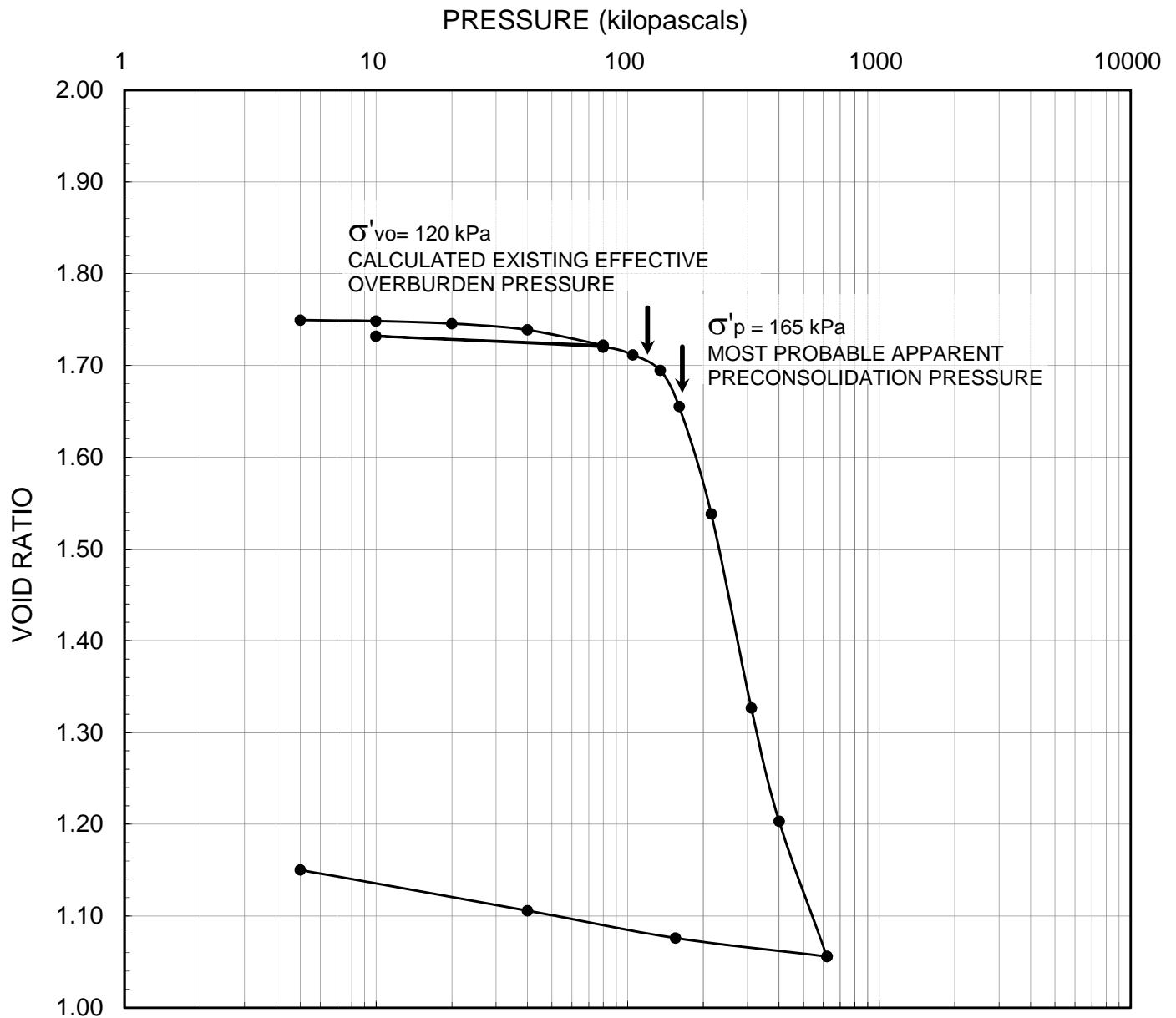
FIGURE B5

UNWEATHERED SILTY CLAY to CLAY



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

Borehole	Sample	Depth (m)
17-1102	19	15.24-15.85
17-1104	6	3.66-4.27



LEGEND

Borehole:	17-1106A	$w_i = 62\%$	$S_o = 100\%$	$\gamma = 16.2 \text{ kN/m}^3$
Sample:	1	$w_f = 42\%$	$e_o = 1.75$	$G_s = 2.79$
Depth (m):	8.1	$w_l = 56\%$	$C_c = 1.33$	
Elevation (m):	62.2	$w_p = 22\%$	$C_r = 0.013$	



GOLDER

SCALE	AS SHOWN
DATE	12/20/17
CADD	N/A
ENTERED	CNM

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

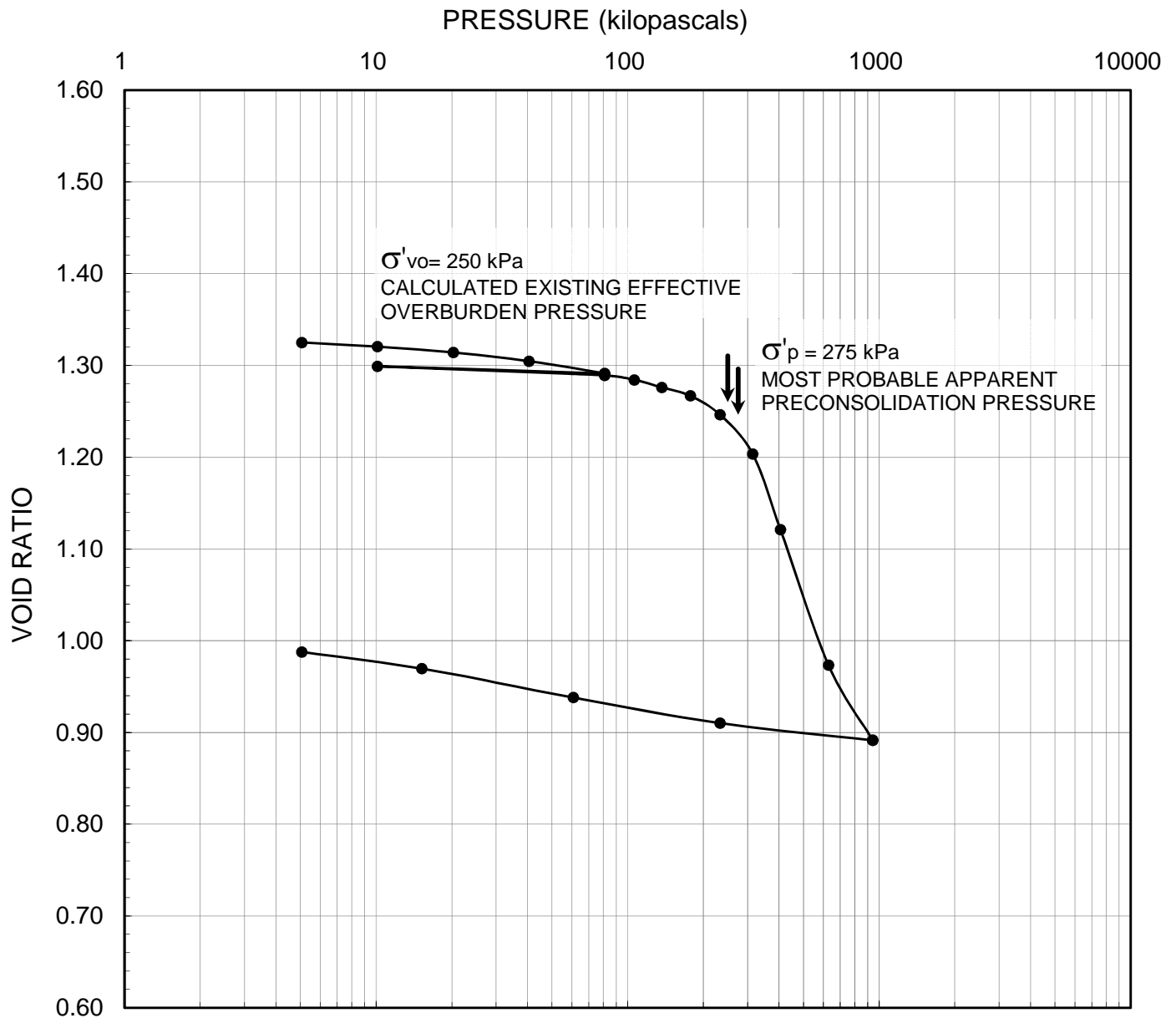
CHECK AC

PROJECT No. 1662565 /1110 REV. 2

REVIEW SAT

FIGURE

B6



LEGEND

Borehole:	17-1116	$w_i = 46\%$	$S_o = 97\%$	$\gamma = 17.1 \text{ kN/m}^3$
Sample:	18	$w_f = 35\%$	$e_o = 1.33$	$G_s = 2.78$
Depth (m):	14.1	$w_l = 49\%$	$C_c = 0.76$	
Elevation (m):	61.8	$w_p = 19\%$	$C_r = 0.011$	



GOLDER

SCALE	AS SHOWN
DATE	12/20/17
CADD	N/A
ENTERED	CNM

TITLE

CONSOLIDATION TEST RESULTS

FILE No. Consolidation summary

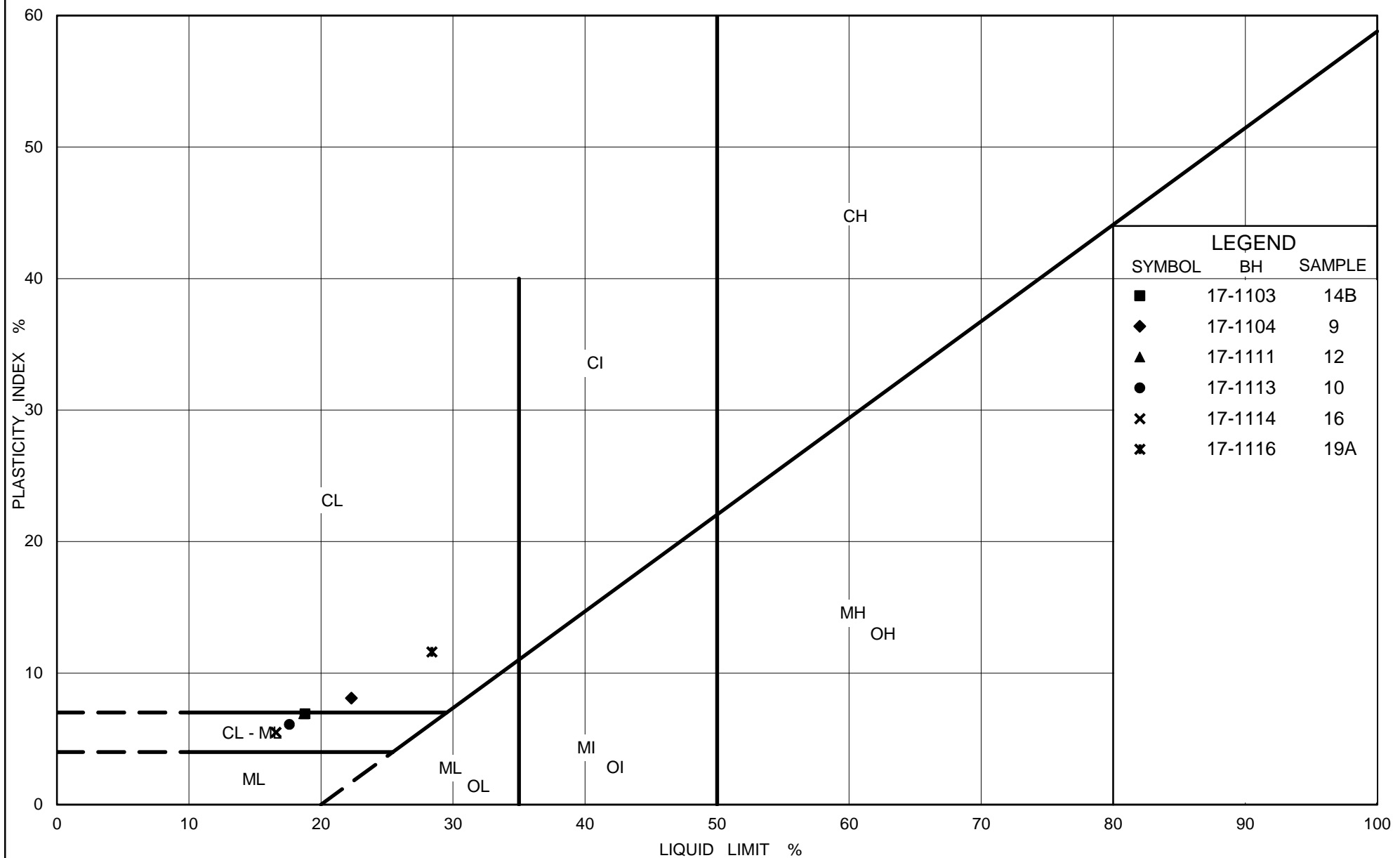
CHECK AC

PROJECT No. 1662565 /1110 REV. 2

REVIEW SAT

FIGURE

B7



Ontario

Ministry of Transportation

PLASTICITY CHART SILT and SAND, Silty SAND, and CLAYEY SILT TILL

FIG No. B8

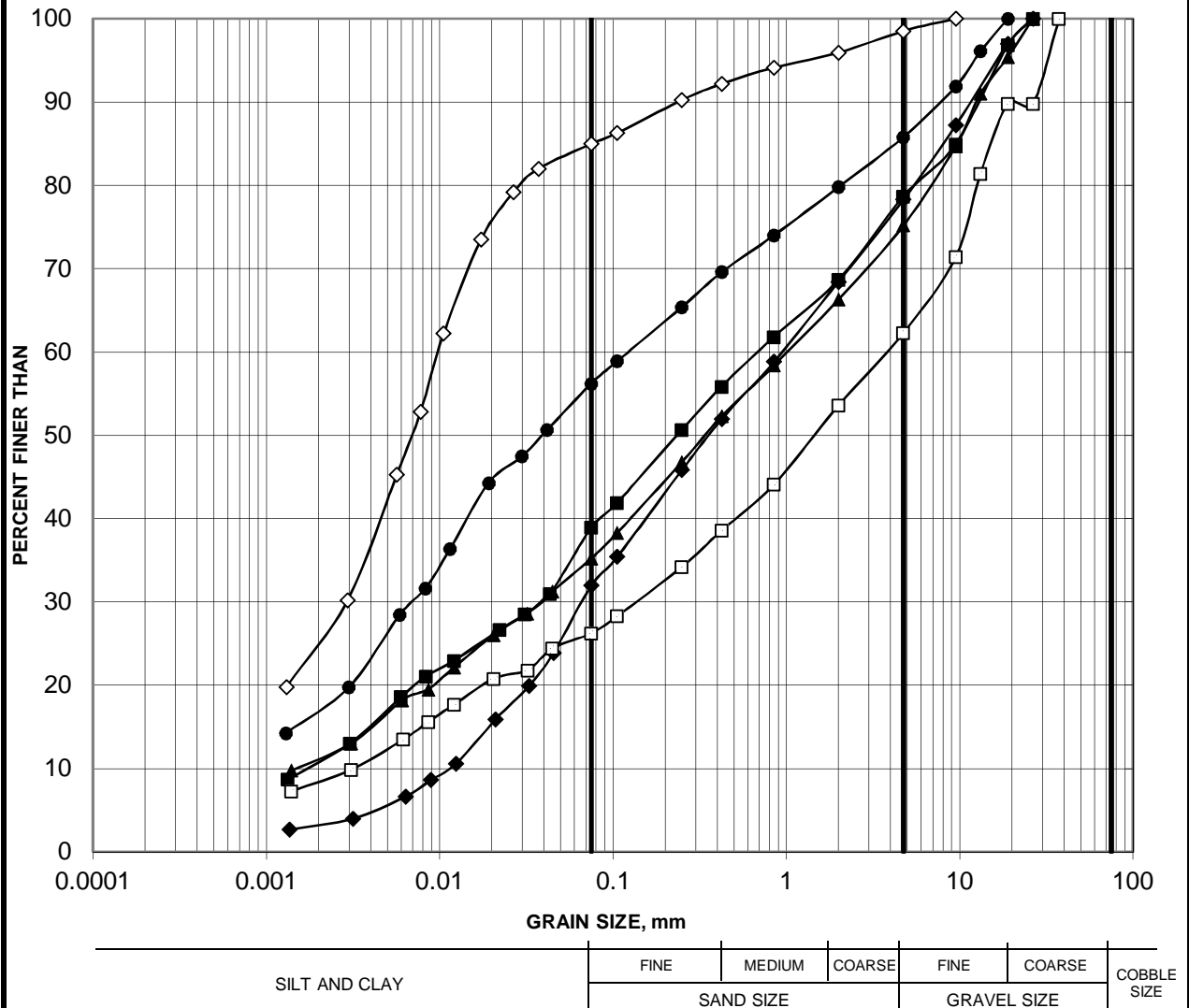
Project No. 1662565 /1110

Compiled By : CNM Checked By : MI

GRAIN SIZE DISTRIBUTION

FIGURE B9a

Silty SAND, Sandy SILT, SAND and GRAVEL, and CLAYEY SILT TILL

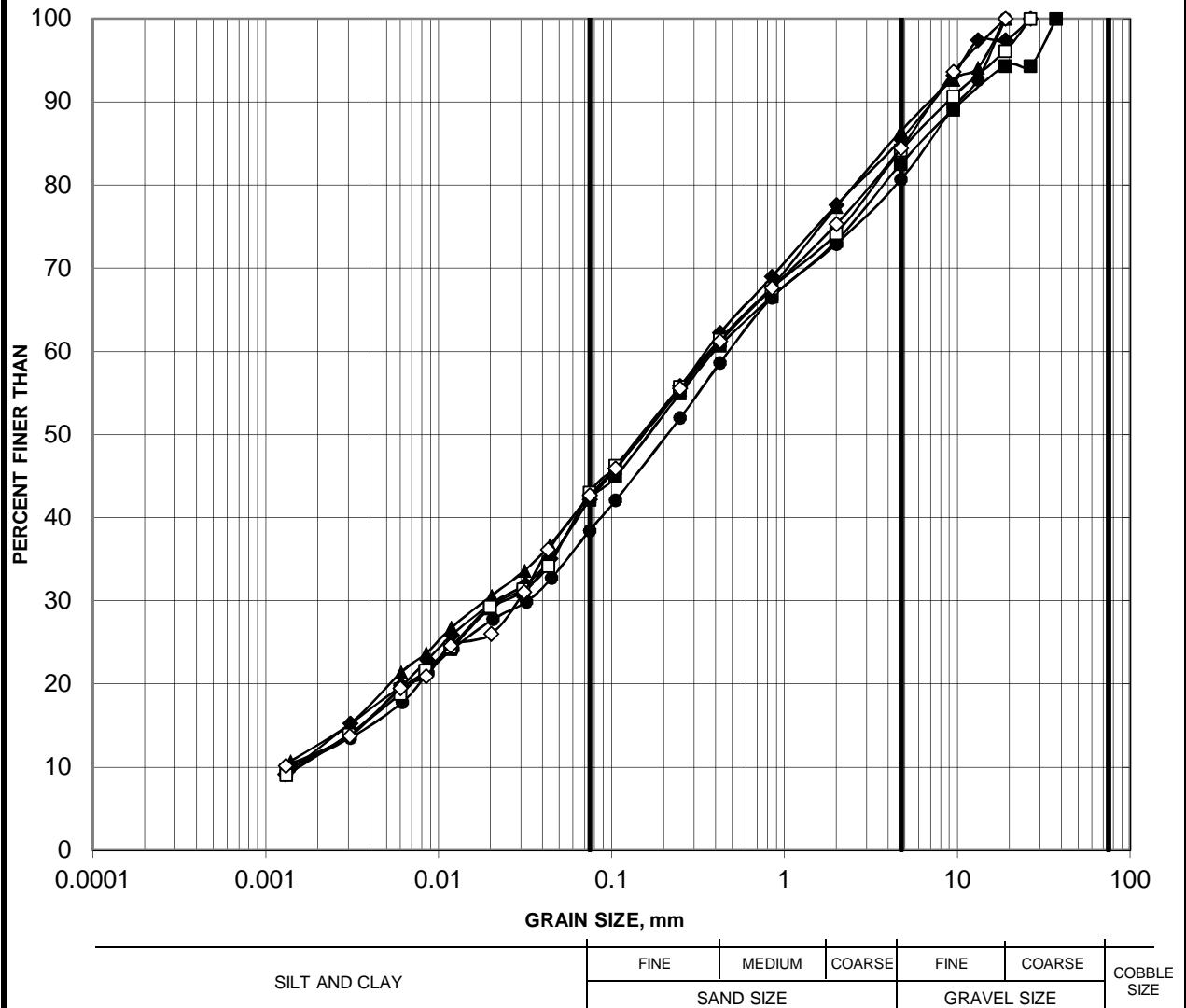


Borehole	Sample	Depth (m)
17-1101	20	16.76-17.37
17-1102	23	19.05-19.66
17-1103	15	11.49-12.10
17-1104	9	6.71-7.32
17-1106	11	10.67-11.10
17-1108	17A	15.24-15.54

GRAIN SIZE DISTRIBUTION

FIGURE B9b

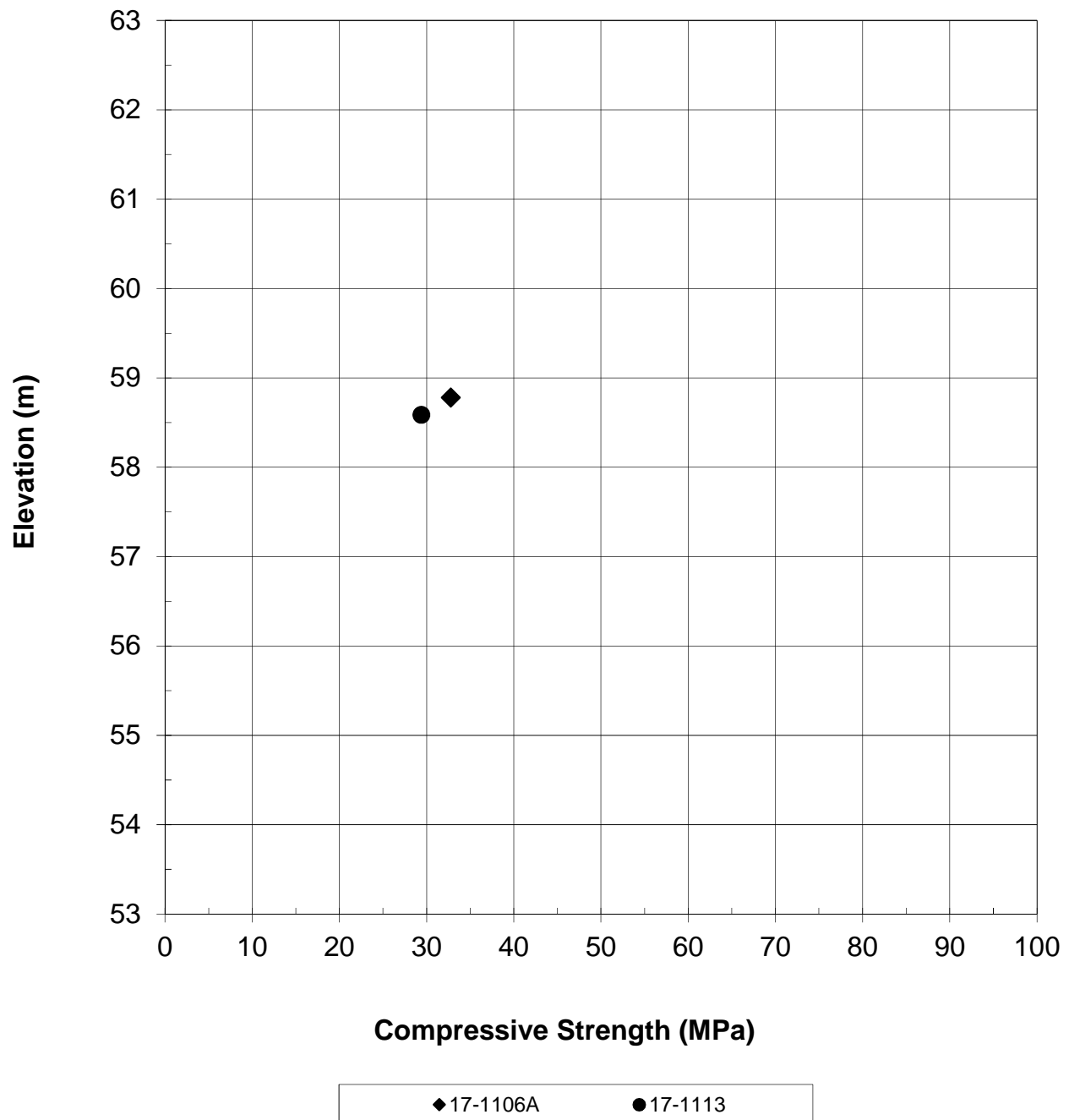
SILT and SAND to Silty SAND TILL



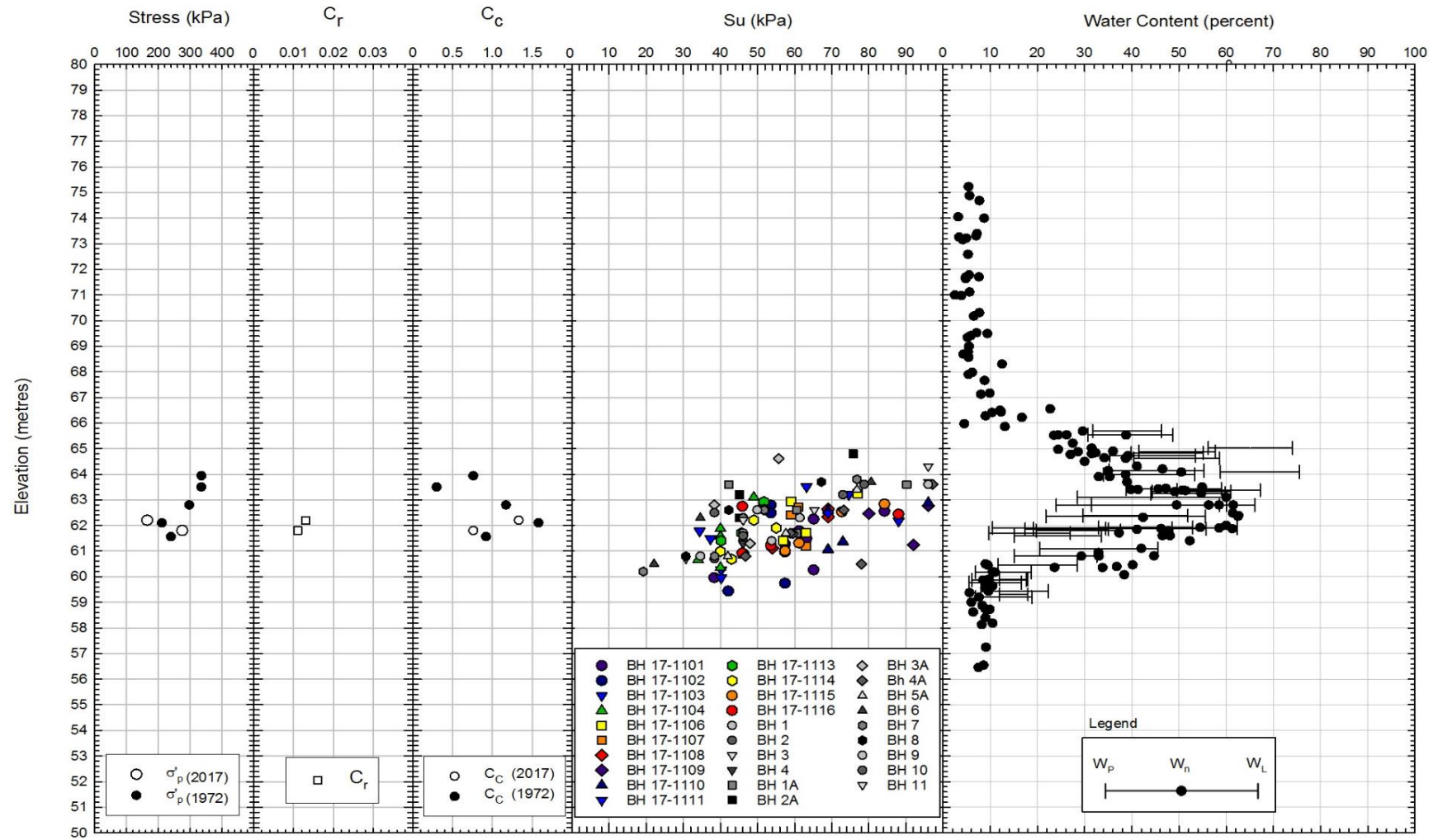
Borehole	Sample	Depth (m)
17-1110	16	15.24-15.85
17-1111	12	9.91-10.52
17-1113	10	6.71-7.32
17-1114	16	10.36-10.97
17-1115	21	16.76-17.37
17-1116	20	16.00-16.61

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

FIGURE B10



SUMMARY OF ENGINEERING PROPERTIES



CNR OVERHEAD WIDENING
SITE NOS. 3-301/1 AND 3-301/2
HIGHWAY 417, OTTAWA, ONTARIO

Project No.	1662565 / 1110
Drawn:	WT/WAM
Date:	2018-05-03
Checked:	WT
Review:	MSS

Figure B11

APPENDIX C

Borehole Record and Laboratory Test Results (Previous Investigation, Geocres No. 31G5-79)

Records of Previous Boreholes BH1 to BH11 and BH1A to BH5A

Laboratory Test Results – Previous Investigation

FOUNDATION SECTION

JOB	71-11124	LOCATION	Co-ords. 495,769 N; 233,682 E.	ORIGINATED BY	WH
W.P.	10-69-03 & 04	BORING DATE	Nov. 10, 1971	COMPILED BY	SO
DATUM	Geodetic	BOREHOLE TYPE	NX Washboring	CHECKED BY	<i>[Signature]</i>

[illegible]

FOUNDATION SECTION

CHECKED BY AK

[illegible]

FOUNDATION SECTION

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %
							20	40	60	80	100						
							UNCONFINED			FIELD VANE							
			QUICK TRIAXIAL			LAB. VANE											
			400			800			1200			1600			2000		
216.5	Ground Level																
0.0	Silty clay to clay																
	Firm to Very Stiff		1	SS	25												
	Grey		2	TW	PH	210											
			3	TW	PH												
			4	TW	PH												
199.5			5	TW	PH	200											
17.0	Het. mix. of silt, sand & gravel, trace of clay.																
	Loose - Compact		6	SS	9												
190.0	Grey		7	SS	12	190											
26.5	Shale Bedrock		8	BX	38%												
	Sound Grey		9	BX	98%												
181.5																	
35.0	End of Borehole					180											

FOUNDATION SECTION

CHECKED BY *AK*

[illegible]

FOUNDATION SECTION

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION BLOWS / FOOT	RESISTANCE	LIQUID LIMIT ——— w_L	PLASTIC LIMIT ——— w_p	WATER CONTENT ——— w	BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		SHEAR STRENGTH P.S.F.						
215.9 0.0	Ground Level						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE						
						210							
						200							
197.5 18.4	End of Cone Test					190							

FOUNDATION SECTION

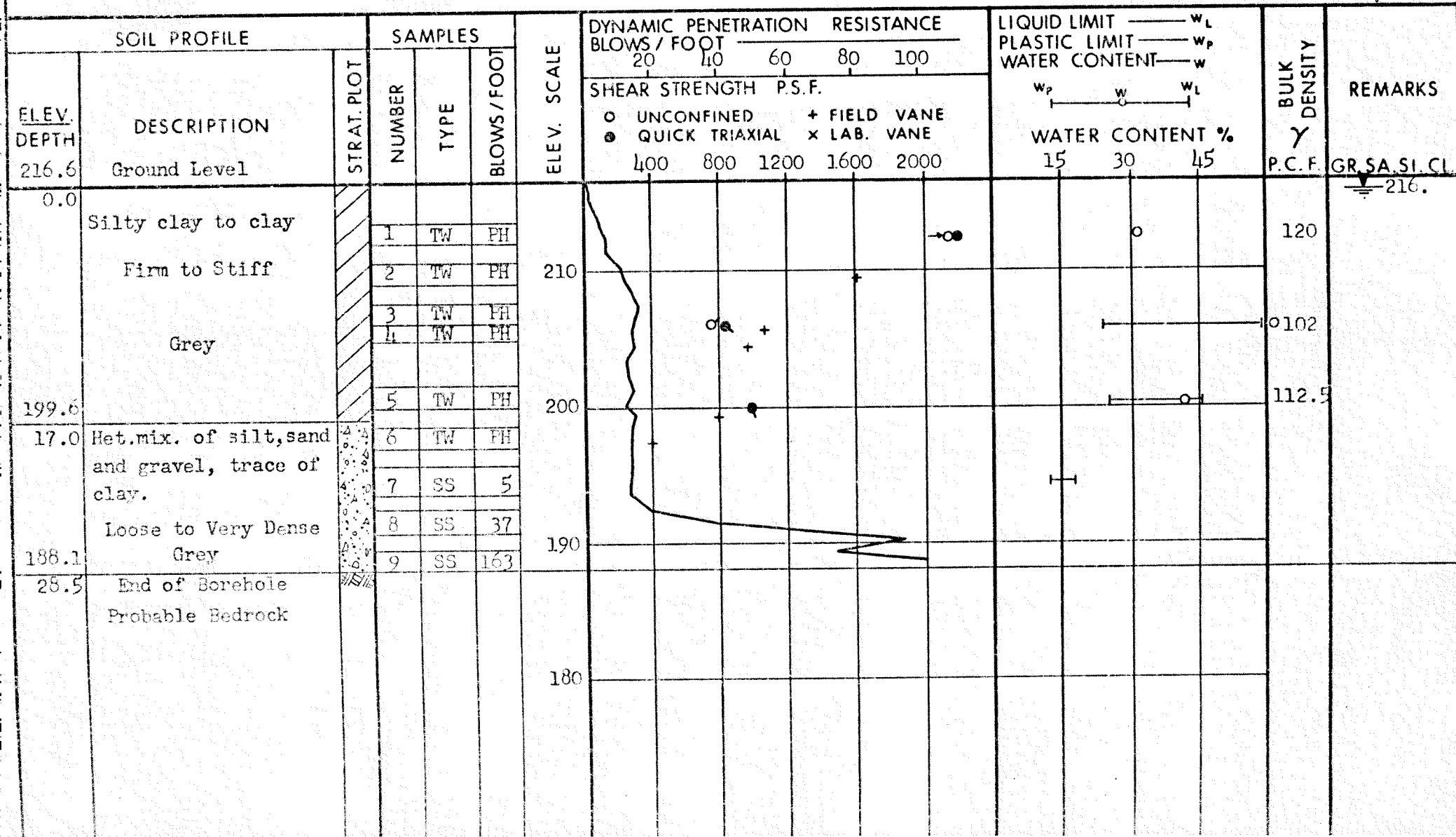
[illegible]

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING OFFICE

RECORD OF BOREHOLE No.7

FOUNDATION SECTION

JOB 71-11124 LOCATION Co-ords. 496,028 N; 233,220 E. ORIGINATED BY JS
W.P. 10-69-03 & 04 BORING DATE Dec. 7, 1971 COMPILED BY SO
DATUM Geodetic BOREHOLE TYPE NX Washboring CHECKED BY SK






FOUNDATION SECTION

CHECKED BY *[Signature]*

[illegible]

FOUNDATION SECTION

CHECKED BY 

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT % w_p ——— w ——— w_L
							20	40	60	80	100						
							400	800	1200	1600	2000						
216.7	Ground surface.																
	Silty clay to clay.					210									El. 215.7		
	Firm to very stiff.		1	SS	16												
	Grey.		2	TW	PH												
			3	TW	PH												
			4	TW	PH												
			5	TW	PH	200											
197.7																	
19.0	Het. mix. of silt, sand and gravel.		6	SS	5	190											
	Trace of clay.		7	SS	14												
	Loose to very dense.		8	SS	23												
	Grey.		9	SS	136												
181.9																	
179.9	Shale bedrock.		10	RC BX	Rec. 100%	180											
36.8	End of borehole.																
						170											

DEPARTMENT OF TRANSPORTATION AND COMMUNICATIONS

RECORD OF BOREHOLE No. 3 A

FOUNDATION SECTION

DESIGN SERVICES BRANCH

JOB 71-11124

LOCATION Co-ord's 496,003 N. 233,610 E.

