

**Foundation Investigation and
Design Report
South Saugeen River Bridge
Replacement, Highway 89,
West of Mount Forest, ON**

MTO Site No. 35-5

G.W.P 3093-12-00

Geocres No. 40P15-49



Prepared for:
Ministry of Transportation

Prepared by:
Stantec Consulting Ltd.
400 – 1331 Clyde Avenue
Ottawa, ON K2C 3G4

Project No. 165001038

January 2018

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1.0 INTRODUCTION

Stantec Consulting Ltd. (Stantec) was retained by the Ministry of Transportation of Ontario (MTO) to undertake the detailed design of the replacement of the existing Highway 89 Bridge over the South Saugeen River located approximately 4.5 km to the west of the community of Mount Forest in the Township of Wellington North, Ontario.

The project involves the replacement of the existing two-lane bridge with a wider and slighter longer structure. A Temporary Modular Bridge (TMB) is planned to be installed on the north side of the existing highway to maintain a single-lane of traffic to pass the site during replacement of the existing bridge.

The purpose of the foundation investigation was to assess the subsurface conditions at the site of the bridge replacement by drilling 6 boreholes to supplement existing borehole information, carrying out in-situ testing including down-hole geophysical testing, and completing a laboratory testing program on selected soil samples obtained from the boreholes.

This draft Foundation Investigation and Design Report (FIDR) has been prepared specifically and solely for the proposed bridge replacement project described above.

2.0 SITE DESCRIPTION AND GEOLOGY

2.1 SITE LOCATION

The site location is approximately 4.5 km to the west of the community of Mount Forest and is shown on the Key Plan inset to the Borehole Locations & Soil Strata Plans, Drawing Nos. 1 and 2 in Appendix A.

2.2 GENERAL SITE DESCRIPTION

At the bridge location, Highway 89 is a two-lane undivided highway with gravel shoulders. Highway 89 runs approximately in an east-west direction while the South Saugeen River has an approximate northwest-southeast orientation.

The existing South Saugeen River Bridge on Highway 89 is a single-span, pony truss bridge. The existing bridge, which is approximately 36.6 m long and 7.8 m wide, was constructed in 1953 and replaced a previous shorter span bridge at the same location. The available structural drawings

for the existing South Saugeen River Bridge indicate the bridge is supported on concrete abutments that have a U-shaped orientation (i.e. a front abutment wall with wingwalls extending back from the abutment parallel to the highway that are supported on a single foundation). The drawings indicate that pile foundations were to be used to support the abutments and wingwalls 'if necessary'.

The existing highway surface at the bridge site is at an elevation of just below 384 m while the original ground surface near the bridge was at an elevation of about 380 m to 381 m. A tableland area, which has ground surface elevations of about 8 m to 12 m higher than the flood plain is present to the west of the site. The site topography on the east side of the river is relatively flat. The adjacent lands and river banks are covered with trees and grasses.

2.3 PHYSIOGRAPHIC DESCRIPTION

The site is located in a physiographic region named the Teeswater Drumlin Field which consists of a drumlinized till plain. Based on the Ontario Geological Survey (OGS) surficial geology map, the surficial geology at the bridge site consists of alluvial deposits (silt, sand and gravel) within and adjacent to the river. Stratified, ice-contact drift deposits, consisting of sand, gravel and silt are present in the upland area west of the bridge site while a stony sandy silt till (Catfish Creek Till) and glaciofluvial outwash deposits are present to the east of the bridge site.

Based on the Ontario Geological Survey (OGS) bedrock geology map, the bedrock geology at the project site may consist of limestone, dolostone and/or gypsum of the Salina Formation. Nearby water well records encountered limestone and/or shale bedrock at depths varying from about 14 m to 40 m below ground surface.

3.0 INVESTIGATION PROCEDURES

3.1 REVIEW OF EXISTING INVESTIGATION

A preliminary foundation investigation was completed for the proposed South Saugeen River Bridge replacement by Golder Associates Ltd. (Golder) in 2013. The results of the preliminary investigation were presented in a report (GEOCREG reference number 40P15-46) titled "South Saugeen River Bridge Replacement, Site No. 35-5, Highway 89 Structure Replacements and Rehabilitation From 6.0 km West of Mount Forest to Shelburne, GWP 3035-11-00" and dated August 2013.

Foundation borehole records from the above noted report were reviewed as part of the current foundation investigation. Two boreholes, designated as BH1 and BH2, were drilled during the preliminary investigation. The borehole locations and a strata plot incorporating information from the previous investigation are shown on Drawing Nos. 1 and 2 (Borehole Location and Soil Strata Drawings) in Appendix A. Copies of borehole records from the previous investigation are included in Appendix B.

Review of the existing information from the previous investigation at the site indicates that the subsurface soils within the vicinity of the bridge replacement consist of surficial topsoil and fill materials underlain by a sand and gravel deposit. These near surface materials are underlain by interlayered glacial till, sand and gravel and silt layers.

3.2 FIELD INVESTIGATION

The current foundation investigation for the proposed bridge replacement consisted of advancing five (5) boreholes, designated as Boreholes BH17-1 to BH17-5, in the vicinity of the proposed Highway 89 Bridge and the TMB structures. These boreholes were drilled from April 10th to 17th, 2017.

Boreholes BH17-1 and BH17-2 were drilled near the east highway bridge abutment and approach embankments, respectively. Boreholes BH17-3 to BH17-5 were drilled in the areas of the proposed west highway and TMB bridge abutments and the western approach embankment. The locations of these boreholes are shown on the Borehole Locations & Soil Strata Plan, Drawing No. 1, in Appendix A.

The boreholes noted above were located based on preliminary design information available at the time of the investigation. Subsequent to the completion of the initial stage of drilling, the location of the TMB structure was shifted to the west. Due to this shift, Borehole 2 from the Golder investigation was located about 10 m east of the proposed location of the east abutment of the TMB. Therefore, an additional borehole designated BH17-7 was drilled at the design location for this abutment. BH17-7 was abandoned at a depth of 5.9 m, due to drilling equipment becoming stuck in the augers, and Borehole BH17-7B was drilled approximately 1 m to the north of BH17-7. These boreholes were drilled on November 30th, 2017.

Prior to carrying out the investigation, Stantec contacted public utility authorities to clear the borehole locations of both private and public utilities.

Drilling was carried out with truck-mounted and track-mounted drill rigs equipped for soil sampling. The boreholes were advanced using continuous flight hollow-stem augers.

The subsurface stratigraphy encountered in each borehole was recorded in the field by a member of Stantec's geotechnical engineering staff. Split spoon samples were collected at regularly spaced intervals (every 760 mm up to 6.5 m below existing ground surface, and every 1.5 m below a depth of 6.5 m). All samples recovered were returned to Stantec's Ottawa laboratory for detailed classification and testing.

A down-hole geophysical testing program was completed in Borehole BH17-1, which was located on the interior of the U-shaped abutment/wing-wall foundation on the east side of the bridge, to try to determine if piles are present beneath the existing abutment/wing-wall foundation. A 50 millimeter (mm) interior diameter PVC casing was installed in this borehole to facilitate the geophysical testing. Further details on the geophysical testing program are provided in Section 4.4.

A 50 mm diameter monitoring well was installed at the location of BH17-5. The monitoring well screen was located at a depth of approximately 4 m to 5.5 m below ground surface. This monitoring well was decommissioned on November 30th, 2017. The remaining boreholes were backfilled with soil cuttings and bentonite. Boreholes advanced on the highway were sealed with cold patch asphalt.

3.3 LOCATION AND ELEVATION SURVEY

The borehole locations and respective ground surface elevations were determined in the field by Stantec personnel with reference to existing site features. Table 3.1 below summarizes the borehole survey information and provides the borehole depths, termination elevations and number of samples collected.

Table 3.1: Borehole Information Summary

	Borehole Number						
	17-1	17-2	17-3	17-4	17-5	17-7	17-7B
MTM Zone 10 Coordinates Northing Easting	4871086 201383	4871088 201399	4871079 201334	4871077 201324	4871098 201337	4871099 201376	4871100 201375
Ground Surface Elevation, m	383.7	383.7	383.7	383.8	380.3	380.4	380.4
Total Depth Drilled, m	15.3	9.6	14.6	9.3	12.3	5.9	6.7
End of Borehole Elevation, m	368.4	374.1	369.1	374.5	368.0	374.5	373.8
Number of Soil Samples	16	11	16	11	13	8	4

3.4 LABORATORY TESTING

All samples were taken to Stantec's Ottawa laboratory and visually reviewed by a Geotechnical Engineer. The geotechnical laboratory testing program for the borehole samples is summarized in Table 3.2.

Table 3.2: Geotechnical Laboratory Testing Program

Test Description	Number of Tests
Moisture Content	75
Atterberg Limits	16 (includes 7 non-plastic results)
Grain Size Distribution (sieve & hydrometer)	22
Organic Content	2
pH, resistivity, soluble sulphate and chloride content tests*	2

*These tests were conducted by Paracel Laboratories in Ottawa. The remaining tests were conducted by Stantec's laboratory in Ottawa.

Samples remaining after testing will be placed in storage for a period of one year after issuance of the final report. After the storage period, the samples will be discarded unless we are directed otherwise by MTO.

4.0 SUBSURFACE CONDITIONS

4.1 OVERVIEW

The detailed subsurface soil and groundwater conditions encountered in the boreholes and the results of in situ and laboratory testing are displayed on the Record of Borehole sheets contained in Appendix B. An explanation of the symbols and terms used to describe the Borehole Records is also provided in Appendix B. Copies of the borehole records for the 2 boreholes drilled at the site by Golder (2013) are also included in Appendix B. The results of geotechnical laboratory testing are presented on Figures C1 to C8 contained in Appendix C.

A borehole location plan and a stratigraphic profile and cross sections of the soils encountered at the borehole locations at the bridge sites are provided on Drawing Nos. 1 and 2 in Appendix A. The stratigraphic boundaries on the borehole records and strata plots are inferred from non-continuous sampling and, therefore, represent transitions between soil types rather than exact boundaries between geological units. The subsoil conditions will vary between and beyond the borehole locations.

In general, the subsurface stratigraphy encountered in the boreholes advanced at this site consisted of asphalt and roadway embankment fill materials; buried topsoil and zones comprised predominantly of wood pieces were encountered immediately beneath and/or intermixed with the bottom of the fill materials in several of the boreholes. The above noted materials are underlain by alluvial deposits consisting predominantly of silt and sand, and then by layered till deposits of variable composition that contain interlayers of sandy silt and sand/sand and gravel.

Detailed descriptions of the subsurface soil layers are provided in the following subsections.

4.2 OVERBURDEN

4.2.1 Pavement Structure Fill

Pavement structure fill was encountered at the surface of the Boreholes BH17-1 to BH17-4 which were drilled from the existing Highway 89 surface. The pavement structure generally consisted of about 230 to 240 mm of asphalt concrete underlain by about 150 mm of sand and gravel (granular base) materials. The granular base materials were underlain by sand to sand and gravel/silty sand and gravel (sub-base) materials that extended to depths of about 1.2 m to 1.5 m below road surface corresponding to elevations of about 382.2 m to 382.5 m.

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Standard Penetration Test (SPT) N-values measured in the pavement structure fill varied from 10 to 28 blows per 0.3 m of penetration indicating the pavement fills are in a compact state.

Samples of the granular fill had moisture contents varying from approximately 3 to 9%. The results of a grain size distribution test conducted on a sample of the granular pavement structure fill materials are displayed on Figure C1 in Appendix C.

4.2.2 Fill including Highway Embankment Fill

Highway embankment fill was encountered under the pavement structure in Boreholes BH17-1 to BH17-4. Fill was also encountered in Borehole 17-7/17-7B and Golder Borehole 2 which were drilled on the north side of the highway to the east of the river. The fill was highly variable in composition varying from sand and gravel with some silt and pockets of clayey silt to sand to sandy clayey silt/silty clay with varying amounts of sand and gravel. The granular fill materials were encountered predominantly at the location of Borehole BH17-1 which was drilled on the interior of the U-shaped foundation for the east abutment and wing walls. Pockets of topsoil were encountered locally within the fill and topsoil layers and wood pieces were encountered at/near the base of the fill (refer to Section 4.2.3 for further details on these materials). Cobbles and boulders were also inferred to be present within the fill material based on drilling resistance.

The photograph below displays fill comprised of a mixture of clayey sand and sandy silt clay containing organic matter that was encountered at a depth of about 0.8 m to 1.3 m in Borehole BH17-7.



The fill at boreholes advanced through the highway embankment extended to depths ranging from about 3.0 m to 6.4 m below road surface corresponding to base of fill elevations of about 380.7 to 377.3 m. Fill materials encountered at Borehole BH17-7/17-7B extended to an elevation of about 378.3 m.

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SPT N-values measured in the fill varied from 1 to 30 blows per 0.3 m of penetration but were generally in the range of 6 to 11 blows. Based on the SPT N-values, the fill is considered to generally be loose to compact or firm to stiff.

Representative grain size distribution plots and a plasticity chart for the fill material are presented in Figures C2 to C4 in Appendix C, respectively. Index tests carried out on representative samples of the fill material yielded the following results:

	Sand and Gravel FILL	Sand Fill	Clayey Silt FILL
Gravel:	44 to 59%	8 to 26%	5 to 9%
Sand:	22 to 38%	51 to 83%	31 to 41%
Fines (silt & clay):	14 to 19%	9 to 23%	50 to 64%
Moisture Content:	3 to 13%	11 to 14%	10 to 37%
Liquid Limit:	20*	-	18 to 26
Plastic Limit:	16*	-	12 to 18

*Note – Atterberg Limit results representative of clayey silt inclusions with the sand and gravel fill.

The Unified Soil Classification System (USCS) group symbol for the embankment fill material varies from GM (silty gravel) to SP (sand) and CL/CL-ML (clayey silt).

4.2.3 Topsoil / Organic Materials

An approximately 0.6 m thick deposit of topsoil containing wood pieces was encountered at ground surface at BH17-5.

Buried topsoil, wood pieces and/or organic matter were frequently encountered either immediately below and/or incorporated into the bottom of the embankment fill in Boreholes BH17-1 to BH17-4. At some locations, the fill materials near the bottom of the embankment were comprised predominantly of wood (e.g. Sample 5 from BH17-3 – refer to photo of this sample below).



The organic materials (i.e. buried topsoil or fill materials containing wood/organic materials) had thicknesses that ranged from about 0.1 m to 1.5 m and were encountered elevations of approximately 377.2 m to 380.7 m.

SPT N-values measured in the topsoil and/or fill containing organic materials typically ranged from 3 to 7 blows per 0.3 m of penetration with higher N-values measured locally.

Moisture contents in the range of 21 to 38% were measured on samples of topsoil and/or embankment fill containing topsoil, wood pieces and organic matter. Laboratory testing conducted on samples BH17-2 Sample 4 and BH17-3 Sample 6 measured organic contents of 7.1% and 4.4%, respectively.

4.2.4 Alluvial Deposits

Alluvial deposits typically comprised of silty gravelly sand and sand/silty sand layers were encountered below the buried topsoil and organic fill in Boreholes BH17-1 to BH17-3. Deposits of gravelly sand to sand and gravel were also encountered beneath the fill in BH17-7/17-7B. A near-surface clayey silt layer was also encountered below the topsoil in BH17-5. These silty sand, sand, and clayey silt deposits contained varying amount of gravel as well as cobbles. Boulders are visible with the river channel near the bridge and are also expected to be present within the alluvial deposits. The thickness of the alluvial layers ranged from about 0.5 to 2.2 m with the base of these deposits encountered at elevations of about 379.1 to 375.8 m.

SPT N-values measured in the sand/silty sand and gravelly sand to sand and gravel layers ranged from 8 to 75 blows per 0.3 m indicating these materials are in a compact to very dense state. SPT N-values varying from 4 to 13 blows were measured in the clayey silt layer in BH17-5; however, it is noted that the higher N-value may have been influenced by gravel within the deposit. Based on the SPT N-values and manual examination of the samples, the clayey silt is considered to have a firm to stiff consistency.

Index tests carried out on representative samples from the granular alluvial layers yielded the following results:

	Silty Gravelly Sand To Sand
Gravel:	17 to 31%
Sand:	44 to 71%
Fines (silt & clay):	12 to 34%
Moisture Content:	6 to 21%

Representative grain size distribution plots for the silty sand to sand layer are presented in Figure C5 in Appendix C. The USCS group symbols for the soils are SM to SP.

The moisture content of samples of the clayey silt layer from Borehole BH17-5 varied from 20 to 21 percent.

4.2.5 Till Deposits

Till deposits of variable composition were encountered below the fill and alluvial deposits in all the boreholes drilled during the current investigation. The surface of the till deposits was encountered at elevations ranging from 375.8 to 379.1 m.

The till deposits can be broadly divided into the following two groups:

- Finer-grained till, varying in composition from clayey silt to sandy silt, that typically contains some gravel. Representative grain size distribution plots and a plasticity chart for samples of these portions of the till are displayed on Figures C6 and C7 in Appendix C; and
- Coarser-grained till varying in composition from gravelly silty sand and gravel to silty gravel. The results of grain size distribution testing of four samples of these till materials are displayed on Figure C8 in Appendix C.

Interlayered deposits of clayey silt till and silt/sandy silt till were encountered at Boreholes BH17-3 to BH17-5 which were located on the west side of the river. The till deposits encountered in Boreholes BH17-1, BH17-2 and BH17-7/17-7B, located on the east side of the river, consisted predominantly of the sandy silt to silty sand and gravel. A very dense, silty gravel till was also encountered near the bottom of Boreholes BH17-1, BH17-3 and BH17-5; this deposit was present on both sides of the river.

Cobbles and boulders were encountered at various locations within the till deposits during drilling and are expected to be present throughout the till at this site.

The SPT N-values measured within the till deposits ranged from 18 to 100 blows per 0.05 m. The lower SPT N-values were generally recorded within the top 1 to 3 m of the till deposits. Below that level, SPT N-values typically varied from 80 to greater than 100 blows. Based on the measured SPT N-values, the till deposits are considered to be very stiff to hard or compact to dense near the interface with the overlying materials and to become hard or very dense at depth.