ORIGINATED BY S.A.A.

W.P. 10-69-4


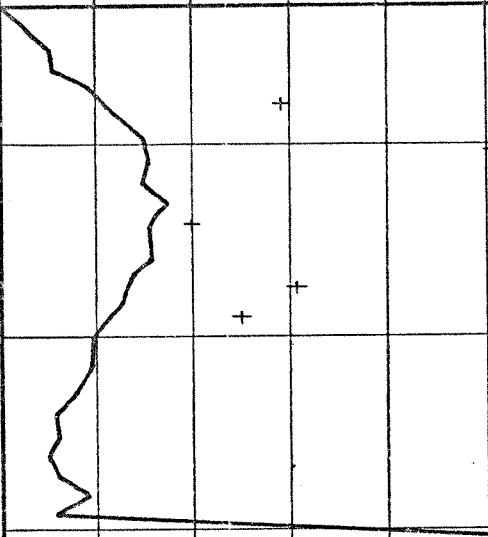
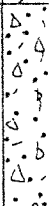


BORING DATE April 20, 1972

COMPILED BY A.T.

DATUM Geodetic

BOREHOLE TYPE Auger & RE Rock Core

CHECKED BY

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE					LIQUID LIMIT ——— w_L PLASTIC LIMIT ——— w_p WATER CONTENT ——— w			BULK DENSITY γ P.C.F.	REMARKS	
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS / FOOT		BLOWS / FOOT					SHEAR STRENGTH P.S.F.					WATER CONTENT %
							20	40	60	80	100	P.S.F.					
												400	800	1200			
						○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB. VANE											
217.0	Ground surface.																
200.5	Silty clay to clay. Firm to very stiff. Grey.		1	SS	12	210											
			2	TW	PH												
			3	TW	PH												
			4	TW	PH												
			5	TW	PH												
16.5	Het. mix. of silt, sand and gravel. Trace of clay.		6	SS	6	200											
189.0	Probable bedrock.		7	SS	12	190											
28.0	End of borehole.																
						180											

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE NO 4A

JOB 71-11124LOCATION Co'ord's 496,069 N. 233,124 E.ORIGINATED BY S.A.A.W.P. 10-69-03BORING DATE June 16, 1972COMPILED BY S.A.A.DATUM GeodeticBOREHOLE TYPE Washboring - NX CasingCHECKED BY SK

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT					LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W W_P W W_L			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F. ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 100 800 1200 1600 2000					WATER CONTENT %				
216.6	Ground surface.															
0.0	Silty clay, with trace of sand.		1	SS	11	210										Elev. 213.6 in open B.H. June 16/72
	Firm to very stiff.		2	TW	PM											
	Grey.		3	TW	PM											
			4	TW	PM											
			5	TW	PM											
198.6						200										
18.0	Het. Mix. of silt, sand and gravel, trace of clay.		6	TW	PM	190										
	Glacial Till.		7	TW	60											
187.1	Very dense.					180										
29.5	Shale bedrock.		8	RC	Rec											
184.6	Sound - grey.		BX	100%												
32.0	End of borehole.															

DESIGN SERVICES BRANCH

FOUNDATION OFFICE

RECORD OF BOREHOLE NO 5A

JOB 71-11124

LOCATION Co-ord's 495,951 N. 233,170 E.

ORIGINATED BY S.A.A.

W.P. 10-69-03

BORING DATE June 19, 1972

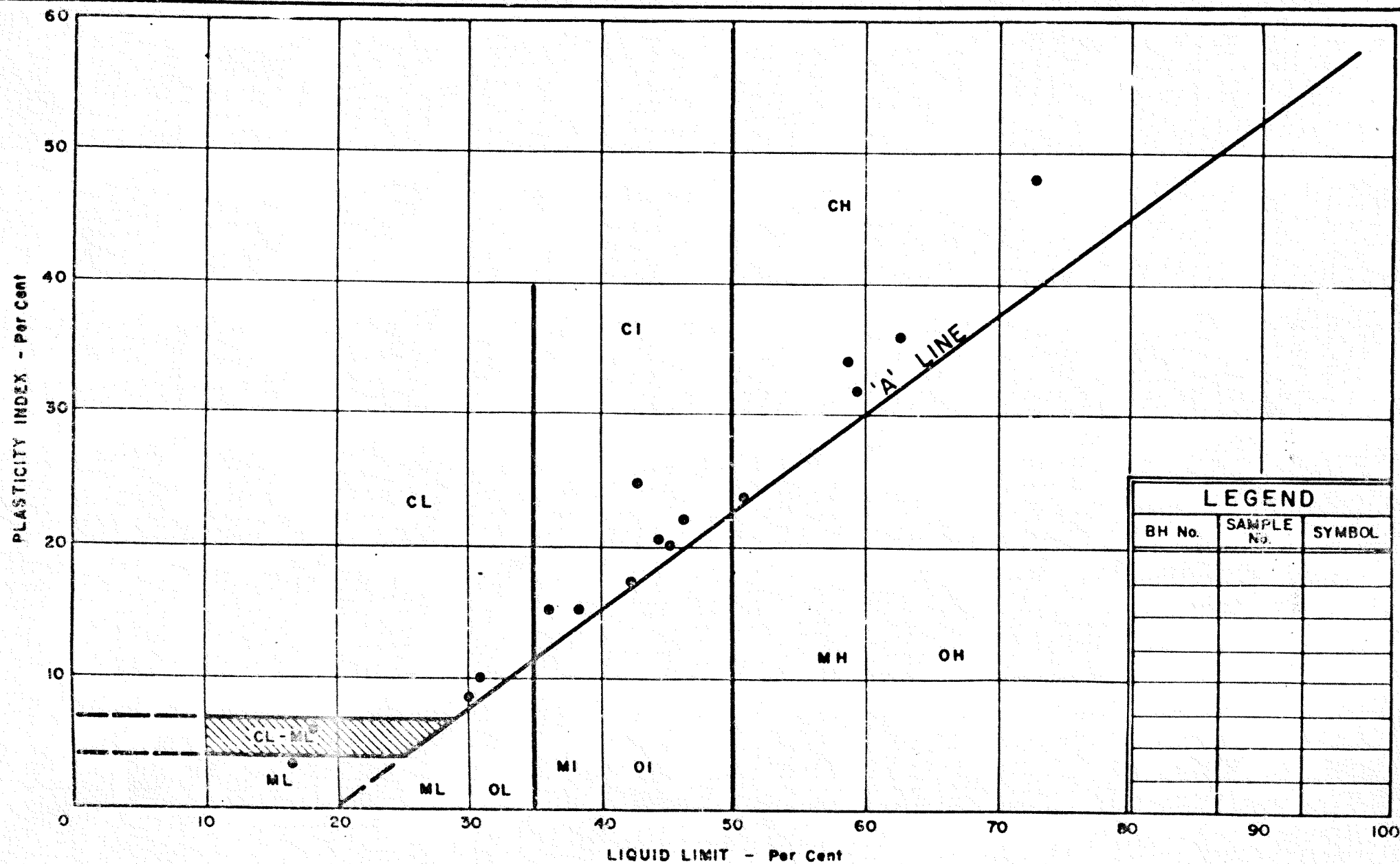
COMPILED BY S.A.A.

DATUM Geodetic

BOREHOLE TYPE Washboring - NX Casing

CHECKED BY *AL*

SOIL PROFILE			SAMPLES			ELEV. SCALE	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT						LIQUID LIMIT W_L PLASTIC LIMIT W_P WATER CONTENT W			BULK DENSITY γ P.C.F.	REMARKS
ELEV. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FOOT		SHEAR STRENGTH P.S.F.						WATER CONTENT %				
						O UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 400 800 1200 1600 2000						W_P — W — W_L WATER CONTENT %					
216.3	Ground surface.																
0.0	Silty clay, with trace of sand. Firm to very stiff. Grey.		1	SS	11												
			2	TW	PM	210											
			3	TW	PM												
			4	TW	PM												
199.3			5	TW	PM	200											
17.0	Het. mix. of silt, sand & gravel, trace of clay. Glacial Till		6	TW	PM												
	Very dense. Probable Bedrock.		7	SS	118	190											
188.3																	
28.0	End of borehole.					180											



DEPARTMENT OF HIGHWAYS
MATERIALS and
TESTING
DIVISION

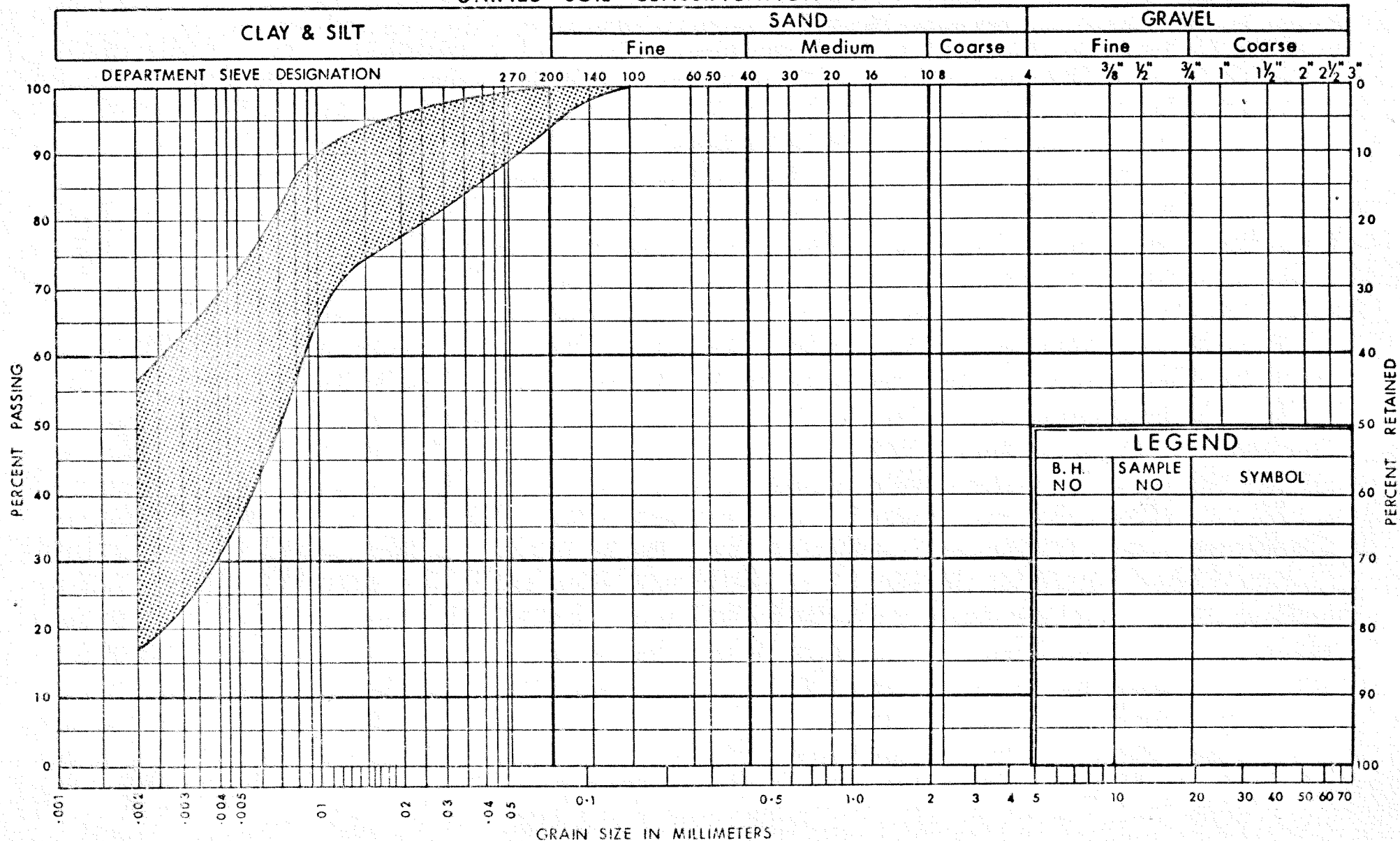
PLASTICITY CHART SILTY CLAY TO CLAY

W.P. No. 10-69-03 & 04


JOB No. 71-11124

FIG. 1

UNIFIED SOIL CLASSIFICATION SYSTEM



DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS

 ONTARIO

DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

W.P. No. 10 - 69 - 03 & 04

JOB No. 71-11124

FIG. 2

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT

SAND

GRAVEL

Fine

Medium

Coarse

Fine

Coarse

DEPARTMENT SIEVE DESIGNATION

270

00

100

605

40.

1. 2

16

10 8

4

 $\frac{3}{8}$ " $\frac{1}{2}$ " $\frac{3}{4}$ "

18 13

 $\frac{1}{2}'' \quad 2''$ $2\frac{1}{2}''$ 3

28

PERCENT PASSING

PERCENT RETAINED

LEGEND

B. H.
NO

SAMPLE NO

SYMBOL

GRAIN SIZE IN MILLIMETERS

DEPARTMENT
OF
TRANSPORTATION AND COMMUNICATIONS



DESIGN SERVICES
BRANCH

GRAIN SIZE DISTRIBUTION
GLACIAL TILL
HET. MIXTURE OF SILT, SAND & GRAVEL, TRACE OF CLAY

W.P. No. 10-69-03 & 04

JOB No. 71-11124

FIG. 3

VOID RATIO-PRESSURE CURVES

JOB NO. 71-11124

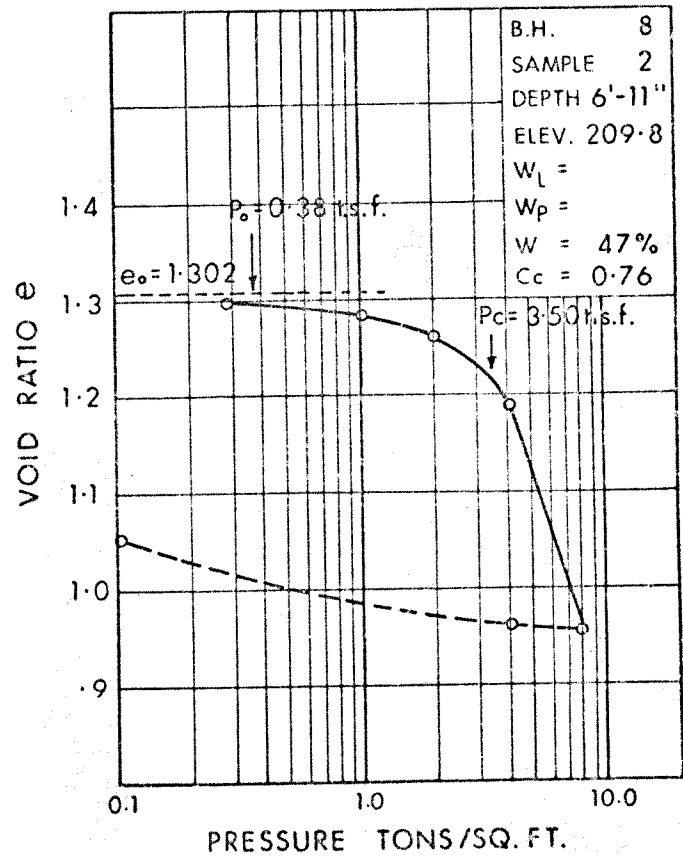
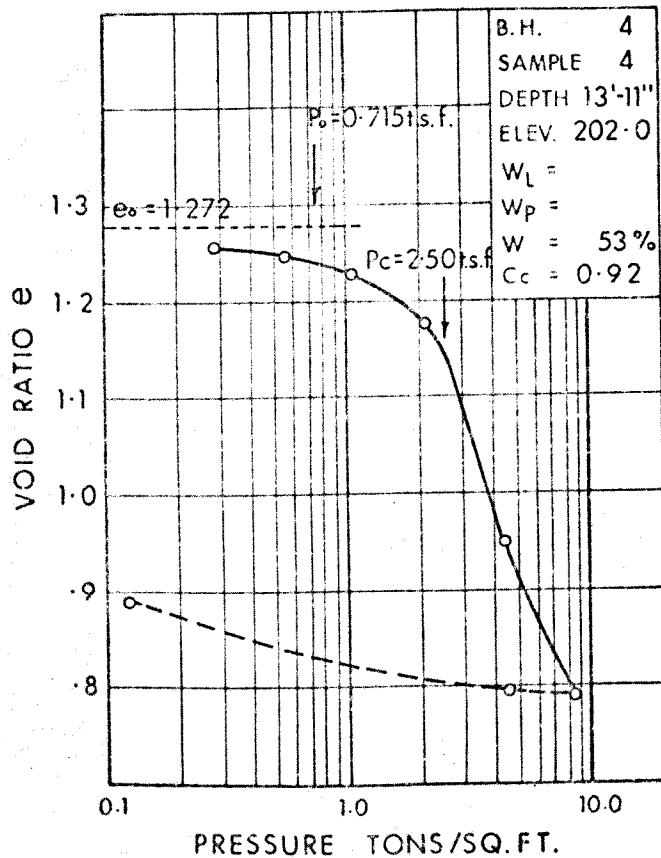
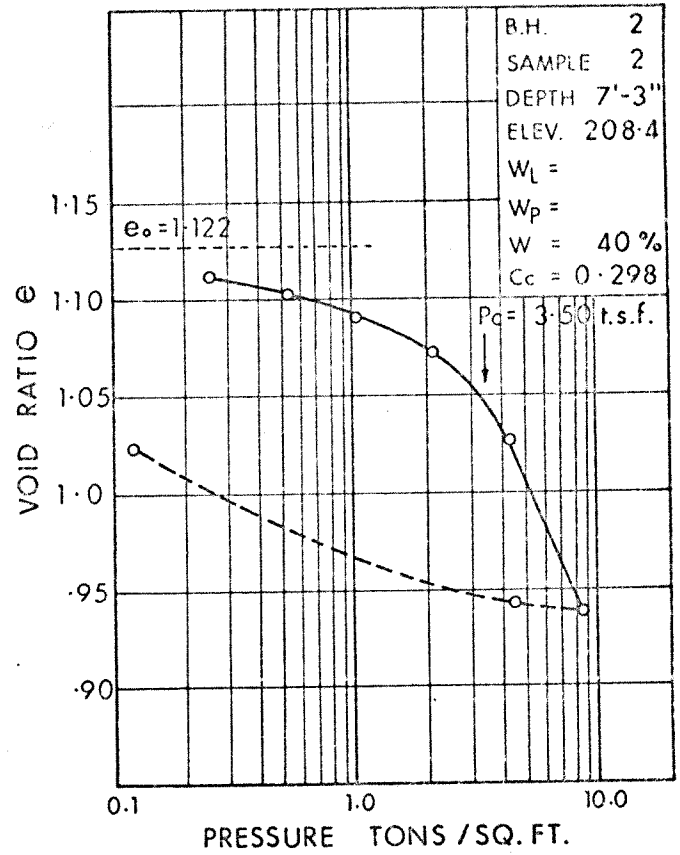
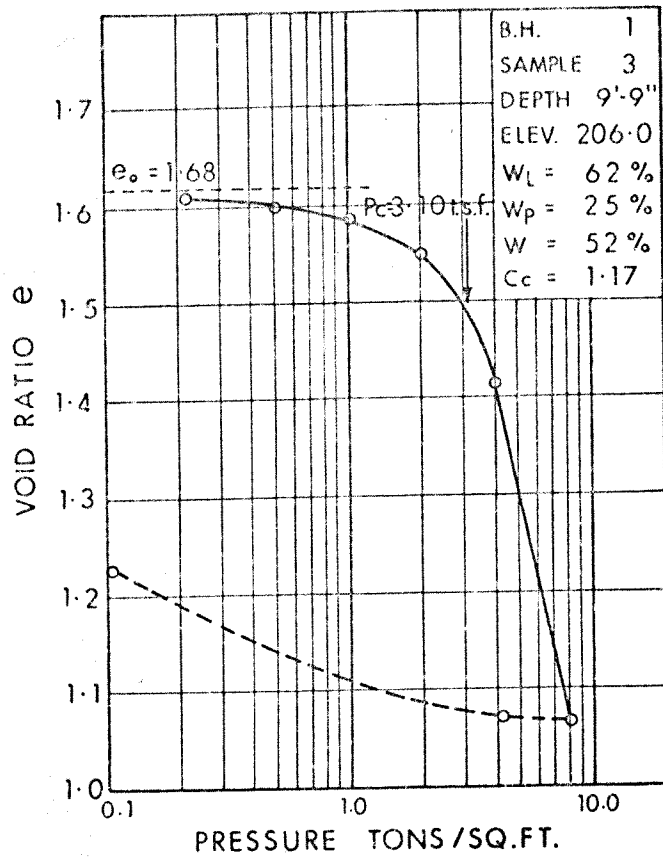
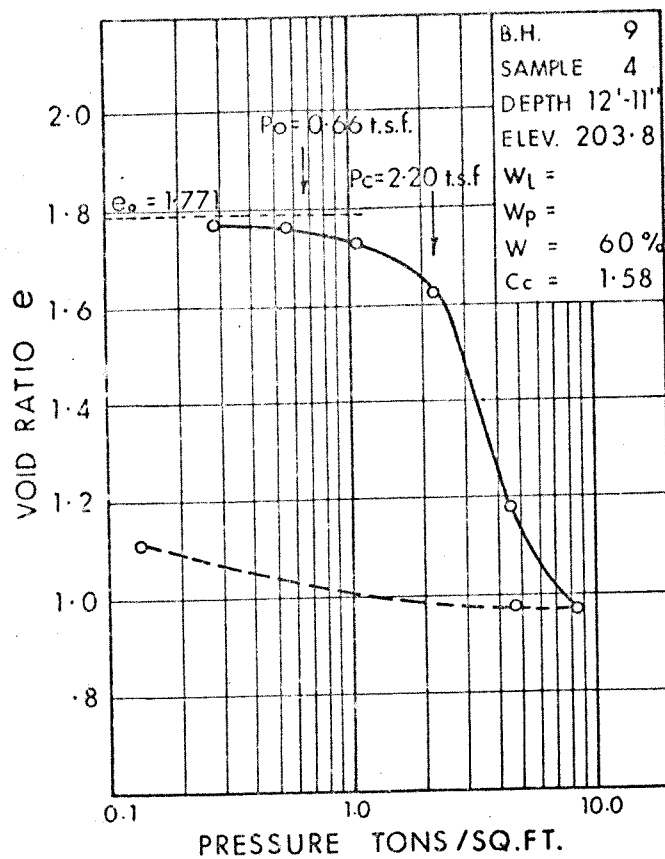


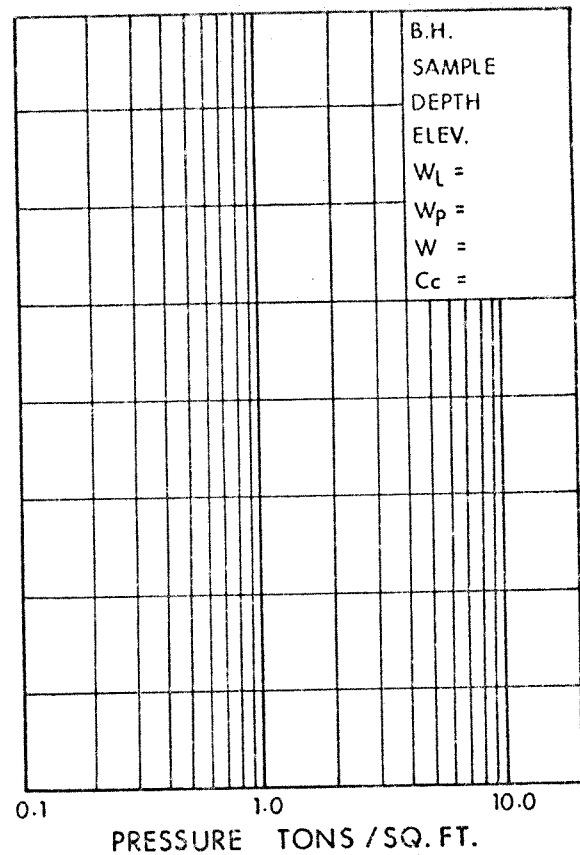
FIG. 4

VOID RATIO - PRESSURE CURVES

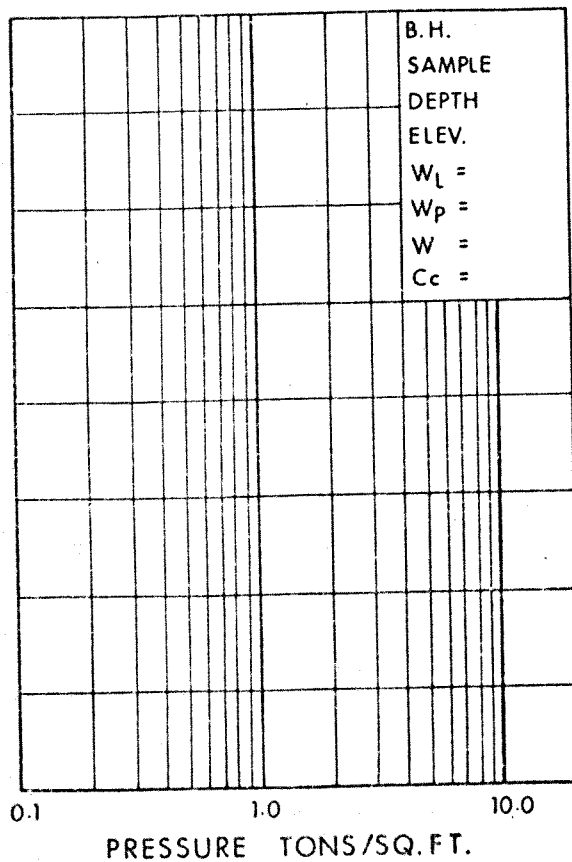
JOB NO. 71-11124



VOID RATIO e



VOID RATIO e



VOID RATIO e

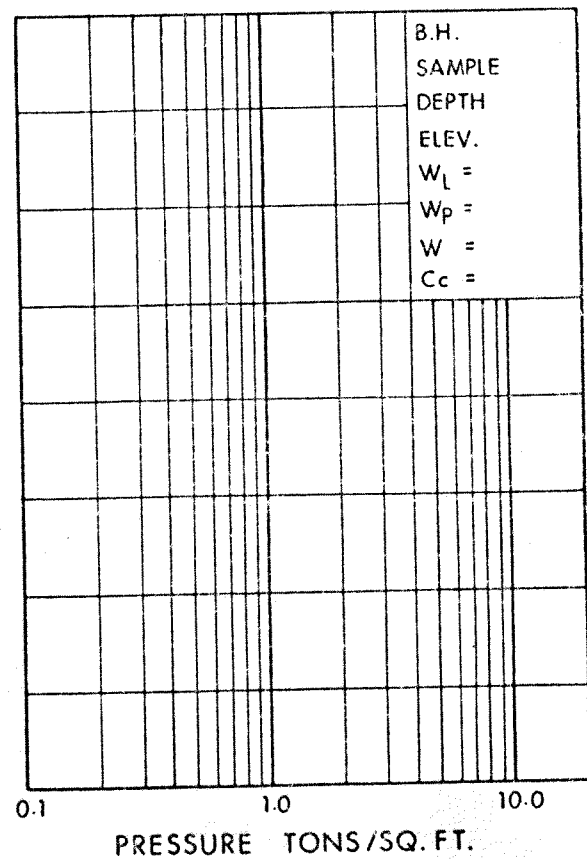


FIG. 5

APPENDIX D

Selected Site Photographs



Photograph 1: Site 3-301/1 (EBL), East side, looking North (June 14, 2017).



Photograph 2: Site 3-301/1 (EBL), CNR Tracks, looking North (June 14, 2017).

CLIENT
WSP CANADA GROUP LIMITED

CONSULTANT



YYYY-MM-DD 2018/02/21

PREPARED SAT

DESIGN SAT

REVIEW MSS

APPROVED FJH

PROJECT
CNR OVERHEAD WIDENING
SITE NOS. 3-301/1 & 3-301/2
HIGHWAY 417, OTTAWA, ONTARIO

TITLE
SELECTED SITE PHOTOGRAPHS

PROJECT No.
1662565

Phase
1110

Rev.
1

Figure
D1

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

1 in



Photograph 3: Site 3-301/2 (WBL), East side, looking North (June 14, 2017).



Photograph 4: Site 3-301/2 (WBL), CNR Tracks, looking North (June 14, 2017).

CLIENT
WSP CANADA GROUP LIMITED

CONSULTANT



GOLDER

YYYY-MM-DD 2018/02/28

PREPARED SAT

DESIGN SAT

REVIEW MSS

APPROVED FJH

PROJECT
CNR OVERHEAD WIDENING
SITE NOS. 3-301/1 & 3-301/2
HIGHWAY 417, OTTAWA, ONTARIO

TITLE
SELECTED SITE PHOTOGRAPHS

PROJECT No.
1662565

Phase
1110

Rev.
1

Figure
D2

APPENDIX E

**Basic Chemical Analysis –
Eurofins Report Numbers 1713269 and 1718216**

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: Ms. Susan Trickey
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1713269
Date Submitted: 2017-07-17
Date Reported: 2017-07-24
Project: 1662565/1110
COC #: 821253

					Lab I.D.	1306727	1306728	1306729	1306730
					Sample Matrix	Soil	Soil	Soil	Soil
					Sample Type	2017-07-05	2017-06-26	2017-07-04	2017-06-28
					Sampling Date	17-1102 SA 15/35-37	17-1107 SA5/10-12	17-1110 SA	17-1115 SA 9/20-22
					Sample I.D.			13/37.5-39.5	
Group	Analyte	MRL	Units	Guideline					
Agri. - Soil	pH	2.0			7.6	9.2	8.3	8.3	
	SO4	0.01	%		<0.01	<0.01	<0.01	<0.01	
General Chemistry	Cl	0.002	%		0.252	0.197	0.006	0.030	
	Electrical Conductivity	0.05	mS/cm		1.65	0.43	0.49	1.00	
	Resistivity	1	ohm-cm		606	2330	2040	1000	

Guideline = *** = Guideline Exceedence**

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

Certificate of Analysis

Client: Golder Associates Ltd. (Ottawa)
1931 Robertson Road
Ottawa, ON
K2H 5B7
Attention: Mr. Alex Meacoe
PO#:
Invoice to: Golder Associates Ltd. (Ottawa)

Report Number: 1718216
Date Submitted: 2017-09-21
Date Reported: 2017-09-27
Project: 1662565/1110
COC #: 823662

					Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1321667 Soil 2017-08-21 17-1103 SA9/16-18	1321668 Soil 2017-09-08 17-1104 SA10/24-25.8	1321669 Soil 2017-08-22 17-1106 SA8/20-22	1321670 Soil 2017-08-26 17-1111 SA11/30-32
Group	Analyte	MRL	Units	Guideline					
Agri. - Soil	pH	2.0			7.6	8.8	7.6	8.7	
	SO4	0.01	%		<0.01	0.02	0.02	0.04	
General Chemistry	Cl	0.002	%		0.032	0.006	0.219	0.008	
	Electrical Conductivity	0.05	mS/cm		0.33	0.20	1.97	0.28	
	Resistivity	1	ohm-cm		3030	5000	508	3570	

					Lab I.D. Sample Matrix Sample Type Sampling Date Sample I.D.	1321671 Soil 2017-08-27 17-11113 SA6/10-12	1321672 Soil 2017-08-21 17-1114 SA 17/36-38
Group	Analyte	MRL	Units	Guideline			
Agri. - Soil	pH	2.0			8.1	8.6	
	SO4	0.01	%		<0.01	0.04	
General Chemistry	Cl	0.002	%		0.015	0.008	
	Electrical Conductivity	0.05	mS/cm		0.38	0.29	
	Resistivity	1	ohm-cm		2630	3450	

Guideline = * = Guideline Exceedence

All analysis completed in Ottawa, Ontario (unless otherwise indicated by ** which indicates analysis was completed in Mississauga, Ontario).
Results relate only to the parameters tested on the samples submitted.
Methods references and/or additional QA/QC information available on request.

MRL = Method Reporting Limit, AO = Aesthetic Objective, OG = Operational Guideline, MAC = Maximum Acceptable Concentration, IMAC = Interim Maximum Acceptable Concentration, STD = Standard, PWQO = Provincial Water Quality Guideline, IPWQO = Interim Provincial Water Quality Objective, TDR = Typical Desired Range

APPENDIX F

Results of MASW and VSP Testing

DATE December 19, 2017**PROJECT No.** 1662565/1110**TO** Susan Trickey
Golder Associates Ltd.**FROM** Stephane Sol, Christopher Phillips**EMAIL** ssol@golder.com;cphillips@golder.com**CHBDC SEISMIC SITE CLASS TESTING RESULTS
CNR OVERHEAD AND HWY 417, OTTAWA, ONTARIO**

This technical memorandum presents the results of two Multichannel Analysis of Surface Waves (MASW) tests performed for the purpose of the Canadian Highway Bridge Design Code (CHBDC 2014) Seismic Site Classification for the widening of the CNR Overhead (Figure 1). The tests are located on each side of the CNR line at the intersection with Highway 417 in Ottawa. The geophysical testing was performed by Golder Associates Ltd. (Golder) personnel on May 25, 2017.

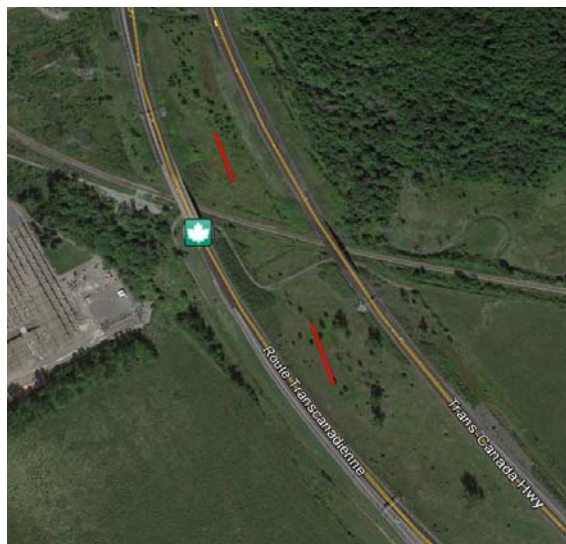


Figure 1: MASW Location Site Map (MASW Lines in red – Line 1 (North) and Line 2 (South))

Golder Associates Ltd.6925 Century Avenue, Suite #100, Mississauga, Ontario, Canada L5N 7K2
Tel: +1 (905) 567 4444 Fax: +1 (905) 567 6561 www.golder.com**Golder Associates: Operations in Africa, Asia, Australasia, Europe, North America and South America**

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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 25, 2017, by personnel from the Golder Mississauga and Ottawa offices. For each MASW line, a series of 24 low frequency (4.5 Hz) geophones were laid out at 3 m intervals. Both active and passive readings were recorded along the 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, and 15 m from the end and collinear to the geophone array. An example of active seismic records collected at each line are shown in Figures 2 and 3, below.

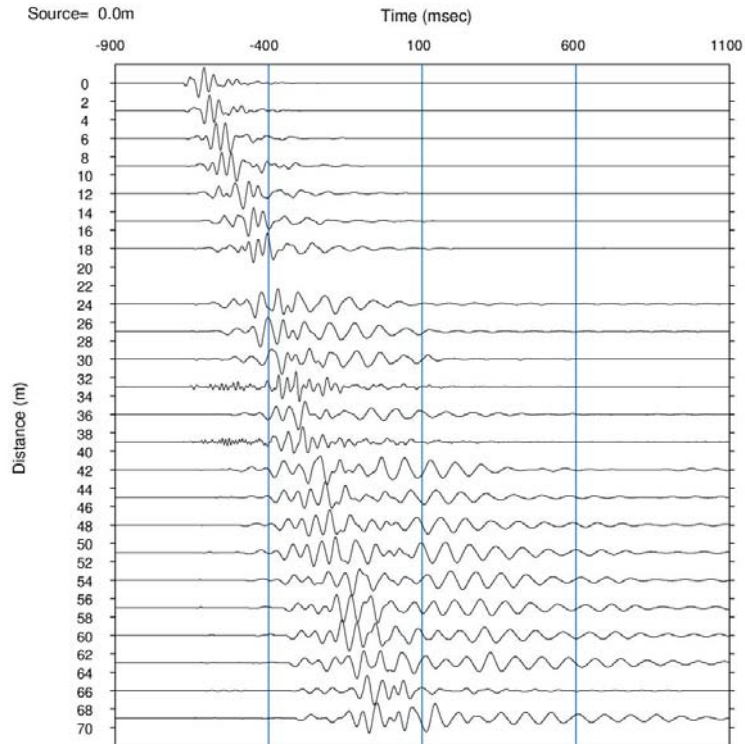


Figure 2: Typical seismic record collected at the site of MASW Line 1 (North).

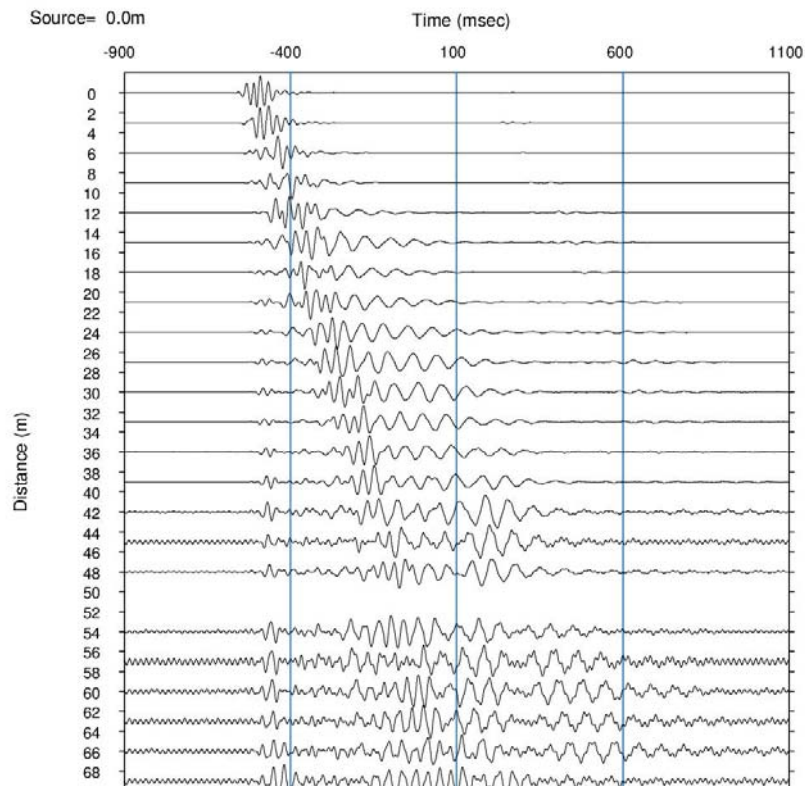


Figure 3: Typical seismic record collected at the site of MASW Line 2 (South).

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 Figure 4 for Line 1 and Figure 5 for Line 2. Shear wave velocity profiles were generated through inverse modelling to best fit the calculated dispersion curves. The active survey of Line 1 provided a dispersion curve with a suitable frequency range (5.8-23 Hz). The active survey of Line 2 provided a dispersion curve with a suitable frequency range (6.8-19.5 Hz). The minimum measured surface wave frequency with sufficient signal-to-noise ratio to accurately measure phase velocity was approximately 5.8 Hz at Line 1 and 6.8 Hz at Line 2.

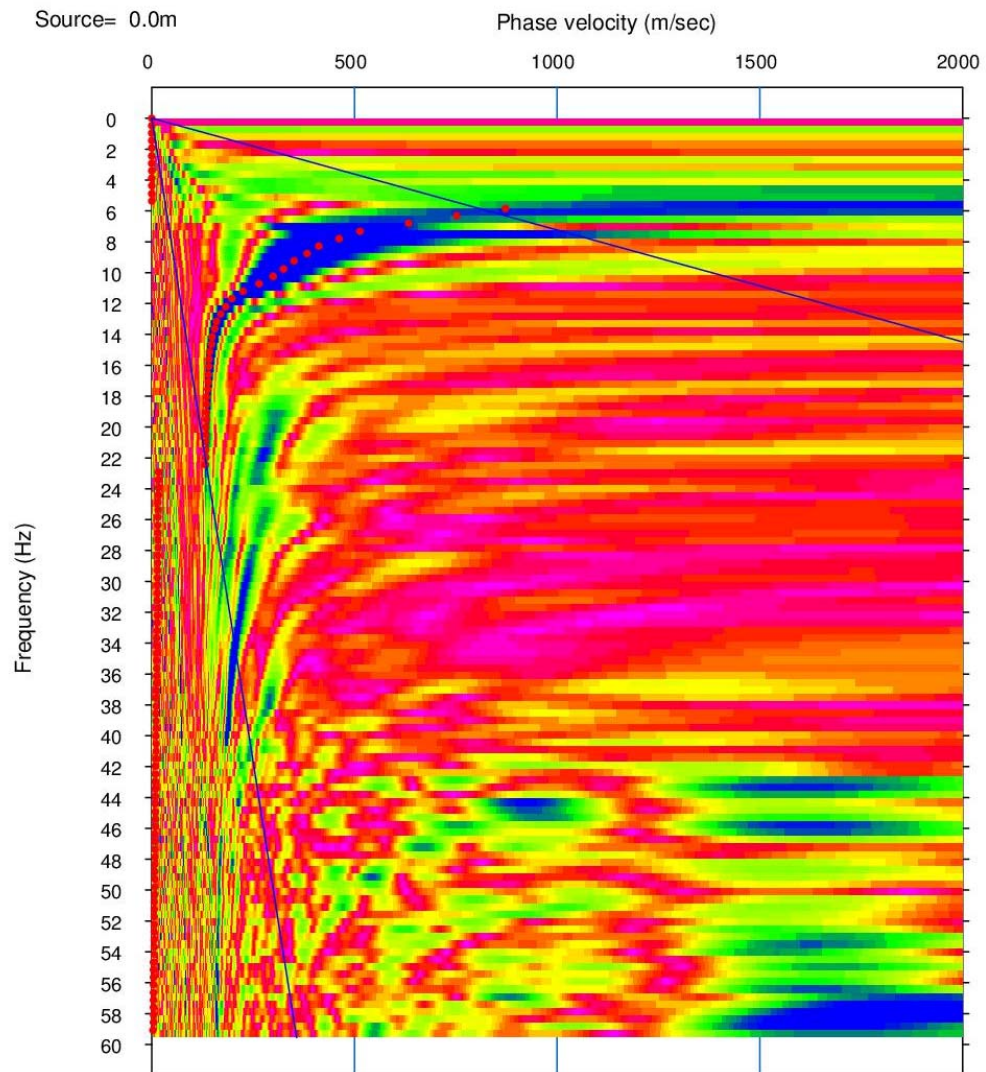


Figure 4: Active MASW Dispersion Curve Picks (red dots) along MASW Line 1

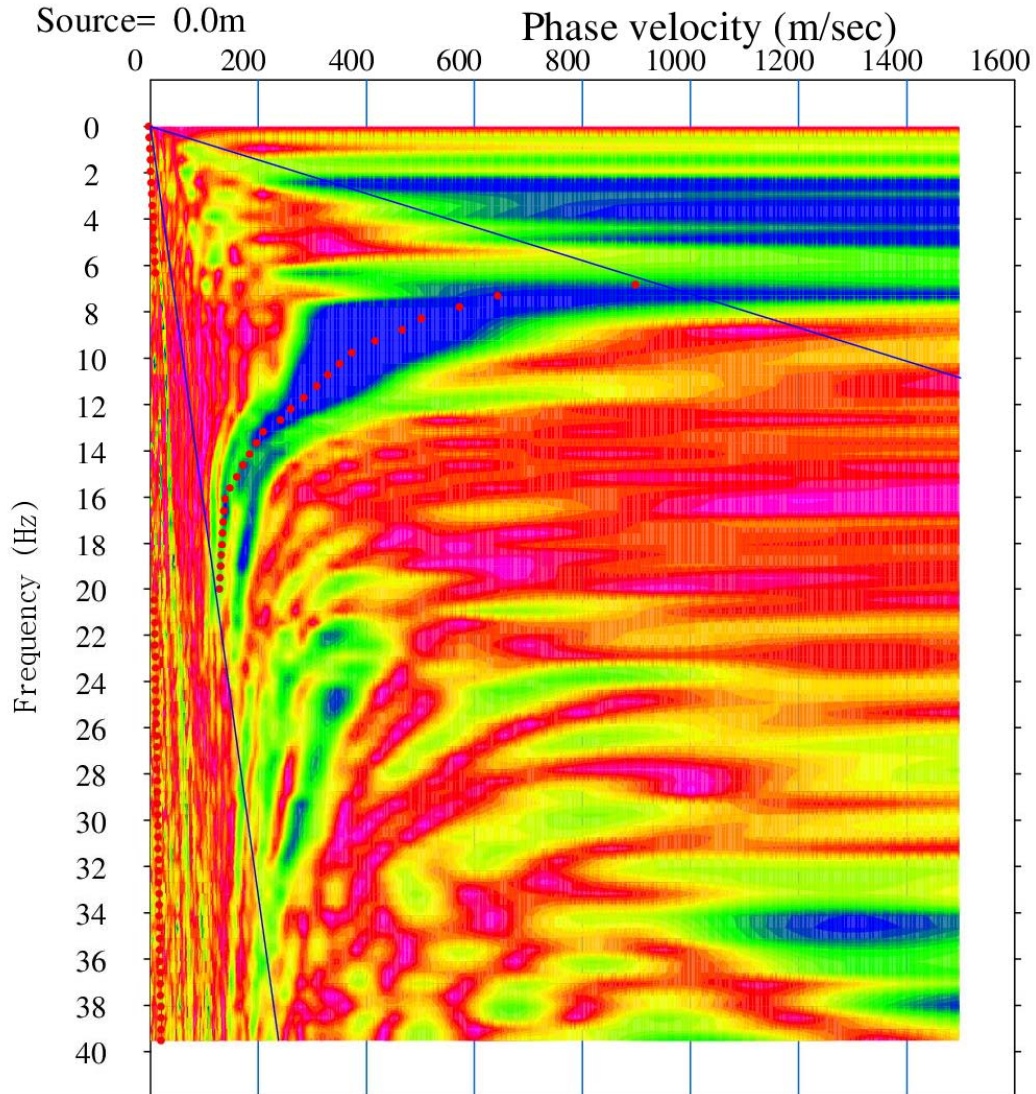


Figure 5: Active MASW Dispersion Curve Picks (red dots) along MASW Line 2

Results

The MASW test results are presented in Figures 6 and 7, which present the calculated shear wave velocity profile derived from the field testing along MASW Lines 1 and 2, respectively. The results along MASW Line 1 have been calculated using a weight-drop located 5 m from the last geophone. The results along MASW Line 2 have also been calculated using a weight-drop located 5 m from the last geophone. The field collected dispersion curves are compared with the model generated dispersion curves on Figures 8 and 9 for MASW Lines 1 and 2, respectively. 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 both lines.

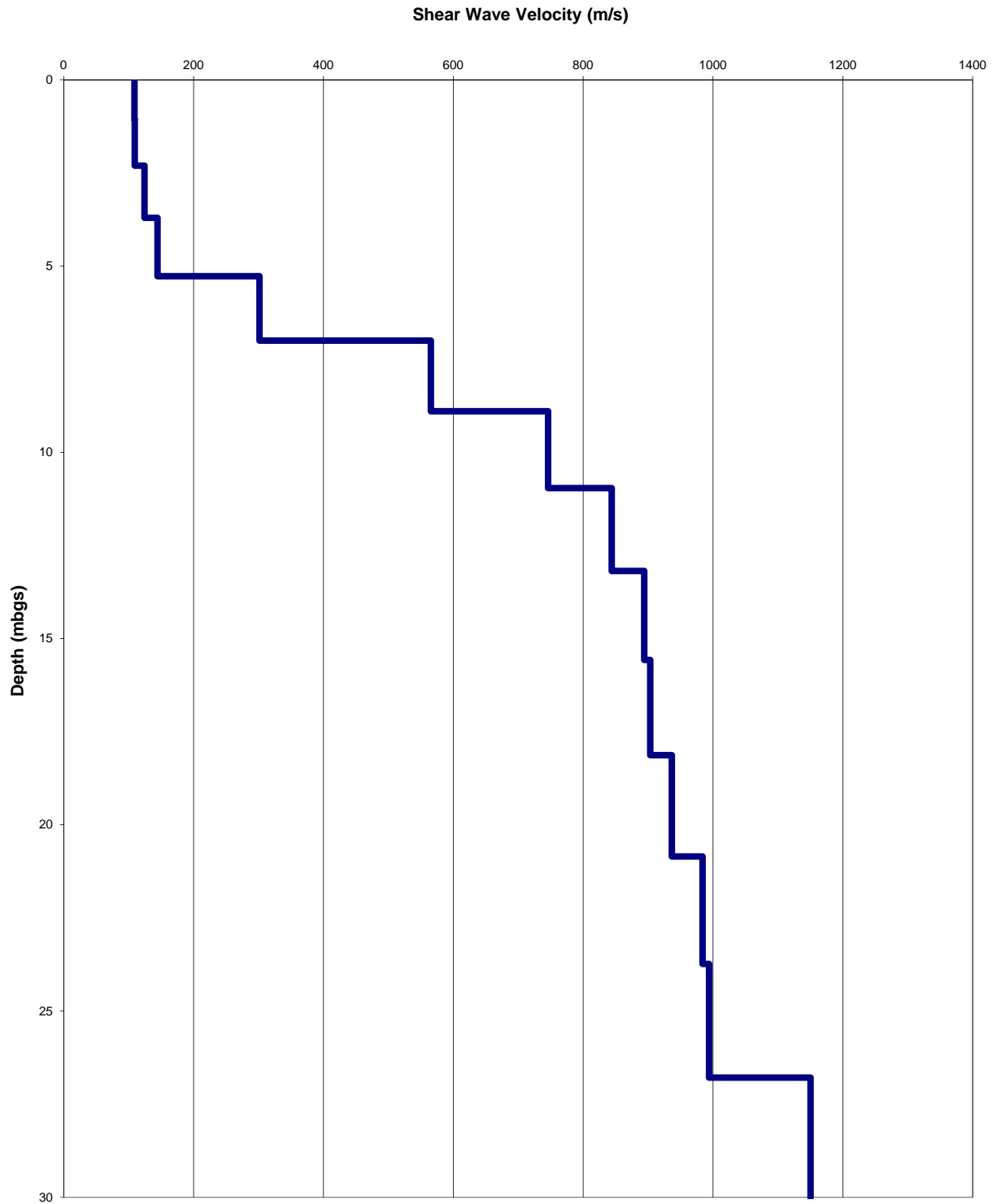


Figure 6: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 1

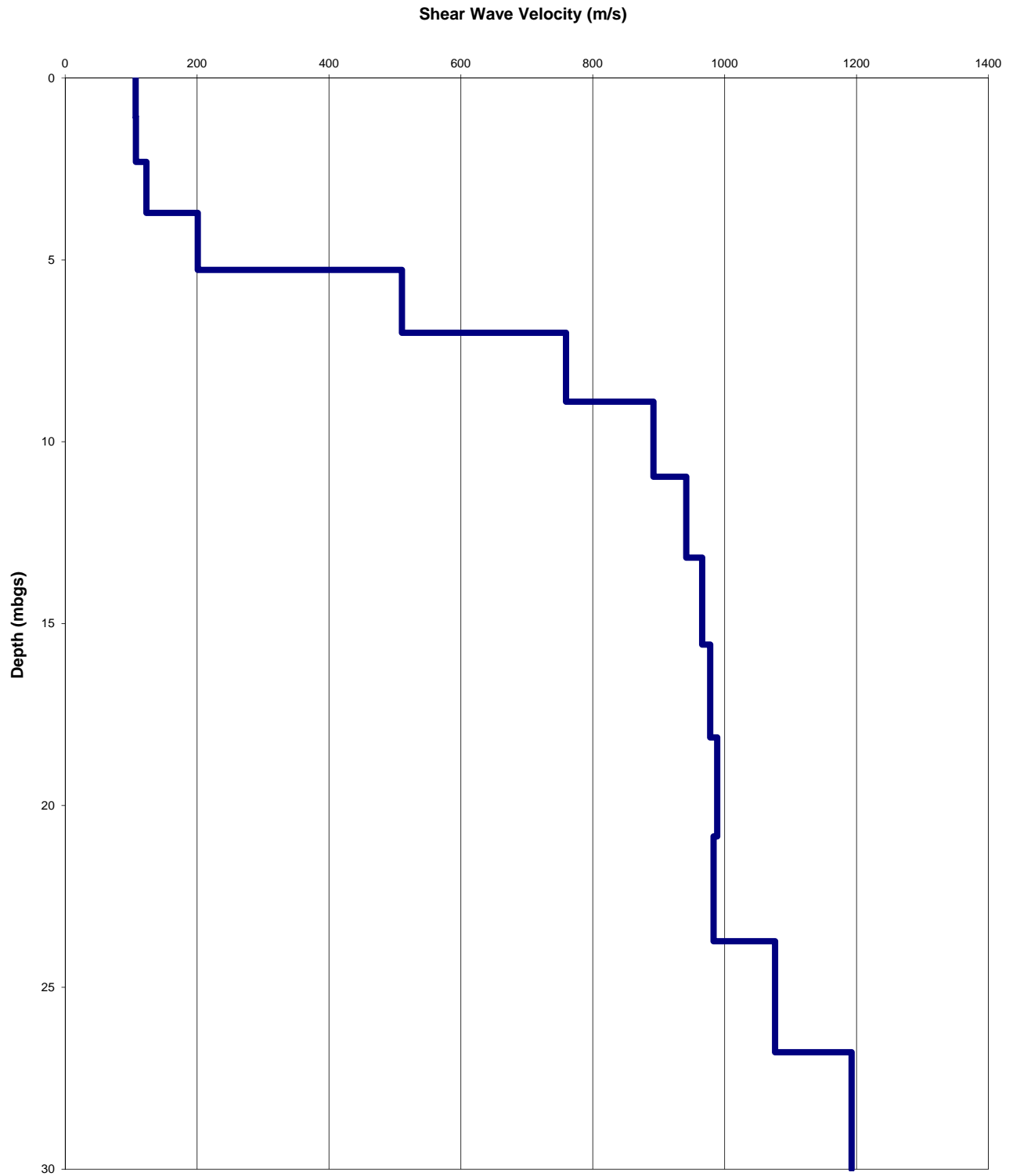


Figure 7: MASW Modelled Shear-Wave Velocity Depth profile along MASW Line 2

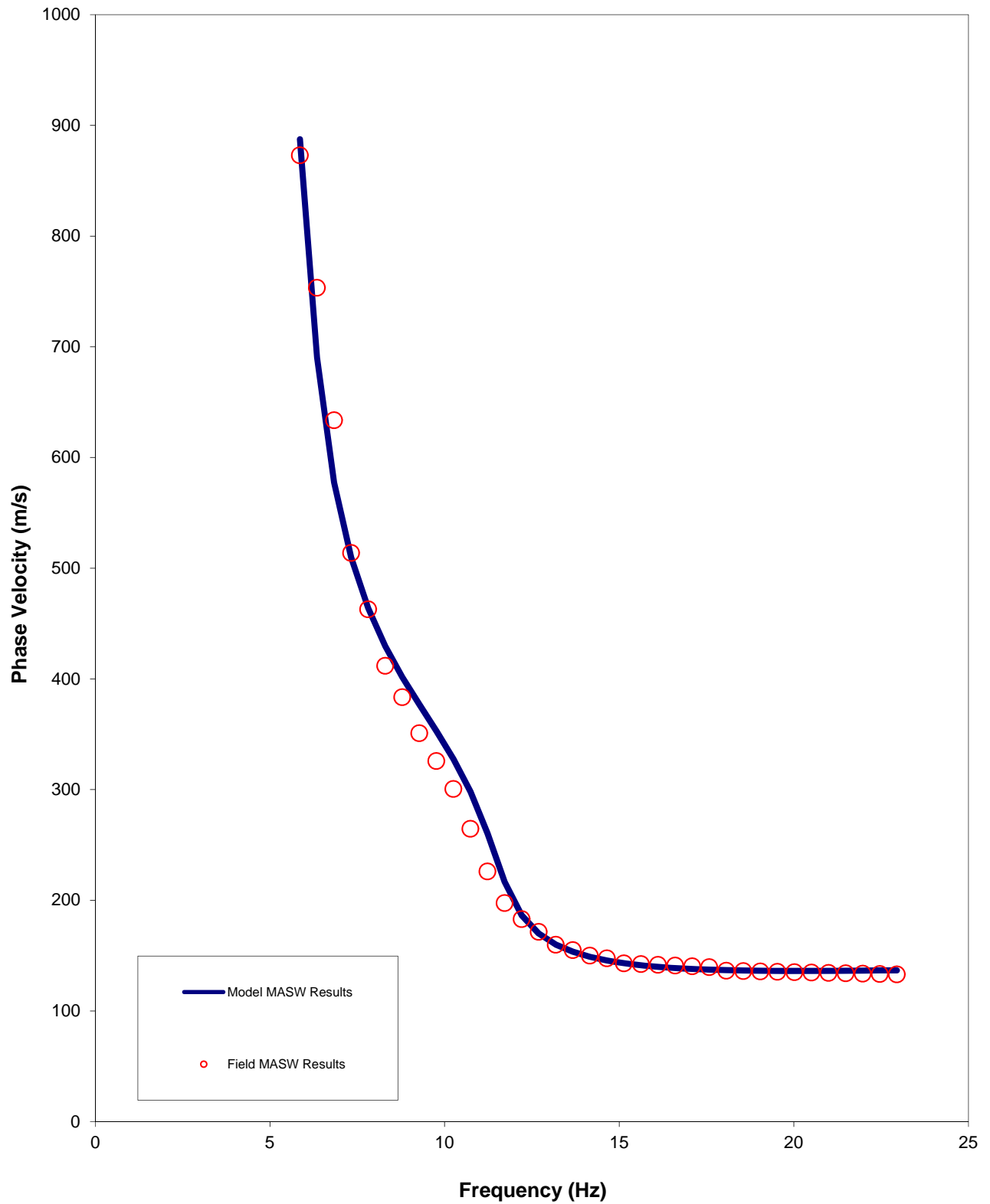


Figure 8: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 1

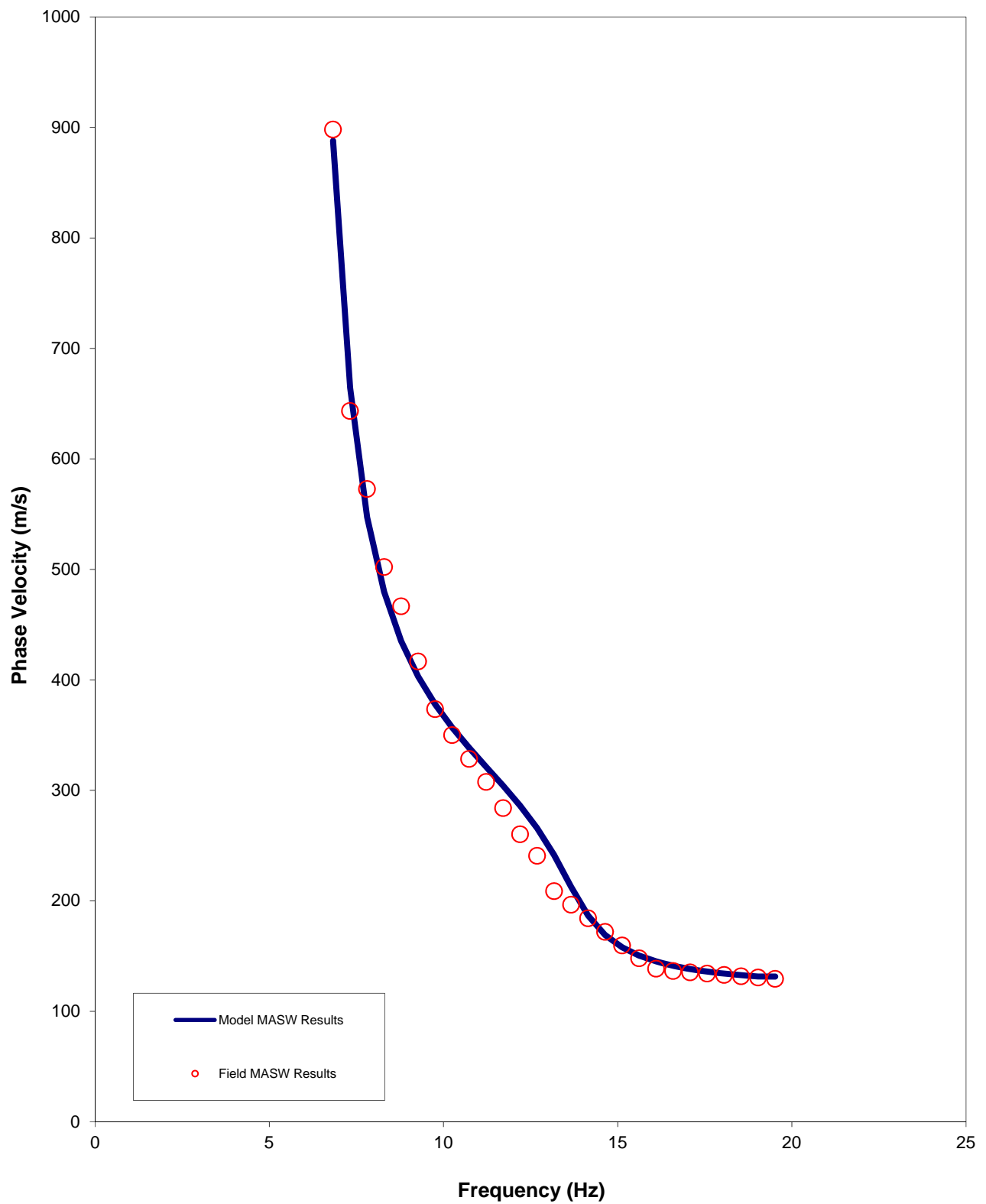


Figure 9: Comparison of Field (red dots) vs. Modelled Data (blue line) along MASW Line 2

To calculate the average shear-wave velocity as required by the CHBDC 2014, the results were modelled to 30 metres below ground surface. The average shear-wave velocity along MASW Line 1 in the north was found to be 400 m/s (Table 1). The average shear-wave velocity along MASW Line 2 in the south was found to be 444 m/s (Table 2).

Table 1: Shear-Wave Velocity Profile along MASW Line 1

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	109	0.009830
1.07	2.31	1.24	109	0.011296
2.31	3.71	1.40	124	0.011266
3.71	5.27	1.56	145	0.010832
5.27	7.01	1.74	302	0.005737
7.01	8.90	1.89	566	0.003352
8.90	10.96	2.06	746	0.002762
10.96	13.19	2.23	844	0.002636
13.19	15.58	2.39	894	0.002673
15.58	18.13	2.55	903	0.002828
18.13	20.85	2.72	937	0.002904
20.85	23.74	2.89	984	0.002931
23.74	26.79	3.05	994	0.003068
26.79	30.00	3.21	1150	0.002794
Vs Average to 30 mbgs (m/s)				400

Table 2: Shear-Wave Velocity Profile along MASW Line 2

Model Layer (mbgs)		Layer Thickness (m)	Shear Wave Velocity (m/s)	Shear Wave Travel Time Through Layer (s)
Top	Bottom			
0.00	1.07	1.07	107	0.010013
1.07	2.31	1.24	107	0.011518
2.31	3.71	1.40	123	0.011356
3.71	5.27	1.56	201	0.007781
5.27	7.01	1.74	511	0.003388
7.01	8.90	1.89	760	0.002496
8.90	10.96	2.06	892	0.002310
10.96	13.19	2.23	942	0.002362
13.19	15.58	2.39	966	0.002474
15.58	18.13	2.55	978	0.002612
18.13	20.85	2.72	989	0.002751
20.85	23.74	2.89	983	0.002933
23.74	26.79	3.05	1077	0.002832
26.79	30.00	3.21	1193	0.002695
Vs Average to 30 mbgs (m/s)				444

The CHBDC 2014 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.

Limitations

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

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

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

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

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

Closure

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

GOLDER ASSOCIATES LTD.



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



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

SS/CRP/jl

[https://golderassociates.sharepoint.com/sites/11263g/shared documents/01_foundations/12_geophysics/cnr-phase 1110 masw/report/final_december 2017/1662565 1110 tech memo 2017dec mto cnr.docx](https://golderassociates.sharepoint.com/sites/11263g/shared%20documents/01_foundations/12_geophysics/cnr-phase%201110_masw/report/final_december%202017/1662565%201110_tech%20memo%202017dec_mto_cnr.docx)

DATE March 06, 2018**PROJECT No.** 1662565/1110**TO** Susan Trickey
Golder Associates Ltd.**FROM** Stephane Sol, Christopher Phillips**EMAIL** ssol@golder.com, cphillips@golder.com**VERTICAL SEISMIC PROFILING TEST RESULTS
CNR OVERPASS ALONG HWY 17, OTTAWA, ONTARIO**

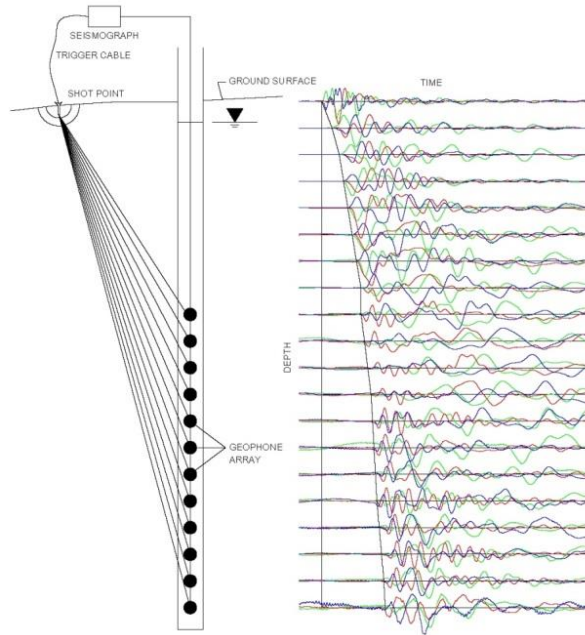
This memorandum presents the results of a Vertical Seismic Profiling (VSP) testing carried out at the CNR Overpass along HWY417 in Ottawa, Ontario. VSP testing was completed in Borehole 17-1110. VSP testing was carried out on July 27, 2017. Borehole 17-1110 was drilled to an approximate depth of 21.9 m below the existing ground surface and then cased with a PVC pipe grouted in place. The overburden consisted of approximately 0.7 m of asphalt and road granular, overlying approximately 10 m of sand fill, 4.2 m of weathered and unweathered silty clay, and 1.3 m of glacial till. At Borehole 17-1110 the shale bedrock was located at approximately 16.2 mbgs to the bottom of the hole.