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Index tests carried out on representative samples from the till deposits yielded the following results:

	Silty Gravel to Silty Sand and Gravel (Till)	Clayey Silt to Sandy Silt (Till)
Gravel:	36 to 43%	2 to 23%
Sand:	19 to 39%	5 to 30%
Fines (silt & clay):	18 to 39%	54 to 93%
Silt:	15 to 35%	44 to 57%
Clay:	3 to 5%	8 to 36%
Moisture Content:	5 to 18%	8 to 18%
Liquid Limit:	-	13 to 28
Plastic Limit:	-	11 to 17

The USCS group symbols for the till deposit vary from CL-ML/CL (clayey silt till), ML (sandy silt), SM (silty sand and gravel) to GM (silty gravel with sand).

Interlayers of sand and sandy silt were encountered within the till deposits at Boreholes BH17-1 and BH17-2, respectively. The sand deposit encountered in Borehole BH17-1 was at approximately the same elevation as a silty sand and gravel layer identified in BH1 and BH2 from the 2013 investigation by Golder. The results of grain size distribution testing of a sample of the sandy silt from Borehole BH17-2 is displayed on Figure C9. Laboratory testing indicated that this sample was non-plastic.

4.3 GROUNDWATER

Groundwater was monitored in all the boreholes during drilling from April 10 to April 17, 2017. The groundwater level in the monitoring well installed at BH17-5 was also measured on May 12th and November 30th, 2017. Information on the measured groundwater level depths and elevations is presented below in Table 4.2 and on the Borehole Records in Appendix B. *Water levels reported for Boreholes BH17-7 and BH17-7B were inferred during drilling on November 30th, 2017.

Table 4.1: Groundwater Levels (Stantec Boreholes)

Borehole No.	Ground Surface Elevation (m)	Approximate Groundwater Level					
		Inferred During Drilling (Apr 10-17, 2017)		May 12, 2017		November 30, 2017	
		Depth (m)	Elevation (m)	Depth (m)	Elevation (m)	Depth (m)	Elevation (m)
BH17-1	383.7	5.5	378.2	N/A	N/A	N/A	N/A
BH17-2	383.7	3.4	380.3	N/A	N/A	N/A	N/A
BH17-3	383.7	3.7	380.0	N/A	N/A	N/A	N/A
BH17-4	383.8	4.9	378.9	N/A	N/A	N/A	N/A
BH17-5	380.3	2.4	377.9	0.0	380.3	0.2	380.1
BH17-7	380.4	N/A	N/A	N/A	N/A	*1.2	*379.2
BH17-7B	380.4	N/A	N/A	N/A	N/A	*1.0	*379.4

The measured groundwater levels outlined above are generally consistent with water levels measured as part of the 2013 investigation by Golder where groundwater level elevations of between about 379 m and 381 m were reported.

Groundwater levels at the site will be subject to fluctuation due to seasonal changes, precipitation and snow melt events as well as variations in the water level in the South Saugeen River. The water levels should be expected to be higher during the spring season or during and following periods of heavy precipitation or snow melt.

4.4 GEOPHYSICAL TESTING

A note contained on the structural drawings for the existing Hwy 89 bridge over the South Saugeen River indicated that pile foundations were to be used to support the abutments and wingwalls 'if necessary'. As no additional information was available on the bridge construction, it is uncertain if piles were used to support the abutment foundations. Therefore, a down-hole geophysical testing program was completed in Borehole BH17-1, which was located on the interior of the U-shaped abutment/wing-wall foundation on the east side of the bridge, to try to determine if piles are present beneath the existing abutment/wing-wall foundation.

The down-hole geophysical investigation was completed by Geophysics GPR International Inc. on April 12, 2017, and consisted of a combination of seismic and magnetometer measurements within Borehole BH17-1. A report providing the methodology and results of the geophysical testing program as well as copy of the existing abutment structural drawings are included in Appendix D. The results and conclusions section of the report identified the following:

- The magnetic data set from the magnetometer survey identified three notable zones: An increasing horizontal magnetic response component from 0 to 5.5 m depth (interpreted to be a response to nearby metallic objects such as reinforced concrete), increasing total field and a peak in the horizontal magnetic response component from about 5.5 m to 7 m depth (interpreted to be a response to the presence of the existing reinforced concrete footing) and a stable magnetic field response below about 7.5 m (interpreted to be background magnetic field values). The report concluded that there does not appear to be steel in the vicinity of the borehole below a depth of about 7.5 m below grade (corresponding to an elevation of about 376.2 m).
- The seismic survey was used to measure the arrival of compression (P) waves transmitted from the surface to a vibration sensor at increasing depths within the borehole. This survey measured slower arrivals from 0 to 4 m depth, faster arrival times between 4 m and 8 m (interpreted to represent refracted arrivals through the concrete footings), and a relatively uniform velocity of about 2,500 m/s below a depth of 8 m. This velocity was described as being towards the upper limits for a dense till but below the typical range for timber piles.
- The seismic and magnetometer measurements do not suggest the presence of timber or steel piles in the immediate vicinity (i.e. within about 2 m to 3 m) of the borehole location.

4.5 CHEMICAL TESTING

Two (2) selected samples retrieved from BH17-1 and BH17-5 were submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of pH, water soluble sulphates and chloride concentrations, and resistivity. The analysis results are summarized in Table 4.3.

Table 4.2: Results of Chemical Analysis

Borehole No	Sample No.	Depth (m)	pH	Chloride (µg/g)	Sulphate (µg/g)	Resistivity (Ohm-m)
BH17-1	SS10	7.6 to 8.2	7.95	34	166	33.9
BH17-5	SS3	1.5 to 2.1	7.83	218	50	17.1

5.0 MISCELLANEOUS

The field work under consideration was carried out under the supervision of Mr. David Lee, P.Eng., under the direction of Kevin Nelson, P.Eng.

The utility locates for the boreholes advanced as part of the foundation investigation were carried out by Stantec personnel.

Truck and track-mounted drilling equipment was supplied and operated by London Soil Testing of London, Ontario.

Location and elevation survey of all the boreholes was carried out by Stantec during the course of drilling.

Traffic control service was provided by On Track Safety Ltd. of Thornhill, Ontario.

Geotechnical laboratory testing was carried out at Stantec's Ottawa laboratory. Chemical testing for pH, soluble sulphate, and chloride content, and resistivity was carried out by Paracel Laboratories of Ottawa.

This report was prepared by Abraham Mineheh, P.Eng., and reviewed by Kevin Nelson, P.Eng., and Raymond Haché, P.Eng., Designated Principal MTO Foundation Contact.

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6.0 CLOSURE

A subsurface investigation is a limited sampling of a site. The subsurface conditions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, we request that we be notified immediately in order to assess the additional information.

Respectfully Submitted;

STANTEC CONSULTING LTD.



Abraham Mineneh, PhD, P.Eng.
Geotechnical Engineer



Kevin Nelson, P.Eng.
Principal, Senior Geotechnical Engineer



Raymond Haché, M.Sc., P. Eng.
Designated Principal MTO Foundation Contact



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For
G.W.P 3093-12-00

Replacement of the South Saugeen River Bridge, Highway 89
West of Mount Forest, ON

7.0 DISCUSSIONS AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design input related to the proposed replacement of the Highway 89 bridge over the South Saugeen River. The recommendations provided herein are based on interpretation of the factual data obtained from subsurface investigations completed at this site.

The interpretation and recommendations provided in this report are intended solely to provide the designers with information to assess foundation options and carry out the design of the bridge foundations and embankments. As such, where comments are made regarding construction, the comments are provided only to highlight those aspects which could affect the design of the project. Those parties requiring information on aspects of construction should make their own interpretation of the factual information provided herein as it may affect equipment selection, proposed construction methods, scheduling and the like.

7.1 PROJECT DESCRIPTION AND BACKGROUND

7.1.1 Project Description

The project involves the replacement of the existing two-lane bridge carrying Highway 89 over the South Saugeen River. A Temporary Modular Bridge (TMB) is planned to be installed on the north side of the existing highway to maintain the ability of a single lane of traffic to pass the site during the replacement of the existing bridge.

7.1.2 Proposed Bridge Structures

A new single-span bridge structure is proposed to be constructed along approximately the same alignment as the existing bridge. The new bridge will be wider and slightly longer than the existing bridge. The available design information for the new structure indicates that the proposed bridge will consist of a 14 m wide structure with an approximate span length of 43 m. The approach embankments are planned to be widened and raised by about 1.5 m above the existing highway grade in the vicinity of the replacement structure. Consideration is being given to using integral abutments in order to minimize excavation below the water table.

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Key information associated with the proposed replacement bridge (based on the use of integral abutments) and existing bridge are as follows:

Existing Highway Grade Elevation at Bridge:	383.7 m
Approximate Proposed Highway Grade at Bridge:	385.2 m
Proposed Underside of Pile Cap Elevation:	380.0 m
Approximate Elevation of Underside of Existing Footings:	~376.5 m

The TMB is planned to be located approximately 4 m north of the replacement bridge and to have a length of about 39.6 m. The TMB and the approach embankments leading up to the TMB are planned to be approximately the same elevation as the existing Highway 89 grade.

7.1.3 Degree of Site and Prediction Model Understanding

The Canadian Highway Bridge Design Code (CHBDC) [December 2014] requires an assessment of the “degree of site and prediction model understanding” as a component of the geotechnical engineering investigation and/or services. The site and prediction model understanding includes the geotechnical properties on the site and the accuracy and degree of confidence regarding the numerical performance prediction models to be used to estimate the geotechnical serviceability limit states reactions and ultimate limit states resistances.

Based on the scope of subsurface investigations completed and available subsurface information related to this site, a “Typical Understanding” has generally been adopted for assessment purposes. A “High” degree of understanding has been adopted for assessment of embankment stability where slip surfaces develop through imported/manufactured granular fill materials. The consequence classification has been selected as “Typical Consequence” as per Section 6.5 of the Commentary on CSA S6-14, Canadian Highway Bridge Design Code (CHBDC), (S6, 1-14).

7.2 GEOTECHNICAL DESIGN PARAMETERS

The soil conditions encountered at this site generally consist of asphalt and roadway embankment fill materials that contain or are underlain by buried topsoil and zones comprised predominantly of wood pieces. These materials are underlain by alluvial deposits consisting predominantly of silt and sand, and then by layered till deposits of variable composition that contain interlayers of sandy silt and sand/sand and gravel.

For design purposes, the soil profiles identified in Tables 7.1 (East Side of River) and 7.2 (West Side of River) below and on Figures E1 to E2 in Appendix E can be used for the east and west abutments of the new bridge structures, respectively. The geotechnical parameters identified in the soil profiles were developed based on the synthesis of measured SPT ‘N’ values and laboratory index test results (including moisture contents) of soil samples obtained from all the current and previous boreholes advanced at the site.

Table 7.1: Representative Soil Profile at East Abutment

Elevation (m)		Soil Type	Design Parameters			
From	To		γ	$^4\phi$	4S_u	E
383.7	378.0	Embankment FILL: Silty sand and gravel to clayey silt; contains organic matter	22 to 23	30	-	15
378.0	376.0	Sandy Silt (TILL) or Sand/Silty Sand, Compact to Dense	22	32	-	30
376.0	374.5	Sandy Silt to Silty Sand and Gravel (TILL), Compact to Very Dense	23	35	-	30
374.5	373.5	Sand to Sand and Gravel, Very Dense	23	35	-	50
373.5	371.0	Sandy Silt (TILL), Very Dense	23	37	-	50
< 371.0		Silty Gravel (TILL)	23	40	-	150

Table 7.2: Representative Soil Profile at West Abutment

Elevation (m)		Soil Type	Design Parameters			
From	To		γ	$^4\phi$	4S_u	E
383.7	380.5	Embankment FILL: Clayey Silt to Silty Sand and Gravel	20	30	-	15
380.5	379.5	Topsoil or Organic Fill	18	28	-	10
379.5	378.0	Alluvial Deposits – Sand to Clayey Silt, Compact / Firm to Stiff	20.5	30	40	15
378.0	375.5	Clayey Silt (TILL), Very Stiff	23	33	125	35
375.5	373.0	Sandy Silt to Silty Sand and Gravel (TILL), Very Dense	24	37	-	75
373.0	370.0	Clayey Silt (TILL), Hard	23	35	200	50
< 370.0		Silty Gravel (TILL), Very Dense	22.5	40	-	150

Note: (1) γ = total unit weight (kN/m^3), ϕ = soil friction angle ($^\circ$), S_u = undrained shear strength (kPa), and E = soil modulus (MPa).

(2) Groundwater will be assumed to be at an Elevation of 380.5 m for design purposes. Submerged unit weights (γ') should be used below the groundwater level.

(3) Cobbles and boulders were present in the fill, and the alluvial and till deposits. These materials should be expected to present within all of these strata at this site.

(4) The friction angles are applicable to drained conditions only and the shear strengths are applicable to undrained conditions only

7.3 FROST PENETRATION

In accordance with OPSD 3090.101, the design frost penetration depth for foundations, f , at the site is 1.6 m. Therefore, all footings and pile caps should be provided with a minimum of 1.6 m of soil cover or equivalent insulation for protection against frost heaving.

This depth of frost penetration should also be considered in the design of frost tapers adjacent to the bridge abutment and retaining wall backfill zones.

7.4 SEISMIC CONDITIONS

7.4.1 Site Class

Based on subsurface information available from nearby water well records, it is inferred that the very dense or hard till deposits encountered during the current investigation either extend to depths in excess of 30 m below ground surface or are underlain directly by bedrock at depths less than 30 m.

Based on these subsurface conditions, it is recommended that Site Class C as defined in Section 4.4.3 of the CHBDC (2014) be used in the seismic design.

7.4.2 Peak Ground Acceleration (PGA)

Seismic hazard values for this site were obtained from Natural Resources Canada (2015 National Building Code). The 2015 NBC Seismic Hazard calculation sheet for this site is provided in Appendix F. Table 7.3 summarizes the parameters based on a 2475-year return period to be used in forced based design.

Table 7.3: Peak Ground Acceleration Data

<i>PGA</i>	<i>S_a(0.2)</i>	<i>PGA_{ref}</i>	Site Class	Site Adjusted <i>PGA</i>
0.053g	0.091g	0.0424g	C	0.053g

7.4.3 Liquefaction Potential

The potential for soil liquefaction was evaluated by comparing the cyclic stress ratio (CSR) caused by the design earthquake with the soil resistance expressed in terms of the cyclic resistance ratio (CRR). The evaluation follows the analysis methodology suggested by Idriss and Boulanger (2008) and is based on the following:

- The blow count data from boreholes.
- A Site Adjusted PGA of 0.053g.
- An earthquake magnitude M_w of 6.55.

The analysis indicates a factor of safety against liquefaction of 2.0 or greater, and therefore liquefaction is not a concern at this site.

7.5 BRIDGE REPLACEMENT FOUNDATION OPTIONS

Both shallow and deep foundation options were evaluated for the proposed replacement bridge structure.

Steel H-pile foundations can be used as part of an integral abutment foundation system. The piles would be driven to practical refusal in the very dense, granular till deposits and would develop most of their load carrying capacity from tip resistance/end-bearing. Where integral abutments are adopted, the upper portion of the piles would need to be installed within sand-

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filled, corrugated steel pipe (CSP) liners to provide suitable flexibility of the steel H-piles. Pre-drilling and/or coring through the existing bridge foundations would be required both to permit installation of the piles and corrugated steel pipe (CSP) liners through the footing and to obtain sufficient pile embedment below the CSP liners.

Shallow foundations, if used, would require removal of the existing bridge footings as well as all existing fill materials in the foundation backfill areas inside of the existing U-shaped footings and would be founded on/within either the very stiff to hard clayey silt till, very dense till or dense alluvial soils. Excavations for removal of the fill and footings would extend to depths of up to about 6 m to 7 m below the existing highway grade and several meters below the water table/river level. These excavations would require extensive temporary protections systems adjacent to the TMB abutments and associated 'detour' highway embankments. In addition, a groundwater cut-off system, such as a sheet pile enclosure, would be required to reduce groundwater inflows through near-surface granular deposits that are hydraulically connected to the river channel. Difficulties may be encountered advancing sheet piles into the underlying very dense till.

The use of drilled piers/caisson foundations is not considered practical or cost-effective at this site given the anticipated difficulties and mitigation measures required for installation of the caissons through the existing footings and into interlayered cohesive and cohesionless till deposits below the water table.

Table 7.4 presents the advantages, disadvantages, relative costs and risks/consequences for various foundation options for the Highway 89 replacement bridge.

Table 7.4: Comparison of Foundation Options for Hwy 89 Bridge Structure

.Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
H-Piles Driven to Practical Refusal in Very Dense Till	<ul style="list-style-type: none"> Allows for use of Integral Abutments Reduced differential settlement Reduces depth of excavations and requirements for temporary support systems. Allows for construction work to be completed primarily above the water level. More suitable than pipe piles for difficult driving conditions 	<ul style="list-style-type: none"> Existing bridge foundations will obstruct installation – specialized methods required for installation of piles through footings Pre-drilling will also be required to advance through till to provide sufficient pile lengths for integral abutments Pile capacity may not be fully utilized due to difficult driving conditions leading to limited pile lengths/shaft friction. 	Medium	<ul style="list-style-type: none"> Difficulties creating holes through existing foundations for piles and CSP liners Pile damage during installation Shallow refusal of piles on cobbles and boulders requires pre-drilling
Shallow foundations - Founded on Till or Dense Alluvial soils	<ul style="list-style-type: none"> Excavation and drilling through difficult deposits avoided Existing foundations are fully exposed to allow for removal. Lower foundation costs than deep foundations; however, the costs for 	<ul style="list-style-type: none"> Founding subgrade conditions are variable increasing potential for differential settlement Requires excavations below water table in granular soils. Groundwater cut-off (e.g. sheet piles) and dewatering systems required. 	Low to medium	<ul style="list-style-type: none"> Differential settlement due to varying subgrade conditions Inability to advance sheet piles into low permeability units due to obstructions leads to large groundwater inflows into excavations

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.Option	Advantages	Disadvantages	Relative Cost	Risks/Consequences
(Semi - Integral Abutment)	groundwater control and temporary protection measures would be higher	<ul style="list-style-type: none"> Higher / more robust temporary support systems needed adjacent to TMB abutments & embankments Not suitable for integral abutments (Semi-integral Abutments possible) 		and disturbance of subgrade soils <ul style="list-style-type: none"> Excessive movements of high temporary support systems.
Drilled Caissons	<ul style="list-style-type: none"> Can transmit very large axial and lateral loads Shorter construction time than shallow foundations 	<ul style="list-style-type: none"> Existing bridge foundations will obstruct installation Difficult to drill and advance liners through interlayered till deposits containing boulders and cobbles Not suitable for integral bridge abutment Requires use of drilling mud to balance water pressures; cannot be visually inspected 	High	<ul style="list-style-type: none"> Existing footings would pose obstruction to caissons. Coring through, or removal of, existing footing required. Installation of drilled piers through saturated granular soils could result in soil disturbance and collapse of sidewalls. Liners and/or drilling mud required to mitigate groundwater issues. Installation of liners may not be practical without specialized equipment.
Piles end-bearing on bedrock	<ul style="list-style-type: none"> High geotechnical resistances Negligible settlement 	<ul style="list-style-type: none"> Excessive pile lengths Pre-drilling would be required throughout till to advance piles to bedrock 	High	<ul style="list-style-type: none"> Bedrock depth is not known but could be up to about 40 m at this site Potential for damage to piles during installation

Based on the comparison presented above in Table 7.4, the preferred option from a geotechnical/foundations perspective is to support the abutments for the bridge replacement on steel H-piles that are driven to practical refusal within the till deposits at the site.