Methodology

For the VSP method, seismic energy is generated at the ground surface by an active seismic source and recorded by a geophone located in a nearby borehole at a known depth. The active seismic source can be either compression or shear wave. The time required for the energy to travel from the source to the receiver (geophone) provides a measurement of the average compression or shear wave seismic velocity of the medium between the source and the receiver. Data obtained from different geophone depths are used to calculate a detailed vertical seismic velocity profile of the subsurface in the immediate vicinity of the test borehole.

The high resolution results of a VSP survey are often used for earthquake engineering site classification, as per the 2014 Canadian Highway Bridge Design Code (CHBDC 2014).





Example 1: Layout and resulting time traces from a VSP survey.

Fieldwork

The fieldwork was carried out on July 27, 2017, by personnel from the Golder Mississauga and Ottawa offices.

Both compression and shear-wave seismic sources were used and both were located 2 m from the borehole. The seismic source for the compression wave test consisted of a 9.9 kg sledge hammer vertically impacted on a metal plate. The seismic source for the shear-wave test consisted of a 2.4 m long, 150 mm by 150 mm wooden beam, weighted by a vehicle and horizontally struck with a 9.9 kilogram sledge hammer on alternate ends of the beam to induce polarized shear waves. The shear source was coupled to the ground surface by parking a vehicle on top of it. Test measurements started at ground surface and were recorded in the borehole with a 3-component receiver spaced at 1-metre intervals below the ground surface to a depth of 12 m and spaced at 0.5 metre intervals from 12 m to 21.5 m.

The seismic records collected for each source location were stacked a minimum of five times to minimize the effects of ambient background seismic noise on the collected data. The data was sampled at 0.020833 ms intervals and a total time window of 0.341 s was collected for each seismic shot.

Data Processing

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

- 1) Combination of seismic records to present seismic traces for all depth intervals on a single plot for each seismic source and for each component;
- 2) Low pass filtering of data to remove spurious high frequency noise;
- 3) First break picking of the compression and shear wave arrivals; and,
- 4) Calculation of the average compression and shear wave velocity to each tested depth interval.

Processing of the VSP data was completed using the SeisImager/SW software package (Geometrics Inc.). The seismic records are presented on the following two plots and show the first break picks of the compression wave at Borehole 17-1110 (Figure 1) and shear wave arrivals at Borehole 17-1110 (Figure 2) overlaid on the seismic waveform traces recorded at the different geophone depths for each borehole. The arrivals were picked on the vertical component for the compression source and on the two horizontal components for the shear source.

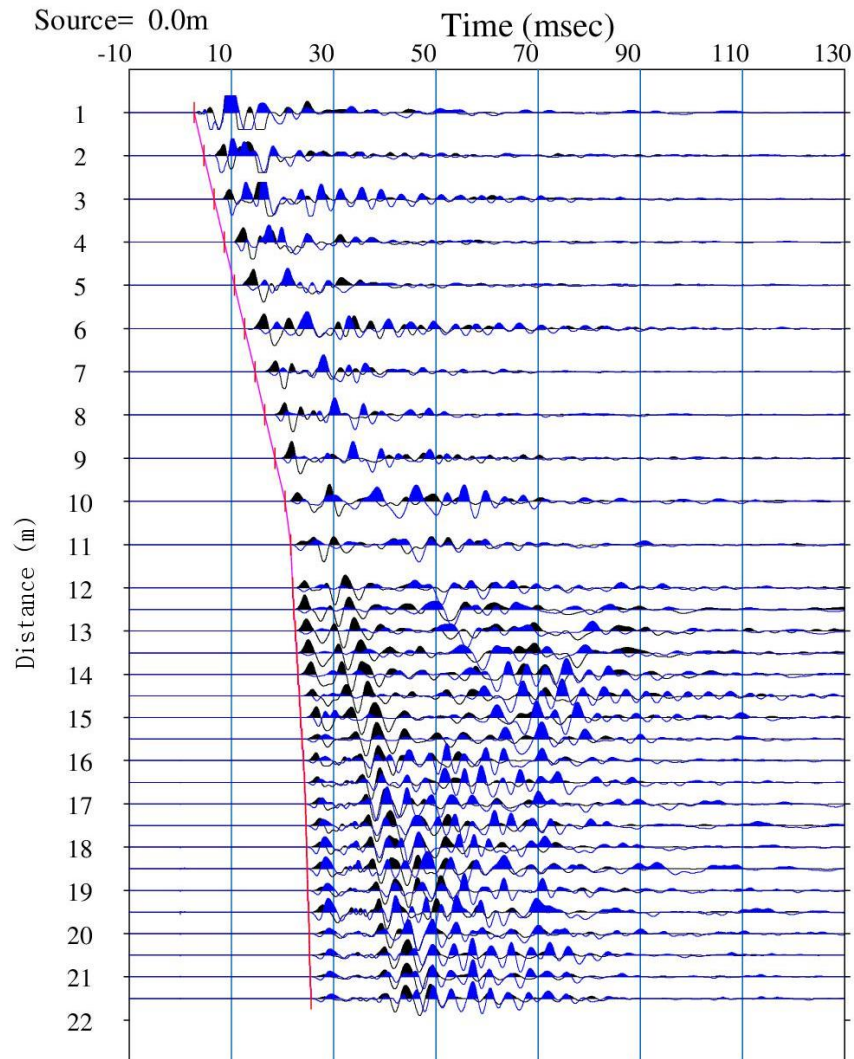


Figure 1: First break picking of compression wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 17-1110.

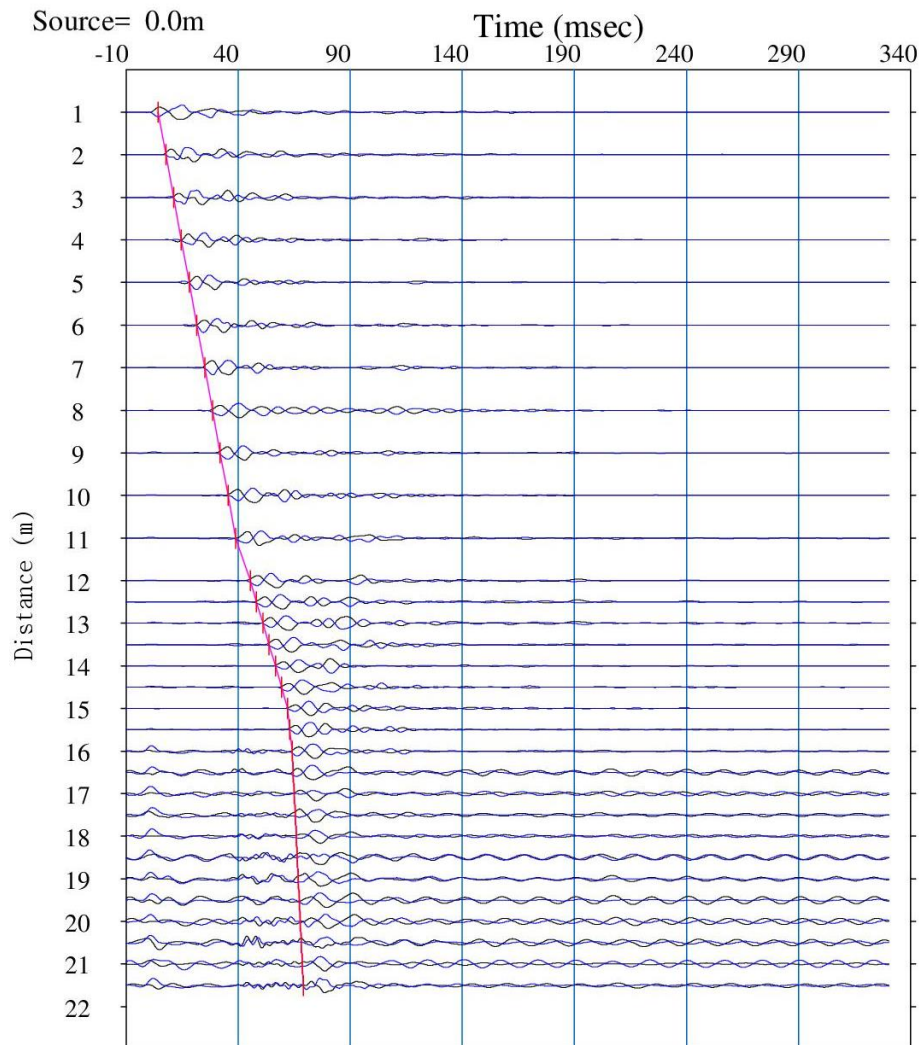


Figure 2: First break picking of shear wave arrivals (red) along the seismic traces recorded at each receiver depth of Borehole 17-1110.

Results

The VSP results at Borehole 17-1110 are summarized in Table 1. The compression and shear wave layer velocities were calculated by best fitting a theoretical travel time model to the field data. The depths presented on the table are relative to ground surface.

The estimated dynamic engineering moduli, based on the calculated wave velocities, are also presented in Table 1. The engineering moduli were calculated using an estimated bulk density of 2,140 kg/m³ for sand fill, 1,835 kg/m³ for the weathered silty clay, 1,750 kg/m³ for unweathered silty clay, 2,365 kg/m³ for glacial till and an estimated shale bedrock bulk density of 2,400 kg/m³ based on the borehole log.

The average shear wave velocity from ground surface to a depth of 30 m (Vs30) was measured to be 393 m/s at Borehole 17-1110. The average velocity was calculated assuming that the velocity from 21.5 m to a depth of

30 m was constant with an average shear wave velocity value of 1,100 m/s which is equal to the velocity of the bedrock at the bottom of the borehole.

Limitations

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

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

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

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

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

Closure

We trust that these results meet your current needs. If you have any questions or require clarification, please contact the undersigned at your convenience.

GOLDER ASSOCIATES LTD.



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



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

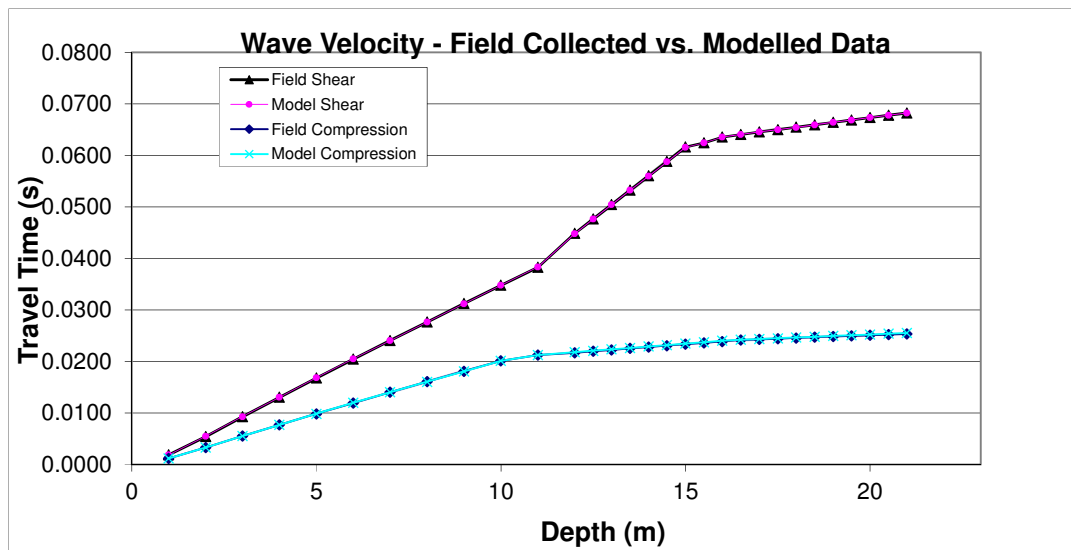
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Attachment: Table 1 – Shear Wave Velocity Profile at Borehole 17-1110

TABLE 1
SHEAR WAVE VELOCITY PROFILE AT BOREHOLE 17-1110

Layer Depth (m)		Velocities (m/s)		Estimated Bulk Density (kg/m ³)	Dynamic Engineering Properties			
Top	Bottom	Compressional Wave	Shear Wave		Poissons Ratio	Shear Modulus (MPa)	Deformation Modulus (MPa)	Bulk Modulus (MPa)
0.0	1.0	830	530	2140	0.16	601	1390	673
1.0	2.0	475	280	2140	0.23	168	414	259
2.0	3.0	450	260	2140	0.25	145	361	240
3.0	4.0	455	265	2140	0.24	150	374	243
4.0	5.0	470	265	2140	0.27	150	381	272
5.0	6.0	470	275	2140	0.24	162	401	257
6.0	7.0	485	280	2140	0.25	168	419	280
7.0	8.0	495	280	2140	0.26	168	424	301
8.0	9.0	490	280	2140	0.26	168	422	290
9.0	10.0	490	280	2140	0.26	168	422	290
10.0	11.0	900	280	2140	0.45	168	485	1510
11.0	12.0	1800	155	1835	0.50	44	132	5887
12.0	12.5	1800	175	1835	0.50	56	168	5870
12.5	13.0	1800	180	1835	0.49	59	178	5866
13.0	13.5	1800	180	1750	0.49	57	170	5594
13.5	14.0	1800	180	1750	0.49	57	170	5594
14.0	14.5	1800	180	1750	0.49	57	170	5594
14.5	15.0	1800	180	1750	0.49	57	170	5594
15.0	15.5	1800	550	2365	0.45	715	2073	6709
15.5	16.0	1800	450	2365	0.47	479	1405	7024
16.0	16.5	2900	1000	2365	0.43	2365	6776	16736
16.5	17.0	3300	1100	2400	0.44	2904	8349	22264
17.0	17.5	3300	1100	2400	0.44	2904	8349	22264
17.5	18.0	3400	1100	2400	0.44	2904	8372	23872
18.0	18.5	3400	1050	2400	0.45	2646	7659	24216
18.5	19.0	3400	1050	2400	0.45	2646	7659	24216
19.0	19.5	3400	1050	2400	0.45	2646	7659	24216
19.5	20.0	3400	1100	2400	0.44	2904	8372	23872
20.0	20.5	3400	1100	2400	0.44	2904	8372	23872
20.5	21.0	3400	1100	2400	0.44	2904	8372	23872
21.0	21.5	3400	1100	2400	0.44	2904	8372	23872

**Notes**

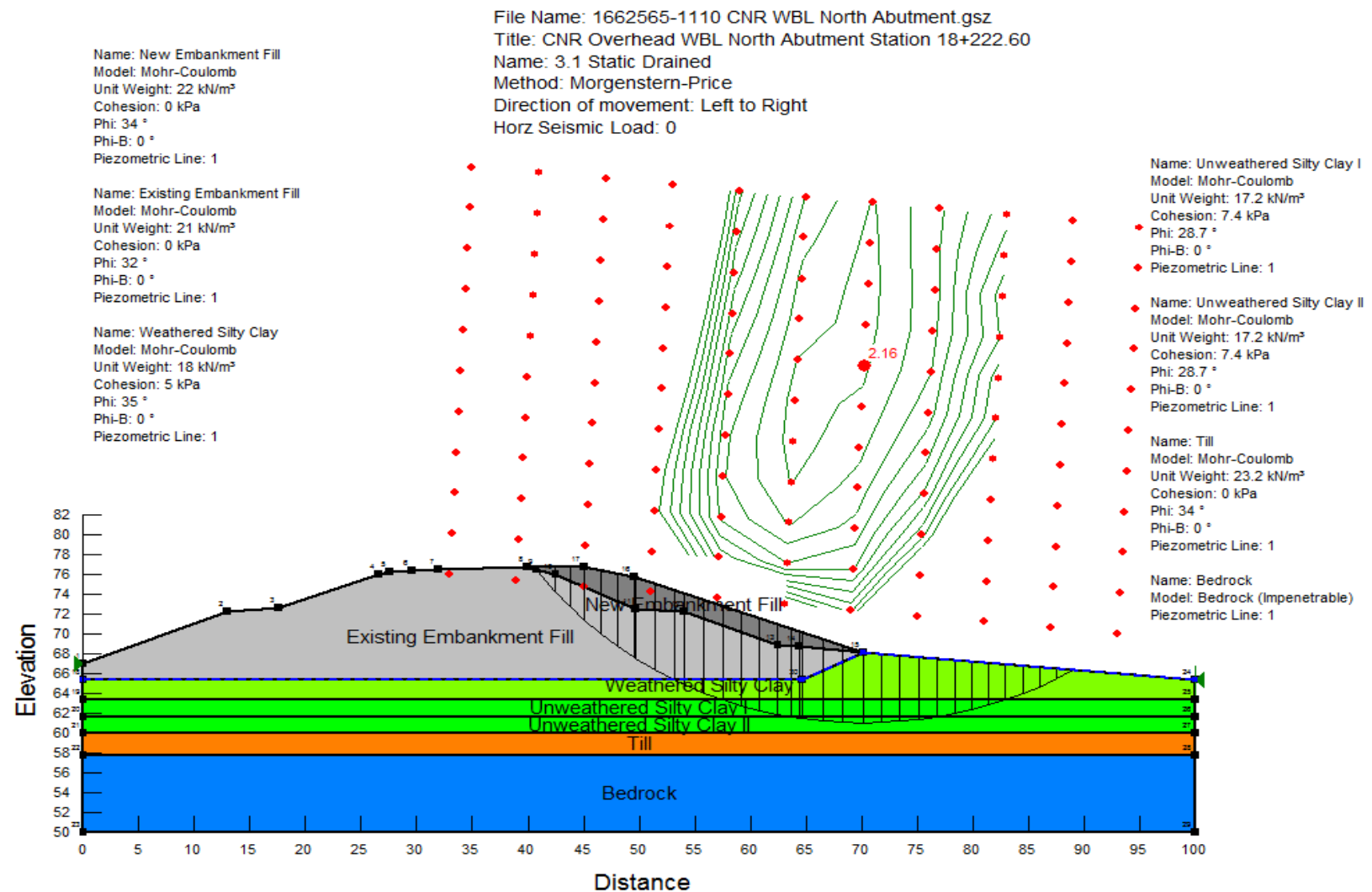
1. Depth Presented relative to ground surface.
2. This Table to be analyzed in conjunction with the accompanying report.

APPENDIX G

Results of Analysis

Figures G1 to G3 - Slope Stability Assessment

Figures G4 to G13 - P-Y Curves



Slope Stability Assessment - Static Drained

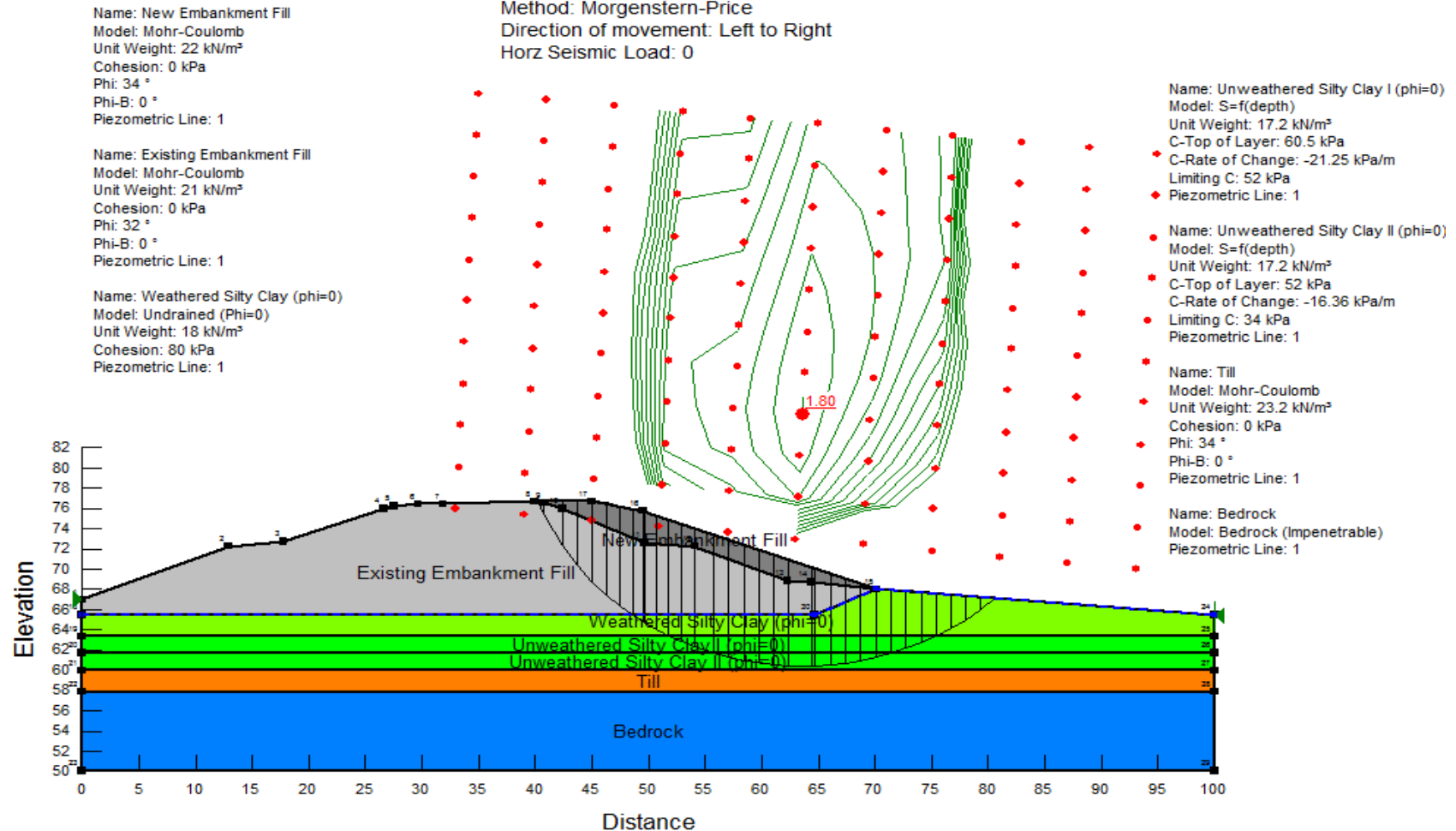
CNR Overhead

Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	WAM
Date:	14/12/2017
Checked:	SAT
Review:	MSS

Figure G1

File Name: 1662565-1110 CNR WBL North Abutment.gsz
 Title: CNR Overhead WBL North Abutment Station 18+222.60
 Name: 3.2 Static Undrained
 Method: Morgenstern-Price
 Direction of movement: Left to Right
 Horz Seismic Load: 0



Slope Stability Assessment - Static Undrained

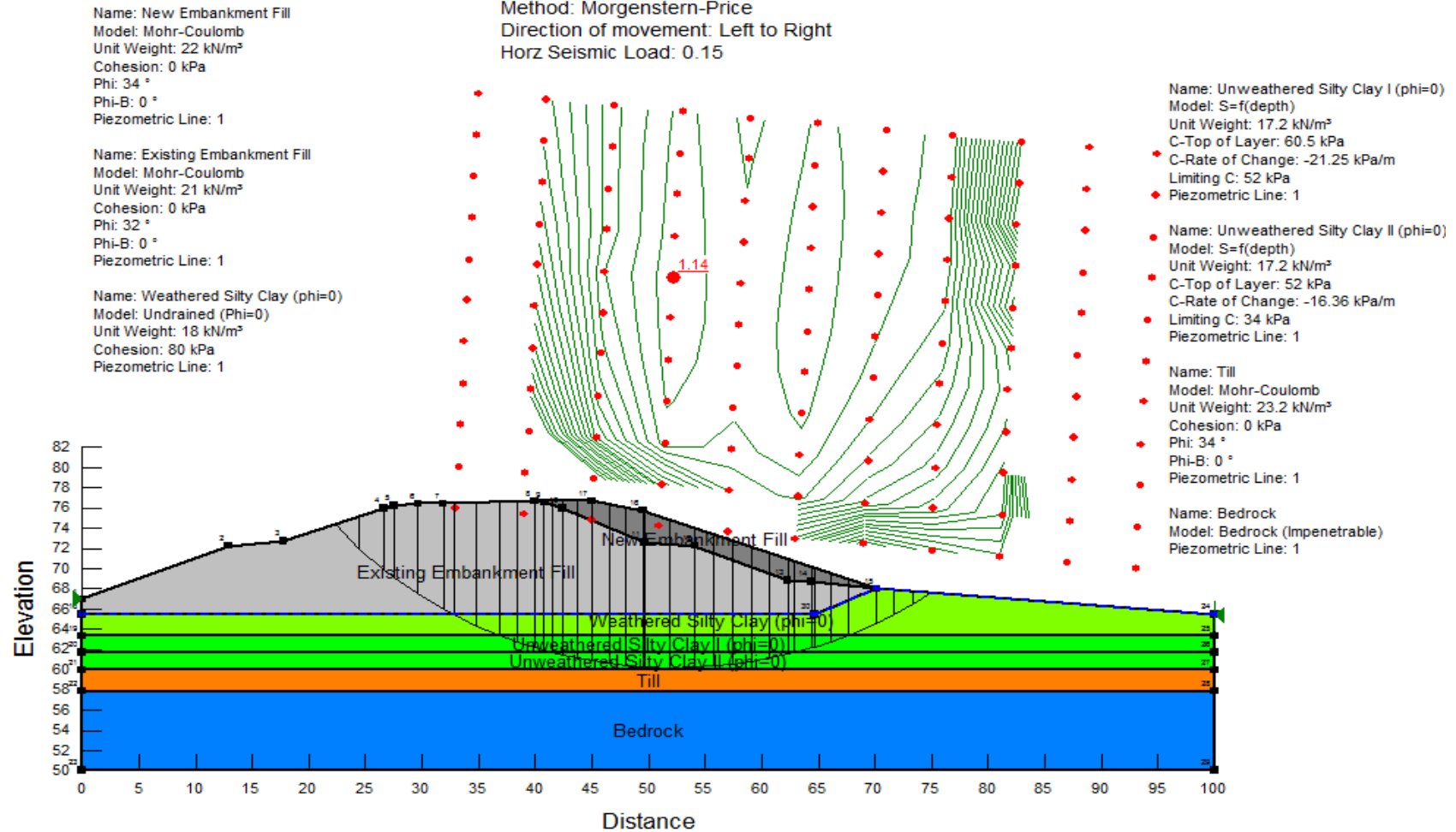
CNR Overhead

Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	WAM
Date:	14/12/2017
Checked:	SAT
Review:	MSS

Figure G2

File Name: 1662565-1110 CNR WBL North Abutment.gsz
 Title: CNR Overhead WBL North Abutment Station 18+222.60
 Name: 3.3 Seismic Undrained
 Method: Morgenstern-Price
 Direction of movement: Left to Right
 Horz Seismic Load: 0.15



Slope Stability Assessment - Seismic Undrained

CNR Overhead

Ottawa, Ontario

Project No.	1662565 / 1110
Drawn:	WAM
Date:	14/12/2017
Checked:	SAT
Review:	MSS

Figure G3

P-Y CURVES - STATIC

CNR Overpass
H-Pile 310x110 - North Abutment (Boreholes 17-1102, 17-1115)

Figure G4

SUMMARY OF P-Y CURVES FOR H-PILE 310x110 - NORTH ABUTMENT

Description		Embankment Fill																								Weathered Silty Clay Crust												Grey Silty Clay			
Depth (z) *	Elevation	z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m		z= 9.5 m		z= 10.0 m		z= 10.5 m		z= 11.0 m		z= 11.5 m					
		Elev. 72.0 m		Elev. 71.5 m		Elev. 71.0 m		Elev. 70.5 m		Elev. 70.0 m		Elev. 69.5 m		Elev. 69.0 m		Elev. 68.5 m		Elev. 68.0 m		Elev. 67.5 m		Elev. 67.0 m		Elev. 66.5 m		Elev. 66.0 m		Elev. 65.5 m		Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m					
		y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)				
P-y Curves		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000			
		0.003	164.370	0.003	219.961	0.004	283.609	0.004	355.312	0.004	403.006	0.004	443.307	0.004	483.608	0.004	523.908	0.004	564.209	0.004	604.509	0.004	644.810	0.004	685.111	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	13.950	0.000	12.028		
		0.006	288.459	0.007	386.018	0.007	497.716	0.008	623.552	0.008	707.252	0.008	777.977	0.008	848.702	0.008	919.427	0.008	990.152	0.008	1060.878	0.008	1131.603	0.008	1202.328	0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	27.900	0.000	24.056				
		0.009	363.700	0.010	486.707	0.011	627.539	0.012	786.198	0.013	891.730	0.013	980.903	0.013	1070.076	0.013	1159.249	0.013	1248.423	0.013	1337.596	0.013	1426.769	0.013	1515.942	0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	41.850	0.000	36.084				
		0.012	403.420	0.013	539.861	0.015	696.074	0.017	872.060	0.017	989.118	0.017	1088.029	0.017	1186.941	0.017	1285.853	0.017	1384.765	0.017	1483.676	0.017	1582.588	0.017	1681.500	0.000	59.520	0.000	59.520	0.000	59.520	0.000	55.800	0.000	48.112						
		0.014	422.863	0.017	565.880	0.019	729.622	0.021	914.090	0.021	1036.789	0.021	1140.468	0.021	1244.147	0.021	1347.826	0.021	1451.505	0.021	1555.184	0.021	1658.863	0.021	1762.542	0.001	74.400	0.001	74.400	0.001	74.400	0.001	74.400	0.001	69.750	0.001	60.140				
		0.017	432.029	0.020	578.146	0.022	745.437	0.025	933.903	0.025	1059.262	0.025	1165.188	0.025	1271.115	0.025	1377.041	0.025	1482.967	0.025	1588.893	0.025	1694.820	0.025	1800.746	0.002	89.280	0.002	89.280	0.002	89.280	0.002	89.280	0.002	83.700	0.002	72.168				
		0.020	436.273	0.023	583.825	0.026	752.760	0.029	943.077	0.030	1069.668	0.030	1176.634	0.030	1283.601	0.030	1390.568	0.030	1497.535	0.030	1604.501	0.030	1711.468	0.030	1818.435	0.004	104.160	0.004	104.160	0.004	104.160	0.004	104.160	0.004	97.650	0.004	84.196				
		0.023	438.222	0.026	586.433	0.030	756.122	0.033	947.290	0.034	1074.446	0.034	1181.890	0.034	1289.335	0.034	1396.779	0.034	1504.224	0.034	1611.668	0.034	1719.113	0.034	1826.558	0.007	119.040	0.007	119.040	0.007	119.040	0.007	119.040	0.007	111.600	0.007	96.224				
		0.026	439.113	0.030	587.626	0.034	757.660	0.037	949.217	0.038	1076.631	0.038	1184.294	0.038	1291.957	0.038	1399.620	0.038	1507.284	0.038	1614.947	0.038	1722.610	0.038	1830.273	0.011	133.920	0.011	133.920	0.011	133.920	0.011	125.550	0.011	108.252						
		0.029	439.520	0.033	588.170	0.037	758.362	0.041	950.096	0.042	1077.629	0.042	1185.392	0.042	1293.155	0.042	1400.918	0.042	1508.681	0.042	1616.444	0.042	1724.206	0.042	1831.969	0.017	148.800	0.017	148.800	0.017	148.800	0.017	139.500	0.017	120.280						
		0.032	439.706	0.036	588.419	0.041	758.683	0.046	950.498	0.047	1078.084	0.047	1185.893	0.047	1293.701	0.047	1401.510	0.047	1509.318	0.047	1617.126	0.047	1724.935	0.047	1832.743	0.025	163.680	0.025	163.680	0.025	163.680	0.025	153.450	0.025	132.308						
		0.035	439.790	0.040	588.532	0.045	758.829	0.050	950.681	0.051	1078.292	0.051	1186.121	0.051	1293.950	0.051	1401.780	0.051	1509.609	0.051	1617.438	0.051	1725.267	0.051	1833.096	0.036	178.560	0.036	178.560	0.036	178.560	0.036	167.400	0.036	144.336						
		0.037	439.829	0.043	588.584	0.048	758.895	0.054	950.764	0.055	1078.387	0.055	1186.225	0.055	1294.064	0.055	1401.903	0.055	1509.741	0.055	1617.580	0.055	1725.419	0.055	1833.257	0.049	193.440	0.049	193.440	0.049	193.440	0.049	181.350	0.049	156.364						
		0.040	439.847	0.046	588.607	0.052	758.926	0.058	950.802	0.059	1078.430	0.059	1186.273	0.059	1294.116	0.059	1401.959	0.059	1509.802	0.059	1617.645	0.059	1725.488	0.059	1833.331	0.066	208.320	0.066	208.320	0.066	208.320	0.066	195.300	0.066	168.392						
		0.043	439.855	0.050	588.618	0.056	758.940	0.062	950.820	0.064	1078.450	0.064	1186.294	0.064	1294.139	0.064	1401.984	0.064	1509.829	0.064	1617.674	0.064	1725.519	0.064	1833.364	0.087	223.200	0.087	223.200	0.087	223.200	0.087	209.250	0.087	180.420						
		0.046	439.858	0.053	588.623	0.060	758.946	0.066	950.828	0.068	1078.459	0.068	1186.304	0.068	1294.150	0.068	1401.996	0.068	1509.842	0.068	1617.688	0.068	1725.534	0.068	1833.379	0.109	223.200	0.109	223.200	0.109	223.200	0.109	209.250	0.109	180.420						