7.6 FOUNDATION RECOMMENDATIONS

The design recommendations presented in the following sections have been developed in accordance with the requirements and methods described in the Canadian Highway Bridge Design Code (CHBDC, 2014).

7.6.1 Driven Pile Foundations – Bridge Replacement

7.6.1.1 Design Considerations

Pile foundations consisting of steel H-piles that are driven to effective refusal in the very dense/hard till deposits, and that derive the majority of their capacity from end-bearing, can be used to support the integral abutments of the proposed replacement bridge. Pipe piles are considered to have a higher risk than H-piles for “hanging up” or being deflected away from their design orientation due to the presence of cobbles and/or boulders within till deposits. Therefore, H-piles are recommended for use at this site.

The use of integral abutments requires relatively flexible piles that extend at least 5 m below the pile cap level. Based on the preliminary GA drawing, the underside of the pile caps at the abutments will be located at elevations of approximately 380 m. As previously indicated, pre-drilling and/or coring would be required to create holes through the existing bridge footings to permit installation of the piles and CSP liners for the integral abutments. Given the variability in both the composition and consistency/density of the till deposits, the depth of pile penetration into the till is expected to vary considerably. Piles are not expected to have to be driven beyond an elevation of about 369 m to 370 m to encounter effective refusal resulting in maximum pile lengths of about 11 m below the pile cap level. However, effective refusal could also be encountered at shallower depth within the very dense portions of the till deposits particularly if cobbles and/or boulders are encountered. Therefore, predrilling is also recommended to be carried out down to an elevation of 375 m (i.e. to a level of 5 m below the pile cap) at all pile locations in order to obtain sufficient pile embedment/satisfy the minimum pile length requirements of 5 m. The tips of all piles are recommended to be driven to an elevation of 373 m or lower. If the predrilling method results in removal of soil in the predrilled zone (rather than the existing soils being left in place), the section of the pre-drilled holes below the bottom of the CSP liners should be filled with sand prior to pile installation.

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS 903 – Construction Specification for Deep Foundations.

7.6.1.2 Geotechnical Axial Resistance

A factored geotechnical resistance at Ultimate Limit States (ULS) of 1,500 kN may be used in the design of HP 310x110 piles driven to practical refusal within the very dense glacial till. This value includes a resistance factor of 0.4 applied to the ultimate capacity.

The estimated geotechnical reaction at SLS (factored) for 25 mm of vertical settlement for a HP 310x110 pile driven to effective refusal in the till exceeds the geotechnical reaction at ULS (factored). Therefore, the ULS (factored) resistance will govern.

Axial Resistance in Tension

For design against uplift, the tensile resistance provided in Table 7.5 is recommended. This value is based on a minimum pile length of 7 m.

Table 7.5: Recommended Tensile Pile Resistance

Pile Type	Minimum Pile Length(m)	Factored Geotechnical Resistance (Tension) at ULS_f (kN)
HP 310 x 110	7	150

A resistance factor, ϕ_{gu} , of 0.3 has been applied to calculate the ULS_f resistance. The factored geotechnical resistance (tension) at ULS_f provided above does not include the self-weight of the pile.

Downdrag

The proposed bridge replacement structure will be constructed in the same location as the existing bridge. The proposed grade raise in the vicinity of the new underpass is typically less than about 1.5 m above existing site grades. In addition, the native site soils at the abutment locations consist predominantly of very stiff to hard clayey silt till or dense to very dense granular fill with interbeds of compact to very dense granular soils. Based on these conditions, the piles are not anticipated to be subjected to significant downdrag loads.

Relaxation of Piles

Relaxation and reduction of pile capacity will not be of concern for H-piles that are driven to the refusal within the very dense till deposits.

However, the site soils include deposits/interlayers of very dense silt/sandy silt. The driving of piles into these soils can result in the generation of reduced (negative) pore water pressure and increased effective stresses due to soil dilation. These effects can lead to an apparent temporary increase in driving resistance and strength leading to a 'false set'. After driving has ended, the reduced pore pressures stabilize with time causing a reduction in the bearing capacity and the energy required to advance the pile. Due to these conditions, retapping of all piles should be completed a minimum of 24 hours after the end of initial driving to confirm that the design pile capacities have been achieved.

Pile Driving and Capacity Testing Considerations

Piles should be supplied and installed/constructed in accordance with the requirements of OPSS 903 – Construction Specification for Deep Foundations.

The site soils generally consist of embankment fill over near-surface alluvial deposits over compact to very dense or very stiff to hard till containing interbeds of compact to very dense granular soils. Cobbles and boulders were observed within the fill and throughout the native overburden soils. Pre-drilling should be carried out to the elevations described previously and the piles should be provided with driving shoes such as Titus "H" Bearing Pile Point (Standard Model) or equivalent to facilitate penetration into/through the dense overburden and reduce the potential for damage to the piles during driving.

The following pile notes should be included in the "Pile Data Table":

- The pile driving equipment shall be appropriate to the driving conditions and capable of delivering a minimum specified hammer energy of 60,000 J.

The following "Pile Driving Note" should be included:

- Piles to be driven in accordance with MTO Standard Drawing SS 103-11 using an ultimate geotechnical resistance 3,000 kN per pile but must be driven below elevation 373 m Geodetic.

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The capacity of each pile should be verified in the field by the use of the Hiley Formula (MTO Standard Structural Drawing SS-103-11) to confirm that the specified ultimate capacity is achieved. In addition, high-strain dynamic testing (i.e. Pile Driving Analyzer (PDA) testing) is to be carried out on 2 piles per abutment. The PDA testing is to be carried out at the end of initial drive (EOID). A Non-Standard Special Provision (NSSP) should be included with the contract documents that notifies the contractor of the requirements for the high-strain dynamic / PDA testing. A sample NSSP has been included in Appendix G.

Driven piles generally gain capacity after driving has been completed and excess pore pressures have dissipated. If the specified ultimate resistance is not achieved during the initial round of testing using the Hiley Formula and PDA methods, retesting should be completed on the same piles after a minimum of 3 days to allow for soil setup to occur.

The "Hiley Formula Pile Resistance" and PDA test results for all piles shall be submitted to the project foundation engineer for comparison. In the event of a discrepancy between the calculated results, the capacities assessed using the Hiley Formula will govern.

7.6.1.3 Geotechnical Lateral Resistance

The geotechnical resistance of the pile against lateral loads is mobilized due to the passive resistance of the surrounding soil. The lateral capacity of 310x110 H-piles was evaluated using the program called LPILE 2016 developed by Ensoft, Inc. (Ensoft, 2016). The input parameters that were used in the analyses for the piles at the east and west abutments are displayed in Tables 7.1 and 7.2, respectively. The pile was modelled with a pile head elevation of 380 m and a total length of 11 m. The moment of inertia of the pile section was calculated to be $77 \times 10^6 \text{ mm}^4$ along the weak axis and $233 \times 10^6 \text{ mm}^4$ along the strong axis. A modulus of elasticity of 200 GPa was used for the steel pile.

Geotechnical lateral resistances at ULS (factored) and SLS (factored) are presented in Table 7.6 below. Similar results were obtained for piles at both abutment locations.

Table 7.6: Geotechnical Lateral Resistance at ULS and SLS – East Abutment

Pile Type	Axis	Pile Head Condition	Factored Resistance at ULS (kN)	Factored SLS Geotechnical Resistance (kN) 10 mm	Factored SLS Geotechnical Resistance (kN) 25 mm
HP310x110	Weak Axis	Free Rotation	85	80	110
HP310x110	Weak Axis	Zero Rotation	140	165	205
HP310x110	Strong Axis	Free Rotation	170	110	165
HP310x110	Strong Axis	Zero Rotation	270	240	360

Notes:

- (1) The factored resistances at ULS were determined using a resistance factor for lateral resistance of 0.5 (Table 6.2 of CHBDC, 2014) and yield moments of 222 kN-m and 606 kN-m for the weak and strong axes of the piles, respectively.

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- (2) SLS geotechnical resistances correspond to displacements of 10 mm and 25 mm and include a resistance factor of 0.8 (Table 6.2 of CHBDC, 2014).

The LPILE results are presented on Figures E3 to E6 and Tables E-1 & E-2.

Figures E3 and E4 present the deformed shapes of the piles for fixed head and free head pile head boundary conditions for lateral deflections of 10 mm, 25 mm and at the unfactored ultimate lateral loading condition.

Figures E5 and E6 present the p-y plots that give the non-linear response of the pile-soil interaction. The p-y curve values versus depth are summarized on Tables E-1 and E-2. These tables provide a series of curves obtained from the LPILE program generated for selected depths below the pile head. The p-y curves can be used in the structural evaluation of the H-piles.

Group Action

Group action of piles (pile interaction) for lateral loading should be considered if centerline spacing of piles is less than 7 pile diameters/widths parallel to the direction of lateral load. Group action reductions would not apply to loads perpendicular to the piles or to the leading pile in a row (load parallel to direction of line of piles) for pile spacings of 4 pile diameters/widths or more.

The effect of interaction between piles can be considered by applying a reduction factor to the soil resistance (p-multiplier). The following reduction factors may be used with ULS and SLS values, as well as with p-y curves to account for pile group action:

Table 7.7: Recommended Reduction Factors for Trailing Piles in a Row

Ratio of pile spacing to pile diameter/width	Reduction Factor (p-multiplier)
Load Parallel to Pile Spacing	
7	1.0
4	0.8
3	0.7
2	0.6

7.6.2 Foundations – Temporary Modular Bridge

7.6.2.1 Temporary Modular Bridge Foundation Options

The following subsurface conditions were encountered at the boreholes advanced from the floodplain level near the abutments of the proposed TMB structure:

- West Abutment - Near surface deposits of topsoil and sand and gravel overlying a firm to stiff clayey silt deposit that was encountered to an elevation of approximately 378 m. The near-surface deposits are underlain by an approximately 1.5 m thick layer of very stiff to hard clayey silt till and then by dense to very dense or hard fill deposits.
- At the location of the west abutment of the TMB, the existing approach embankment for the current bridge is wider than at other locations in order to support a small water monitoring station building. Therefore, several meters of fill are present above these soils at this location.

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- East Abutment - Surficial fill materials were encountered to depths of about 2.1 m at the locations of Boreholes 17-7 and 17-7/B which were drilled in the footprint of the proposed east abutment. The fill materials varied in composition from topsoil to a mixture of silty sand/clayey sand and sandy clay, were typically brown to black in colour and contained organic matter. The fill materials extended below the measured water level. SPT N-values of between 1 and 4 (typically 1 and 2) were measured within the uppermost of 1.5 m of the fill materials suggesting these materials are soft to very soft or very loose. The fill materials are underlain by deposits of loose to compact, gravelly sand/sand and gravel and then by dense to very dense granular fill varying in composition from sandy silt to silty sand and gravel.

Based on the results of the foundation investigation, the soils present in the areas of the abutments for the proposed TMB are highly variable in composition. Very soft to soft or very loose fill materials extending below the water/river level were encountered at the east abutment location while topsoil and firm clayey soils were encountered at the west abutment. Boreholes drilled from the highway surface adjacent to the TMB abutments also encountered existing embankment fill materials that were variable in composition and contained zones of wood and organic matter. In addition, grubbing materials are known to have been placed in the toe of embankment areas at some MTO bridge sites and may have been incorporated into the embankments at this site.

Based on the conditions noted above, the use of driven pile foundations is recommended over the use of shallow foundations to support the TMB abutments for the following reasons:

- Shallow foundations should not be founded within or over top of the existing fill materials due to their inherent variability and very soft/loose nature and due to the presence of organic matter in the fill. Similarly, the near-surface, soft native cohesive soils encountered at the west abutment locations will compress and consolidate when subjected to new loads, which could lead to unacceptable settlement and performance of shallow TMB abutment foundations.
- Consideration could be given to removing the fill materials and/or soft/loose soils to expose the underlying competent, native till and/or the compact to very dense granular soil deposits and then supporting the TMB on shallow spread footings bearing either on these native materials or perched within pads of granular engineered fill constructed above these soils. However, the fill and unsuitable materials extended below the water/river level and the abutments are located close to the river. Therefore, the removal of the unsuitable materials would require the installation of a perimeter cut-off system, such as a sheet pile enclosure, around the excavation areas in conjunction with extensive internal dewatering within the excavation (where granular soils are present at the base of the excavation) which would be expected to require a Permit to Take Water from the Ministry of the Environment and Climate Change (MOECC). Difficulties installing a water-tight sheet pile enclosure may also be encountered due to obstructions, including cobbles and boulders, within the fill and underlying native soils.
- A pile cap for driven piles can be constructed above the river water level thereby reducing risks and costs associated with excavations for removal of unsuitable materials below the river water level.

Based on the above, it is recommended that the TMB abutment foundations be supported on driven steel 310x110 H-piles. The structural design team has indicated that if pile foundations are used, each abutment would be supported on 4 piles (2 under each truss) and that each pile would be required to support factored loads of approximately 350 kN at SLS and 500 kN at ULS. The design input provided in Section 7.6.1 can be used in the design of the piles for the TMB.

7.7 LATERAL EARTH PRESSURES

7.7.1 Abutment Backfill

Ontario Provincial Standard Drawing (OPSD) 3101.150 outlines the required extent of the granular backfill zone at the bridge abutments. The materials used as backfill behind the proposed bridge abutments should consist of free-draining granular fill placed and compacted using methods and equipment appropriate to the type of structure. For the purpose of this report, it is assumed that backfill materials meeting the requirements of OPSS Granular B (Type I or Type II) or Granular A materials will be used.

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures. Backfill materials should be placed and compacted in accordance with the requirements of OPSS.PROV 206 and OPSS.PROV 501, respectively.

7.7.2 Static Lateral Earth Pressures

Static lateral earth pressures will need to be considered in the design of abutments, retaining walls (wingwalls) and retained soil systems (if any).

Computation of earth pressures should be in accordance with Section 6.17.3 of the CHBDC. For retaining walls that are designed to allow rotation, active earth pressure may be used for design. For rigidly tied and unyielding structures, the at-rest earth pressure should be used for design. The effects of compaction should be accounted for by applying a compaction surcharge as shown in Figure 6.6 of the CHBDC.

The total at rest, (P_O) active (P_A) and passive (P_P) thrusts can be calculated using the following equations:

$$P_O = \frac{1}{2} K_O \gamma H^2$$

$$P_A = \frac{1}{2} K_A \gamma H^2$$

$$P_P = \frac{1}{2} K_P \gamma H^2$$

where H is the height of the wall and γ is the unit weight of the backfill soil. Values for K_A , K_P , K_O and γ are provided in Tables 7.9 and 7.10 for horizontal and sloping (2H:1V) backfill conditions, respectively. The thrust acts at a point one third up the height of the wall.

Table 7.8: Recommended Static Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, γ (kN/m ³)	21	22	22
Effective Friction Angle	32°	35°	30°
Coefficient of Earth Pressure at Rest (K_o)	0.47	0.43	0.50
Coefficient of Active Earth Pressure (K_a)	0.31	0.27	0.33
Coefficient of Passive Earth Pressure (K_p)	3.25	3.7	3.0

Table 7.9: Recommended Static Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, γ (kN/m ³)	21	22	22
Effective Friction Angle	32°	35°	30°
Coefficient of Earth Pressure at Rest (K_o)	0.68	0.62	0.72
Coefficient of Active Earth Pressure (K_a)	0.47	0.39	0.54

7.7.3 Seismic Lateral Earth Pressures

The following design parameters are provided for use in assessing the earth pressures induced on the bridge abutment and wingwalls under seismic loading conditions under seismic loading conditions. The seismic earth pressures may be calculated using the parameters detailed in Tables 7.12 and 7.13 for horizontal and 2H:1V backfills, respectively.

The total active and passive thrusts under seismic loading conditions can be calculated using the following equations:

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v)$$

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

where:

- K_{AE} = active earth pressure coefficient (combined static and seismic)
- K_{PE} = passive earth pressure coefficient (combined static and seismic)
- H = height of wall
- k_h = horizontal acceleration coefficient
- k_v = vertical acceleration coefficient
- γ = total unit weight

For this site, the following design parameters were used to develop the recommended K_{AE} and K_{PE} values as per CHBDC 2014.

Table 7.10: Seismic Design Parameters to Estimate Lateral Earth Pressures

Site Adjusted PGA	Horizontal Acceleration Coefficient, k_{ho}	Horizontal Acceleration Coefficient, k_h
	Non-Yielding	Yielding (<i>wall movements of 25 mm to 50 mm</i>)
0.053g	0.053	0.0265

Note: k_{ho} is the seismic horizontal acceleration coefficient that corresponds to zero wall movement and is equal to the site-adjusted **PGA** estimated at ground surface. The vertical acceleration coefficient (k_v) should be ignored in the calculations as per CHBDC 2014, section C4.6.5.

The angle of friction between the soil and the wall has been set at 0° to provide a conservative estimate.

Table 7.11: Recommended Seismic Earth Pressure Parameters (Horizontal Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, γ (kN/m ³)	21	22	22
Effective Friction Angle	32°	35°	30°
Passive Earth Pressure, (K_{PE})	3.21	3.64	2.95
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.329	0.330	0.329
Yielding Wall			
Active Earth Pressure (K_{AE}) for Yielding Wall	0.32	0.29	0.35
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.346	0.347	0.345
Non-Yielding Wall			
Active Earth Pressure (K_{AE}) for Non-Yielding Wall	0.34	0.30	0.37
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Non-Yielding Wall	0.358	0.359	0.357

Table 7.12: Recommended Seismic Earth Pressure Parameters (2H:1V Backfill)

Parameter	OPSS Gran B Type I	OPSS Gran A and Gran B Type II	Existing Embankment Fill
Bulk Unit Weight, γ (kN/m ³)	21	22	22
Effective Friction Angle	32°	35°	30°
Passive Earth Pressure, (K_{PE})	8.55	10.75	7.41
Height of Application of P_{PE} from base as a ratio of wall height, (H)	0.332	0.332	0.331
Yielding Wall			
Active Earth Pressure (K_{AE}) for Yielding Wall	0.51	0.43	0.60
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Yielding Wall	0.357	0.354	0.361
Non-Yielding Wall			
Active Earth Pressure (K_{AE}) for Non-Yielding Wall	0.57	0.46	0.70
Height of Application of P_{AE} from base as a ratio of wall height, (H) for Non-Yielding Wall	0.381	0.375	0.397

7.8 EMBANKMENT DESIGN CONSIDERATIONS

The existing approach embankments are proposed to be raised by approximately 1.5 m with associated widening on both sides of the embankment. New, temporary embankments will also be built on the north side of the existing highway to serve as approach embankments for the TMB. Typically, all new or widened embankments will have maximum embankment heights of about 4 m to 5 m or less and embankment sideslopes of 2H:1V or flatter.

All new/widened embankments are recommended to be constructed using granular fill materials meeting the requirements of OPSS Granular B Type 1 or Type 2 materials or Select Subgrade materials. Any embankment widening associated with the grade raise should be carried out in accordance with OPSD 208.010 Benching of Earth Slopes.

Buried topsoil layers and/or zones of organic materials (e.g. wood pieces) were encountered within or beneath the existing embankment fill materials at several of the borehole locations. No signs of substantial differential settlement (e.g. dips in the highway surface leading up to the bridge, buried or exceptionally thick asphalt layers indicative of previous padding activities etc.) were noted between the bridge structure and approach embankments during the foundation investigation. Ongoing settlements of these topsoil and organic soils/materials associated with the existing embankment loads, which are understood to have been in place for over 60 years, are also expected to be nominal. Given that the organic materials below the embankments are present in relatively thin layers and typically consist of wood pieces that are less prone to consolidation than other organic materials (e.g. peat, organic silt etc.), such settlements are not expected to have significant effects on the long-term performance of the highway. However,

some differential settlement may occur, due to initial compression of the organic materials resulting from raising of the embankments as well as ongoing creep associated with decay of the organic materials, in the approach slab areas which are located partially over granular, abutment backfill materials and partially over existing embankment fill containing/underlain by organic soils. The approach slabs to bridge are understood to be designed to accommodate settlement and, as such, consideration can be given to leaving the existing fill and organic soils in place if some long-term settlement, and the associated potential need for increased maintenance such as asphalt padding, is tolerable in these areas.

If settlement of the approach slabs are not acceptable, all existing organic soils located beneath the highway surface within the zone of influence of the approach slabs (i.e., within a zone extending back 10 m from the abutment walls) should be subexcavated and replaced with engineered fill to limit the potential for such differential settlement to occur. Based on the subsurface conditions present at Boreholes BH17-2 to BH17-4, the base of the organic materials requiring subexcavation is expected to vary from approximately 379 m on the west abutment to 380.5 m at the east abutment.

7.8.1 Embankment Settlements

Settlement of the underlying soil due to the raising and widening of the embankments was evaluated. The following assumptions were made in evaluating the settlement of the site soil under the proposed embankment:

- The typical soil profile at the west abutment given in Table 7.1 was incorporated into the settlement analyses. Settlements on the east side of the bridge are expected to be less.
- The maximum height of the embankment grade raise is limited to about 1.5 m.
- The new embankment platform, estimated to have a crest-to-crest width of about 14 m, has been assumed to be centered over the existing embankment platform.
- The load from the bridge abutments will be transferred to deeper and more competent strata by the piles and hence will not contribute to the settlement of the site soil.
- The estimated preconsolidation pressures of the clayey till deposits are expected to be higher than the anticipated post-construction stresses in these deposits. Therefore, substantial consolidation settlements of the cohesive native soils are not expected to occur and only immediate (elastic) settlement was considered in the analyses.

Evaluation of soil settlements due to the new embankment loading was carried out using the commercial program Settle3D (Rocscience 2009). The results of the analyses indicate that for the conditions presented herein, the maximum total vertical settlement of the native soils beneath the approach embankments leading up to the new bridge is expected to be less than 25 mm due to the additional loading imposed by the proposed grade raise of 1.5 m in these areas. Settlements of less than 15 mm are expected at the abutment locations. These settlements are anticipated to take place relatively rapidly and to be predominantly completed during construction of the embankments.

Additional analyses were carried out to estimate the magnitude of settlement that would occur beneath the temporary approach embankments to the TMB. Based on the analysis results,

indicate that the maximum total vertical settlement of the native soils is estimated is also expected to be less than 20 mm.

Self-weight settlement due to compression of this embankment fill during the construction process is expected to be less than 20 mm (0.5 % strain). More than half of this settlement is expected to be completed almost immediately after the fill has achieved its full height.

7.8.2 Slope Stability Evaluation

A slope stability evaluation was carried out for a critical cross-section of the new roadway embankment near the west abutment (i.e. a section corresponding to the greatest embankment height and weakest subgrade soils) using a commercial program Slope/W (Geo-Slope, 2010). The analyses included allowance for dynamic loading due to traffic by considering a static surcharge load equivalent to 0.8 m of additional fill as per Section 6.9.5 of the CHBDC.

Minimum factors of safety under static conditions of 1.4 (corresponding to a ϕ_{gu} of 0.7) and 1.25 (corresponding to a ϕ_{gu} of 0.8) are considered acceptable for permanent and temporary embankments, respectively, for slip surfaces extending entirely through portions of the embankments constructed out of imported granular fill materials based on the 'High' degree of understanding of these materials. A minimum factor of safety of 1.5 is considered acceptable against deeper-seated failure surfaces under static conditions.

The results of the slope stability analysis for static, drained conditions for the temporary and permanent embankment geometries are provided in Figures E7 and E8 in Appendix E. The results of these stability analyses indicate that for the proposed embankments, with a slope angle of 2H:1V, would provide a factor of safety against instability of 1.4 for a shallow critical failure surface extending through granular fill under static conditions. A factor of safety of greater than 1.1 is also calculated under seismic conditions for the embankments. Stability analyses carried out using undrained parameters provided similar or higher factors of safety.

Given the higher resistance factor (lower factor of safety) that can be applied for temporary embankments, additional analyses were completed for a temporary embankment configuration incorporating a 1.5H:1V sideslope. The results of this analysis, which are presented in Figure E9 in Appendix E, indicated that the factor of safety against instability for this embankment configuration is approximately 1.1 which is less than the required factor of safety for temporary conditions. Therefore, the sideslopes of the temporary embankments are also recommended to be constructed using 2H:1V sideslopes.

7.9 EROSION AND SCOUR PROTECTION

The near-surface site soils present in the vicinity of the abutment locations include highly variable fill materials, ranging in composition from silty clay to clayey sand to sand and gravel, as well as native soils comprised of topsoil and clayey silt till. These materials are expected to be susceptible to erosion and scour under the design flood/flow velocities. The requirements for

design of erosion/scour protection should be assessed by the hydraulic design engineer. As a minimum, it is recommended that erosion protection (e.g., rip-rap or granular sheeting) be provided for any embankment fill slopes present in the abutment foreslope areas to protect the foundations/pile caps from being exposed. The rip-rap or granular sheeting should be consistent with the requirements of OPSS.PROV 1004 (Aggregates – Miscellaneous) with the type/size of material approved by the hydraulic design engineer.

Vegetation on the new highway embankment slopes should be established as soon as possible after completion of the embankment construction to minimize the potential for surficial erosion.

7.10 CEMENT TYPE AND CORROSION POTENTIAL

Two samples of the native soil were submitted to Paracel Laboratories in Ottawa, Ontario, for analysis of pH, water soluble sulphate and chloride concentrations, and resistivity. The testing was completed to provide data for assessing the potential for degradation of the concrete in the presence of soluble sulphates and the potential for corrosion of exposed steel used in foundations and buried infrastructure. The analysis results together with the results for the sample from the bottom part of the fill are summarized in Table 4.2.

The concentration of soluble sulphate provides an indication of the degree of sulphate attack that is expected for concrete in contact with soil and groundwater at the site. The maximum soluble sulphate concentration for all the samples tested was 166 µg/g. Soluble sulphate concentrations less than 1000 µg/g generally indicate that a low degree of sulphate attack is expected for concrete in contact with soil and groundwater. Type GU (General Use) Portland Cement should therefore be suitable for use in concrete at this site.

The pH, resistivity and chloride concentration provide an indication of the degree of corrosiveness of the sub-surface environment. The soil pH values were between 7.8 and 8.0 which are within what is considered the normal range for soil pH of 5.5 to 9.0. The pH levels of the tested soil do not indicate a highly corrosive environment. The test results provided in Table 4.2 should be used by the designers in assessing the potential for corrosion of steel elements and may be used to aid in the selection of coatings and corrosion protection systems for buried steel objects.

8.0 CONSTRUCTION CONSIDERATIONS

8.1 CONSTRUCTION STAGING AND DETOUR

A Temporary Modular Bridge (TMB) is planned to be installed on the north side of the existing highway to maintain the ability of a single lane of traffic to pass the site during the replacement of the existing bridge.

The installation of the TMB will require construction of new approach embankments leading up to the TMB. The highway grade at the TMB will be at approximately the same elevation as the

existing Highway 89 grade. Therefore, the new approach embankments will have maximum heights of about 3 m to 4 m above existing grades.

8.2 TEMPORARY ROADWAY PROTECTION

Temporary roadway protection is anticipated to form part of the staged construction approach that will be required to protect traffic on Hwy 89 during excavations for the construction of the abutments for the TMB and to protect traffic on the TMB detour during construction of the new bridge particularly if fill and organic soils in the area of the approach slabs for the Hwy 89 replacement bridge are to be removed.

The contractor will ultimately be responsible to develop and implement a roadway protection system meeting the requirements of OPSS.PROV 539, including establishing appropriate geotechnical design parameters.

The following table compares the available roadway protection options considered for the proposed rehabilitation:

Table 8.1: Comparison of Roadway Protection Systems

Option	Advantages	Disadvantages	Relative Cost	Risk & Consequences
Steel sheet piles (SSP)	<ul style="list-style-type: none"> • Simple installation process • Provides cut-off to groundwater seepage from sides of excavation • Can be incorporated with groundwater cut-off system for excavation for removal of weak soils at west abutment of TMB 	<ul style="list-style-type: none"> • Difficult to drive/install in very hard or very dense fill particularly where cobbles/boulders are present • May require large sections where cantilever design is adopted 	Medium	<ul style="list-style-type: none"> • Possible damage to sheet piles during driving
Soldier piles with timber lagging; (struts/rakers as required)	<ul style="list-style-type: none"> • Simple installation process 	<ul style="list-style-type: none"> • Additional labour required • Groundwater seepage into the excavation can occur without groundwater control • Removal of soldier piles can be difficult 	Low	<ul style="list-style-type: none"> • Potential for groundwater seepage and loss of ground unless groundwater control measures are implemented • Potential for minor loss of ground at rear of lagging

Both support systems are considered feasible for use at this site. However, the use of soldier pile and lagging is considered to be more practical for excavations above the water table while steel sheet pile walls presents itself as the more viable option where excavations will extend below the groundwater level as it will reduce dewatering requirements.

Roadway protection design should meet the requirements of Performance Level 2 in accordance with OPSS.PROV 539 and should consider traffic loading. Performance Level 2 specifies a Maximum Angular Distortion of 1:200 and a Maximum Horizontal Displacement of 25 mm. Strut, raker or tie-back design, if and as required, must be designed not to exceed these limits.

Horizontal movement of the temporary roadway protection system should be monitored throughout the culvert replacement process as described in OPSS.PROV 539. The monitoring requirements are outlined in OPSS.PROV 539, including the milestone inspections to be completed by the Quality Verification Engineer.

8.3 EXCAVATION AND BACKFILLING

Excavation and backfill for the new bridge structure should be carried out in accordance with OPSS 902 Construction Specification for Excavation and Backfilling – Structures.

Any vegetation, fill and organic soils and other deleterious materials must be removed from beneath the new approach embankments for the proposed TMB. In addition, the compressible clayey silt deposit encountered to an elevation of about 178 m at the west abutment of the TMB may also be removed to reduce settlements if shallow foundations are to be used to support the TMB. Similarly, organic soils/fill may also need to be removed to limit settlements beneath the approach slabs for the new Hwy 89 replacement bridge. Where deleterious materials are encountered at foundation subgrade level, the materials should be excavated, removed and replaced with compacted granular fill materials. The lateral extent of the zone of sub-excavation (and replacement) should include all deleterious material within the influence zone of the bridge foundations and approach slabs and any retaining walls structures.

Grading work should be carried out in accordance with OPSS.PROV 206 Construction Specification for Grading and SP 206S03. Where existing embankments are to be widened, the new fill should be benched into the existing embankments in accordance with OPSD 208.010.

All side slopes for open cut excavations should conform to Occupational Health and Safety Act regulations for Construction Projects (OHSA). The excavations required for the new abutments would be developed through the existing approach embankment fill and would extend to depths in the order of 4 m below the existing Hwy 89 road grade. The excavations required for construction of the TMB foundations are expected to encounter topsoil, fill materials and native soils varying from firm clayey silt to loose to compact granular soils.

Where space permits, these excavations may be developed using open-cut methods. The existing fill materials, clayey silt and alluvial deposits above the water table would be classified as Type 3 soils. OHSA indicates that temporary excavations made within these materials above the water table should be developed with side slopes no steeper than 1H:1V. Granular soils (fill materials and native overburden) below the water table would be classified as Type 4 soil and excavations in these materials should be sloped no steeper than 3H:1V based on OSHA requirements.

8.4 OBSTRUCTIONS

Cobbles and/or boulders are present in the fill, the alluvial materials and the fill deposits at this site. These materials could obstruct excavations and the installation of temporary roadway protections systems. In addition, the existing concrete bridge foundations will also obstruct the installation of some the piles for the replacement bridge.

A Non-Standard Special Provision (NSSP) should be included in the contract to address this issue. A draft of the NSSP is provided in Appendix H.

8.5 PILE INSTALLATION

It is essential that the compatibility of the pile driving equipment, the soil conditions, and the pile type being driven is properly accounted for in order to achieve the required pile penetration and a satisfactory pile foundation.

Piles shall be provided with bearing points such as Titus 'H' bearing Pile Point (Standard Model or equivalent).

The pile driving equipment shall be appropriate to the driving conditions and capable of delivering a minimum specified energy of 60 kJ.

As indicated in Section 7.6, pre-augering/drilling and/or coring through both the existing bridge foundation and soil deposits containing cobbles and boulders will be required to 5 m below the base of the pile cap before the installation of CSP. The diameter of the pre-auger/drill/core hole should be large enough to permit installation of the CSP; below the base of the CSP, the pre-drilled hole can be maintained at the same diameter as used above or reduced to diameter as small as 300 mm if selected by the contractor. If spoil is removed from the portion of the predrilled zone below the CSP, this zone should be backfilled with sand prior to CSP placement. After placement of CSP in the hole, the annular space around the CSP should also be backfilled before the commencement of pile driving.

As outlined in MTO's Structural Manual (MTO, 2014), MTO's principle pile driving control tool is the Hiley Formula. This manual indicates that Ultimate Pile Resistance (R) calculated in the field using the Hiley Formula must be greater than twice the Design Load at ULS determined by structural engineer (which is less than or equal to two times the factored geotechnical resistance at ULS). The factored geotechnical resistance at ULS applicable to 310x110 H-piles driven to refusal within the fill at this site is 1,500kN. The geotechnical resistance of each pile should be determined using the Hiley Formula. As noted in Section 7.6.1.2, PDA testing on two piles at each abutment of the replacement bridge is also to be carried out in conjunction with the Hiley Formula testing for this project.

8.6 TEMPORARY GROUNDWATER CONTROL

The most recent water levels measured in monitoring wells installed Boreholes 2 and 17-5 were at elevations of 180.3 m. Excavations for removal of the firm clayey soil at the west abutment of

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the TMB and organic soils/fill beneath the approach slabs for bridge will require excavations extending below level. In addition, it is noted that water levels at the site will fluctuate with the water levels in the South Saugeen River. Temporary unwatering, using conventional sump and pump techniques, is considered appropriate for excavations that are developed entirely within clayey silt soils.

However, the near surface soils present at the site are highly variable in composition and contain alluvial deposits including coarse granular soils. Large groundwater inflows would occur through such deposits. Given the proximity of the excavations to the river and the presence of lower-permeability clayey soils at relative shallow depth, isolating the excavation zones from the river by means of a cofferdam or sheet pile enclosure system extending through the surficial granular soils is considered to be more practical than the installation of a dewatering system to control groundwater inflows.

All groundwater control and temporary flow passage systems required for the construction of the TMP and new Highway 89 bridge should be designed and implemented in accordance with OPSS 517 Construction Specification for Dewatering and SP No. 517F01 (July 2017). It is understood that pile cap levels for the new structures will be located above the river and groundwater level and that no extensive excavations below the groundwater level are planned. Based on these conditions, a preconstruction survey related to dewatering activities is not required and the Design Engineer Requirements box in the Dewatering Systems Section of Table A from SP No. 517F01 can be input as "No".

9.0 SPECIFICATIONS

The following specifications are referenced in this report:

Table 9.1: Specifications Referenced in Report

Document	Title
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.101	Foundation Frost Depths for Southern Ontario
OPSD 3101.150	Walls – Abutment, Backfill – Minimum Granular Requirement
OPSS 206	Construction Specification for Grading
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
OPSS.PROV 539	Construction Specification for Temporary Protections Systems
SP 206S03	Earth Excavation, Grading

10.0 REFERENCES

Canadian Foundation Engineering Manual (CFEM). 2006. Fourth Edition. Canadian Geotechnical Society, 488 p.

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11.0 CLOSURE

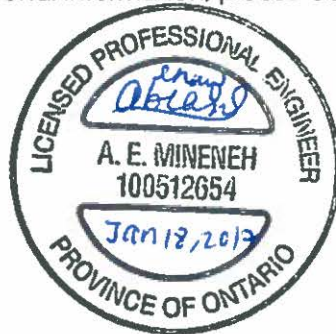
The recommendations made in this report were made based on our our current understanding of the project. Stantec should be given the opportunity to review, and if necessary revise, the recommendations contained herein when the drawings and specifications are complete.