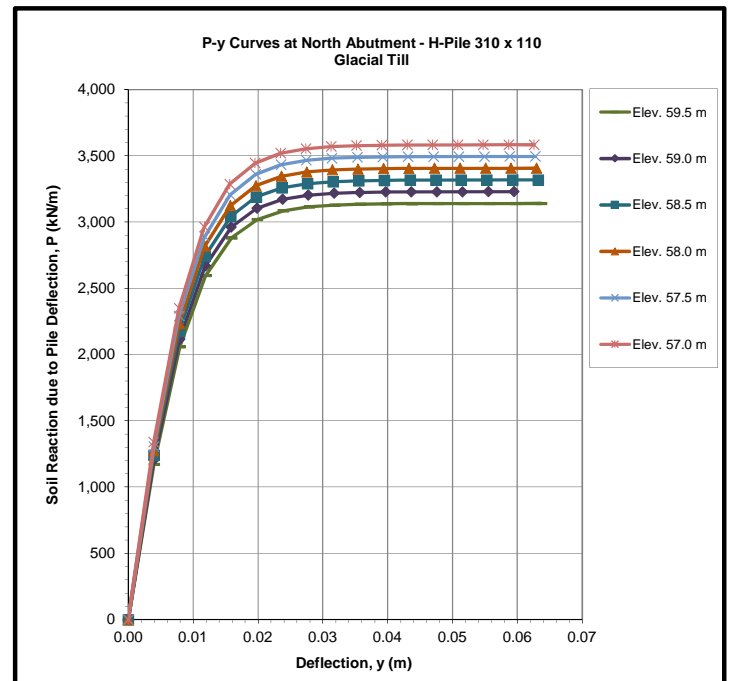
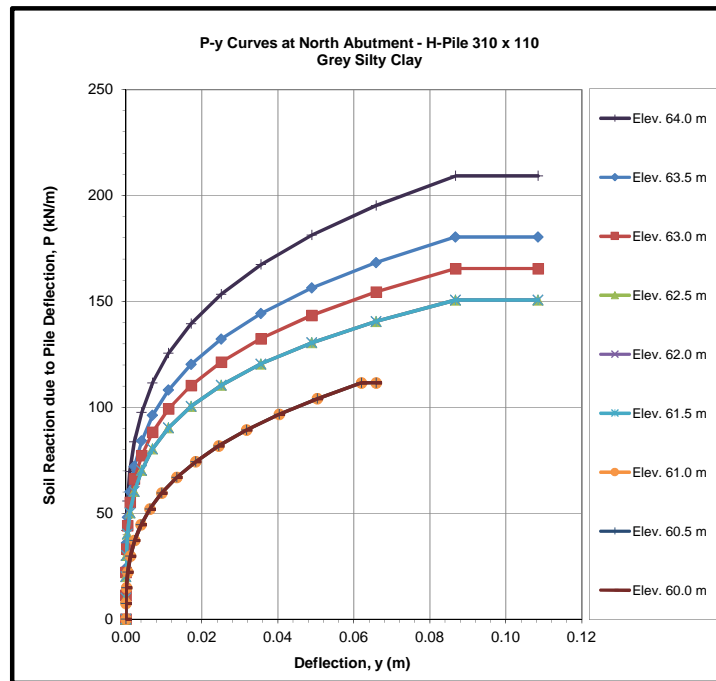
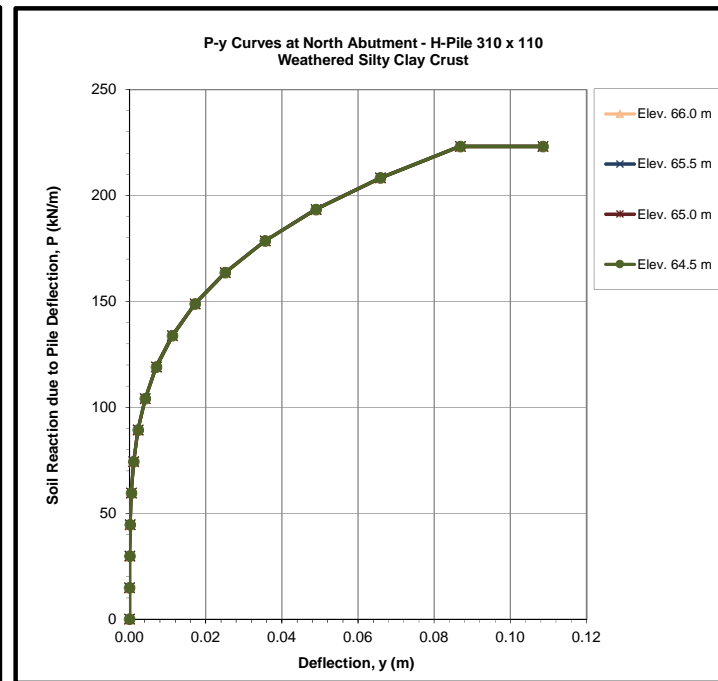
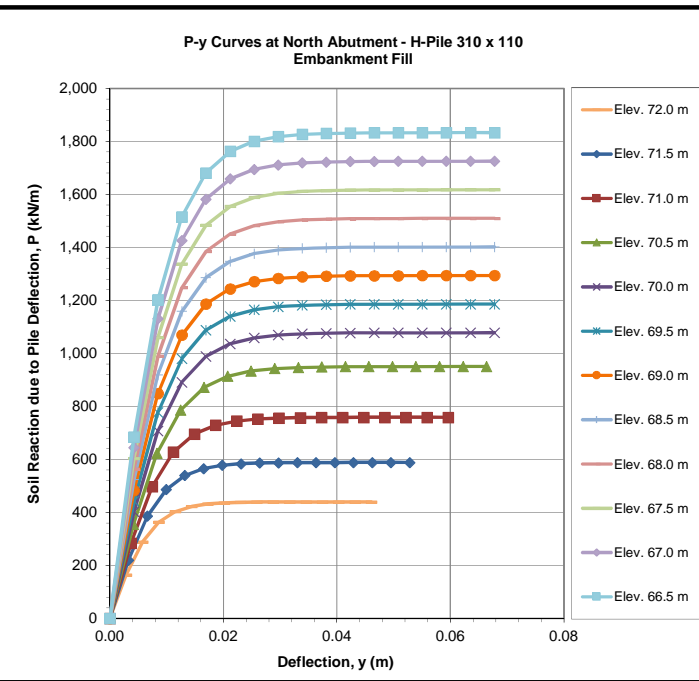
Description		Grey Silty Clay (continued)																								Glacial Till											
Depth (z) *	Elevation	z= 12.0 m		z= 12.5 m		z= 13.0 m		z= 13.5 m		z= 14.0 m		z= 14.5 m		z= 15.0 m		z= 15.5 m		z= 16.0 m		z= 16.5 m		z= 17.0 m		z= 17.5 m		z= 18.0 m											
		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 57.5 m		Elev. 57.0 m											
		y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)								
P-y Curves		0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		
		0.000	11.036	0.000	10.044	0.000	10.044	0.000	10.044	0.000	7.440	0.000	7.440	0.000	7.440	0.004	1173.854	0.004	1206.927	0.004	1240.000	0.004	1273.073	0.004	1306.147	0.004	1339.220										
		0.000	22.072	0.000	20.088	0.000	20.088	0.000	20.088	0.000	14.880	0.000	14.880	0.000	14.880	0.008	2060.042	0.008	2118.084	0.008	2176.126	0.008	2234.167	0.008	2292.209	0.008	2350.250										
		0.000	33.108	0.000	30.132	0.000	30.132	0.000	30.132	0.000	22.320	0.000	22.320	0.000	22.320	0.012	2597.381	0.012	2670.562	0.012	2743.743	0.012	2816.924	0.012	2890.105	0.012	2963.286										
		0.000	44.144	0.000	40.176	0.000	40.176	0.000	40.176	0.001	29.760	0.001	29.760	0.001	29.760	0.016	2881.045	0.016	2962.218	0.016	3043.392	0.016	3124.565	0.016	3205.738	0.016	3286.912										
		0.001	55.180	0.001	50.220	0.001	50.220	0.001	50.220	0.002	37.200	0.002	37.200	0.002	37.200	0.020	3019.901	0.020	3104.986	0.020	3190.072	0.020	3275.157	0.020	3360.243	0.020	3445.328										
		0.002	66.216	0.002	60.264	0.002	60.264	0.002	60.264	0.004	44.640	0.004	44.640	0.004	44.640	0.024	3085.358	0.024	3172.288	0.024	3259.218	0.024	3346.148	0.024	3433.077	0.023	3520.007										
		0.004	77.252	0.004	70.308	0.004	70.308	0.004	70.308	0.006	52.080	0.006	52.080	0.006	52.080	0.028	3115.667	0.028	3203.450	0.028	3291.234	0.028	3379.018	0.027	3466.802	0.027	3554.585										
		0.007	88.288	0.007	80.352	0.007	80.352	0.007	80.352	0.009	59.520	0.009	59.520	0.009	59.520	0.032	3129.584	0.032	3217.759	0.032	3305.935	0.031	3394.111	0.031	3482.287	0.031	3570.463										
		0.011	99.324	0.011	90.396	0.011	90.396	0.011	90.396	0.013	66.960	0.013	66.960	0.013	66.960	0.036	3135.949	0.036	3224.305	0.036	3312.660	0.035	3401.015	0.035	3489.370	0.035	3577.725										
		0.017	110.360	0.017	100.440	0.017	100.440	0.017	100.440	0.018	74.400	0.018	74.400	0.018	74.400	0.040	3138.856	0.040	3227.939	0.039	3315.730	0.039	3404.167	0.039	3492.604	0.039	3581.041										
		0.025	121.396	0.025	110.484	0.025	110.484	0.025	110.484	0.024	81.840	0.024	81.840	0.024	81.840	0.044	3140.182	0.044	3228.657	0.043	3317.131	0.043															

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

1. P-y curves have been generated for vertical piles (i.e. no inclination) with a ground slope angle of zero.
2. Static loading condition is considered. Lateral loading is considered normal to the strong axis.
3. There are no pile group effects (i.e., analysis is based on a single pile).
4. The effects of construction disturbance are not considered.

5. Groundwater table is assumed at the base of embankment fill / top of the weathered silty clay crust.
6. Top of pile elevation is assumed to be 72 m.



SUMMARY OF P-Y CURVES FOR H-PILE 310x110 - PIERS 1 & 4

Description Depth (z) * Elevation P-y Curves	Embankment Fill								Weathered Silty Clay Crust								Grey Silty Clay														
	z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m		
	Elev. 68.0 m		Elev. 67.5 m		Elev. 67.0 m		Elev. 66.5 m		Elev. 66.0 m		Elev. 65.5 m		Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
0.002	77.355	0.002	116.834	0.003	164.370	0.003	219.961	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	13.299	0.000	10.881	0.000	10.044	0.000	10.044	0.000	10.044	0.000	10.044	0.000	7.440
0.004	135.753	0.005	205.037	0.006	288.459	0.007	386.018	0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	26.598	0.000	21.762	0.000	20.088	0.000	20.088	0.000	20.088	0.000	20.088	0.000	14.880
0.006	171.163	0.007	258.519	0.009	363.700	0.010	486.707	0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	39.897	0.000	32.643	0.000	30.132	0.000	30.132	0.000	30.132	0.000	30.132	0.000	22.320
0.008	189.856	0.010	286.752	0.012	403.420	0.013	539.861	0.000	59.520	0.000	59.520	0.000	59.520	0.000	59.520	0.000	59.520	0.000	53.196	0.000	43.524	0.000	40.176	0.000	40.176	0.000	40.176	0.000	40.176	0.001	29.760
0.010	199.007	0.012	300.572	0.014	422.863	0.017	565.880	0.001	74.400	0.001	74.400	0.001	74.400	0.001	74.400	0.001	74.400	0.001	66.495	0.001	54.405	0.001	50.220	0.001	50.220	0.001	50.220	0.001	50.220	0.002	37.200
0.012	203.320	0.015	307.087	0.017	432.029	0.020	578.146	0.002	89.280	0.002	89.280	0.002	89.280	0.002	89.280	0.002	89.280	0.002	79.794	0.002	65.286	0.002	60.264	0.002	60.264	0.002	60.264	0.002	60.264	0.004	44.640
0.014	205.317	0.017	310.104	0.020	436.273	0.023	583.825	0.004	104.160	0.004	104.160	0.004	104.160	0.004	104.160	0.004	104.160	0.004	93.093	0.004	76.167	0.004	70.308	0.004	70.308	0.004	70.308	0.004	70.308	0.006	52.080
0.016	206.235	0.020	311.489	0.023	438.222	0.026	586.433	0.007	119.040	0.007	119.040	0.007	119.040	0.007	119.040	0.007	119.040	0.007	106.392	0.007	87.048	0.007	80.352	0.007	80.352	0.007	80.352	0.007	80.352	0.009	59.520
0.018	206.654	0.022	312.123	0.026	439.113	0.030	587.626	0.011	133.920	0.011	133.920	0.011	133.920	0.011	133.920	0.011	133.920	0.011	119.691	0.011	97.929	0.011	90.396	0.011	90.396	0.011	90.396	0.011	90.396	0.013	66.960
0.020	206.846	0.025	312.412	0.029	439.520	0.033	588.170	0.017	148.800	0.017	148.800	0.017	148.800	0.017	148.800	0.017	148.800	0.017	132.990	0.017	108.810	0.017	100.440	0.017	100.440	0.017	100.440	0.017	100.440	0.018	74.400
0.022	206.933	0.027	312.544	0.032	439.706	0.036	588.419	0.025	163.680	0.025	163.680	0.025	163.680	0.025	163.680	0.025	163.680	0.025	146.289	0.025	119.691	0.025	110.484	0.025	110.484	0.025	110.484	0.025	110.484	0.024	81.840
0.024	206.973	0.029	312.604	0.035	439.790	0.040	588.532	0.036	178.560	0.036	178.560	0.036	178.560	0.036	178.560	0.036	178.560	0.036	159.588	0.036	130.572	0.036	120.528	0.036	120.528	0.036	120.528	0.036	120.528	0.032	89.280
0.026	206.991	0.032	312.631	0.037	439.829	0.043	588.584	0.049	193.440	0.049	193.440	0.049	193.440	0.049	193.440	0.049	193.440	0.049	172.887	0.049	141.453	0.049	130.572	0.049	130.572	0.049	130.572	0.049	130.572	0.040	96.720
0.028	206.999	0.034	312.644	0.040	439.847	0.046	588.607	0.066	208.320	0.066	208.320	0.066	208.320	0.066	208.320	0.066	208.320	0.066	186.186	0.066	152.334	0.066	140.616	0.066	140.616	0.066	140.616	0.066	140.616	0.050	104.160
0.030	207.003	0.037	312.650	0.043	439.855	0.050	588.618	0.087	223.200	0.087	223.200	0.087	223.200	0.087	223.200	0.087	223.200	0.087	199.485	0.087	163.215	0.087	150.660	0.087	150.660	0.087	150.660	0.087	150.660	0.062	111.600
0.033	207.005	0.039	312.652	0.046	439.858	0.053	588.623	0.109	223.200	0.109	223.200	0.109	223.200	0.109	223.200	0.109	223.200	0.109	199.485	0.109	163.215	0.109	150.660	0.109	150.660	0.109	150.660	0.109	150.660	0.066	111.600

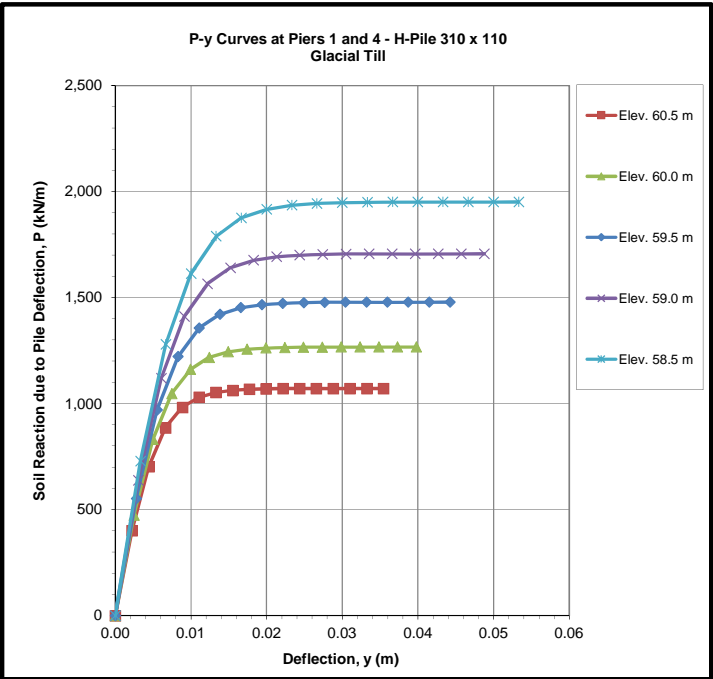
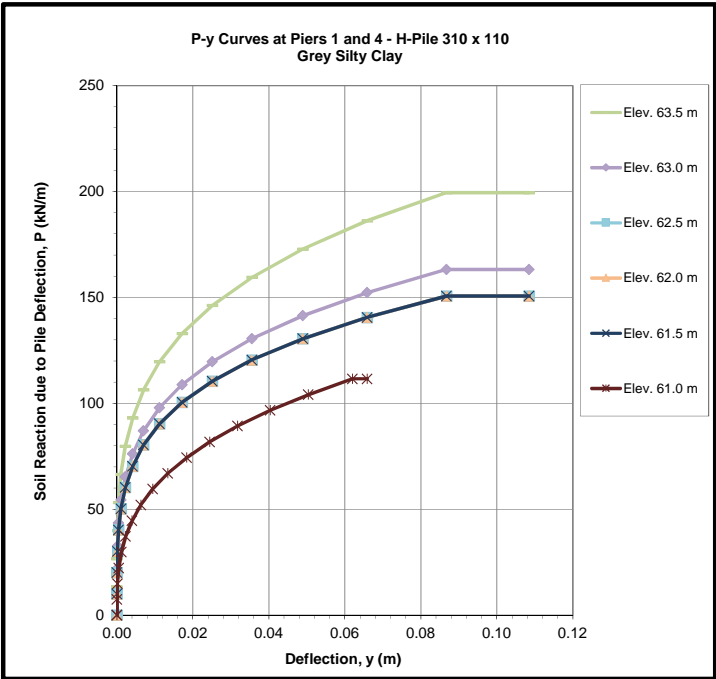
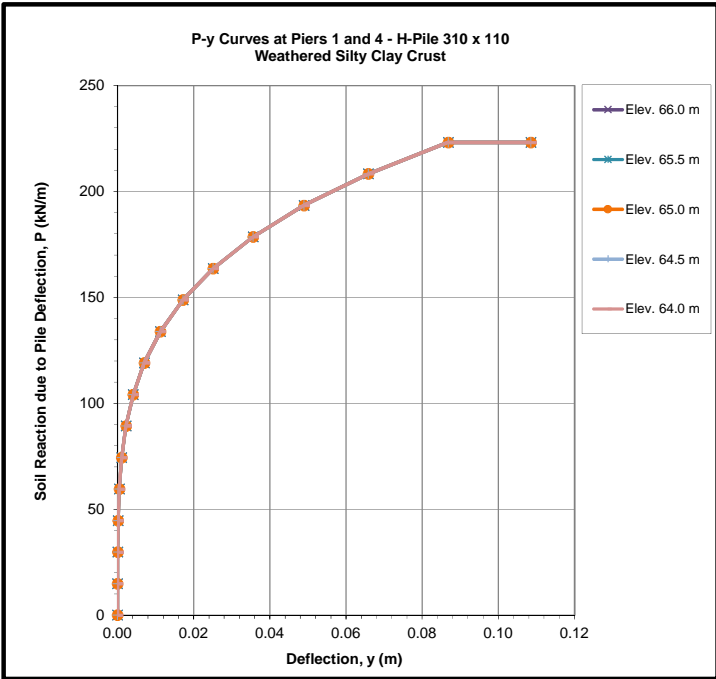
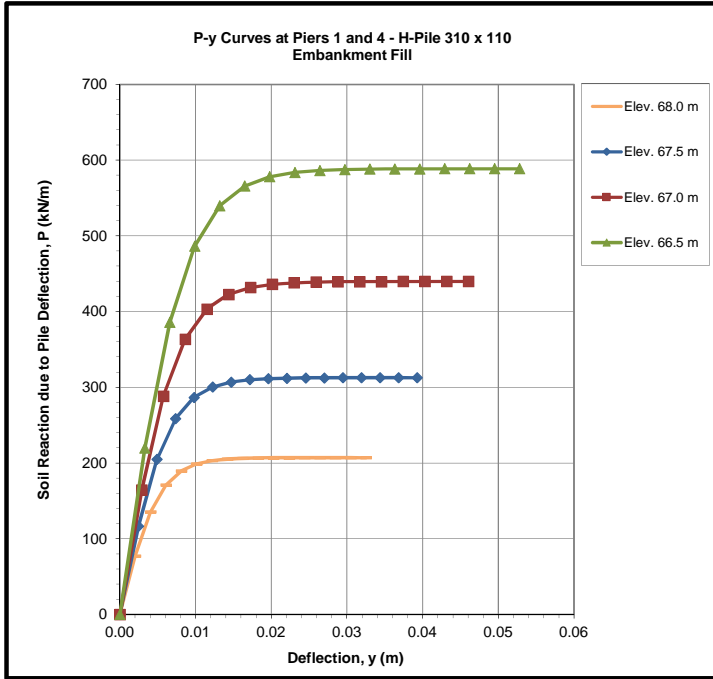
Description Depth (z) * Elevation P-y Curves	Glacial Till									
	z= 9.5 m		z= 10.0 m		z= 10.5 m		z= 11.0 m		z= 11.5 m	
	Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.002	400.391	0.002	473.420	0.003	552.571	0.003	637.844	0.003	729.242	0.003
0.004	702.663	0.005	830.823	0.006	969.728	0.006	1119.379	0.007	1279.776	0.007
0.007	885.944	0.007	1047.534	0.008	1222.671	0.009	1411.356	0.010	1613.592	0.010
0.009	982.700	0.010	1161.937	0.011	1356.200	0.012	1565.492	0.013	1789.814	0.013
0.011	1030.062	0.012	1217.938	0.014	1421.564	0.015	1640.943	0.017	1876.077	0.017
0.013	1052.389	0.015	1244.337	0.017	1452.377	0.018	1676.511	0.020	1916.741	0.020
0.016	1062.727	0.017	1256.561	0.019	1466.644	0.021	1692.980	0.023	1935.570	0.023
0.018	1067.474	0.020	1262.174	0.022	1473.195	0.024	1700.542	0.027	1944.216	0.027
0.020	1069.645	0.022	1264.741	0.025	1476.192	0.027	1704.001	0.030	1948.171	0.030
0.022	1070.637	0.025	1265.913	0.028	1477.560	0.030	1705.581	0.033	1949.976	0.033
0.024	1071.089	0.027	1266.448	0.030	1478.185	0.034	1706.301	0.037	1950.800	0.037
0.027	1071.295	0.030	1266.692	0.033	1478.469	0.037	1706.630	0.040	1951.176	0.040
0.029	1071.389	0.032	1266.803	0.036	1478.599	0.040	1706.780	0.043	1951.347	0.043
0.031	1071.432	0.035	1266.854	0.039	1478.658	0.043	1706.848	0.047	1951.425	0.047
0.033	1071.452	0.037	1266.877	0.041	1478.685	0.046	1706.879	0.050	1951.461	0.050
0.035	1071.461	0.040	1266.888	0.044	1478.698	0.049	1706.893	0.053	1951.477	0.053

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

1. P-y curves have been generated for vertical piles (i.e. no inclination) with a ground slope angle of zero.
2. Static loading condition is considered. Lateral loading is considered normal to the strong axis.
3. There are no pile group effects (i.e., analysis is based on a single pile).
4. The effects of construction disturbance are not considered.

5. Groundwater table is assumed at the base of embankment fill / top of the weathered silty clay crust.
6. Top of pile elevation is assumed to be 68 m.



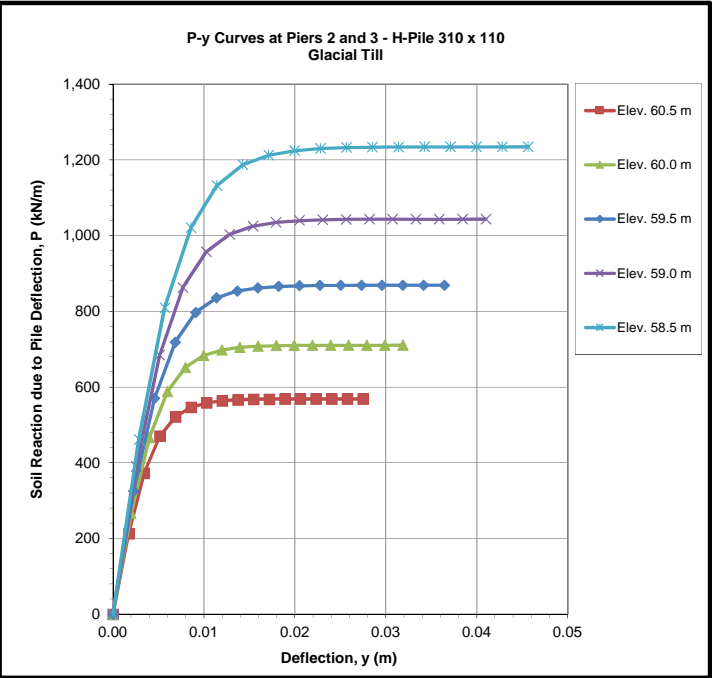
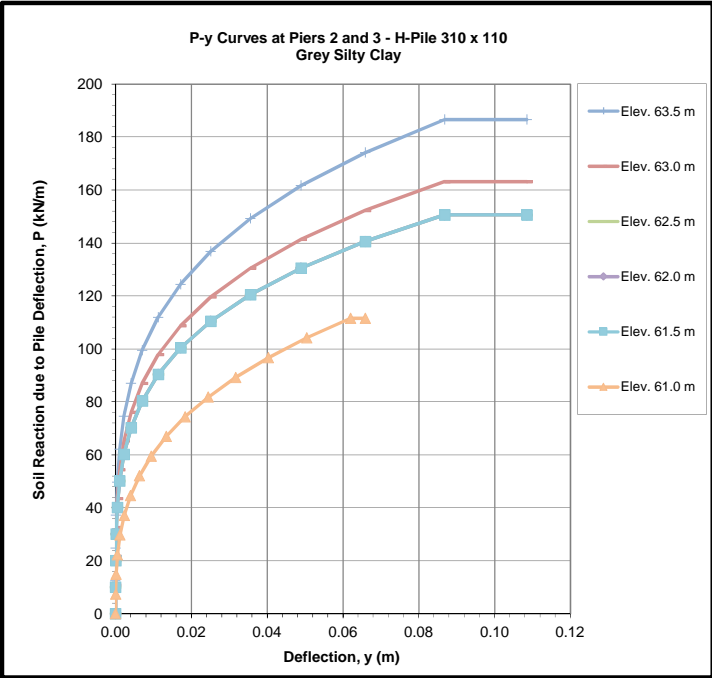
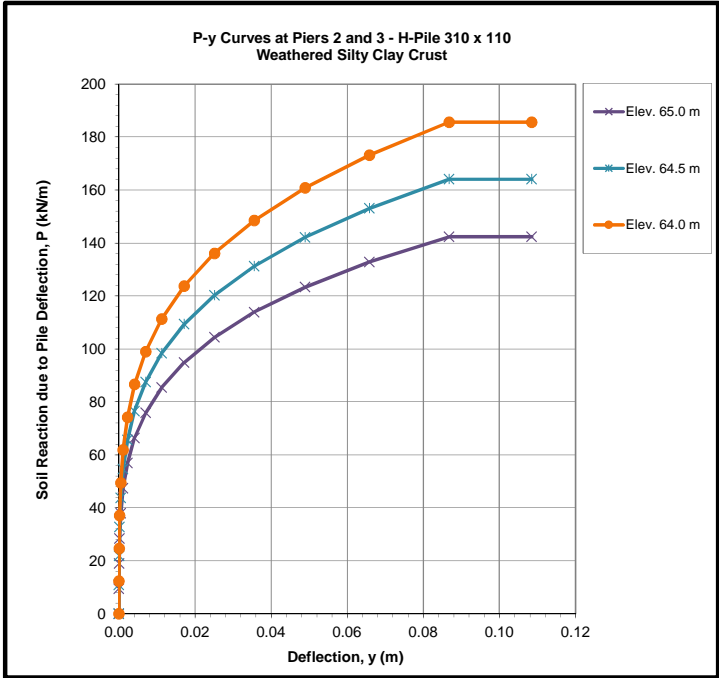
SUMMARY OF P-Y CURVES FOR H-PILE 310x110 - PIERS 2 & 3																											
Description Depth (z) * Elevation P-y Curves	Weathered Silty Clay Crust									Grey Silty Clay																	
	z= 2.0 m			z= 2.5 m			z= 3.0 m			z= 3.5 m			z= 4.0 m			z= 4.5 m			z= 5.0 m			z= 5.5 m			z= 6.0 m		
	Elev. 65.0 m			Elev. 64.5 m			Elev. 64.0 m			Elev. 63.5 m			Elev. 63.0 m			Elev. 62.5 m			Elev. 62.0 m			Elev. 61.5 m			Elev. 61.0 m		
	y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)	
	0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000	
	0.000	9.493		0.000	10.940		0.000	12.377		0.000	12.442		0.000	10.881		0.000	10.044		0.000	10.044		0.000	10.044		0.000	7.440	
	0.000	18.986		0.000	21.879		0.000	24.754		0.000	24.884		0.000	21.762		0.000	20.088		0.000	20.088		0.000	20.088		0.000	14.880	
	0.000	28.480		0.000	32.819		0.000	37.130		0.000	37.326		0.000	32.643		0.000	30.132		0.000	30.132		0.000	30.132		0.000	22.320	
	0.000	37.973		0.000	43.759		0.000	49.507		0.000	49.769		0.000	43.524		0.000	40.176		0.000	40.176		0.000	40.176		0.001	29.760	
	0.001	47.466		0.001	54.699		0.001	61.884		0.001	62.211		0.001	54.405		0.001	50.220		0.001	50.220		0.001	50.220		0.002	37.200	
	0.002	56.959		0.002	65.638		0.002	74.261		0.002	74.653		0.002	65.286		0.002	60.264		0.002	60.264		0.002	60.264		0.004	44.640	
	0.004	66.452		0.004	76.578		0.004	86.637		0.004	87.095		0.004	76.167		0.004	70.308		0.004	70.308		0.004	70.308		0.006	52.080	
	0.007	75.946		0.007	87.518		0.007	99.014		0.007	99.537		0.007	87.048		0.007	80.352		0.007	80.352		0.007	80.352		0.009	59.520	
	0.011	85.439		0.011	98.458		0.011	111.391		0.011	111.979		0.011	97.929		0.011	90.396		0.011	90.396		0.011	90.396		0.013	66.960	
	0.017	94.932		0.017	109.397		0.017	123.768		0.017	124.422		0.017	108.810		0.017	100.440		0.017	100.440		0.017	100.440		0.018	74.400	
	0.025	104.425		0.025	120.337		0.025	136.145		0.025	136.864		0.025	119.691		0.025	110.484		0.025	110.484		0.025	110.484		0.024	81.840	
	0.036	113.918		0.036	131.277		0.036	148.521		0.036	149.306		0.036	130.572		0.036	120.528		0.036	120.528		0.036	120.528		0.032	89.280	
	0.049	123.412		0.049	142.217		0.049	160.898		0.049	161.748		0.049	141.453		0.049	130.572		0.049	130.572		0.049	130.572		0.040	96.720	
	0.066	132.905		0.066	153.156		0.066	173.275		0.066	174.190		0.066	152.334		0.066	140.616		0.066	140.616		0.066	140.616		0.050	104.160	
	0.087	142.398		0.087	164.096		0.087	185.652		0.087	186.632		0.087	163.215		0.087	150.660		0.087	150.660		0.087	150.660		0.062	111.600	
	0.109	142.398		0.109	164.096		0.109	185.652		0.109	186.632		0.109	163.215		0.109	150.660		0.109	150.660		0.109	150.660		0.066	111.600	
Description Depth (z) * Elevation P-y Curves	Glacial Till																										
	z= 6.5 m			z= 7.0 m			z= 7.5 m			z= 8.0 m			z= 8.5 m														
	Elev. 60.5 m			Elev. 60.0 m			Elev. 59.5 m			Elev. 59.0 m			Elev. 58.5 m														
	y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)		y (m)	P (kN/m)	
	0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000		0.000	0.000	
	0.002	212.461		0.002	265.586		0.002	324.778		0.003	390.050		0.003	461.411		0.003	539.749		0.003	624.515		0.006	809.749		0.006	1002.962	
	0.003	372.857		0.004	466.087		0.005	569.966		0.005	684.515		0.006	809.749		0.007	947.319		0.009	1102.962		0.011	1274.463		0.014	1461.044	
	0.005	470.112		0.006	587.660		0.007	718.635		0.008	863.063		0.009	1020.962		0.011	1192.463		0.014	1387.044		0.017	1572.773		0.020	1757.687	
	0.007	521.454		0.008	651.840		0.009	797.119		0.010	957.319		0.011	1132.463		0.014	1312.463		0.017	1497.044		0.020	1677.687		0.023	1857.157	
	0.009	546.586		0.010	683.256		0.011	835.537		0.013	1003.458		0.014	1187.044		0.017	1372.463		0.020	1562.044		0.023	1747.687		0.026	1932.659	
	0.010	558.434		0.012	698.066		0.014	853.647		0.015	1025.209		0.017	1212.773		0.020	1402.019		0.023	1587.044		0.026	1772.687		0.029	1952.802	
	0.012	563.919		0.014	704.923		0.016	862.033		0.018	1035.280		0.020	1224.687		0.023	1414.019		0.026	1599.044		0.029	1784.687		0.031	1964.323	
	0.014	566.438		0.016	708.072		0.018	865.883		0.020	1039.904		0.023	1230.157		0.026	1420.019		0.029	1605.044		0.031	1790.687		0.033	1970.323	
	0.015	567.590		0.018	709.512		0.020	867.645		0.023	1042.019		0.026	1232.659		0.029	1422.019		0.031	1607.044		0.033	1792.687		0.034	1972.323	
	0.017	568.117		0.020	710.170		0.023	868.449		0.026	1042.985		0.029	1233.802		0.031	1423.019		0.033	1608.044		0.034	1793.687		0.036	1974.323	
	0.019	568.357		0.022	710.470		0.025	868.816		0.028	1043.426		0.031	1234.323		0.033	1423.323		0.034	1608.323		0.036	1794.687		0.037	1974.669	
	0.021	568.466		0.024	710.607		0.027	868.983		0.031	1043.627		0.034	1234.561		0.036	1423.561		0.037	1608.561		0.038	1794.861		0.040	1974.719	
	0.022	568.516		0.026	710.669		0.030	869.060		0.033	1043.718		0.037	1234.669		0.040	1423.669		0.043	1608.669		0.046	1794.941		0.049	1974.991	
	0.024	568.539		0.028	710.698		0.032	869.094		0.036	1043.760		0.040	1234.719		0.043	1423.719		0.046	1608.719		0.049	1794.991		0.052	1975.041	
	0.026	568.549		0.030	710.711		0.034	869.110		0.038	1043.779		0.043	1234.741		0.046	1423.741		0.049	1608.741		0.052	1795.021		0.055	1975.071	
	0.027	568.554		0.032	710.716		0.036	869.117		0.041	1043.788		0.046	1234.752		0.049	1423.752		0.052	1608.752		0.055	1795.071		0.058	1975.121	

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

1. P-y curves have been generated for vertical piles (i.e. no inclination) with a ground slope angle of zero.
2. Static loading condition is considered. Lateral loading is considered normal to the strong axis.
3. There are no pile group effects (i.e., analysis is based on a single pile).
4. The effects of construction disturbance are not considered.

5. Groundwater table is assumed at the base of embankment fill / top of the weathered silty clay crust.
6. Top of pile elevation is assumed to be 65 m.



SUMMARY OF P-Y CURVES FOR H-PILE 310x110 - SOUTH ABUTMENT

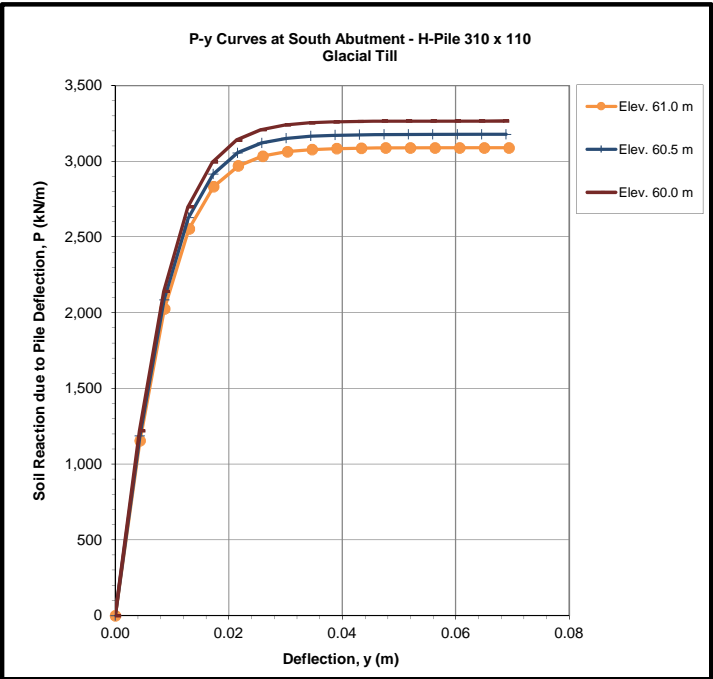
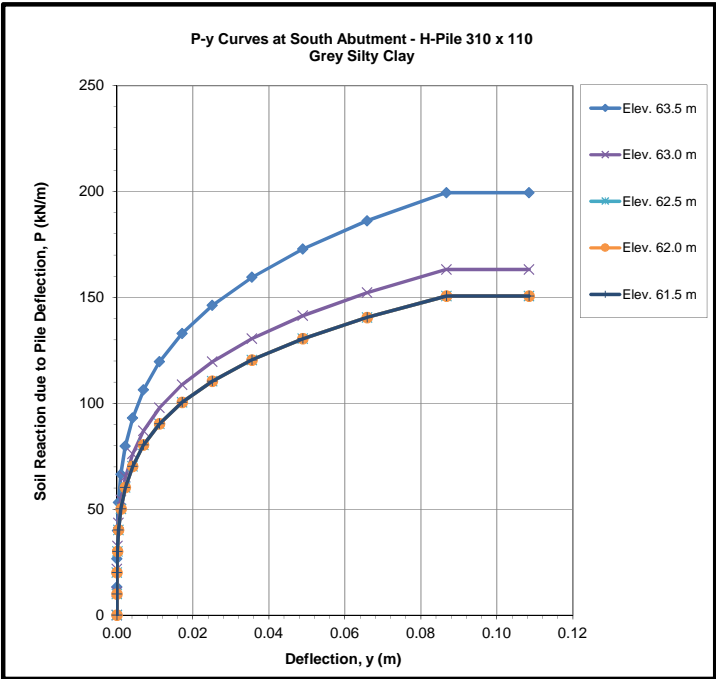
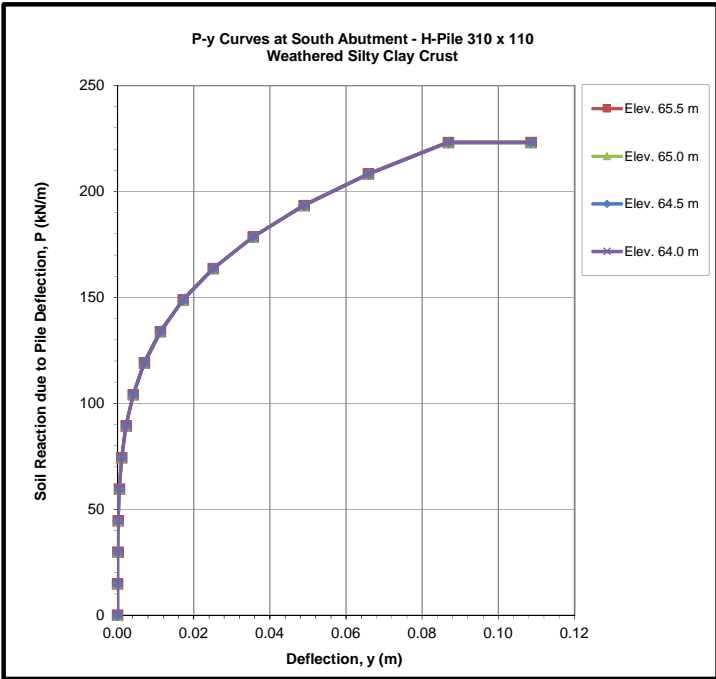
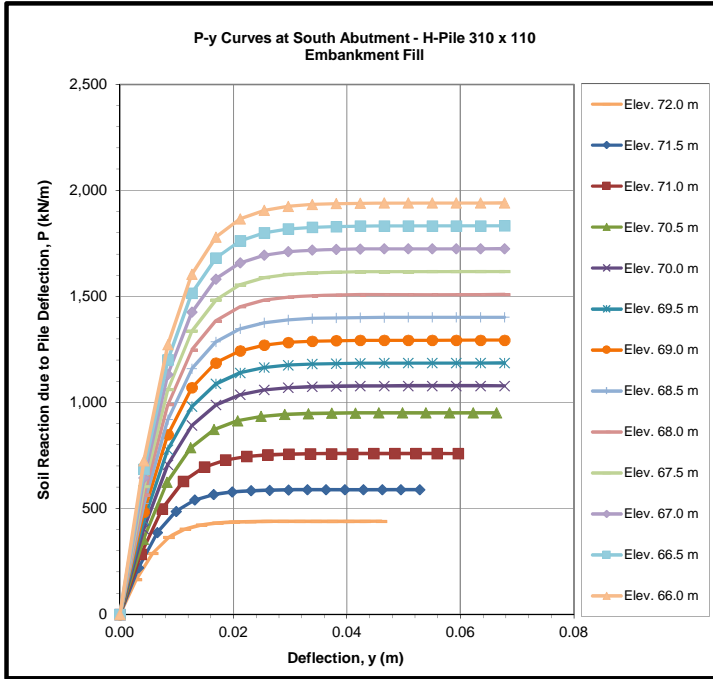
Description Depth (z) * Elevation P-y Curves	Embankment Fill																			
	z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m	
	Elev. 72.0 m		Elev. 71.5 m		Elev. 71.0 m		Elev. 70.5 m		Elev. 70.0 m		Elev. 69.5 m		Elev. 69.0 m		Elev. 68.5 m		Elev. 68.0 m		Elev. 67.5 m	
P-y Curves	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.003	164.370	0.003	219.961	0.004	283.609	0.004	355.312	0.004	403.006	0.004	443.307	0.004	483.608	0.004	523.908	0.004	564.209	0.004	604.509
	0.006	288.459	0.007	386.018	0.007	497.716	0.008	623.552	0.008	707.252	0.008	777.977	0.008	848.702	0.008	919.427	0.008	990.152	0.008	1060.878
	0.009	363.700	0.010	486.707	0.011	627.539	0.012	786.198	0.013	891.730	0.013	980.903	0.013	1070.076	0.013	1159.249	0.013	1248.423	0.013	1337.596
	0.012	403.420	0.013	539.861	0.015	696.074	0.017	872.060	0.017	989.118	0.017	1088.029	0.017	1186.941	0.017	1285.853	0.017	1384.765	0.017	1483.676
	0.014	422.863	0.017	565.880	0.019	729.622	0.021	914.090	0.021	1036.789	0.021	1140.468	0.021	1244.147	0.021	1347.826	0.021	1451.505	0.021	1555.184
	0.017	432.029	0.020	578.146	0.022	745.437	0.025	933.903	0.025	1059.262	0.025	1165.188	0.025	1271.115	0.025	1377.041	0.025	1482.967	0.025	1588.893
	0.020	436.273	0.023	583.825	0.026	752.760	0.029	943.077	0.030	1069.668	0.030	1176.634	0.030	1283.601	0.030	1390.568	0.030	1497.535	0.030	1604.501
	0.023	438.222	0.026	586.433	0.030	756.122	0.033	947.290	0.034	1074.446	0.034	1181.890	0.034	1289.335	0.034	1396.779	0.034	1504.224	0.034	1611.668
	0.026	439.113	0.030	587.626	0.034	757.660	0.037	949.217	0.038	1076.631	0.038	1184.294	0.038	1291.957	0.038	1399.620	0.038	1507.284	0.038	1614.947
	0.029	439.520	0.033	588.170	0.037	758.362	0.041	950.096	0.042	1077.629	0.042	1185.392	0.042	1293.155	0.042	1400.918	0.042	1508.681	0.042	1616.444
	0.032	439.706	0.036	588.419	0.041	758.683	0.046	950.498	0.047	1078.084	0.047	1185.893	0.047	1293.701	0.047	1401.510	0.047	1509.318	0.047	1617.126
	0.035	439.790	0.040	588.532	0.045	758.829	0.050	950.681	0.051	1078.292	0.051	1186.121	0.051	1293.950	0.051	1401.780	0.051	1509.609	0.051	1617.438
	0.037	439.829	0.043	588.584	0.048	758.895	0.054	950.764	0.055	1078.387	0.055	1186.225	0.055	1294.064	0.055	1401.903	0.055	1509.741	0.055	1617.580
	0.040	439.847	0.046	588.607	0.052	758.926	0.058	950.802	0.059	1078.430	0.059	1186.273	0.059	1294.116	0.059	1401.959	0.059	1509.802	0.059	1617.645
	0.043	439.855	0.050	588.618	0.056	758.940	0.062	950.820	0.064	1078.450	0.064	1186.294	0.064	1294.139	0.064	1401.984	0.064	1509.829	0.064	1617.674
	0.046	439.858	0.053	588.623	0.060	758.946	0.066	950.828	0.068	1078.459	0.068	1186.304	0.068	1294.150	0.068	1401.996	0.068	1509.842	0.068	1617.688
	Weathered Silty Clay Crust										Grey Silty Clay									
	z= 9.5 m		z= 10.0 m		z= 10.5 m		z= 11.0 m		z= 11.5 m		z= 12.0 m		z= 12.5 m		z= 13.0 m		z= 13.5 m		z= 14.0 m	
	Elev. 65.5 m		Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	13.299	0.000	10.881	0.000	10.044	0.000	10.044	0.000	10.044	0.004	1154.602
	0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	26.598	0.000	21.762	0.000	20.088	0.000	20.088	0.000	20.088	0.009	2026.257
	0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	39.897	0.000	32.643	0.000	30.132	0.000	30.132	0.000	30.132	0.013	2554.783
	0.000	59.520	0.000	59.520	0.000	59.520	0.000	59.520	0.000	53.196	0.000	43.524	0.000	40.176	0.000	40.176	0.000	40.176	0.017	2833.795
	0.001	74.400	0.001	74.400	0.001	74.400	0.001	74.400	0.001	66.495	0.001	54.405	0.001	50.220	0.001	50.220	0.001	50.220	0.022	2970.373
	0.002	89.280	0.002	89.280	0.002	89.280	0.002	89.280	0.002	79.794	0.002	65.286	0.002	60.264	0.002	60.264	0.002	60.264	0.026	3034.757
	0.004	104.160	0.004	104.160	0.004	104.160	0.004	104.160	0.004	93.093	0.004	76.167	0.004	70.308	0.004	70.308	0.004	70.308	0.030	3064.569
	0.007	119.040	0.007	119.040	0.007	119.040	0.007	119.040	0.007	106.392	0.007	87.048	0.007	80.352	0.007	80.352	0.007	80.352	0.035	3078.257
	0.011	133.920	0.011	133.920	0.011	133.920	0.011	133.920	0.011	119.691	0.011	97.929	0.011	90.396	0.011	90.396	0.011	90.396	0.039	3084.519
	0.017	148.800	0.017	148.800	0.017	148.800	0.017	148.800	0.017	132.990	0.017	108.810	0.017	100.440	0.017	100.440	0.017	100.440	0.043	3087.378
	0.025	163.680	0.025	163.680	0.025	163.680	0.025	163.680	0.025	146.289	0.025	119.691	0.025	110.484	0.025	110.484	0.025	110.484	0.048	3088.682
	0.036	178.560	0.036	178.560	0.036	178.560	0.036	178.560	0.036	159.588	0.036	130.572	0.036	120.528	0.036	120.528	0.036	120.528	0.052	3089.277
	0.049	193.440	0.049	193.440	0.049	193.440	0.049	193.440	0.049	172.887	0.049	141.453	0.049	130.572	0.049	130.572	0.049	130.572	0.056	3089.548
	0.066	208.320	0.066	208.320	0.066	208.320	0.066	208.320	0.066	186.186	0.066	152.334	0.066	140.616	0.066	140.616	0.066	140.616	0.061	3089.672
	0.087	223.200	0.087	223.200	0.087	223.200	0.087	223.200	0.087	199.485	0.087	163.215	0.087	150.660	0.087	150.660	0.087	150.660	0.065	3089.729
	0.109	223.200	0.109	223.200	0.109	223.200	0.109	223.200	0.109	199.485	0.109	163.215	0.109	150.660	0.109	150.660	0.109	150.660	0.069	3089.754

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

1. P-y curves have been generated for vertical piles (i.e. no inclination) with a ground slope angle of zero.
2. Static loading condition is considered. Lateral loading is considered normal to the strong axis.
3. There are no pile group effects (i.e., analysis is based on a single pile).
4. The effects of construction disturbance are not considered.

5. Groundwater table is assumed at the base of embankment fill / top of the weathered silty clay crust.
6. Top of pile elevation is assumed to be 72 m.



CYCLIC P-Y CURVES

CNR Overpass
H-Pile 310x110 - North Abutment (Boreholes 17-1102, 17-1115)

Figure G8

SUMMARY OF CYCLIC P-Y CURVES FOR H-PILE 310x110 - NORTH ABUTMENT

Description Depth (z) * Elevation P-y Curves	Embankment Fill																				Weathered Silty Clay Crust								Grey Silty Clay									
	z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m		z= 10.0 m		z= 10.5 m		z= 11.0 m		z= 11.5 m					
	Elev. 72.0 m		Elev. 71.5 m		Elev. 71.0 m		Elev. 70.5 m		Elev. 70.0 m		Elev. 69.5 m		Elev. 69.0 m		Elev. 68.5 m		Elev. 68.0 m		Elev. 67.5 m		Elev. 67.0 m		Elev. 66.5 m		Elev. 66.0 m		Elev. 65.5 m		Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m			
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)		
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		
0.003	164.370	0.003	219.961	0.004	283.609	0.004	355.312	0.004	403.006	0.004	443.307	0.004	483.608	0.004	523.908	0.004	564.209	0.004	604.509	0.004	644.810	0.004	685.111	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	13.950	0.000	12.028	0.000	12.028	0.000
0.006	288.459	0.007	386.018	0.007	497.716	0.008	623.552	0.008	707.252	0.008	777.977	0.008	848.702	0.008	919.427	0.008	990.152	0.008	1060.878	0.008	1131.603	0.008	1202.328	0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	27.900	0.000	24.056	0.000	24.056	0.000
0.009	363.700	0.010	486.707	0.011	627.539	0.012	786.198	0.013	891.730	0.013	980.903	0.013	1070.076	0.013	1159.249	0.013	1248.423	0.013	1337.596	0.013	1426.769	0.013	1515.942	0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	41.850	0.000	36.084	0.000	36.084	0.000
0.012	403.420	0.013	539.861	0.015	696.074	0.017	872.060	0.017	989.118	0.017	1088.029	0.017	1186.941	0.017	1285.853	0.017	1384.765	0.017	1483.676	0.017	1582.588	0.017	1681.500	0.001	59.520	0.001	59.520	0.001	59.520	0.001	59.520	0.001	55.800	0.001	48.112	0.001	48.112	0.001
0.014	422.863	0.017	565.880	0.019	729.622	0.021	914.090	0.021	1036.789	0.021	1140.468	0.021	1244.147	0.021	1347.826	0.021	1451.505	0.021	1555.184	0.021	1658.863	0.021	1762.542	0.003	74.400	0.003	74.400	0.003	74.400	0.003	74.400	0.003	69.750	0.003	60.140	0.003	60.140	0.003
0.017	432.029	0.020	578.146	0.022	745.437	0.025	933.903	0.025	1059.262	0.025	1165.188	0.025	1271.115	0.025	1377.041	0.025	1482.967	0.025	1588.893	0.025	1694.820	0.025	1800.746	0.006	89.280	0.006	89.280	0.006	89.280	0.006	89.280	0.006	83.700	0.006	72.168	0.006	72.168	0.006
0.020	436.273	0.023	583.825	0.026	752.760	0.029	943.077	0.030	1069.668	0.030	1176.634	0.030	1283.601	0.030	1390.568	0.030	1497.535	0.030	1604.501	0.030	1711.468	0.030	1818.435	0.011	104.160	0.011	104.160	0.011	104.160	0.011	104.160	0.011	97.650	0.011	84.196	0.011	84.196	0.011
0.023	438.222	0.026	586.433	0.030	756.122	0.033	947.290	0.034	1074.446	0.034	1181.890	0.034	1289.335	0.034	1396.779	0.034	1504.224	0.034	1611.668	0.034	1719.113	0.034	1826.558	0.018	119.040	0.018	119.040	0.018	119.040	0.018	119.040	0.018	111.600	0.018	96.224	0.018	96.224	0.018
0.026	439.113	0.030	587.626	0.034	757.660	0.037	949.217	0.038	1076.631	0.038	1184.294	0.038	1291.957	0.038	1399.620	0.038	1507.284	0.038	1614.947	0.038	1722.610	0.038	1830.273	0.029	133.920	0.029	133.920	0.029	133.920	0.029	133.920	0.029	125.550	0.029	108.252	0.029	108.252	0.029
0.029	439.520	0.033	588.170	0.037	758.362	0.041	950.096	0.042	1077.629	0.042	1185.392	0.042	1293.155	0.042	1400.918	0.042	1508.681	0.042	1616.444	0.042	1724.206	0.042	1831.969	0.045	148.800	0.045	148.800	0.045	148.800	0.045	148.800	0.045	139.500	0.045	120.280	0.045	120.280	0.045
0.032	439.706	0.036	588.419	0.041	758.683	0.046	950.498	0.047	1078.084	0.047	1185.893	0.047	1293.701	0.047	1401.510	0.047	1509.318	0.047	1617.126	0.047	1724.935	0.047	1832.743	0.066	163.680	0.066	163.680	0.066	163.680	0.066	163.680	0.066	153.450	0.066	132.308	0.066	132.308	0.066
0.035	439.790	0.040	588.532	0.045	758.829	0.050	950.681	0.051	1078.292	0.051	1186.121	0.051	1293.950	0.051	1401.780	0.051	1509.609	0.051	1617.438	0.051	1725.267	0.051	1833.096	0.093	178.560	0.093	178.560	0.093	178.560	0.093	178.560	0.093	167.400	0.093	144.336	0.093	144.336	0.093
0.037	439.829	0.043	588.584	0.048	758.895	0.054	950.764	0.055	1078.387	0.055	1186.225	0.055	1294.064	0.055	1401.903	0.055	1509.741	0.055	1617.580	0.055	1725.419	0.055	1833.257	0.128	193.440	0.128	193.440	0.128	193.440	0.128	193.440	0.128	181.350	0.128	156.364	0.128	156.364	0.128
0.040	439.847	0.046	588.607	0.052	758.926	0.058	950.802	0.059	1078.430	0.059	1186.273	0.059	1294.116	0.059	1401.959	0.059	1509.802	0.059	1617.645	0.059	1725.488	0.059	1833.331	0.173	208.320	0.173	208.320	0.173	208.320	0.173	208.320	0.173	195.300	0.173	168.392	0.173	168.392	0.173
0.043	439.855	0.050	588.618	0.056	758.940	0.062	950.820	0.064	1078.450	0.064	1186.294	0.064	1294.139	0.064	1401.984	0.064	1509.829	0.064	1617.674	0.064	1725.519	0.064	1833.364	0.227	223.200	0.227	223.200	0.227	223.200	0.227	223.200	0.227	209.250	0.227	180.420	0.227	180.420	0.227
0.046	439.858	0.053	588.623	0.060	758.946	0.066	950.828	0.068	1078.459	0.068	1186.304	0.068	1294.150	0.068	1401.996	0.068	1509.842	0.068	1617.688	0.068	1725.534	0.068	1833.379	0.284	223.200	0.284	223.200	0.284	223.200	0.284	223.200	0.284	209.250	0.284	180.420	0.284	180.420	0.284
Description Depth (z) * Elevation P-y Curves	Grey Silty Clay (continued)																				Glacial Till																	
	z= 12.0 m		z= 12.5 m		z= 13.0 m		z= 13.5 m		z= 14.0 m		z= 14.5 m		z= 15.0 m		z= 15.5 m		z= 16.0 m		z= 16.5 m		z= 17.0 m		z= 17.5 m		z= 18.0 m													
	Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 57.5 m		Elev. 57.0 m													
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)				
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000				
0.000	11.036	0.000	10.044	0.000	10.044	0.000	10.044	0.000	5.748	0.000	5.748	0.000	5.748	0.004	1173.854	0.004	1206.927	0.004	1240.000	0.004	1273.073	0.004	1306.147	0.004	1339.220	0.000	22.072	0.000	20.088	0.000	20.088	0.000	20.088	0.000	20.088	0.000		
0.000	22.072	0.000	20.088	0.000	20.088	0.000	20.088	0.000	11.497	0.000	11.497	0.000	11.497	0.008	2060.042	0.008	2118.084	0.008	2176.126	0.008	2234.167	0.008	2292.209	0.008	2350.250	0.000	33.108	0.000	30.132	0.000	30.132	0.000	30.132	0.000	30.132	0.000		
0.000	33.108	0.000	30.132	0.000	30.132	0.000	30.132	0.000	17.245	0.000	17.245	0.000	17.245	0.012	2597.381	0.012	2670.562	0.012	2743.743	0.012	2816.924	0.012	2890.105	0.012	2963.286	0.000	44.144	0.001	40.176	0.001	40.176	0.001	40.176	0.001	40.176	0.001		
0.001	44.144	0.001	40.176	0.001	40.176	0.001	40.176	0.001	22.994	0.001	22.994	0.001	22.994	0.016	2881.045	0.016	2962.218	0.016	3043.392	0.016	3124.565	0.016	3205.738	0.016	3286.912	0.000	55.180	0.003	50.220	0.003	50.220	0.003	50.220	0.001	28.742	0.001		
0.003	55.180	0.003	50.220	0.003	50.220	0.003	50.220	0.001	28.742	0.001	28.742	0.001	28.742	0.020	3019.901	0.020	3104.986	0.020	3190.072	0.020	3275.157	0.020	3360.243	0.020	3445.328	0.006	66.216	0.006	60.264	0.006	60.264	0.006	60.264	0.002	34.490	0.002		
0.006	66.216	0.006	60.264	0.006	60.264	0.006	60.264	0.003	34.490	0.002	34.490	0.002	34.490	0.024	3085.358	0.024	3172.288	0.024	3259.218	0.024	3346.148	0.024	3433.077	0.023	3520.007	0.011	77.252	0.011	70.308	0.011	70.308	0.011	70.308	0.				

CYCLIC P-Y CURVES

CNR Overpass
H-Pile 310x110 - Piers 1 & 4 (Boreholes 17-1103, 17-1106, 17-1111, 17-1114)

Figure G9

SUMMARY OF CYCLIC P-Y CURVES FOR H-PILE 310x110 - PIERS 1 & 4

Description Depth (z) * Elevation P-y Curves	Embankment Fill								Weathered Silty Clay Crust								Grey Silty Clay													
	z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m	
	Elev. 68.0 m		Elev. 67.5 m		Elev. 67.0 m		Elev. 66.5 m		Elev. 66.0 m		Elev. 65.5 m		Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.002	77.355	0.002	116.834	0.003	164.370	0.003	219.961	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	13.299	0.000	10.881	0.000	10.044	0.000	10.044	0.000	10.044	0.000	5.748	0.000
0.004	135.753	0.005	205.037	0.006	288.459	0.007	386.018	0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	26.598	0.000	21.762	0.000	20.088	0.000	20.088	0.000	20.088	0.000	11.497	0.000
0.006	171.163	0.007	258.519	0.009	363.700	0.010	486.707	0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	39.897	0.000	32.643	0.000	30.132	0.000	30.132	0.000	30.132	0.000	17.245	0.000
0.008	189.856	0.010	286.752	0.012	403.420	0.013	539.861	0.001	59.520	0.001	59.520	0.001	59.520	0.001	59.520	0.001	59.520	0.001	53.196	0.001	43.524	0.001	40.176	0.001	40.176	0.001	40.176	0.001	22.994	0.001
0.010	199.007	0.012	300.572	0.014	422.863	0.017	565.880	0.003	74.400	0.003	74.400	0.003	74.400	0.003	74.400	0.003	74.400	0.003	66.495	0.003	54.405	0.003	50.220	0.003	50.220	0.003	50.220	0.003	28.742	0.001
0.012	203.320	0.015	307.087	0.017	432.029	0.020	578.146	0.006	89.280	0.006	89.280	0.006	89.280	0.006	89.280	0.006	89.280	0.006	79.794	0.006	65.286	0.006	60.264	0.006	60.264	0.006	60.264	0.006	34.490	0.002
0.014	205.317	0.017	310.104	0.020	436.273	0.023	583.825	0.011	104.160	0.011	104.160	0.011	104.160	0.011	104.160	0.011	104.160	0.011	93.093	0.011	76.167	0.011	70.308	0.011	70.308	0.011	70.308	0.011	40.239	0.003
0.016	206.235	0.020	311.489	0.023	438.222	0.026	586.433	0.018	119.040	0.018	119.040	0.018	119.040	0.018	119.040	0.018	119.040	0.018	106.392	0.018	87.048	0.018	80.352	0.018	80.352	0.018	80.352	0.018	45.987	0.004
0.018	206.654	0.022	312.123	0.026	439.113	0.030	587.626	0.029	133.920	0.029	133.920	0.029	133.920	0.029	133.920	0.029	133.920	0.029	119.691	0.029	97.929	0.029	90.396	0.029	90.396	0.029	90.396	0.029	51.736	0.006
0.020	206.846	0.025	312.412	0.029	439.520	0.033	588.170	0.045	148.800	0.045	148.800	0.045	148.800	0.045	148.800	0.045	148.800	0.045	132.990	0.045	108.810	0.045	100.440	0.045	100.440	0.045	100.440	0.045	57.484	0.008
0.022	206.933	0.027	312.544	0.032	439.706	0.036	588.419	0.066	163.680	0.066	163.680	0.066	163.680	0.066	163.680	0.066	163.680	0.066	146.289	0.066	119.691	0.066	110.484	0.066	110.484	0.066	110.484	0.066	63.232	0.011
0.024	206.973	0.029	312.604	0.035	439.790	0.040	588.532	0.093	178.560	0.093	178.560	0.093	178.560	0.093	178.560	0.093	178.560	0.093	159.588	0.093	130.572	0.093	120.528	0.093	120.528	0.093	120.528	0.093	68.981	0.015
0.026	206.991	0.032	312.631	0.037	439.829	0.043	588.584	0.128	193.440	0.128	193.440	0.128	193.440	0.128	193.440	0.128	193.440	0.128	172.887	0.128	141.453	0.128	130.572	0.128	130.572	0.128	130.572	0.128	74.729	0.019
0.028	206.999	0.034	312.644	0.040	439.847	0.046	588.607	0.173	208.320	0.173	208.320	0.173	208.320	0.173	208.320	0.173	208.320	0.173	186.186	0.173	152.334	0.173	140.616	0.173	140.616	0.173	140.616	0.173	80.478	0.023
0.030	207.003	0.037	312.650	0.043	439.855	0.050	588.618	0.227	223.200	0.227	223.200	0.227	223.200	0.227	223.200	0.227	223.200	0.227	199.485	0.227	163.215	0.227	150.660	0.227	150.660	0.227	150.660	0.227	80.478	0.116
0.033	207.005	0.039	312.652	0.046	439.858	0.053	588.623	0.284	223.200	0.284	223.200	0.284	223.200	0.284	223.200	0.284	223.200	0.284	199.485	0.284	163.215	0.284	150.660	0.284	150.660	0.284	150.660	0.284	80.478	0.124

*To account for cyclic softening behaviour, cyclic p-y curves in the Glacial Till are taken as 75% of the static soil reaction for a given pile deflection.

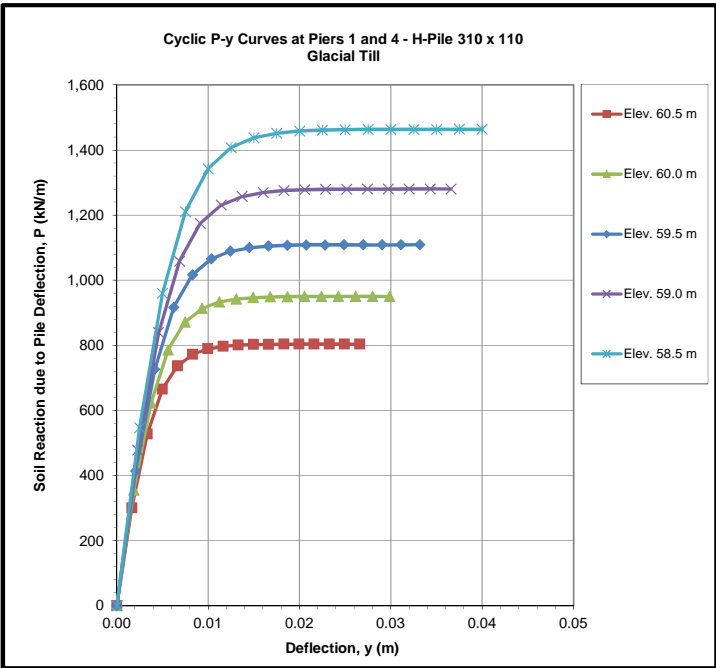
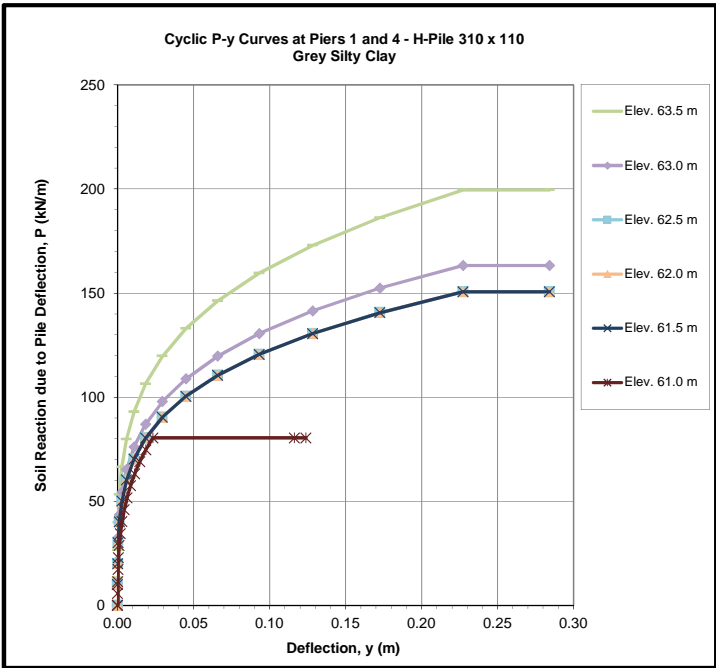
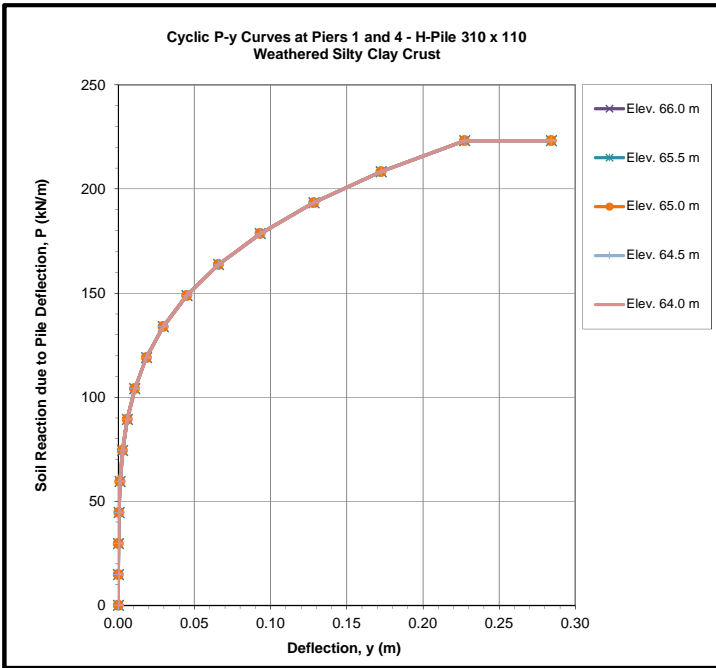
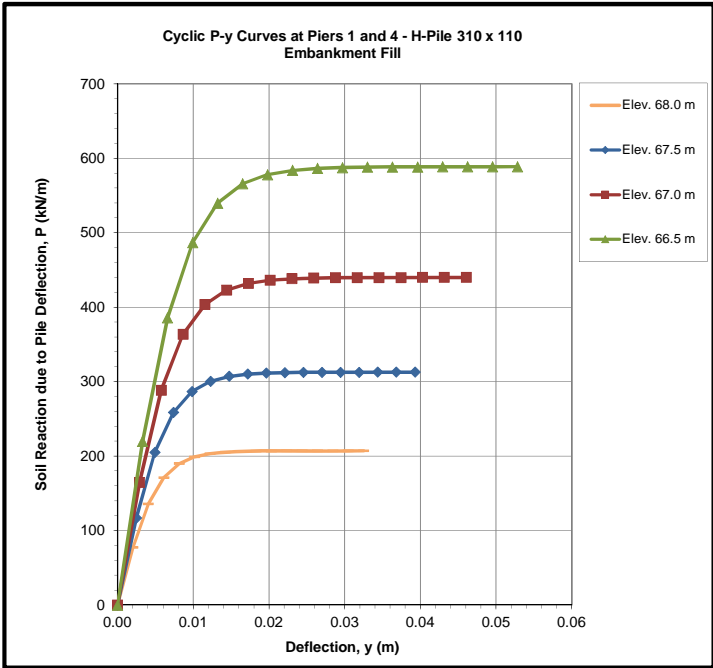
Description Depth (z) * Elevation P-y Curves	Glacial Till									
	z= 9.5 m		z= 10.0 m		z= 10.5 m		z= 11.0 m		z= 11.5 m	
	Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.002	300.293	0.002	355.065	0.002	414.428	0.002	478.383	0.002	546.932	0.002
0.003	526.997	0.004	623.118	0.004	727.296	0.005	839.534	0.005	959.832	0.005
0.005	664.458	0.006	785.651	0.006	917.003	0.007	1058.517	0.008	1210.194	0.008
0.007	737.025	0.007	871.453	0.008	1017.150	0.009	1174.119	0.010	1342.361	0.010
0.008	772.547	0.009	913.454	0.010	1066.173	0.011	1230.707	0.012	1407.058	0.012
0.010	789.292	0.011	933.253	0.012	1089.283	0.014	1257.383	0.015	1437.556	0.015
0.012	797.045	0.013	942.421	0.015	1099.983	0.016	1269.735	0.017	1451.678	0.017
0.013	800.605	0.015	946.630	0.017	1104.897	0.018	1275.407	0.020	1458.162	0.020
0.015	802.234	0.017	948.556	0.019	1107.144	0.021	1278.001	0.022	1461.128	0.022
0.017	802.977	0.019	949.435	0.021	1108.170	0.023	1279.185	0.025	1462.482	0.025
0.018	803.317	0.021	949.836	0.023	1108.638	0.025	1279.726	0.027	1463.100	0.027
0.020	803.471	0.022	950.019	0.025	1108.852	0.027	1279.972	0.030	1463.382	0.030
0.022	803.542	0.024	950.102	0.027	1108.949	0.030	1280.085	0.032	1463.510	0.032
0.023	803.574	0.026	950.140	0.029	1108.994	0.032	1280.136	0.035	1463.569	0.035
0.025	803.589	0.028	950.158	0.031	1109.014	0.034	1280.159	0.037	1463.596	0.037
0.027	803.596	0.030	950.166	0.033	1109.023	0.037	1280.170	0.040	1463.608	0.040

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

- Cyclic p-y curves have been generated for vertical piles (i.e. no inclination) with a ground slope angle of zero.
- Cyclic loading condition is considered and 500 cycles of loading are assumed. Lateral loading is considered normal to the strong axis.
- There are no pile group effects (i.e., analysis is based on a single pile).
- The effects of construction disturbance are not considered.