A soil investigation is a limited sampling of a site. The conclusions given herein are based on information gathered at the specific borehole locations. Should any conditions at the site be encountered which differ from those at the borehole locations, Stantec should be notified immediately in order to assess the additional information and its effects on the above recommendations.

We trust the information presented herein meets your present requirements. Should you have any questions or require additional information, please do not hesitate to contact us.

Respectfully submitted,

STANTEC CONSULTING LTD.



Abraham Mineneh, PhD, P.Eng.
Geotechnical Engineer



Kevin Nelson, P.Eng.
Principal, Senior Geotechnical Engineer



Raymond Haché, M.Sc., P.Eng.
Designated Principal MTO Foundations Contact

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APPENDIX A

Drawing Nos. 1 and 2 – Borehole Location Plan and Soil Strata Plot

APPENDIX B

Symbols and Terms Used on Borehole Records

Borehole Records

Borehole Records from a Previous Investigation (Geocres Report No. 40P15-46)

SYMBOLS AND TERMS USED ON BOREHOLE AND TEST PIT RECORDS

SOIL DESCRIPTION

Terminology describing common soil genesis:

<i>Topsoil</i>	- mixture of soil and humus capable of supporting vegetative growth
<i>Peat</i>	- mixture of visible and invisible fragments of decayed organic matter
<i>Till</i>	- unstratified glacial deposit which may range from clay to boulders
<i>Fill</i>	- material below the surface identified as placed by humans (excluding buried services)

Terminology describing soil structure:

<i>Desiccated</i>	- having visible signs of weathering by oxidization of clay minerals, shrinkage cracks, etc.
<i>Fissured</i>	- having cracks, and hence a blocky structure
<i>Varved</i>	- composed of regular alternating layers of silt and clay
<i>Stratified</i>	- composed of alternating successions of different soil types, e.g. silt and sand
<i>Layer</i>	- > 75 mm in thickness
<i>Seam</i>	- 2 mm to 75 mm in thickness
<i>Parting</i>	- < 2 mm in thickness

Terminology describing soil types:

The classification of soil types are made on the basis of grain size and plasticity in accordance with the Unified Soil Classification System (USCS) (ASTM D 2487 or D 2488). The classification excludes particles larger than 76 mm (3 inches). The USCS provides a group symbol (e.g. SM) and group name (e.g. silty sand) for identification.

Terminology describing cobbles, boulders, and non-matrix materials (organic matter or debris):

Terminology describing materials outside the USCS, (e.g. particles larger than 76 mm, visible organic matter, construction debris) is based upon the proportion of these materials present:

<i>Trace, or occasional</i>	Less than 10%
<i>Some</i>	10-20%
<i>Frequent</i>	> 20%

Terminology describing compactness of cohesionless soils:

The standard terminology to describe cohesionless soils includes compactness (formerly "relative density"), as determined by the Standard Penetration Test N-Value (also known as N-Index). A relationship between compactness condition and N-Value is shown in the following table.

Compactness Condition	SPT N-Value
<i>Very Loose</i>	<4
<i>Loose</i>	4-10
<i>Compact</i>	10-30
<i>Dense</i>	30-50
<i>Very Dense</i>	>50

Terminology describing consistency of cohesive soils:

The standard terminology to describe cohesive soils includes the consistency, which is based on undrained shear strength as measured by *in situ* vane tests, penetrometer tests, or unconfined compression tests.

Consistency	Undrained Shear Strength	
	kips/sq.ft.	kPa
<i>Very Soft</i>	<0.25	<12.5
<i>Soft</i>	0.25 - 0.5	12.5 - 25
<i>Firm</i>	0.5 - 1.0	25 - 50
<i>Stiff</i>	1.0 - 2.0	50 - 100
<i>Very Stiff</i>	2.0 - 4.0	100 - 200
<i>Hard</i>	>4.0	>200

ROCK DESCRIPTION

Terminology describing rock quality:

RQD	Rock Mass Quality
0-25	<i>Very Poor</i>
25-50	<i>Poor</i>
50-75	<i>Fair</i>
75-90	<i>Good</i>
90-100	<i>Excellent</i>

Rock quality classification is based on a modified core recovery percentage (RQD) in which all pieces of sound core over 100 mm long are counted as recovery. The smaller pieces are considered to be due to close shearing, jointing, faulting, or weathering in the rock mass and are not counted. RQD was originally intended to be done on NW core; however, it can be used on different core sizes if the bulk of the fractures caused by drilling stresses are easily distinguishable from *in situ* fractures. The terminology describing rock mass quality based on RQD is subjective and is underlain by the presumption that sound strong rock is of higher engineering value than fractured weak rock.

Terminology describing rock mass:

Spacing (mm)	Joint Classification	Bedding, Laminations, Bands
> 6000	<i>Extremely Wide</i>	-
2000-6000	<i>Very Wide</i>	<i>Very Thick</i>
600-2000	<i>Wide</i>	<i>Thick</i>
200-600	<i>Moderate</i>	<i>Medium</i>
60-200	<i>Close</i>	<i>Thin</i>
20-60	<i>Very Close</i>	<i>Very Thin</i>
<20	<i>Extremely Close</i>	<i>Laminated</i>
<6	-	<i>Thinly Laminated</i>

Terminology describing rock strength:

Strength Classification	Unconfined Compressive Strength (MPa)
<i>Extremely Weak</i>	< 1
<i>Very Weak</i>	1 – 5
<i>Weak</i>	5 – 25
<i>Medium Strong</i>	25 – 50
<i>Strong</i>	50 – 100
<i>Very Strong</i>	100 – 250
<i>Extremely Strong</i>	> 250

Terminology describing rock weathering:

Term	Description
<i>Fresh</i>	No visible signs of rock weathering. Slight discolouration along major discontinuities
<i>Slightly Weathered</i>	Discolouration indicates weathering of rock on discontinuity surfaces. All the rock material may be discoloured.
<i>Moderately Weathered</i>	Less than half the rock is decomposed and/or disintegrated into soil.
<i>Highly Weathered</i>	More than half the rock is decomposed and/or disintegrated into soil.
<i>Completely Weathered</i>	All the rock material is decomposed and/or disintegrated into soil. The original mass structure is still largely intact.

STRATA PLOT

Strata plots symbolize the soil or bedrock description. They are combinations of the following basic symbols. The dimensions within the strata symbols are not indicative of the particle size, layer thickness, etc.



Boulders
Cobbles
Gravel



Sand



Silt



Clay



Organics



Asphalt



Concrete



Fill



Bedrock

SAMPLE TYPE

SS	Split spoon sample (obtained by performing the Standard Penetration Test)
ST	Shelby tube or thin wall tube
DP	Direct-Push sample (small diameter tube sampler hydraulically advanced)
PS	Piston sample
BS	Bulk sample
WS	Wash sample
HQ, NQ, BQ, etc.	Rock core samples obtained with the use of standard size diamond coring bits.

WATER LEVEL MEASUREMENT



measured in standpipe,
piezometer, or well



inferred

RECOVERY

For soil samples, the recovery is recorded as the length of the soil sample recovered. For rock core, recovery is defined as the total cumulative length of all core recovered in the core barrel divided by the length drilled and is recorded as a percentage on a per run basis.

N-VALUE

Numbers in this column are the field results of the Standard Penetration Test: the number of blows of a 140 pound (64 kg) hammer falling 30 inches (760 mm), required to drive a 2 inch (50.8 mm) O.D. split spoon sampler one foot (305 mm) into the soil. For split spoon samples where insufficient penetration was achieved and N-values cannot be presented, the number of blows are reported over sampler penetration in millimetres (e.g. 50/75). Some design methods make use of N value corrected for various factors such as overburden pressure, energy ratio, borehole diameter, etc. No corrections have been applied to the N-values presented on the log.

DYNAMIC CONE PENETRATION TEST (DCPT)

Dynamic cone penetration tests are performed using a standard 60 degree apex cone connected to A size drill rods with the same standard fall height and weight as the Standard Penetration Test. The DCPT value is the number of blows of the hammer required to drive the cone one foot (305 mm) into the soil. The DCPT is used as a probe to assess soil variability.

OTHER TESTS

S	Sieve analysis
H	Hydrometer analysis
k	Laboratory permeability
γ	Unit weight
G_s	Specific gravity of soil particles
CD	Consolidated drained triaxial
CU	Consolidated undrained triaxial with pore pressure measurements
UU	Unconsolidated undrained triaxial
DS	Direct Shear
C	Consolidation
Q_u	Unconfined compression
I_p	Point Load Index (I_p on Borehole Record equals $I_p(50)$ in which the index is corrected to a reference diameter of 50 mm)

	Single packer permeability test; test interval from depth shown to bottom of borehole
	Double packer permeability test; test interval as indicated
	Falling head permeability test using casing
	Falling head permeability test using well point or piezometer



RECORD OF BOREHOLE No BH17-1

1 OF 2

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Mount Forest N: 4 871 086 E: 201 383 ORIGINATED BY DL
DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
DATUM Geodetic DATE 2017 10 04 - 2017 11 04 CHECKED BY KN

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		
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE					
383.7 0.0	ASPHALT							20 40 60 80 100						GR SA SI CL
383.5 0.4	SAND and GRAVEL (SP-GP) (FILL)		1	SS	-									
383.2 0.4	SAND and GRAVEL (GP-GM), some silt (FILL) Compact Brown Moist		2	SS	10									
382.4 1.3	SAND and GRAVEL (GM), some silt (FILL) Contains cobbles and pockets of clayey silt (CL-ML) Very loose to loose Brown Moist		3	SS	7									53 33 10 4
			4	SS	7									44 38 12 6
			5	SS	6									
			6	SS	6									
	Becomes wet below 4.5 m		7	SS	6									
378.5 5.2	SAND (SP), trace silt and gravel (Probable FILL) Very loose Brown Wet SPT value inferred to have been influenced by drilling disturbance		8	SS	1									8 83 (9)
377.3 376.2 6.5	Woodpieces and fibrous organic matter Silty gravelly SAND (SM) Dense Grey Wet		9	SS	42									22 44 27 7
376.1 7.6	SAND (SP), trace silt, gravel and organic matter Dense Grey-brown Wet		10	SS	75/ 0.29m									
375.8 7.9	SAND and GRAVEL (SM-GM), some silt to silty (TILL) Contains cobbles and boulders Very dense Grey Wet		11	SS	50/ 0.14m									36 38 21 5 Non-plastic
374.6 9.1	SAND, some gravel (SP) Very dense Grey Wet		12	SS	100									
374.2 9.5	Sandy SILT (ML), some gravel (TILL) Contains cobbles, boulders and zones of clayey silt with sand Very dense Grey to brown Wet													

Continued Next Page

WH - Weight of Hammer

 \times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

STN13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT GPJ ONTARIO MOT.GDT 1/16/18

W.P.	3093-12-00	LOCATION	Hwy 89 West of Mount Forest	N: 4 871 086 E: 201 383	ORIGINATED BY	DL	
DIST	West	HWY	89	BOREHOLE TYPE	Hollow Stem Auger - Split Spoon	COMPILED BY	SR
DATUM	Geodetic	DATE	2017 10 04 - 2017 11 04		CHECKED BY	KN	

[illegible]

STN13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT.GPJ ONTARIO MOT.GDT 1/16/18

WH - Weight of Hammer

x³, x³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



RECORD OF BOREHOLE No BH17-2

1 OF 1

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Mount Forest N: 4 871 088 E: 201 399 ORIGINATED BY DL
DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
DATUM Geodetic DATE 2017 11 04 CHECKED BY KN

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									
								20	40	60	80	100					
								20	40	60	80	100					

STN13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT GPJ ONTARIO MOT.GDT 1/16/18

WH - Weight of Hammer

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

○ 3% STRAIN AT FAILURE



RECORD OF BOREHOLE No BH17-3

1 OF 2

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Mount Forest N: 4 871 079 E: 201 334 ORIGINATED BY DL
DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
DATUM Geodetic DATE 2014 12 04 CHECKED BY KN

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 (%)		
								○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL	✕ LAB VANE	20						40	60	80
383.7	ASPHALT																			
0.0																				
383.5																				
383.2	SAND and GRAVEL (SP-GP) (FILL)																			
0.4	Silty SAND and GRAVEL (GP-GM) (FILL) Compact Brown Moist		1	AS	-		383										44 38 (18)			
382.5			2	SS	24															
1.2	Clayey SILT (CL), some sand and gravel (FILL) Contains silt seams, cobbles and boulders Firm to very stiff Brown Moist to wet		3	SS	29		382													
			4	SS	6		381													
380.7																				
3.1	Mixed silty SAND and GRAVEL, and woodpieces (FILL) Sample 5 consists predominantly of wood Loose Brown Wet		5	SS	27		380													
			6	SS	7															
379.1																				
4.6	SAND (SP), some silt and gravel Compact Brown Wet		7	SS	22		379													
378.6																	17 71 (12)			
5.1	SILT (ML), some sand and gravel (TILL) Contains cobbles and boulders Dense Light brown Wet		8	SS	45		378													
377.9																				
5.8	CLAYEY SILT (CL-ML) with sand and gravel (TILL) Contains cobbles and boulders Occasional sand seams Very stiff to hard Brown Wet Frequent grinding of augers noted		9	SS	24		377										16 27 47 15			
			10	SS	100/ 0.15m		376													
			11	SS	100/ 0.18m		375													
374.6																				
9.1	Sandy SILT (ML), some gravel (TILL) Contains cobbles, boulders and sand seams Very dense Grey Wet		12	SS	100/ 0.18m		374													
373.0																				
10.7					101/		373													

Continued Next Page

WH - Weight of Hammer

 \times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE

STN/13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT GPJ ONTARIO MOT.GDT 1/16/18

W.P.	3093-12-00	LOCATION	Hwy 89 West of Mount Forest	N: 4 871 079 E: 201 334	ORIGINATED BY	DL	
DIST	West	HWY	89	BOREHOLE TYPE	Hollow Stem Auger - Split Spoon	COMPILED BY	SR
DATUM	Geodetic	DATE	2014 12 04		CHECKED BY	KN	

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STN13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT.GPJ ONTARIO MOT.GDT 1/16/18

WH - Weight of Hammer

x³, x³. Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



RECORD OF BOREHOLE No BH17-4

1 OF 1

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Mount Forest N: 4 871 077 E: 201 324 ORIGINATED BY DL
 DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
 DATUM Geodetic DATE 2017 12 04 CHECKED BY KN

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 (%)		
								○ UNCONFINED ● QUICK TRIAXIAL	✕ FIELD VANE ✕ LAB VANE	20	40	60						80	100	10
383.8	ASPHALT																			
383.6																				
383.4	SAND and GRAVEL (SP-GP) (FILL)																			
0.4	Silty SAND and GRAVEL (GM) (FILL) Contains cobbles Compact Brown Moist		1	AS	-															
			2	SS	12															
382.4																				
1.4	Sandy CLAYEY SILT (CL-ML), some gravel (FILL) Contains topsoil pockets Firm to stiff Grey-brown to dark brown Sample 5 contains woodpieces up to 75 mm long		3	SS	11															
			4	SS	8															
380.3			5	SS	30															
3.5	Wood debris (FILL)																			
380.0																				
3.8	No split spoon sample recovery (2 attempts) - Took sample of auger cuttings SILTY CLAY (CI) Possible buried (topsoil) layer Soft Dark brown		6	AS	1															
379.1																				
4.7	Sandy SILT (ML), some gravel (Possible TILL) Contains cobbles and/or boulders Small sample recovery		7	SS	42															
378.5																				
5.3	CLAYEY SILT (CL), some sand and gravel (TILL) Very stiff to hard Grey Moist		8	SS	37															
377.7																				
6.1	Sandy SILT (ML), trace to some gravel (TILL) Contains cobbles and/or boulders Very dense Grey Wet		9	SS	100/ 0.18															
			10	SS	105/ 0.13m															

STN13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT GPJ ONTARIO MOT.GDT 1/16/18

WH - Weight of Hammer

×³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE



RECORD OF BOREHOLE No BH17-5

1 OF 2

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Mount Forest N: 4 871 098 E: 201 337 ORIGINATED BY DL
DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
DATUM Geodetic DATE 2017 04 17 CHECKED BY KN

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 (%)	
								○ UNCONFINED	✕ FIELD VANE						● QUICK TRIAXIAL	✕ LAB VANE
380.3							20 40 60 80 100									
0.0	TOPSOIL with wood pieces		1	SS	3											
379.7																
379.6	SAND and GRAVEL (SP-GP), trace silt															
0.8	Clayey SILT (CL), some sand and gravel Contains cobbles Firm to stiff Brown to dark grey Wet SPT 'N' value for sample 3 inferred to have been influenced by gravel		2	SS	4											
			3	SS	13											
378.0																
2.3	Clayey SILT (CL), trace sand and gravel (TILL) Contains cobbles Very stiff to hard Grey Moist to wet		4	SS	18											
			5	SS	19											
376.5																
3.8	Gravelly silty SAND (SM) to silty SAND and GRAVEL (GM-SM), trace clay (TILL) Contains cobbles and boulders Dense to very dense Brown Wet		6	SS	40											
			7	SS	55											
			8	SS	69											
			9	SS	80											
			10	SS	100/ 0.27m											

Continued Next Page

WH - Weight of Hammer

 \times^3, \times^3 : Numbers refer to
Sensitivity \circ 3% STRAIN AT FAILURE

STN/13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT.GPJ ONTARIO.MOT.GDT 1/16/18



RECORD OF BOREHOLE No BH17-5

2 OF 2

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Mount Forest N: 4 871 098 E: 201 337 ORIGINATED BY DL
DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
DATUM Geodetic DATE 2017 04 17 CHECKED BY KN

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			"N" VALUES	SHEAR STRENGTH kPa					WATER CONTENT (%)			
							20	40	60	80	100	W _p	W	W _L		
							20	40	60	80	100					
368.0	Silty GRAVEL (GM), some sand (TILL) Contain cobbles and boulders Very dense Light brown Wet Note: No sample recovery from SS12 : Strata boundary with unit above is approximate (<i>continued</i>)		12	SS	0.05m											
12.3	End of Borehole Water level in open borehole at 2.4 m depth on completion of drilling. Water level in monitoring well at ground surface on May 12, 2017. Water level in monitoring well at depth of 0.2 m (Elev. 380.1 m) on Nov. 30, 2017. Well re-drilled and re-installed 1.0 m west of BH17-5.		13	SS	0.08m											

WH - Weight of Hammer

 \times^3, \times^3 : Numbers refer to Sensitivity \circ 3% STRAIN AT FAILURE



RECORD OF BOREHOLE No BH17-7

1 OF 1

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Wellington Road 6 N: 4 871 099 E: 201 375 ORIGINATED BY DL
DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
DATUM Geodetic DATE 2017 11 30 CHECKED BY

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 (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
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380.4 0.0	SILTY CLAY (CI), some organic material (TOPSOIL FILL) Very soft Dark brown		1	SS	1	▽	380																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				</

STN13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT.GPJ ONTARIO MTO GDT 1/16/18

WH - Weight of Hammer

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

○ 3% STRAIN AT FAILURE







RECORD OF BOREHOLE No BH17-7B

1 OF 1

METRIC

W.P. 3093-12-00 LOCATION Hwy 89 West of Wellington Road 6 N: 4 871 100 E: 201 375 ORIGINATED BY DL
DIST West HWY 89 BOREHOLE TYPE Hollow Stem Auger - Split Spoon COMPILED BY SR
DATUM Geodetic DATE 2017 11 30 CHECKED BY

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 (%)		
								20	40	60	80	100						○ UNCONFINED	✕ FIELD VANE	● QUICK TRIAXIAL
380.4	0.0	Sandy SILTY CLAY (Cl), trace organic matter (TOPSOIL FILL)		1	SS	1	▽	380									43	26 51 20 3 Non-plastic		
379.6	0.8	Mixture of silty SAND (SM), trace gravel and clay and sandy CLAY (CL) (Probable FILL) Occasional cobbles and boulders Very loose/very soft Dark brown to black Wet Becomes compact and transitioning to gravelly sand below 1.5 m		2	SS	2		379												
378.3				3	SS	15		378												
376.0	2.1	SAND and GRAVEL (SP/GP), some silt Compact Brown Wet						377												
								376												
								375												
								374												
373.7	6.7	End of Borehole Water level in borehole at 1.0 m depth on completion of drilling.																		