- Groundwater table is assumed at the base of embankment fill / top of the weathered silty clay crust.
- Top of pile elevation is assumed to be 68 m.



SUMMARY OF CYCLIC P-Y CURVES FOR H-PILE 310x110 - PIERS 2 & 3

Description Depth (z) * Elevation P-y Curves	Weathered Silty Clay Crust						Grey Silty Clay											
	z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m	
	Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
P-y Curves	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	9.296	0.000	10.746	0.000	12.185	0.000	12.270	0.000	10.881	0.000	10.044	0.000	10.044	0.000	10.044	0.000	5.748
	0.000	18.592	0.000	21.492	0.000	24.370	0.000	24.540	0.000	21.762	0.000	20.088	0.000	20.088	0.000	20.088	0.000	11.497
	0.000	27.888	0.000	32.238	0.000	36.555	0.000	36.810	0.000	32.643	0.000	30.132	0.000	30.132	0.000	30.132	0.000	17.245
	0.001	37.184	0.001	42.983	0.001	48.741	0.001	49.081	0.001	43.524	0.001	40.176	0.001	40.176	0.001	40.176	0.001	22.994
	0.003	46.480	0.003	53.729	0.003	60.926	0.003	61.351	0.003	54.405	0.003	50.220	0.003	50.220	0.003	50.220	0.001	28.742
	0.006	55.776	0.006	64.475	0.006	73.111	0.006	73.621	0.006	65.286	0.006	60.264	0.006	60.264	0.006	60.264	0.002	34.490
	0.011	65.072	0.011	75.221	0.011	85.296	0.011	85.891	0.011	76.167	0.011	70.308	0.011	70.308	0.011	70.308	0.003	40.239
	0.018	74.368	0.018	85.967	0.018	97.481	0.018	98.161	0.018	87.048	0.018	80.352	0.018	80.352	0.018	80.352	0.004	45.987
	0.029	83.664	0.029	96.713	0.029	109.666	0.029	110.431	0.029	97.929	0.029	90.396	0.029	90.396	0.029	90.396	0.006	51.736
	0.045	92.960	0.045	107.459	0.045	121.851	0.045	122.702	0.045	108.810	0.045	100.440	0.045	100.440	0.045	100.440	0.008	57.484
	0.066	102.256	0.066	118.204	0.066	134.037	0.066	134.972	0.066	119.691	0.066	110.484	0.066	110.484	0.066	110.484	0.011	63.232
	0.093	111.552	0.093	128.950	0.093	146.222	0.093	147.242	0.093	130.572	0.093	120.528	0.093	120.528	0.093	120.528	0.015	68.981
	0.128	120.847	0.128	139.696	0.128	158.407	0.128	159.512	0.128	141.453	0.128	130.572	0.128	130.572	0.128	130.572	0.019	74.729
	0.173	130.143	0.173	150.442	0.173	170.592	0.173	171.782	0.173	152.334	0.173	140.616	0.173	140.616	0.173	140.616	0.023	80.478
	0.227	139.439	0.227	161.188	0.227	182.777	0.227	184.052	0.227	163.215	0.227	150.660	0.227	150.660	0.227	150.660	0.116	80.478
	0.284	139.439	0.284	161.188	0.284	182.777	0.284	184.052	0.284	163.215	0.284	150.660	0.284	150.660	0.284	150.660	0.124	80.478

Description Depth (z) * Elevation P-y Curves	Glacial Till										*To account for cyclic softening behaviour, cyclic p-y curves in the Glacial Till are taken as 75% of the static soil reaction for a given pile deflection.
	z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		
	Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	
P-y Curves	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	
	0.001	159.346	0.001	199.189	0.002	243.584	0.002	292.538	0.002	346.058	
	0.003	279.643	0.003	349.565	0.003	427.475	0.004	513.386	0.004	607.311	
	0.004	352.584	0.004	440.745	0.005	538.977	0.006	647.297	0.006	765.722	
	0.005	391.091	0.006	488.880	0.007	597.839	0.008	717.989	0.009	849.347	
	0.006	409.940	0.007	512.442	0.009	626.653	0.010	752.594	0.011	890.283	
	0.008	418.825	0.009	523.549	0.010	640.236	0.012	768.907	0.013	909.580	
	0.009	422.940	0.010	528.692	0.012	646.525	0.013	776.460	0.015	918.515	
	0.010	424.829	0.012	531.054	0.014	649.413	0.015	779.928	0.017	922.618	
	0.012	425.693	0.013	532.134	0.015	650.734	0.017	781.514	0.019	924.494	
	0.013	426.087	0.015	532.627	0.017	651.337	0.019	782.239	0.021	925.351	
	0.014	426.267	0.016	532.852	0.019	651.612	0.021	782.569	0.024	925.742	
	0.015	426.350	0.018	532.955	0.020	651.737	0.023	782.720	0.026	925.921	
	0.017	426.387	0.019	533.002	0.022	651.795	0.025	782.789	0.028	926.002	
	0.018	426.404	0.021	533.023	0.024	651.821	0.027	782.820	0.030	926.039	
	0.019	426.412	0.022	533.033	0.026	651.833	0.029	782.834	0.032	926.056	
	0.021	426.415	0.024	533.037	0.027	651.838	0.031	782.841	0.034	926.064	

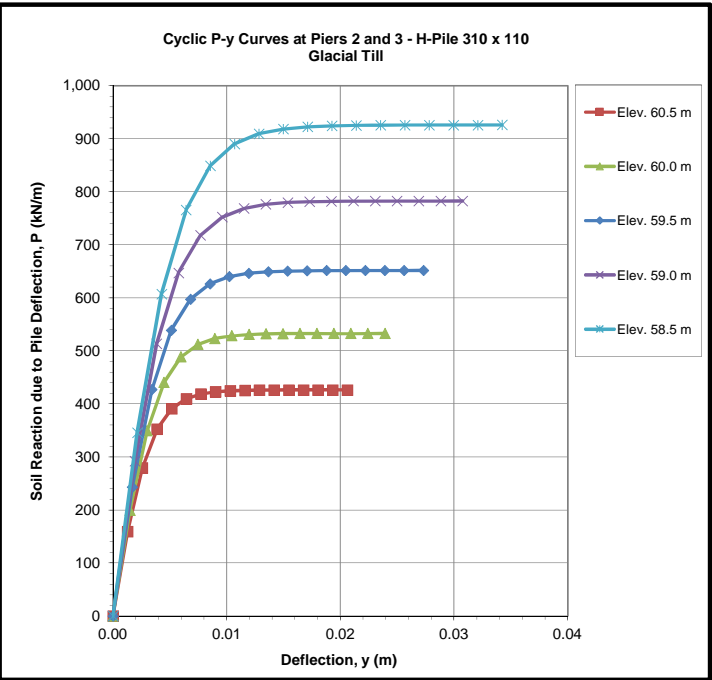
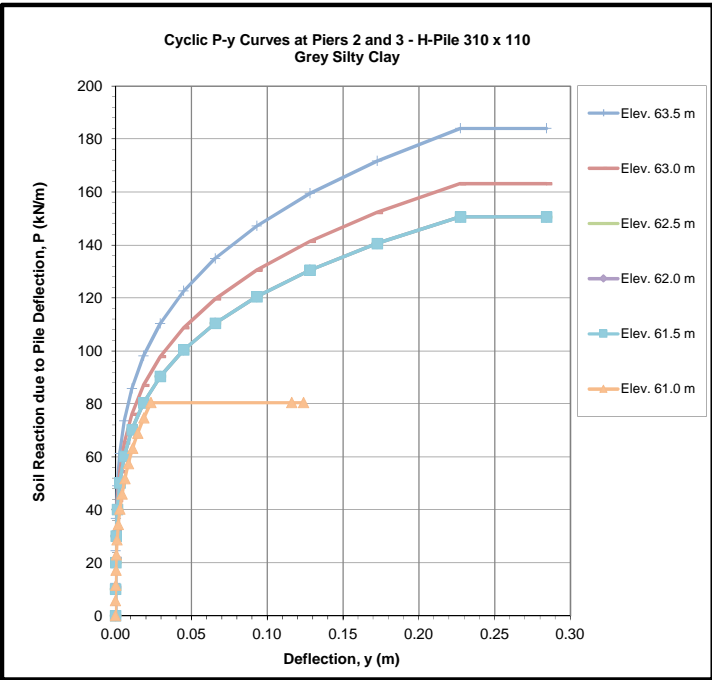
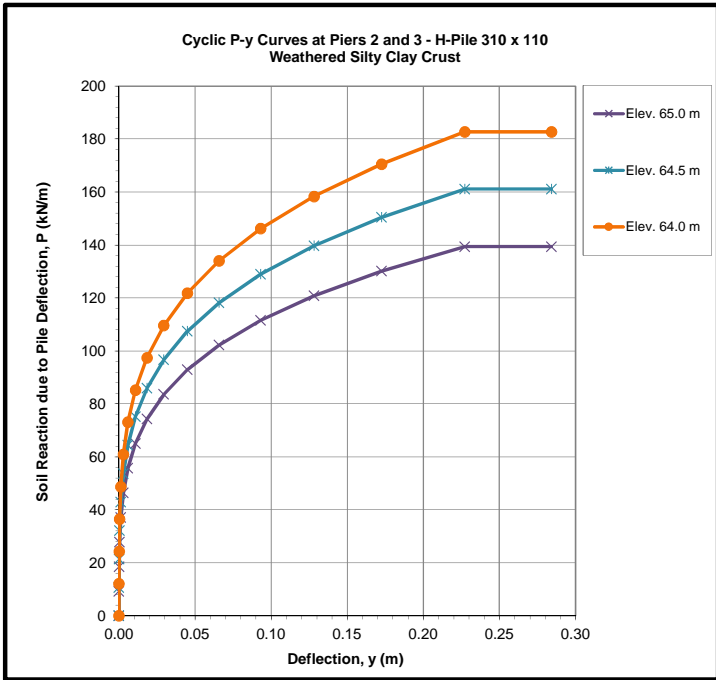
*To account for cyclic softening behaviour, cyclic p-y curves in the Glacial Till are taken as 75% of the static soil reaction for a given pile deflection.

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

1. Cyclic p-y curves have been generated for vertical piles (i.e. no inclination) with a ground slope angle of zero.
2. Cyclic loading condition is considered and 500 cycles of loading are assumed. Lateral loading is considered normal to the strong axis.
3. There are no pile group effects (i.e., analysis is based on a single pile).
4. The effects of construction disturbance are not considered.

5. Groundwater table is assumed at the base of embankment fill / top of the weathered silty clay crust.
6. Top of pile elevation is assumed to be 65 m.



SUMMARY OF CYCLIC P-Y CURVES FOR H-PILE 310x110 - SOUTH ABUTMENT

Description Depth (z) * Elevation P-y Curves	Embankment Fill																									
	z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m		z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m	
	Elev. 72.0 m		Elev. 71.5 m		Elev. 71.0 m		Elev. 70.5 m		Elev. 70.0 m		Elev. 69.5 m		Elev. 69.0 m		Elev. 68.5 m		Elev. 68.0 m		Elev. 67.5 m		Elev. 67.0 m		Elev. 66.5 m		Elev. 66.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
0.003	164.370	0.003	219.961	0.004	283.609	0.004	355.312	0.004	403.006	0.004	443.307	0.004	483.608	0.004	523.908	0.004	564.209	0.004	604.509	0.004	644.810	0.004	685.111	0.004	725.411	0.004
0.006	288.459	0.007	386.018	0.007	497.716	0.008	623.552	0.008	707.252	0.008	777.977	0.008	848.702	0.008	919.427	0.008	990.152	0.008	1060.878	0.008	1131.603	0.008	1202.328	0.008	1273.053	0.008
0.009	363.700	0.010	466.707	0.011	627.539	0.012	786.198	0.013	891.730	0.013	980.903	0.013	1070.076	0.013	1159.249	0.013	1248.423	0.013	1337.596	0.013	1426.769	0.013	1515.942	0.013	1605.115	0.013
0.012	403.420	0.013	539.861	0.015	696.074	0.017	872.060	0.017	989.118	0.017	1088.029	0.017	1186.941	0.017	1285.853	0.017	1384.765	0.017	1483.676	0.017	1582.588	0.017	1681.500	0.017	1780.412	0.017
0.014	422.863	0.017	565.880	0.019	729.622	0.021	914.090	0.021	1036.789	0.021	1140.468	0.021	1244.147	0.021	1347.826	0.021	1451.505	0.021	1555.184	0.021	1658.863	0.021	1762.542	0.021	1866.221	0.021
0.017	432.029	0.020	578.146	0.022	745.437	0.025	933.903	0.025	1059.262	0.025	1165.188	0.025	1271.115	0.025	1377.041	0.025	1482.967	0.025	1588.893	0.025	1694.820	0.025	1800.746	0.025	1906.672	0.025
0.020	436.273	0.023	583.825	0.026	752.760	0.029	943.077	0.030	1069.668	0.030	1176.634	0.030	1283.601	0.030	1390.568	0.030	1497.535	0.030	1604.501	0.030	1711.468	0.030	1818.435	0.030	1925.402	0.030
0.023	438.222	0.026	586.433	0.030	756.122	0.033	947.290	0.034	1074.446	0.034	1181.890	0.034	1289.335	0.034	1396.779	0.034	1504.224	0.034	1611.668	0.034	1719.113	0.034	1826.558	0.034	1934.002	0.034
0.026	439.113	0.030	587.626	0.034	757.660	0.037	949.217	0.038	1076.631	0.038	1184.294	0.038	1291.957	0.038	1399.620	0.038	1507.284	0.038	1614.947	0.038	1722.610	0.038	1830.273	0.038	1937.936	0.038
0.029	439.520	0.033	588.170	0.037	758.362	0.041	950.096	0.042	1077.629	0.042	1185.392	0.042	1293.155	0.042	1400.918	0.042	1508.681	0.042	1616.444	0.042	1724.206	0.042	1831.969	0.042	1939.732	0.042
0.032	439.706	0.036	588.419	0.041	758.683	0.046	950.498	0.047	1078.084	0.047	1185.893	0.047	1293.701	0.047	1401.510	0.047	1509.318	0.047	1617.126	0.047	1724.935	0.047	1832.743	0.047	1940.552	0.047
0.035	439.790	0.040	588.532	0.045	758.829	0.050	950.681	0.051	1078.292	0.051	1186.121	0.051	1293.950	0.051	1401.780	0.051	1509.609	0.051	1617.438	0.051	1725.267	0.051	1833.096	0.051	1940.926	0.051
0.037	439.829	0.043	588.584	0.048	758.895	0.054	950.764	0.055	1078.387	0.055	1186.225	0.055	1294.064	0.055	1401.903	0.055	1509.741	0.055	1617.580	0.055	1725.419	0.055	1833.257	0.055	1941.096	0.055
0.040	439.847	0.046	588.607	0.052	758.926	0.058	950.802	0.059	1078.430	0.059	1186.273	0.059	1294.116	0.059	1401.959	0.059	1509.802	0.059	1617.645	0.059	1725.488	0.059	1833.331	0.059	1941.174	0.059
0.043	439.855	0.050	588.618	0.056	758.940	0.062	950.820	0.064	1078.450	0.064	1186.294	0.064	1294.139	0.064	1401.984	0.064	1509.829	0.064	1617.674	0.064	1725.519	0.064	1833.364	0.064	1941.209	0.064
0.046	439.858	0.053	588.623	0.060	758.946	0.066	950.828	0.068	1078.459	0.068	1186.304	0.068	1294.150	0.068	1401.996	0.068	1509.842	0.068	1617.688	0.068	1725.534	0.068	1833.379	0.068	1941.225	0.068
Description Depth (z) * Elevation P-y Curves	Weathered Silty Clay Crust												Grey Silty Clay						Glacial Till							
	z= 9.5 m		z= 10.0 m		z= 10.5 m		z= 11.0 m		z= 11.5 m		z= 12.0 m		z= 12.5 m		z= 13.0 m		z= 13.5 m		z= 14.0 m		z= 14.5 m		z= 15.0 m			
	Elev. 65.5 m		Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m		Elev. 60.5 m		Elev. 60.0 m			
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)		
0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000		
0.000	14.880	0.000	14.880	0.000	14.880	0.000	14.880	0.000	13.299	0.000	10.881	0.000	10.044	0.000	10.044	0.000	10.044	0.004	1154.602	0.004	1187.675	0.004	1220.749	0.004		
0.000	29.760	0.000	29.760	0.000	29.760	0.000	29.760	0.000	26.598	0.000	21.762	0.000	20.088	0.000	20.088	0.000	20.088	0.009	2026.257	0.009	2084.299	0.009	2142.340	0.009		
0.000	44.640	0.000	44.640	0.000	44.640	0.000	44.640	0.000	39.897	0.000	32.643	0.000	30.132	0.000	30.132	0.000	30.132	0.013	2554.783	0.013	2627.964	0.013	2701.145	0.013		
0.001	59.520	0.001	59.520	0.001	59.520	0.001	59.520	0.001	53.196	0.001	43.524	0.001	40.176	0.001	40.176	0.001	40.176	0.017	2833.795	0.017	2914.968	0.017	2996.142	0.017		
0.003	74.400	0.003	74.400	0.003	74.400	0.003	74.400	0.003	66.495	0.003	54.405	0.003	50.220	0.003	50.220	0.003	50.220	0.022	2970.373	0.022	3055.459	0.021	3140.544	0.021		
0.006	89.280	0.006	89.280	0.006	89.280	0.006	89.280	0.006	79.794	0.006	65.286	0.006	60.264	0.006	60.264	0.006	60.264	0.026	3034.757	0.026	3121.687	0.026	3208.617	0.026		
0.011	104.160	0.011	104.160	0.011	104.160	0.011	104.160	0.011	93.093	0.011	76.167	0.011	70.308	0.011	70.308	0.011	70.308	0.030	3064.569	0.030	3152.352	0.030	3240.136	0.030		
0.018	119.040	0.018	119.040	0.018	119.040	0.018	119.040	0.018	106.392	0.018	87.048	0.018	80.352	0.018	80.352	0.018	80.352	0.035	3078.257	0.034	3166.433	0.034	3254.609	0.034		
0.029	133.920	0.029	133.920	0.029	133.920	0.029	133.920	0.029	119.691	0.029	97.929	0.029	90.396	0.029	90.396	0.029	90.396	0.039	3084.519	0.039	3172.874	0.038	3261.229	0.038		
0.045	148.800	0.045	148.800	0.045	148.800	0.045	148.800	0.045	132.990	0.045	108.810	0.045	100.440	0.045	100.440	0.045	100.440	0.043	3087.378	0.043	3175.815	0.043	3264.252	0.043		
0.066	163.680	0.066	163.680	0.066	163.680	0.066	163.680	0.066	146.289	0.066	119.691	0.066	110.484	0.066	110.484	0.066	110.484	0.048	3088.682	0.047	3177.157	0.047	3265.631	0.047		
0.093	178.560	0.093	178.560	0.093	178.560	0.093	178.560	0.093	159.588	0.093	130.572	0.093	120.528	0.093	120.528	0.093	120.528	0.052	3089.277	0.052	3177.769	0.051	3266.260	0.051		
0.128	193.440	0.128	193.440	0.128	193.440	0.128	193.440	0.128	172.887	0.128	141.453	0.128	130.572	0.128	130.572	0.128	130.572	0.056	3089.548	0.056	3178.048	0.056	3266.547	0.056		
0.173	208.320	0.173	208.320	0.173	208.320	0.173	208.320	0.173	186.186	0.173	152.334	0.173	140.616	0.173	140.616	0.173	140.616	0.061	3089.672	0.060	3178.175	0.060	3266.678	0.060		
0.227	223.200	0.227	223.200	0.227	223.200	0.227	223.200	0.227	199.485	0.227	163.215	0.227	150.660	0.227	150.660	0.227	150.660	0.065	3089.729	0.065	3178.233	0.064	3266.737	0.064		
0.284	223.200	0.284	223.200	0.284	223.200	0.284	223.200	0.284	199.485	0.284	163.215	0.284	150.660	0.284	150.660	0.284	150.660	0.069	3089.754	0.069	3178.259	0.068	3266.765	0.068		

SUMMARY OF P-Y CURVES FOR CAISSON (OD = 1.0 m) - PIERS 2 & 3

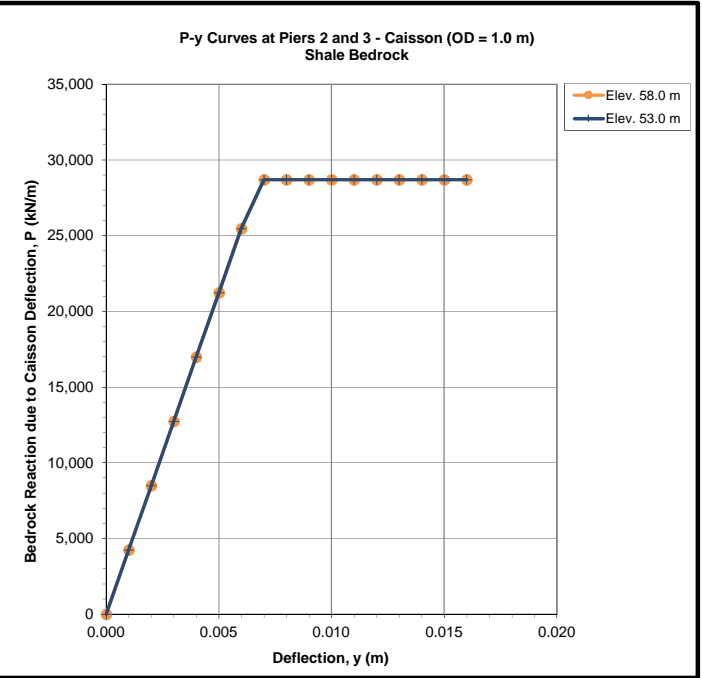
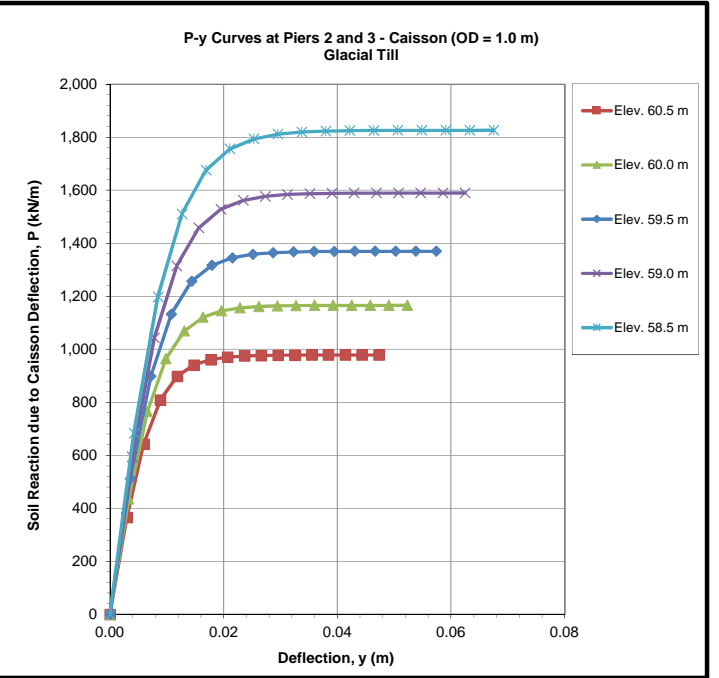
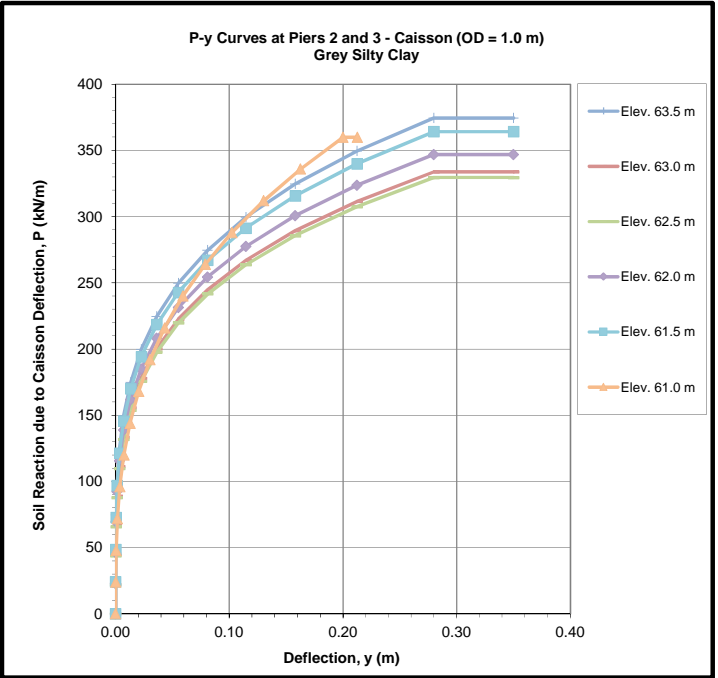
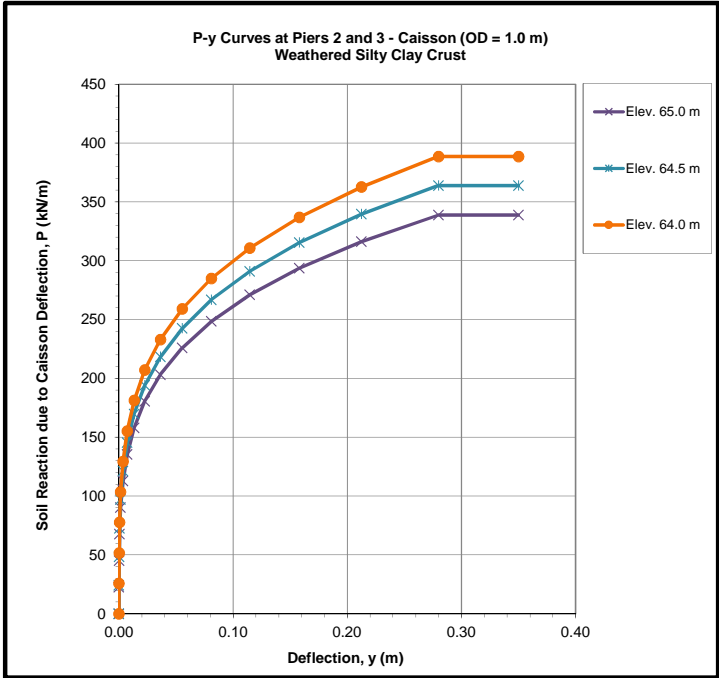
Description Depth (z) * Elevation P-y Curves	Weathered Silty Clay Crust						Grey Silty Clay											
	z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m	
	Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
P-y Curves	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	22.593	0.000	24.265	0.000	25.914	0.000	24.967	0.000	22.256	0.000	21.973	0.000	23.130	0.000	24.283	0.000	24.000
	0.000	45.187	0.000	48.529	0.000	51.829	0.000	49.935	0.000	44.512	0.000	43.947	0.000	46.259	0.000	48.566	0.000	48.000
	0.000	67.780	0.000	72.794	0.000	77.743	0.000	74.902	0.000	66.768	0.000	65.920	0.000	69.389	0.000	72.849	0.002	72.000
	0.001	90.373	0.001	97.058	0.001	103.657	0.001	99.870	0.001	89.024	0.001	87.893	0.001	92.518	0.001	97.132	0.004	96.000
	0.003	112.966	0.003	121.323	0.003	129.572	0.003	124.837	0.003	111.280	0.003	109.866	0.003	115.648	0.003	121.415	0.007	120.000
	0.007	135.560	0.007	145.587	0.007	155.486	0.007	149.804	0.007	133.536	0.007	131.840	0.007	138.778	0.007	145.698	0.013	144.000
	0.013	158.153	0.013	169.852	0.013	181.400	0.013	174.772	0.013	155.792	0.013	153.813	0.013	161.907	0.013	169.981	0.020	168.000
	0.023	180.746	0.023	194.116	0.023	207.314	0.023	199.739	0.023	178.049	0.023	175.786	0.023	185.037	0.023	194.264	0.030	192.000
	0.036	203.339	0.036	218.381	0.036	233.229	0.036	224.706	0.036	200.305	0.036	197.760	0.036	208.166	0.036	218.547	0.043	216.000
	0.055	225.933	0.055	242.646	0.055	259.143	0.055	249.674	0.055	222.561	0.055	219.733	0.055	231.296	0.055	242.830	0.059	240.000
	0.081	248.526	0.081	266.910	0.081	285.057	0.081	274.641	0.081	244.817	0.081	241.706	0.081	254.426	0.081	267.113	0.079	264.000
	0.115	271.119	0.115	291.175	0.115	310.972	0.115	299.609	0.115	267.073	0.115	263.679	0.115	277.555	0.115	291.396	0.102	288.000
	0.158	293.712	0.158	315.439	0.158	336.886	0.158	324.576	0.158	289.329	0.158	285.653	0.158	300.685	0.158	315.679	0.130	312.000
	0.212	316.306	0.212	339.704	0.212	362.800	0.212	349.543	0.212	311.585	0.212	307.626	0.212	323.815	0.212	339.962	0.163	336.000
	0.280	338.899	0.280	363.968	0.280	388.715	0.280	374.511	0.280	333.841	0.280	329.599	0.280	346.944	0.280	364.244	0.200	360.000
	0.350	338.899	0.350	363.968	0.350	388.715	0.350	374.511	0.350	333.841	0.350	329.599	0.350	346.944	0.350	364.244	0.213	360.000
P-y Curves	Glacial Till						Shale Bedrock											
	z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m		z= 14.0 m					
	Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 53.0 m					
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.003	365.674	0.003	435.718	0.004	511.892	0.004	594.195	0.004	682.628	0.001	4245	0.001	4245				
	0.006	641.736	0.007	764.659	0.007	898.339	0.008	1042.776	0.008	1197.970	0.002	8489	0.002	8489				
	0.009	809.126	0.010	964.112	0.011	1132.661	0.012	1314.772	0.013	1510.447	0.003	12734	0.003	12734				
	0.012	897.491	0.013	1069.404	0.014	1256.360	0.016	1458.361	0.017	1675.405	0.004	16979	0.004	16979				
	0.015	940.747	0.016	1120.945	0.018	1316.912	0.020	1528.648	0.021	1756.154	0.005	21224	0.005	21224				
	0.018	961.138	0.020	1145.242	0.022	1345.457	0.023	1561.782	0.025	1794.219	0.006	25468	0.006	25468				
	0.021	970.580	0.023	1156.492	0.025	1358.674	0.027	1577.124	0.030	1811.844	0.007	28700	0.007	28700				
	0.024	974.915	0.026	1161.658	0.029	1364.742	0.031	1584.169	0.034	1819.937	0.008	28700	0.008	28700				
	0.027	976.898	0.029	1164.021	0.032	1367.518	0.035	1587.391	0.038	1823.639	0.009	28700	0.009	28700				
	0.030	977.804	0.033	1165.100	0.036	1368.786	0.039	1588.862	0.042	1825.329	0.010	28700	0.010	28700				
	0.033	978.217	0.036	1165.592	0.039	1369.364	0.043	1589.534	0.046	1826.101	0.011	28700	0.011	28700				
	0.035	978.405	0.039	1165.817	0.043	1369.628	0.047	1589.840	0.051	1826.452	0.012	28700	0.012	28700				
	0.038	978.491	0.043	1165.919	0.047	1369.748	0.051	1589.980	0.055	1826.613	0.013	28700	0.013	28700				
	0.041	978.530	0.046	1165.966	0.050	1369.803	0.055	1590.043	0.059	1826.686	0.014	28700	0.014	28700				
	0.044	978.548	0.049	1165.987	0.054	1369.828	0.059	1590.072	0.063	1826.719	0.015	28700	0.015	28700				
	0.047	978.556	0.052	1165.997	0.057	1369.840	0.062	1590.085	0.066	1826.734	0.016	28700	0.016	28700				

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

1. P-y curves have been generated a vertical caisson (i.e., no inclination) with a ground slope angle of zero.
2. Static loading condition is considered.
3. There are no caisson group effects (i.e., analysis is based on a single caisson).
4. The effects of construction disturbance are not considered.

5. Pier locations as per WSP drawing file nos. CNR_EBL_001_GA_I_NEW_ALIGNEMENT.dwg and CNR_WBL_001_GA_I.dwg, received January 8, 2018.
6. Groundwater table assumed at base of embankment fill / top of weathered silty clay crust (Elevation 65.0 m).
7. Top of caisson assumed at Elevation 65.0 m.



SUMMARY OF CYCLIC P-Y CURVES FOR CAISSON (OD = 1.0 m) - PIERS 2 & 3

Description Depth (z) * Elevation P-y Curves	Weathered Silty Clay Crust						Grey Silty Clay											
	z= 2.0 m		z= 2.5 m		z= 3.0 m		z= 3.5 m		z= 4.0 m		z= 4.5 m		z= 5.0 m		z= 5.5 m		z= 6.0 m	
	Elev. 65.0 m		Elev. 64.5 m		Elev. 64.0 m		Elev. 63.5 m		Elev. 63.0 m		Elev. 62.5 m		Elev. 62.0 m		Elev. 61.5 m		Elev. 61.0 m	
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
Piers 2 & 3	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.000	19.462	0.000	21.264	0.000	23.002	0.000	22.336	0.000	20.011	0.000	19.885	0.000	21.075	0.000	22.256	0.000	18.543
	0.000	38.923	0.000	42.529	0.000	46.003	0.000	44.671	0.000	40.023	0.000	39.771	0.000	42.150	0.000	44.511	0.000	37.086
	0.001	58.385	0.001	63.793	0.001	69.005	0.001	67.007	0.001	60.034	0.001	59.656	0.001	63.225	0.001	66.767	0.001	55.630
	0.004	77.846	0.004	85.057	0.004	92.007	0.004	89.342	0.004	80.045	0.004	79.542	0.004	84.300	0.004	89.022	0.002	74.173
	0.009	97.308	0.009	106.322	0.009	115.008	0.009	111.678	0.009	100.056	0.009	99.427	0.009	105.375	0.009	111.278	0.003	92.716
	0.019	116.769	0.019	127.586	0.019	138.010	0.019	134.013	0.019	120.068	0.019	119.313	0.019	126.450	0.019	133.533	0.006	111.259
	0.035	136.231	0.035	148.850	0.035	161.012	0.035	156.349	0.035	140.079	0.035	139.198	0.035	147.525	0.035	155.789	0.009	129.802
	0.059	155.693	0.059	170.114	0.059	184.013	0.059	178.684	0.059	160.090	0.059	159.084	0.059	168.600	0.059	178.044	0.014	148.346
	0.095	175.154	0.095	191.379	0.095	207.015	0.095	201.020	0.095	180.101	0.095	178.969	0.095	189.675	0.095	200.300	0.020	166.889
	0.145	194.616	0.145	212.643	0.145	230.017	0.145	223.355	0.145	200.113	0.145	198.855	0.145	210.750	0.145	222.555	0.027	185.432
	0.212	214.077	0.212	233.907	0.212	253.018	0.212	245.691	0.212	220.124	0.212	218.740	0.212	231.825	0.212	244.811	0.036	203.975
	0.300	233.539	0.300	255.172	0.300	276.020	0.300	268.026	0.300	240.135	0.300	238.626	0.300	252.900	0.300	267.066	0.047	222.519
	0.414	253.000	0.414	276.436	0.414	299.022	0.414	290.362	0.414	260.146	0.414	258.511	0.414	273.975	0.414	289.322	0.060	241.062
	0.557	272.462	0.557	297.700	0.557	322.023	0.557	312.697	0.557	280.158	0.557	278.397	0.557	295.050	0.557	311.578	0.075	259.605
	0.733	291.924	0.733	318.965	0.733	345.025	0.733	335.033	0.733	300.169	0.733	298.282	0.733	316.125	0.733	333.833	0.375	259.605
	0.917	291.924	0.917	318.965	0.917	345.025	0.917	335.033	0.917	300.169	0.917	298.282	0.917	316.125	0.917	333.833	0.400	259.605
Piers 2 & 3	Glacial Till - Reduced Dynamic Stiffness*						Shale Bedrock											
	z= 6.5 m		z= 7.0 m		z= 7.5 m		z= 8.0 m		z= 8.5 m		z= 9.0 m		z= 9.0 m		z= 14.0 m			
	Elev. 60.5 m		Elev. 60.0 m		Elev. 59.5 m		Elev. 59.0 m		Elev. 58.5 m		Elev. 58.0 m		Elev. 58.0 m		Elev. 53.0 m			
	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)	y (m)	P (kN/m)
	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
	0.003	274.256	0.003	326.789	0.004	383.919	0.004	445.646	0.004	511.971	0.001	4245	0.001	4245	0.002	8489	0.002	8489
	0.006	481.302	0.007	573.494	0.007	673.754	0.008	782.082	0.008	898.478	0.002	8489	0.002	8489	0.002	8489	0.002	8489
	0.009	606.844	0.010	723.084	0.011	849.496	0.012	986.079	0.013	1132.835	0.003	12734	0.003	12734	0.003	12734	0.003	12734
	0.012	673.119	0.013	802.053	0.014	942.270	0.016	1093.771	0.017	1256.554	0.004	16979	0.004	16979	0.004	16979	0.004	16979
	0.015	705.560	0.016	840.709	0.018	987.684	0.020	1146.486	0.021	1317.115	0.005	21224	0.005	21224	0.005	21224	0.005	21224
	0.018	720.854	0.020	858.932	0.022	1009.093	0.023	1171.337	0.025	1345.664	0.006	25468	0.006	25468	0.006	25468	0.006	25468
	0.021	727.935	0.023	867.369	0.025	1019.005	0.027	1182.843	0.030	1358.883	0.007	28700	0.007	28700	0.007	28700	0.007	28700
	0.024	731.186	0.026	871.243	0.029	1023.557	0.031	1188.127	0.034	1364.953	0.008	28700	0.008	28700	0.008	28700	0.008	28700
	0.027	732.674	0.029	873.016	0.032	1025.639	0.035	1190.543	0.038	1367.729	0.009	28700	0.009	28700	0.009	28700	0.009	28700
	0.030	733.353	0.033	873.825	0.036	1026.589	0.039	1191.647	0.042	1368.997	0.010	28700	0.010	28700	0.010	28700	0.010	28700
	0.033	733.663	0.036	874.194	0.039	1027.023	0.043	1192.150	0.046	1369.575	0.011	28700	0.011	28700	0.011	28700	0.011	28700
	0.035	733.804	0.039	874.362	0.043	1027.221	0.047	1192.380	0.051	1369.839	0.012	28700	0.012	28700	0.012	28700	0.012	28700
	0.038	733.868	0.043	874.439	0.047	1027.311	0.051	1192.485	0.055	1369.960	0.013	28700	0.013	28700	0.013	28700	0.013	28700
	0.041	733.898	0.046	874.474	0.050	1027.352	0.055	1192.532	0.059	1370.014	0.014	28700	0.014	28700	0.014	28700	0.014	28700
	0.044	733.911	0.049	874.490	0.054	1027.371	0.059	1192.554	0.063	1370.039	0.015	28700	0.015	28700	0.015	28700	0.015	28700
	0.047	733.917	0.052	874.497	0.057	1027.380	0.062	1192.564	0.068	1370.051	0.016	28700	0.016	28700	0.016	28700	0.016	28700

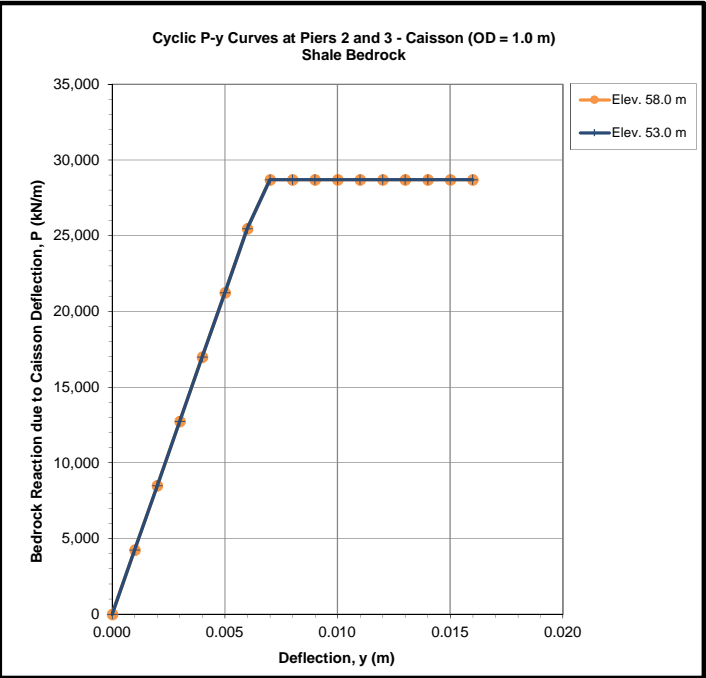
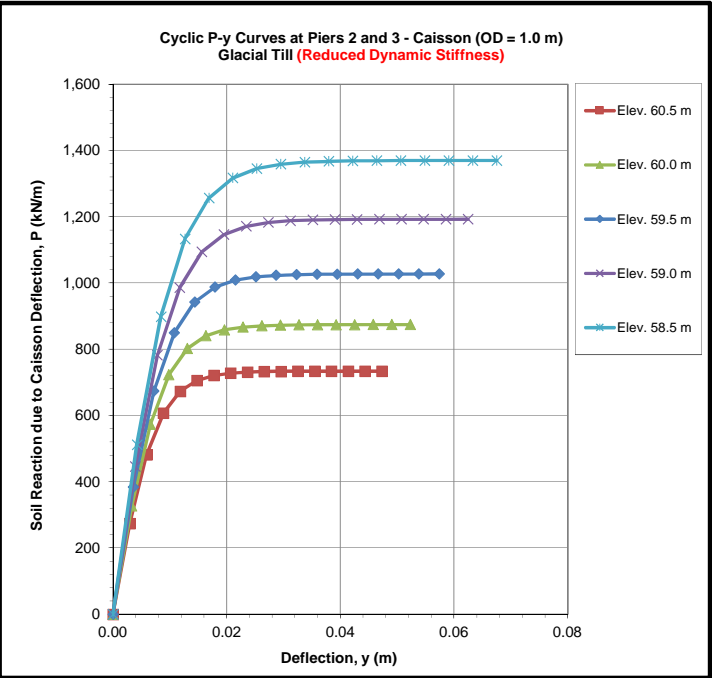
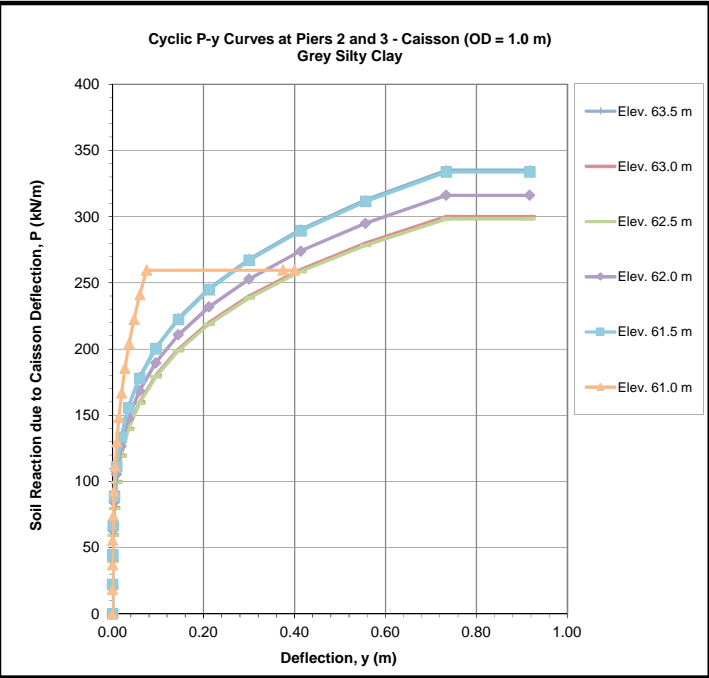
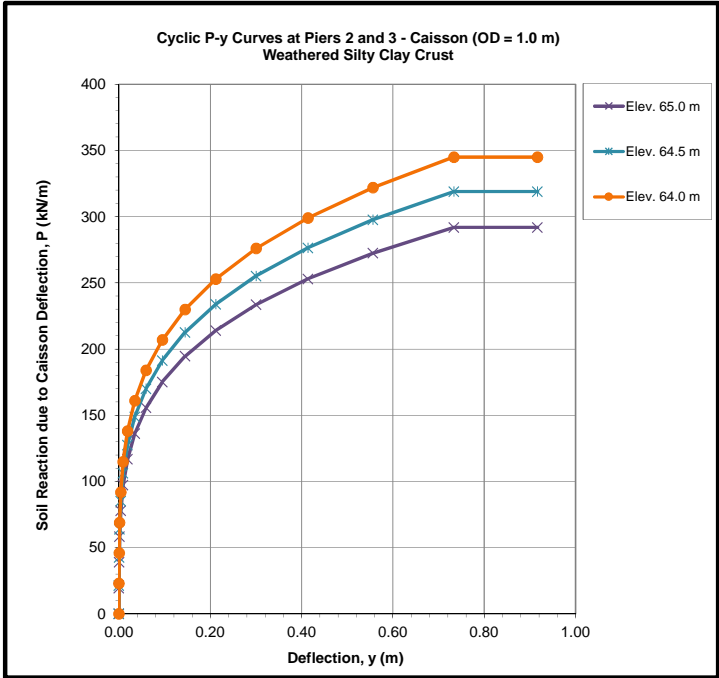
*To account for cyclic softening behaviour, cyclic p-y curves in the Glacial Till are taken as 75% of the static soil reaction for a given caisson deflection.

NOTES: * Depth (z) is measured to be positive below the ground surface.

Please note the following assumptions:

- Cyclic p-y curves have been generated a vertical caisson (i.e., no inclination) with a ground slope angle of zero.
- Cyclic loading condition is considered and 500 cycles of loading are assumed.
- There are no caisson group effects (i.e., analysis is based on a single caisson).
- The effects of construction disturbance are not considered.

- Pier locations as per WSP drawing file nos. CNR_EBL_001_GA_I_NEW_ALIGNEMENT.dwg and CNR_WBL_001_GA_I.dwg, received January 8, 2018.
- Groundwater table assumed at base of embankment fill / top of weathered silty clay crust (Elevation 65.0 m).
- Top of caisson assumed at Elevation 65.0 m.



APPENDIX H

Non-Standard Special Provisions

Obstructions During Piling

Vibration Monitoring During Piling

Driving Piles Adjacent to Existing Piles

Expanded Polystyrene Embankment Fill

Settlement Monitoring During Grading

Excavating Through Obstructions - Structures

Deep Foundations - Item No.

Special Provision

Amendment to OPSS 903, April 2016

Obstructions During Piling

This special provision describes requirements for pile installation through obstructions and natural cobbles and boulders.

Definitions

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

Submission Requirements

The Contractor shall submit details for advancing the piles through obstructions, cobbles and boulders. The submittals shall satisfy the specifications and at a minimum contain specific information on their approach to advancing the piles in the event such conditions are encountered.

Pile Driving Through Obstacles, Cobbles, Boulders

The soils at the site are glacially-derived and are known to contain cobbles and boulders within the till deposits. The embankment fills at the site may also contain some obstructions. The Contractor is advised that appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of deep foundations.

Basis of Payment

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

END OF SECTION

Deep Foundations - Item No.

Special Provision

Amendment to OPSS 903, April 2016

Vibration Monitoring During Piling

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

Definitions

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

Submission Requirements

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

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

The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the FES.

Monitoring

The FES shall take readings during driving of each pile. The readings should be taken and recorded during the entire length of driving and during seating of the pile on the bedrock (if applicable). As a minimum, one vibration monitoring point shall be installed on the nearest existing abutment wall to the pile driving activities.

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

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

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

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

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

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

Basis of Payment

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

END OF SECTION

Deep Foundations - Item No.

Special Provision

Amendment to OPSS 903, April 2016

Driving Piles Adjacent to Existing Piles

This special provision describes requirements for pile installation adjacent to existing piles.

Definitions

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

Pile Driving Adjacent to Existing Piles

For new piles driven within the potential zone of interference with the existing abutment or wing wall piles (defined as a distance around the existing pile tip at depth equal to 10% of the pile length) the driving operations shall be continuously monitored by the FES.

The contractor shall cease driving of the pile if the FES indicates that the driven pile may have come in contact with an existing pile.

If contact between the new and existing piles is believed to exist the contractor shall take remedial action as directed by the Contract Administrator, which may include extracting the pile and re-driving or replacing the pile.

Basis of Payment

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

END OF SECTION

Expanded Polystyrene Embankment - Item No.

Special Provision

Amendment to OPSS 206, November 2014

Requirements for Expanded Polystyrene Embankment Fill

206.01 Scope

Section 206.01 of OPSS 206 shall be amended by the addition of the following:

Shall also include the supply and installation of expanded polystyrene embankment fill, including foundation preparation, excavation, leveling pad, polyethylene sheeting and associated works as shown on the contract drawings.

206.03 Definitions

Section 206.03 of OPSS 206 shall be amended by the addition of the following:

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

Rigid Expanded Polystyrene: shall consist of molded rigid blocks produced by a process of pre-expansion, aging and forming of a petroleum based raw material.

Production Lot: shall consist of a quantity of rigid polystyrene blocks produced in a continuous period of manufacturing the same grade and thickness of product within the same production day.

206.07.05.01 Earth Embankments

Section 206.07.05.01 of OPSS 206 shall be amended by the addition of the following:

Qualifications

The Contractor shall have on site at the commencement of the work a representative of the supplier of the rigid expanded polystyrene to advise on recommended construction procedure.

The Contractor shall maintain liaison with the supplier throughout the construction of the embankment for advice and guidance as required. Periodic site visits by the supplier should be coordinated as required.

Submission and Design Requirements

Submission of Shop Drawings

At least three weeks before the commencement of work, the Contractor shall submit to the Contract Administrator and FES six copies of the shop drawings and a method statement that provides full details of the materials and construction procedure.

Delivery, Storage, Handling and Protection

The Contractor shall submit the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the rigid expanded polystyrene manufacturer's requirements.

Construction

The contractor shall submit full details of the following.

- a) The method of foundation excavation and preparation.
- b) Construction of granular leveling pad.
- c) The method of placement of expanded polystyrene including temporary ballasting (if required) and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer by layer basis.
- d) The method and limits of placement of polyethylene sheeting.
- e) The method of placement of protective concrete slab.
- f) The method of placement of subbase material.
- g) The method of placement of side slope cover.

Quality Assurance

The Contractor shall submit to the Contract Administrator and FES a Certificate of Conformance sealed and signed, a minimum of one week prior to the commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents. Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision, shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation.

Materials

Granular Leveling Pad

The leveling pad shall consist of a Granular 'A' material with gradation and physical requirements as specified in OPSS.PROV 1010.

Rigid Expanded Polystyrene

General

The Contractor shall submit:

1. A general statement as to the type, composition, and method of production of the material.
2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the rigid expanded polystyrene.
3. Certification of compliance of physical and mechanical properties.
4. An identification of a laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the expanded polystyrene.
5. The physical and mechanical properties of the rigid expanded polystyrene including:
 1. Geometry
 2. Nominal Density
 3. Compressive Strength
 4. Flexural Strength
 5. Dimensional Stability
 6. Oxygen Index
 7. Water Absorption
6. Aging and durability characteristics of the polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.
7. A sample of the expanded polystyrene material to the Contract Administrator for review.
8. To the Contract Administrator, a Certificate of Conformance sealed and signed by the Quality Verification Engineer. The Certificate shall state that the expanded polystyrene material is in conformance with the requirements and specifications of the contract documents. Certificate to be submitted a minimum of one week prior to commencement of work under this item.

Each block of the same production lot shall be stamped with the same production code showing plant identification, type and date of production. The polystyrene shall be free from defects affecting serviceability.

Detail Requirements

The polystyrene shall meet the requirements for EPS22, as defined by ASTM D6817/D6817M-17, as follows:

TABLE 1 – MATERIAL PROPERTIES

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear - Flatness - Squareness - Thickness	mm	1200 x 600 x 100 ± 0.5%	
Compressive Strength at 5% strain	kPa (min)	115	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240	ASTM C203
Dimensional Stability	% linear change (max)	1.5	ASTM D2126
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	4	ASTM D2842

Geometry

The expanded polystyrene shall be supplied in the form of rectangular parallel sheets bundled into minimum acceptable dimensions of 1200 mm x 600 mm x 100 mm.

The maximum deviation from the specified linear dimensions, flatness, squareness and thickness shall be ± 0.5%.

Compressive Strength

The minimum compressive strength, measured in accordance with ASTM D1621, Procedure A, shall be 115 kPa at a strain of not more than 5%. The maximum design permanent stress level must not exceed 30% of the compressive strength of the material at 5% strain.

Flexural Strength

The minimum flexural strength of the polystyrene shall be 240 kPa. The flexural strength shall be determined in accordance to ASTM C203, Method 1, Procedure B.2.7.4 Dimensional Stability.

Dimensional Stability

Dimensional Stability shall be determined in accordance with ASTM D2126, Procedure G. A tolerance of 1.5% shall be satisfied.

Flammability

The expanded polystyrene shall be classified as to surface burning characteristics in accordance with CAN/ULC - 51022 having a flame spread rating less than 500. The expanded polystyrene shall have a minimum limiting oxygen index measured in accordance with ASTM D2863

Water Absorption

The water absorption as measured by ASTM D2842 shall be limited to 4% by volume.

Chemical Resistance

The expanded polystyrene shall be resistant to common inorganic acids and alkalies. A table identifying the chemical resistance as either resistant, limited or not resistant shall be submitted.

Biological Resistance

The expanded polystyrene shall be resistant to biological degradation caused by organisms or enzymes.

Environmental

The expanded polystyrene shall be inert, non-nutritive and highly stable and shall not produce undesirable gases or leachate.

Construction**Delivery, Storage and Handling**

The product shall be suitably marked to identify its type, number and the manufacturer's name or trademark.

The Contractor shall protect the expanded polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer's recommendation.

Protection of materials and works from damage by weather, traffic, construction staging, fire or vandalism and other causes shall be the responsibility of the Contractor.

Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with Granular 'A' material.

Leveling Pad

Place, level and compact a 300 mm thick layer of Granular 'A' material in accordance with OPSS.PROV 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks. The leveling pad shall not be placed on frozen ground.

Installation of Polystyrene

- 1) The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.
- 2) Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90° to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.
- 3) A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with a maximum joint opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer should not exceed 5 mm.
- 4) Sloping end adjustments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.
- 5) Temporary ballast shall be provided as necessary to prevent movement of expanded polystyrene both in storage and as placed due to windy conditions. Timber fasteners or equivalent shall be used as necessary.
- 6) The expanded polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the expanded polystyrene.
- 7) The expanded polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction.
- 8) Exposed blocks shall be covered immediately to avoid possible burrowing by animals.
- 9) Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.
- 10) The top surface and side surfaces of the expanded polystyrene shall be covered with 10 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.
- 11) The side slope of the rigid expanded polystyrene embankment shall be covered with fill material as detailed elsewhere in this contract.

Equipment

All cutting of polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the expanded polystyrene as per the manufacturer's requirement.

Quality Assurance

General

Quality test certificates for each production lot supplied, showing compliance with all requirements of this special provision shall be obtained by the Contractor and submitted to the Contract Administrator prior to installation.

Sampling and Testing

The Contract Administrator may undertake an independent testing program of the expanded polystyrene. Sampling and testing will be carried out in conformance with the relevant test procedure. The physical and thermal property testing identified in Table 1 may be conducted. The testing shall be conducted by a recognized testing laboratory accredited by the Standards Council of Canada.

Sampling Frequency

Sufficient sample material shall be obtained from blocks randomly selected by the Contract Administrator from each production lot as soon as the material arrives on site. As a minimum, one (1) block shall be tested for the full suite of tests and three (3) blocks shall be tested for compressive strength.

Acceptance/Rejection

Failure of any one of the sample blocks to comply with any requirements of this special provision shall be cause for rejection of the production lot from which it was taken. Replacement of the blocks shall be at the Contractor's expense.

Measurement of Payment

Measurement will be by volume in cubic metres measured in its original position and based on cross-sections.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work as described above and no extra payments will be made.

END OF SECTION

Settlement Rods - Item No.

Special Provision

Amendment to OPSS 206, November 2014

Settlement Monitoring During Grading

206.01 *Scope*

Section 206.01 of OPSS 206 shall be amended by the addition of the following:

Shall also include the supply and installation of settlement rods and data collection during construction. The purpose of the settlement rods is to directly monitor settlement of the soils in areas of the widened approach embankments. Settlement is to be measured by survey of the top of the rod with reference to stable, non-settling benchmarks.

206.03 *Definitions*

Section 206.03 of OPSS 206 shall be amended by the addition of the following:

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

206.07.05.01 *Earth Embankments*

Section 206.07.05.01 of OPSS 206 shall be amended by the addition of the following:

General Procedure Settlement Rods

Settlement rods shall be installed in pre-excavated holes through the bench of the existing embankments and into the top of the underlying native silty clay deposit. The hole shall be at least 100 mm in inner diameter, with the bottom of the rods concreted in-place at about Elevation 64.5 m. The settlement rods shall be installed before any new embankment fill placement. As the embankment construction proceeds, the rods shall be extended above the new ground level.

Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod as the embankment settles. As embankment construction proceeds, the sleeves shall also be extended above the new ground level.

A protective surround shall be extended with the rods and sleeves as embankment construction proceeds above the existing embankment grade.

The holes and protective surround shall be backfilled with uniform sand around the rods and sleeves. NO sand is to be placed within the sleeves.

Once the embankment preload has been constructed to its final grade, the rods and sleeves shall be cut down to just below subgrade level and covered with a flush mount casing.

Materials

Rod

The Contractor shall supply a steel rod with an outside diameter of at least 19 mm.

The top of the rod shall be capped in such a way that a single survey point can be clearly identified and returned to.

Concrete

The Contractor shall supply concrete, with a compressive strength of about 20 MPa, to concrete the bottom of the rod in-place within each pre-excavated hole.

Friction Reducing Sleeve

The Contractor shall supply a PVC pipe, friction reducing sleeve with an internal diameter of at least 25 mm and at least 25 per cent larger than the rod outside diameter.

Sand

The Contractor shall supply uniform medium-grained sand to backfill the hole once the rod and friction reducing sleeve are in place as well as within the protective surround as the rod and sleeve are extended as embankment construction proceeds above the existing embankment grade.

Protective Surround

The Contractor shall supply a protective surround for the portion of the rod and sleeve above the existing embankment grade. The surround shall consist of a 300 mm diameter corrugated metal pipe (CMP).

Monitoring Equipment

An experienced registered surveyor, retained by the Contractor to provide the datum readings, shall survey the elevation of the top of the settlement points. The surveyor shall provide suitable equipment capable of surveying settlement point elevations to an accuracy of +/- 2 mm or better.

Location

The Contractor shall install a total of 20 settlement rods at the locations shown on the Contract Drawings, as follows:

Instrument Number	Structure	Location	Approximate Offset from Centreline
SR1	EBL	5 m from N abutment	See drawing detail
SR2	EBL	15 m from N abutment	See drawing detail
SR3	EBL	30 m from N abutment	See drawing detail
SR4	EBL	50 m from N abutment	See drawing detail
SR5	EBL	75 m from N abutment	See drawing detail
SR6	EBL	5 m from S abutment	See drawing detail
SR7	EBL	15 m from S abutment	See drawing detail
SR8	EBL	30 m from S abutment	See drawing detail
SR9	EBL	50 m from S abutment	See drawing detail
SR10	EBL	75 m from S abutment	See drawing detail
SR11	WBL	5 m from N abutment	See drawing detail
SR12	WBL	15 m from N abutment	See drawing detail
SR13	WBL	30 m from N abutment	See drawing detail
SR14	WBL	50 m from N abutment	See drawing detail
SR15	WBL	75 m from N abutment	See drawing detail
SR16	WBL	5 m from S abutment	See drawing detail
SR17	WBL	15 m from S abutment	See drawing detail
SR18	WBL	30 m from S abutment	See drawing detail
SR19	WBL	50 m from S abutment	See drawing detail
SR20	WBL	75 m from S abutment	See drawing detail

Reporting

Installation Records

The Contractor shall record and report relevant installation details to the Contract Administrator. These include, but are not limited to:

- Settlement rod location, easting, northing;
- Elevation of top of rod;
- Distance between bottom of hole and top of rod;
- Dates of installation and datum readings;
- Installation notes / sketches;
- Description of settlement rod and backfill.

Monitoring Records

The party responsible for monitoring the settlement points shall record and report the readings to the Contract Administrator within 24 hours of completion of the survey. Each report shall include all survey data collected in tabular and graphical format as plots of time versus settlement in comparison to survey data collected prior to commencement of the work.

Installation

Rods

The settlement rod may be installed after stripping of topsoil or existing fill, and shall be placed prior to placement of any new embankment/preload fill.

Friction Reducing Sleeve

The friction reducing sleeve shall extend over the entire length of the rod, with a 25 to 50 mm gap between the bottom of the sleeve and the top of the concrete.

The settlement rod shall be in the centre of the sleeve.

Protective Surround

The sand or CMP protective surround shall be extended in 1.5 m increments with the rods.

Backfill

The annulus between the ground or CMP and the friction reducing sleeve shall be filled with sand to a level no higher than the top of the sleeve.

Installation Details

The elevation, easting and northing of the top of the rod shall be surveyed by the Contractor.

The total distance from the base of the rod to the top of the rod shall be measured and recorded to an accuracy of +/- 2 mm or better.

The contractor is responsible for preventing damage to the settlement rods during the embankment construction. If the rod assembly is damaged or the location/inclination of the point is altered during the filling or other construction activities, the rods and protective casing shall be replaced and surveyed before resuming the filling.

Monitoring

The settlement rods shall be monitored by a licensed surveyor, under the direction of the Contract Administrator. The Contractor shall meet with the Contract Administrator and staff responsible for the on-going monitoring immediately after installation of the instruments and before completing the embankment widening. This meeting is referred to as the “hand-over” meeting.

At the meeting, the Contractor shall hand over to the Contract Administrator all records pertaining to the installation of the instruments and all equipment to be supplied by the Contractor.

Monitoring by others for the baseline readings shall commence within two working days after the “hand over” meeting and prior to placing embankment widening fill. The monitoring shall continue on a schedule described herein throughout the completion of the embankment construction, and for approximately up to 12 months following the completion of construction or as dictated by the instrumentation readings.

Baseline Readings

Monitoring of the settlement point shall commence within seven (7) working days after the “hand over” meeting as described elsewhere in the Contract Documents.

Prior to the start of the embankment construction, a minimum of three baseline readings must be obtained. Anomalous readings which cannot be repeated are to be discarded and the average of the remaining readings used as a baseline datum.

Monitoring Frequency

Each settlement point shall be monitored at the following minimum frequencies:

PERIOD	MINIMUM FREQUENCY
During construction of the embankment widening	Daily
First month following end of filling	Twice per week
Up to 11 months after completion of embankment widening and grade raise	Every second week

Anomalous readings should be flagged, checked and discarded, if necessary. The reason for the anomalous reading should be identified and corrected, if possible. Damaged settlement rods shall be reported to the Contract Administrator.

The monitoring data should be reviewed and analysed monthly in order to assess the progression of settlement, and to determine if adjustment of the monitoring schedule or construction methodology or schedule is necessary.

Removal

After completion of the settlement monitoring period, the settlement points shall be removed. Removal shall extend to at least 0.3 m below the subgrade by excavating and cutting of the rods, sleeves, protective surround and surface casings. The voids resulting from the removal of the settlement rods should be backfilled with compacted granular.

Measurement of Payment

Measurement is by Plan Quantity, as may be revised by Adjusted Plan Quantity, of the number of settlement rods placed. The unit of measurement is each.

Basis of Payment

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material to do the work.

END OF SECTION

Deep Foundations - Item No.

Special Provision

Amendment to OPSS 903, April 2016

Obstructions During Piling

This special provision describes requirements for pile installation through obstructions and natural cobbles and boulders.

Definitions

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

Submission Requirements

The Contractor shall submit details for advancing the piles through obstructions, cobbles and boulders. The submittals shall satisfy the specifications and at a minimum contain specific information on their approach to advancing the piles in the event such conditions are encountered.

Pile Driving Through Obstacles, Cobbles, Boulders

The soils at the site are glacially-derived and are known to contain cobbles and boulders within the till deposits. The embankment fills at the site may also contain some obstructions. The Contractor is advised that appropriate equipment and construction procedures will be required to penetrate or remove obstructions, such as cobbles and boulders, to permit installation of deep foundations.

Basis of Payment

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

END OF SECTION



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