STN13-ONTARIO MTO STANTEC 165001038 - HWY 89 - MOST RECENT GPJ ONTARIO MOT.GDT 1/16/18

WH - Weight of Hammer

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

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1

1 OF 1

METRIC

PROJECT 11-1132-0109-2000

W.P. 3035-11-00

LOCATION N 4871073.1, E 201350.4

ORIGINATED BY MA

DIST HWY 89

BOREHOLE TYPE POWER AUGER, HOLLOW STEM

COMPILED BY LMK

DATUM GEODETIC

DATE October 1, 2012

CHECKED BY

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 (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE								
379.97	GROUND SURFACE					▽	379									14 36 37 13			
0.00	TOPSOIL, silty to sandy with gravel, cobbles and boulders																		
379.36	SAND AND GRAVEL, trace silt, with cobbles Compact Brown																		
0.61			1	SS	11														
378.60																			
1.37			2	SS	33														
	SANDY SILT TILL, some clay, some gravel Dense to very dense Brown								378										
			3	SS	75														
			4	SS	112														
			5	SS	100/ 250mm														
							377												
			6	SS	100/ 225mm														
374.79	SILTY SAND AND GRAVEL, cobbles, trace clay Very dense Brown						376												
5.18			7	SS	100/ 250mm														
			8	SS	100/ 250mm														
372.81	SANDY SILT TILL, some clay, trace gravel, cobbles Very dense Grey						375												
7.16																			
							374												
							373												
							372												
370.80	END OF BOREHOLE		10	SS	100/ 25mm		371												
9.17	Groundwater encountered at about elev. 378.8m during drilling on October 1, 2012.																		

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

APPENDIX C

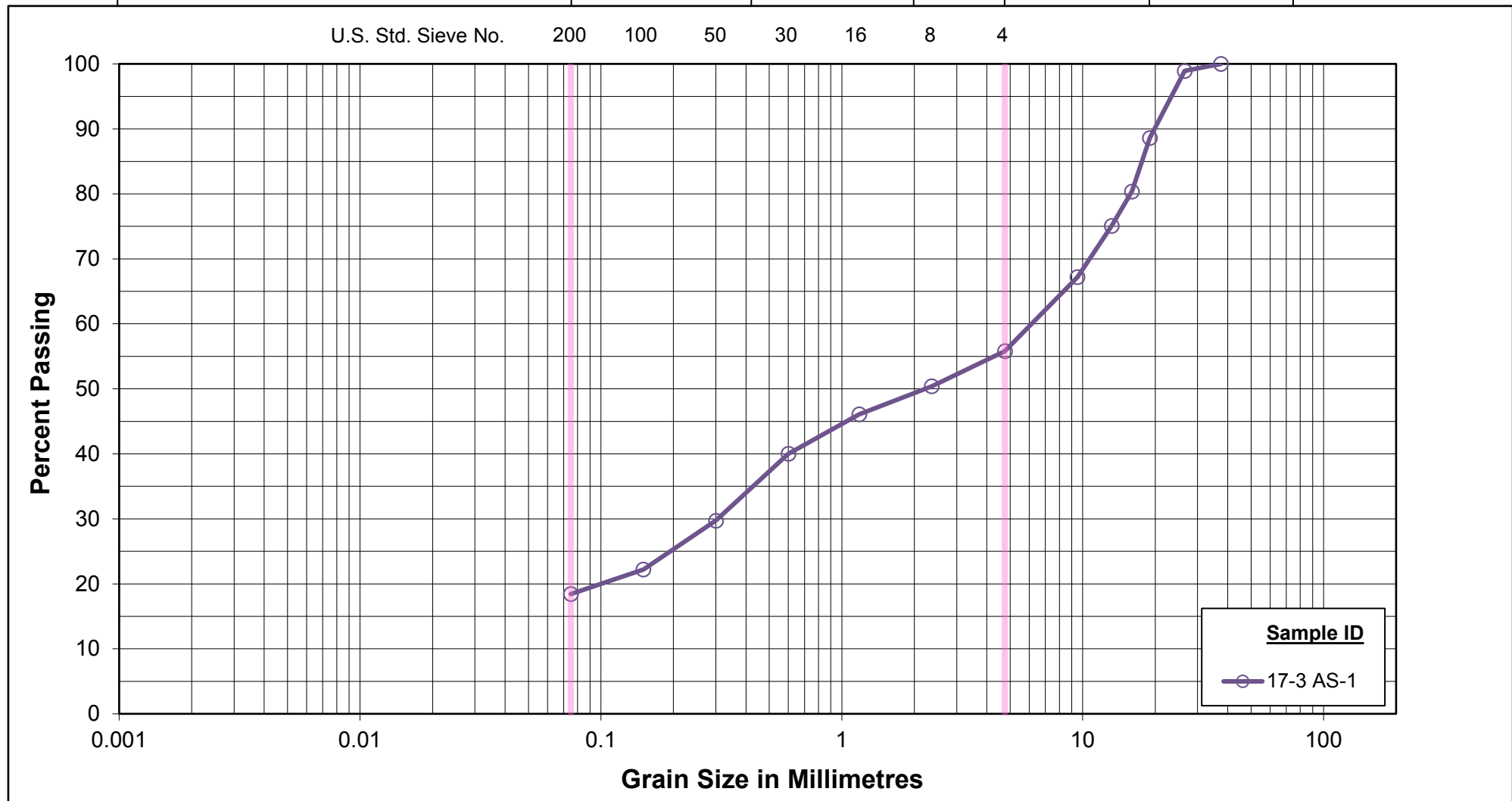
Laboratory Test Results

Figures C1 to C9: Grain Size Distribution Plots and Plasticity Charts

Corrosivity Testing Results

Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse

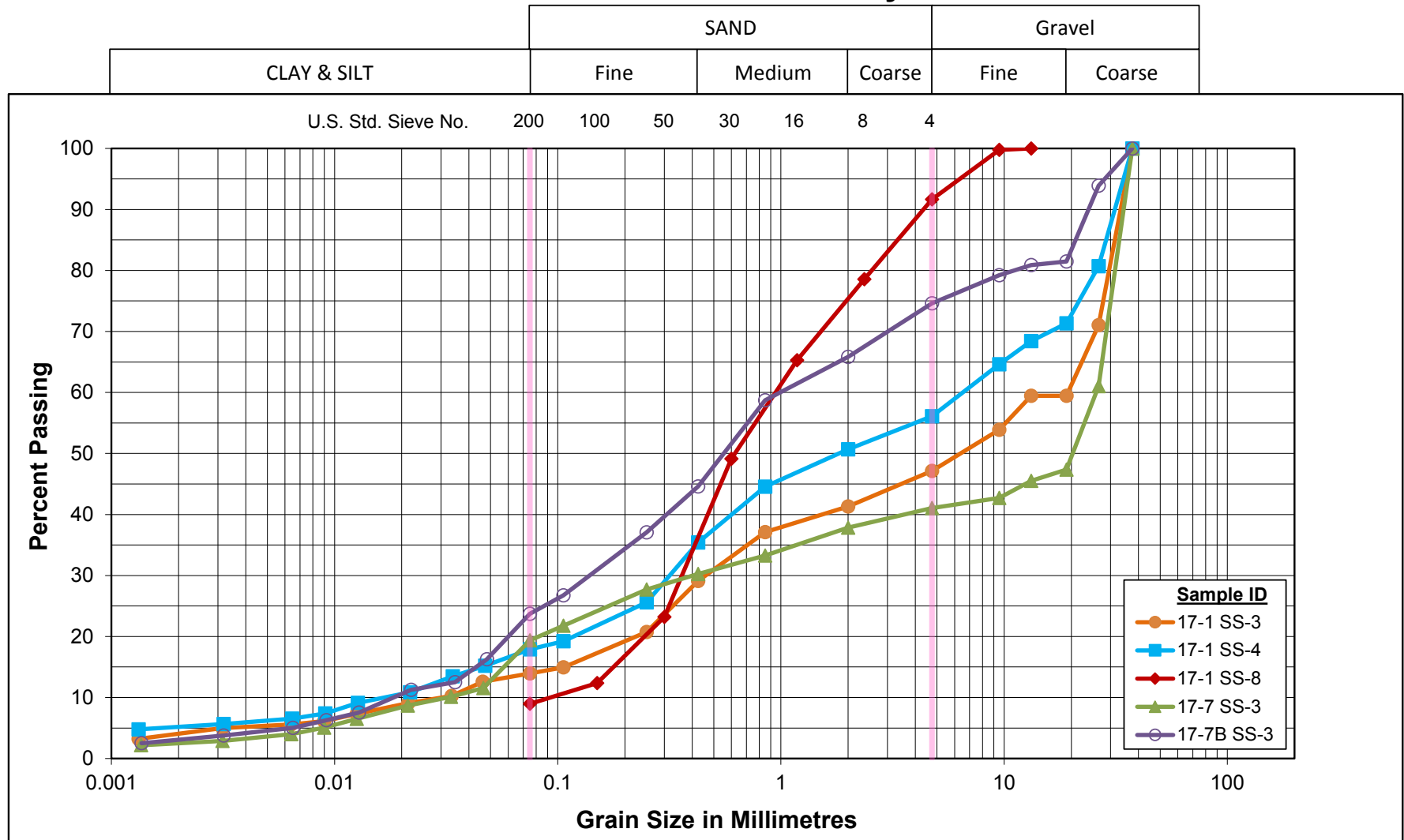


GRAIN SIZE DISTRIBUTION FILL: SAND and GRAVEL

Figure No. C1

Project No. 165001038
HWY 89 - SOUTH SAUGREEN RIVER

Unified Soil Classification System



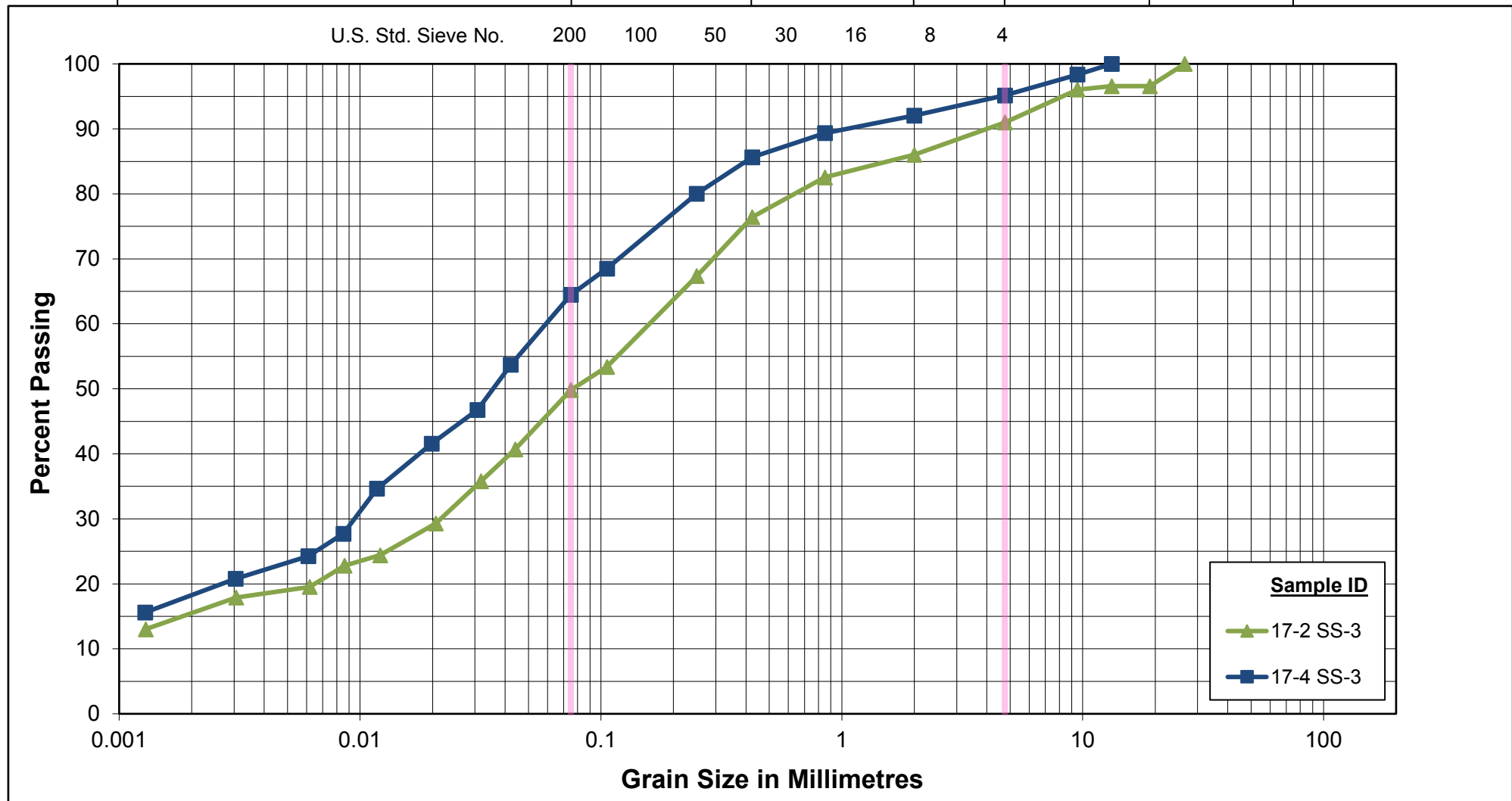
GRAIN SIZE DISTRIBUTION
 SILTY SAND to SANDY GRAVEL (FILL)

Figure No. C2

Project No. 165001038
 HWY 89 - SOUTH SAUGEEN RIVER

Unified Soil Classification System

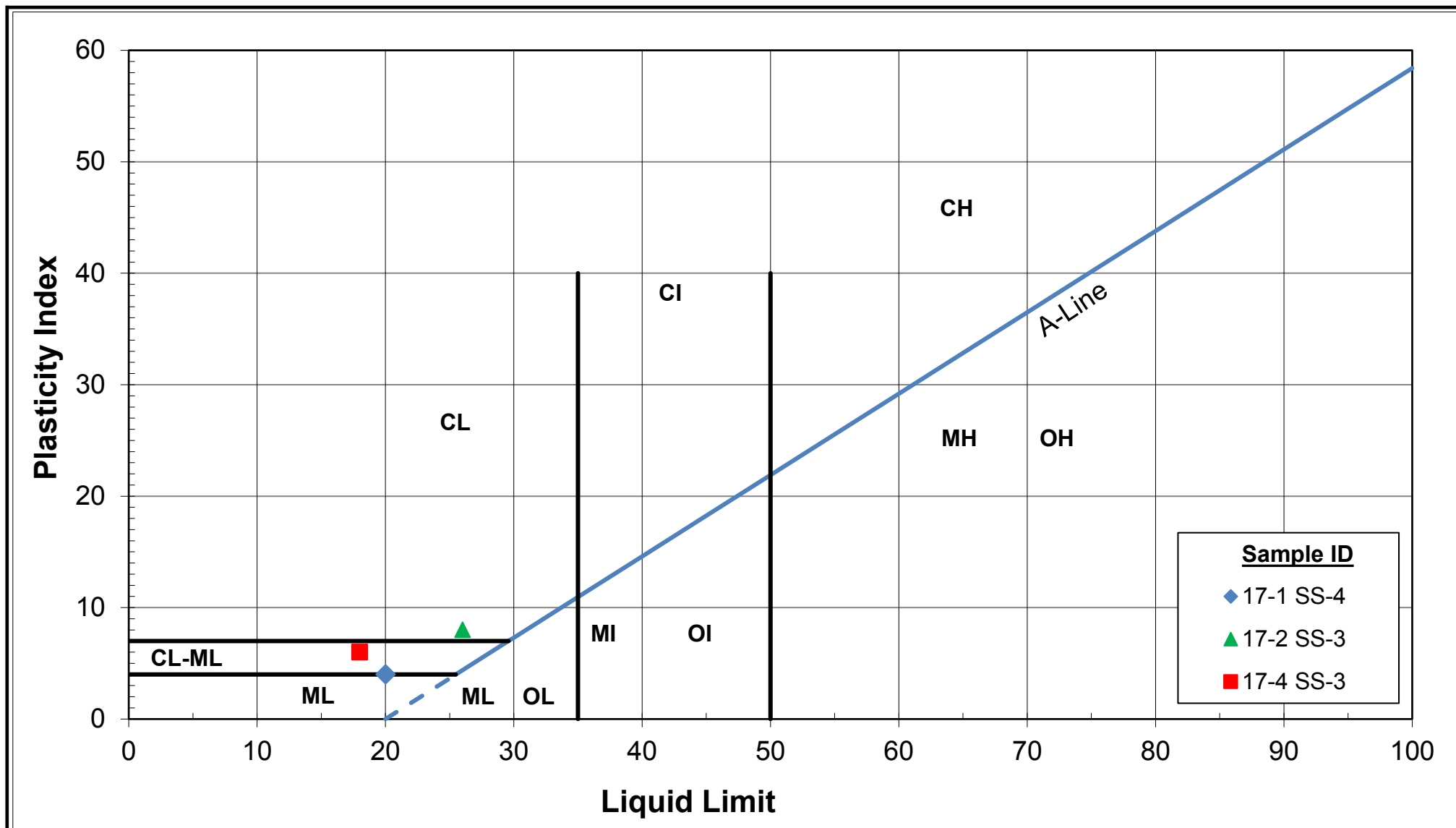
CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse



GRAIN SIZE DISTRIBUTION
FILL: CLAYEY SILT

Figure No. C3

Project No. 165001038
HWY 89 - SOUTH SAUGEEN RIVER



PLASTICITY CHART

FILL: CLAYEY SILT (CL/CL-ML)

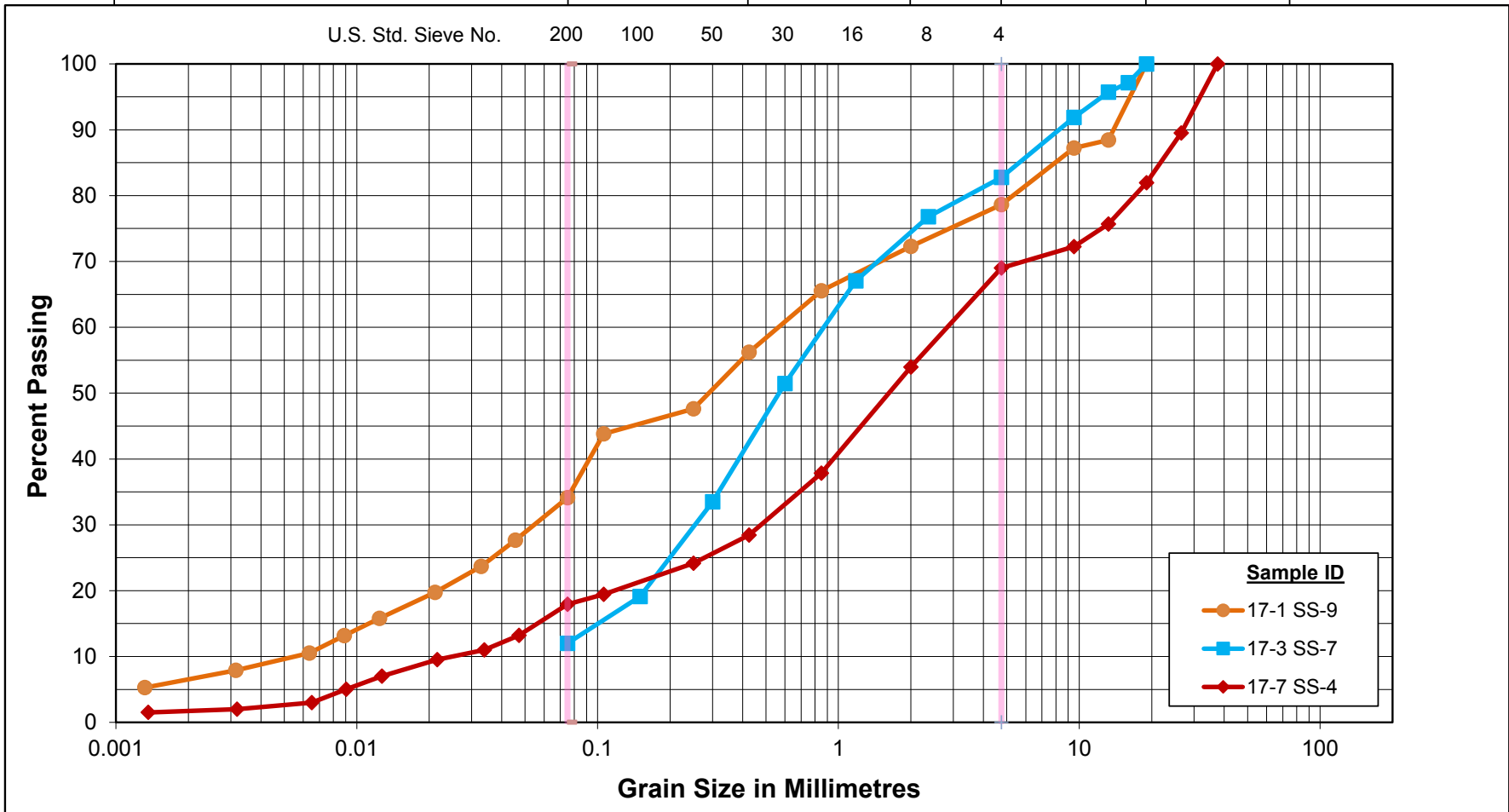
Figure No. C4

Project No. 165001038

HWY 89 - SOUTH SAUGREEN RIVER

Unified Soil Classification System

CLAY & SILT	SAND			Gravel	
	Fine	Medium	Coarse	Fine	Coarse

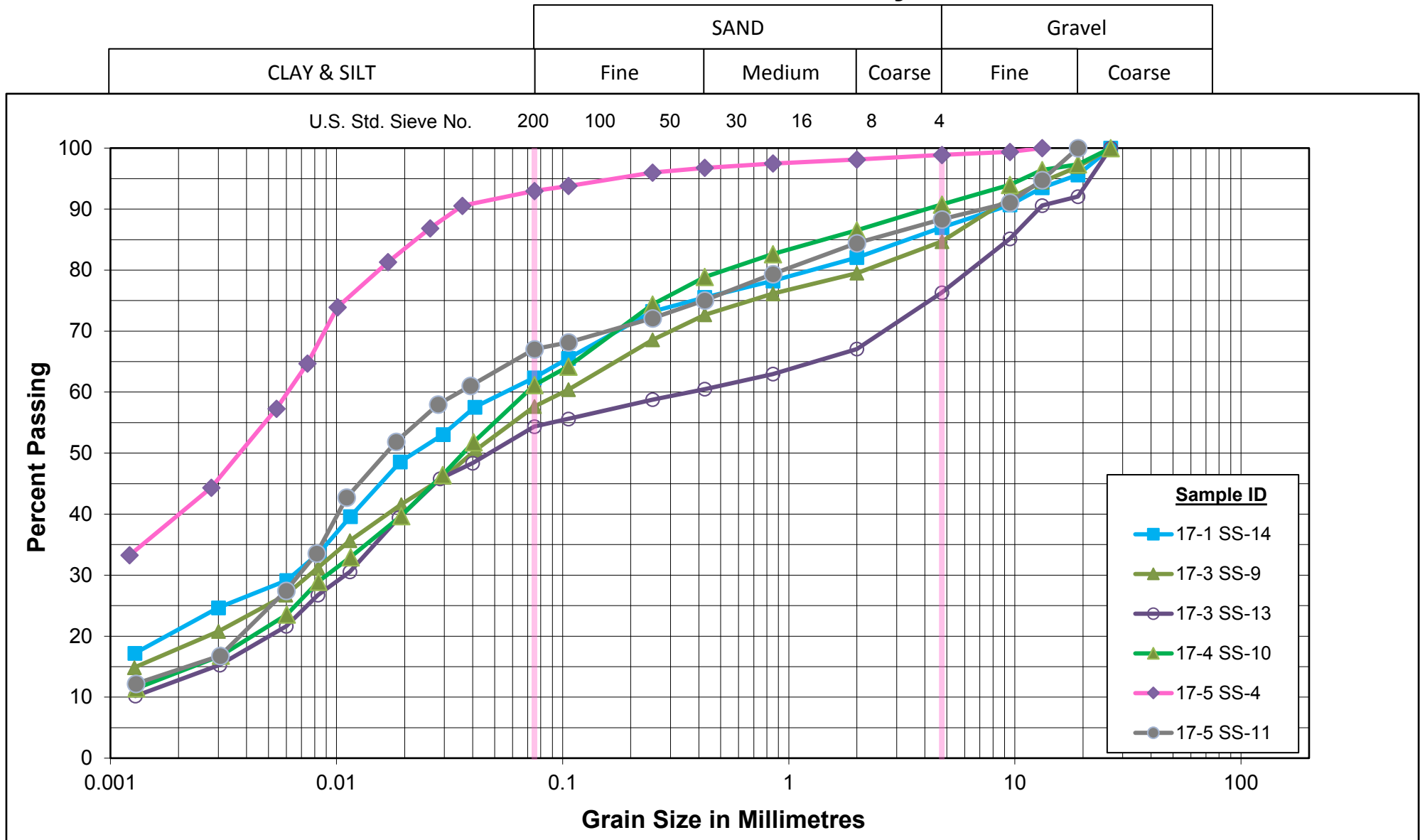


GRAIN SIZE DISTRIBUTION
SAND to Silty, Gravelly SAND

Figure No. C5

Project No. 165001038
HWY 89 - SOUTH SAUGEEN RIVER

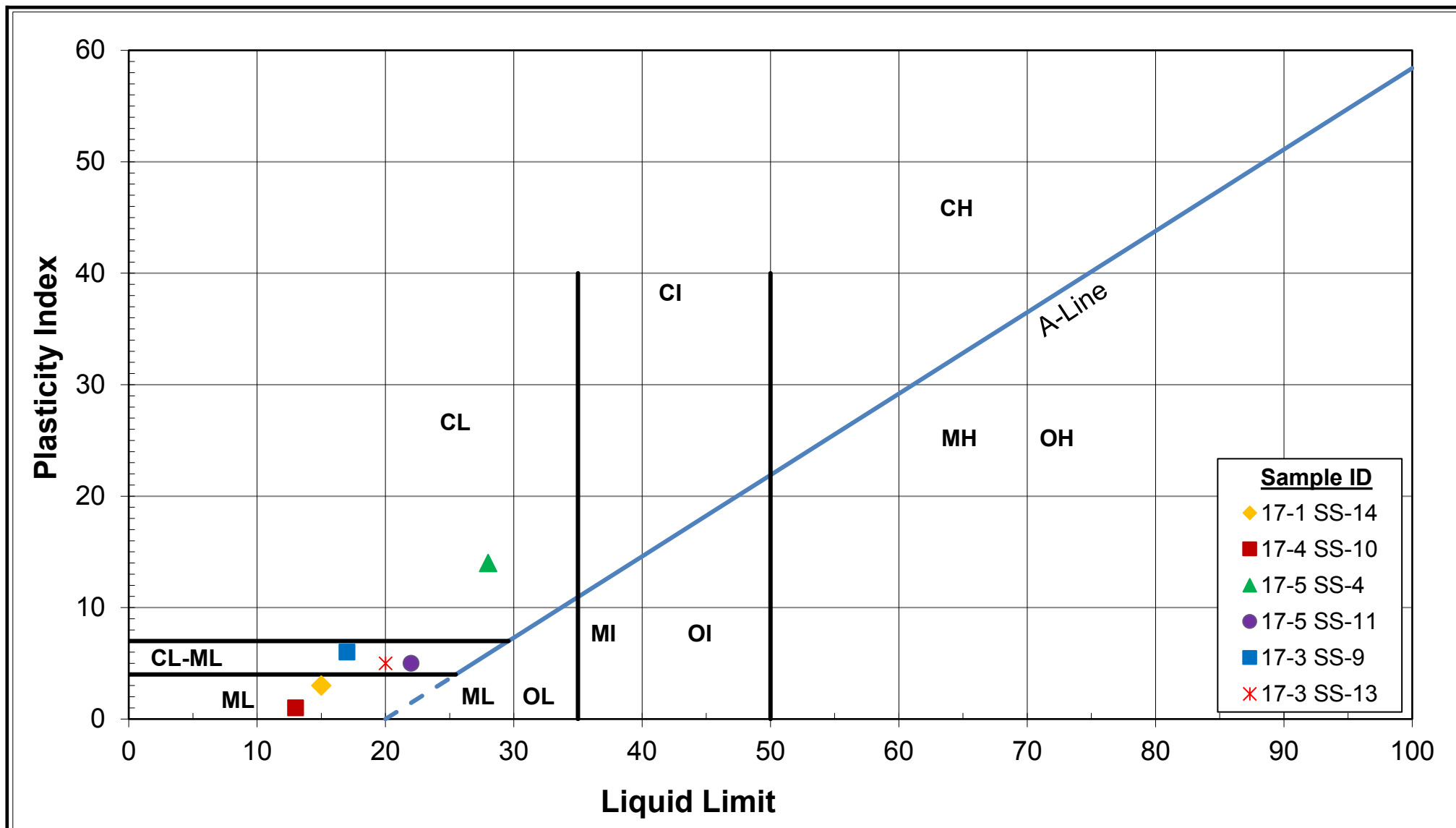
Unified Soil Classification System



GRAIN SIZE DISTRIBUTION
SANDY SILT to CLAYEY SILT (TILL)

Figure No. C6

Project No. 165001038
HWY 89 - SOUTH SAUGEEEN RIVER



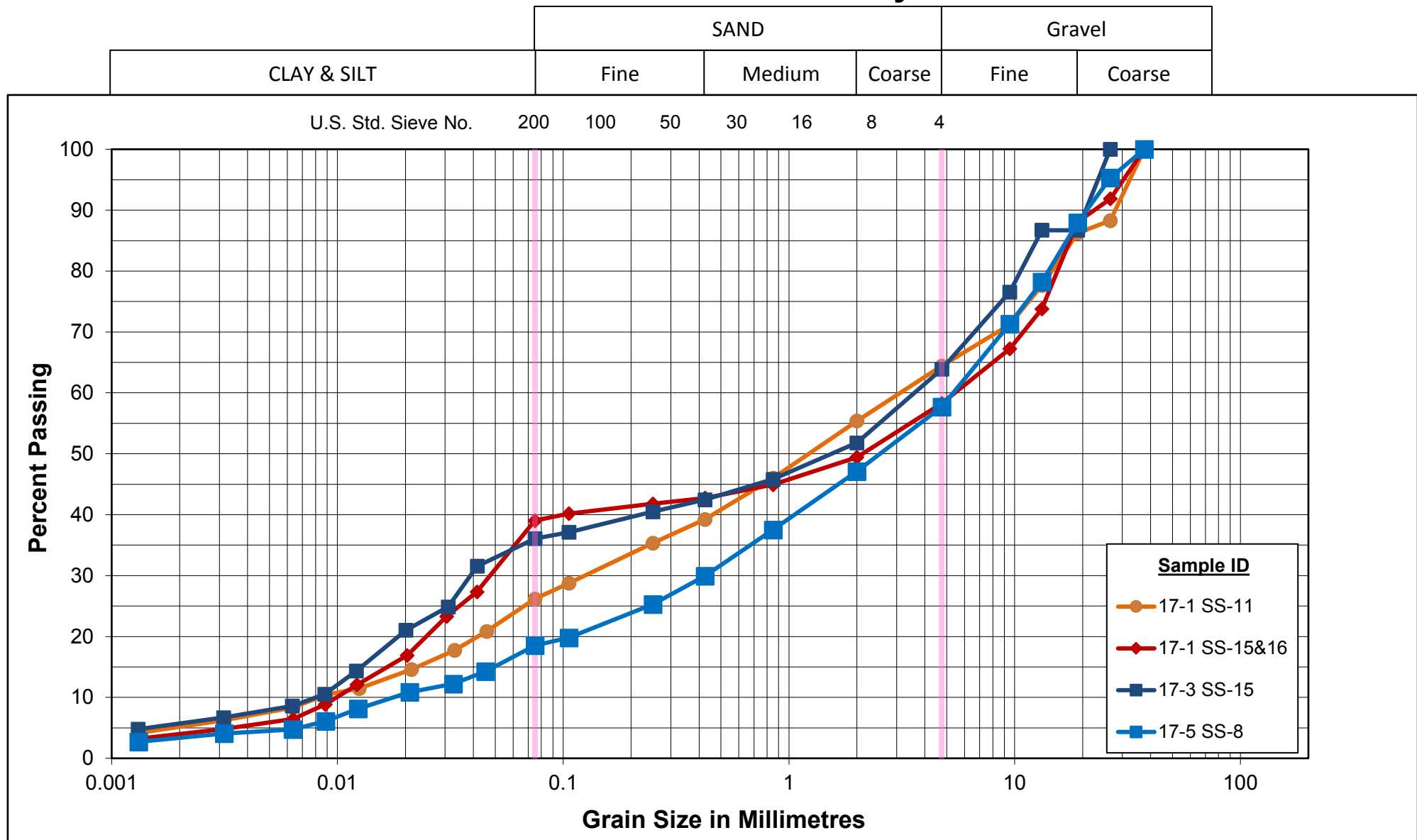
PLASTICITY CHART

SANDY SILT to CLAYEY SILT (TILL)

Figure No. C7

Project No. 165001038
HWY 89 - SOUTH SAUGEE RIVER

Unified Soil Classification System

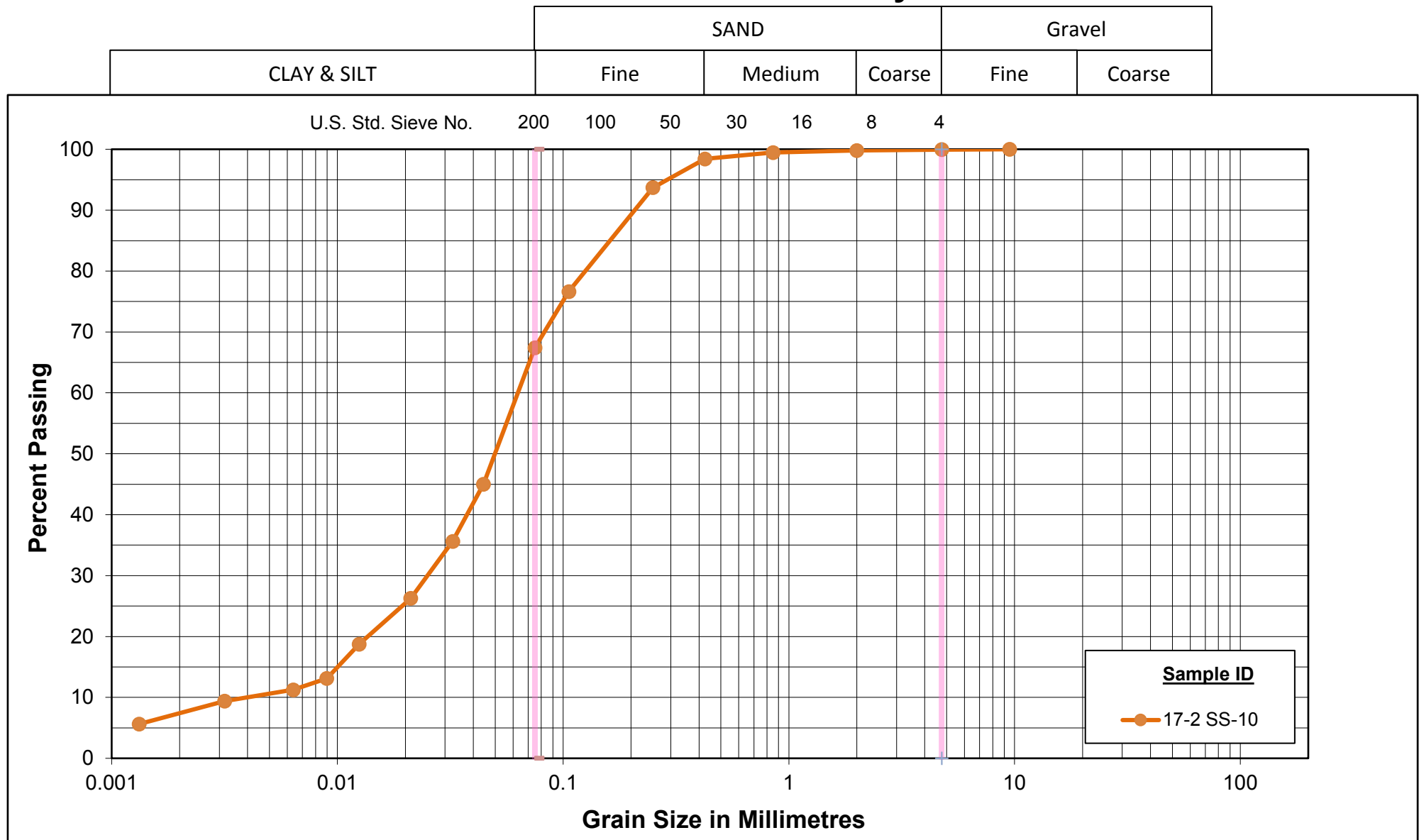


GRAIN SIZE DISTRIBUTION
 SILTY GRAVEL to SILTY SAND and
 GRAVEL (TILL)

Figure No. C8

Project No. 165001038
 HWY 89 - SOUTH SAUGEEN RIVER

Unified Soil Classification System



GRAIN SIZE DISTRIBUTION SANDY SILT

Figure No. C9

Project No. 165001038

HWY 89 - SOUTH SAUGEEEN RIVER

Certificate of Analysis

Stantec Consulting Ltd. (Ottawa)

1331 Clyde Avenue, Suite 400
Ottawa, ON K2C 3G4
Attn: Kevin Nelson

Client PO: Hwy 89
Project: 165001038.141
Custody:

Report Date: 18-May-2017
Order Date: 12-May-2017

Order #: 1719544

This Certificate of Analysis contains analytical data applicable to the following samples as submitted:

Paracel ID	Client ID
1719544-01	BH17-1, SS10, 25'-27'
1719544-02	BH17-5, SS3, 5'-7'

Approved By:



Mark Foto, M.Sc.
Lab Supervisor

Certificate of Analysis
Client: Stantec Consulting Ltd. (Ottawa)
Client PO: Hwy 89

Report Date: 18-May-2017
Order Date: 12-May-2017
Project Description: 165001038.141

Analysis Summary Table

Analysis	Method Reference/Description	Extraction Date	Analysis Date
Anions	EPA 300.1 - IC, water extraction	16-May-17	16-May-17
pH, soil	EPA 150.1 - pH probe @ 25 °C, CaCl buffered ext.	16-May-17	16-May-17
Resistivity	EPA 120.1 - probe, water extraction	18-May-17	18-May-17
Solids, %	Gravimetric, calculation	18-May-17	18-May-17

Certificate of Analysis
Client: Stantec Consulting Ltd. (Ottawa)
Client PO: Hwy 89

Report Date: 18-May-2017
 Order Date: 12-May-2017
Project Description: 165001038.141

Client ID:	BH17-1, SS10, 25'-27'	BH17-5, SS3, 5'-7'	-	-
Sample Date:	10-Apr-17	17-Apr-17	-	-
Sample ID:	1719544-01	1719544-02	-	-
MDL/Units	Soil	Soil	-	-

Physical Characteristics

% Solids	0.1 % by Wt.	90.6	90.5	-	-
----------	--------------	------	------	---	---

General Inorganics

pH	0.05 pH Units	7.95 [1]	7.83	-	-
Resistivity	0.10 Ohm.m	33.9	17.1	-	-

Anions

Chloride	5 ug/g dry	34 [1]	218	-	-
Sulphate	5 ug/g dry	166 [1]	50	-	-

Certificate of Analysis
 Client: Stantec Consulting Ltd. (Ottawa)
 Client PO: Hwy 89

Report Date: 18-May-2017
 Order Date: 12-May-2017
 Project Description: 165001038.141

Method Quality Control: Blank

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	ND	5	ug/g						
Sulphate	ND	5	ug/g						
General Inorganics									
Resistivity	ND	0.10	Ohm.m						

Certificate of Analysis
Client: Stantec Consulting Ltd. (Ottawa)
Client PO: Hwy 89

Report Date: 18-May-2017
Order Date: 12-May-2017
Project Description: 165001038.141

Method Quality Control: Duplicate

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	38.4	5	ug/g dry	34.1			11.6	20	
Sulphate	193	5	ug/g dry	166			15.2	20	
General Inorganics									
pH	7.04	0.05	pH Units	7.05			0.1	10	
Resistivity	11.0	0.10	Ohm.m	11.0			0.1	20	
Physical Characteristics									
% Solids	92.2	0.1	% by Wt.	90.6			1.8	25	

Certificate of Analysis
Client: Stantec Consulting Ltd. (Ottawa)
Client PO: Hwy 89

Report Date: 18-May-2017
Order Date: 12-May-2017
Project Description: 165001038.141

Method Quality Control: Spike

Analyte	Result	Reporting Limit	Units	Source Result	%REC	%REC Limit	RPD	RPD Limit	Notes
Anions									
Chloride	137	5	ug/g	34.1	103	78-113			
Sulphate	266	5	ug/g	166	101	78-111			

Certificate of Analysis
Client: Stantec Consulting Ltd. (Ottawa)
Client PO: Hwy 89

Report Date: 18-May-2017
Order Date: 12-May-2017
Project Description: 165001038.141

Qualifier Notes:

Login Qualifiers :

Sample - One or more parameter received past hold time - Chloride, pH, Sulphate.

Applies to samples: BH17-1, SS10, 25'-27'

Sample Qualifiers :

1 : Holding time had been exceeded upon receipt of the sample at the laboratory.

Sample Data Revisions

None

Work Order Revisions / Comments:

None

Other Report Notes:

n/a: not applicable

ND: Not Detected

MDL: Method Detection Limit

Source Result: Data used as source for matrix and duplicate samples

%REC: Percent recovery.

RPD: Relative percent difference.

Soil results are reported on a dry weight basis when the units are denoted with 'dry'.

Where %Solids is reported, moisture loss includes the loss of volatile hydrocarbons.

Parcel ID: 1719544



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Page ____ of ____

OTTAWA * KINGSTON * NIAGARA * MISSISSAUGA * SARNIA

Client Name: Stantec Consulting Ltd.	Project Reference: Hwy 89	TAT: <input checked="" type="checkbox"/> Regular <input type="checkbox"/> 3 Day
Contact Name: Kevin Nelson	Task #: 141	<input type="checkbox"/> 2 Day <input type="checkbox"/> 1 Day
Address: 2781 Lancaster Road., Suite 101. Ottawa ON. K1B-1A7	PO #: 165001038	Date Required: _____
Telephone: 613-738-6075	Email Address: kevin.nelson@stantec.com	

Criteria: ☐ O. Reg. 153/04 Table ____ ☐ O. Reg. 153/11 (Current) Table ____ ☐ RSC Filing ☐ O. Reg. 558/00 ☐ PWQO ☐ CCME ☐ SUB (Storm) ☐ SUB (Sanitary) Municipality: _____ ☐ Other: _____

Matrix Type: S (Soil/Sed.) GW (Ground Water) SW (Surface Water) SS (Storm/Sanitary Sewer) P (Paint) A (Air) O (Other)

Required Analyses

Parcel Order Number:

1719544

Parcel Order Number:		Matrix	Air Volume	# of Containers	Sample Taken		Resistivity	PH	Sulphate & Chloride										
Sample ID/Location Name					Date	Time													
1	BH17-1, SS10. 2S-27'	S		1	10-Apr-17		X	X	X										
2	BH17-5, SS3. 5-7'	S		1	17-Apr-17		X	X	X										
3																			
4																			
5																			
6																			
7																			
8																			
9																			
10																			

Comments:

Proceed with ~~ex~~ parameters past hold time as per Donna. *Lu*

Method of Delivery:

Swift

Relinquished By (Print & Sign):

Denis Rodriguez

Received by Driver/Depot:

Received at Lab:

SUNDBORN ROKMAI

Verified By:

Date/Time:

Date/Time:

12-May-17

Temperature: _____ °C

Temperature: 22.1 °C

pH Verified [] By: _____

Parcel Form.xlsx

APPENDIX D

Geophysical Testing Results



GEOPHYSICS GPR INTERNATIONAL INC.

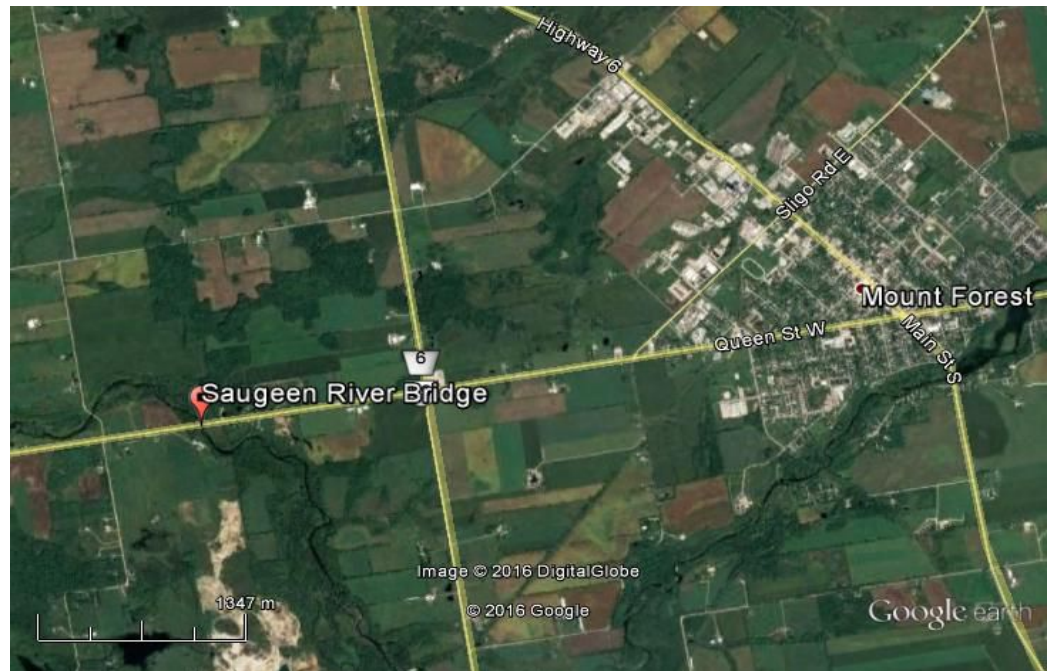
GEOPHYSICAL DOWN-HOLE INVESTIGATION OF THE SAUGEEN RIVER BRIDGE ON HWY 89 WEST OF MOUNT FOREST, ONTARIO

Presented to :



Stantec

**400-1331 Clyde Avenue,
Ottawa, Ontario
K2C 3G4**



Geophysics GPR International Inc.

6741 Columbus Road, Unit 14

Mississauga (Ontario) L5T 2G9

Tel. : +1 905.696.0656

info@geophysicsgpr.com

APRIL 2017 T-17998c

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1	INTRODUCTION.....	1
2	METHODOLOGY.....	2
2.1	Borehole Magnetometry.....	2
2.2	Seismic Down-hole.....	2
3	RESULTS & CONCLUSIONS.....	3

ILLUSTRATION INDEX

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Figure 2:	Down-hole seismic time-distance plot.....	6



1 INTRODUCTION

Geophysics GPR International Inc. has been requested by the Stantec to carry out a geophysical down-hole survey for the purpose of determining the presence of piles beneath the footings of the east abutment of the Saugeen River bridge on Hwy 89 west of Mount Forest, Ontario.

The survey was performed on April 12, 2017.

The investigations involved a combination of seismic and magnetometer measurements within a borehole to attempt to determine the presence of piles beneath the footing of the bridge abutment.

The following paragraphs describe the survey design, the principles of the test method, the methodology for interpreting the data, and provide a culmination of the results in chart format.



2 METHODOLOGY

2.1 Borehole Magnetometry

The total magnetic field is a sum of all magnetic fields generated by all conductive materials including the earth itself. A target will generate a magnetic field that will alter the earth's field in a localized area. Typically the largest influence is from the ferromagnetic metals. By recording the magnetic field data, the perturbations in the earth's field can be quantified.

The range of influence for a magnetometer is anomaly dependent. This means that a larger metallic object can be detected at a greater distance than a smaller object.

A BVM-03 three-component borehole fluxgate magnetometer was employed for this survey. The magnetometer was used to record the magnetic field data. The magnetic data along with the borehole tool depth were recorded at 0.5 s intervals.

2.2 Seismic Down-hole

The seismic down-hole method relies on the accurate measurement of the transit time for a generated wave to travel from a shot-point on the surface to a receiver (geophone) at sequential depths within a borehole. The velocities at which the waves propagate are then determined from the arrival times of the impulse signals. Arrival time records are recorded separately for waves with preferential shear (S) wave components and compressional (P) wave components. The seismic "P" wave velocity depends mainly on volumetric elastic ratio of the constituent soil particles and pore water. The seismic "S" wave velocity depends more on the structural elasticity of the material, which is influenced by the size, form and tightness of the particles (for the case of unconsolidated sediments). Unlike the P-wave, a polarized S-wave is easily generated.

A tri-axial geophone, containing two orthogonal horizontal geophones, for detecting the shear (S) wave arrivals, and a vertical geophone for detecting the compressional (P) wave arrivals, was used as the receiver. The geophone was held firm to the borehole casing by a motorized wall-lock.

Data were recorded with an ABEM Terraloc Mark 6 seismograph. The sampling interval was set to 25 μ s with 8192 samples for a total record length of 204 ms with a pre-trigger delay of 10 ms. The seismic source is typically offset from the borehole by 1 to 3 m.

Interpretation of the down-hole seismic data involves identifying the first arrival times of the P-waves and/or S-waves from the shot records at each depth interval.

The preferred method for analyzing down-hole data is to produce time-distance plots and calculate the velocities from the slope of the best-fit lines. The selection of the best-fit lines can be visually interpreted by the analyst or can be computer aided.



3 RESULTS & CONCLUSIONS

At the request of Stantec, Geophysics GPR carried out a series of geophysical measurements within a borehole prepared by the client. The purpose of the investigation was to assess the potential presence of piles beneath the footings of the east abutment of the Saugeen River Bridge on Hwy 89, west of Mount Forest, Ontario.

The testing was completed in Borehole 17-1 which was located on the east side of the bridge approximately 3.6 m east of the bridge joint and approximately 1.6 m north of the concrete curb. The cased and grouted borehole was drilled to a depth of approximately 15 m below grade corresponding to a depth of approximately 8 m below the existing abutment foundation (based on available drawings).

Two different methodologies were employed to determine the presence of piles.

The magnetometer survey was used to locate metal content (e.g. steel piles) at depth. Figure 2 plots the magnetic response (total magnetic field, horizontal and vertical components of the magnetic field, and the magnetic susceptibility) versus depth. The regional background magnetic field values for the site location, as calculated from Natural Resources Canada Magnetic field calculator, are overlain on the data plot. The total field value is expected to be on the order of 54,170 nT. This is in very good agreement with the background magnetic field levels measured on site at depth. Within the magnetic data set there are three notable zones:

- 1) 0 to 5 m: Lower total field and vertical component values and increasing horizontal component –
Interpreted to be a response to variety of nearby metal (e.g. guard rails, reinforced concrete);
- 2) 5 to 7 m: Increasing total field and vertical components and a peak in the horizontal component –
Interpreted to be a response to the existing reinforced concrete footing;
- 3) Below 7.5 m: Relatively stable magnetic field response -
Interpreted to be background magnetic field values.

Based on the magnetic data set, there not appear to be steel in the vicinity of the borehole beyond approximately 6.75 m to 7.5 m below grade.

The seismic survey was used to measure the arrival time of a wave transmitted from the surface to a vibration sensor at increasing depths. The arrival times of the waves allow a velocity of wave propagation to be measured. Shot records with preferential shear-waves (S-waves) and compressional (P-waves) were recorded; however, the compressional arrivals were much better defined. Accordingly, the time-distance plot and velocities presented represent the compressional (P) wave velocities. Steel piles would be expected to have a velocity on the order of >5000 m/s. Competent wood piles would be expected to have a velocity on the order of > 2800 m/s. Typical soil velocities range from 300 to 2500 m/s.

Figure 2 presents the results of the seismic down-hole measurements. The upper 4 m is interpreted to represent a combination of direct wave arrivals through the overburden/fill material and refracted arrivals through the concrete bridge abutment. The borehole SPT N values of approximately 6-7



within this depth interval indicated relatively loose material. The arrivals between 4 m and 8 m are interpreted to represent primarily refracted arrivals through the concrete footings. Below 8 m, a uniform velocity of approximately 2500 m/s was measured. This velocity is towards the upper limit for a dense till but below the typical range for competent timber piles. Borehole data provided by the client indicates very dense till material (SPT N > 80 blows) were encountered below depths of about 7.5 m. The likelihood of poor quality timber piles (velocity < 2800 m/s) being used in the very dense till material encountered within the borehole is highly unlikely, suggesting that there are no wooden piles present in the vicinity of the borehole below 8 m.



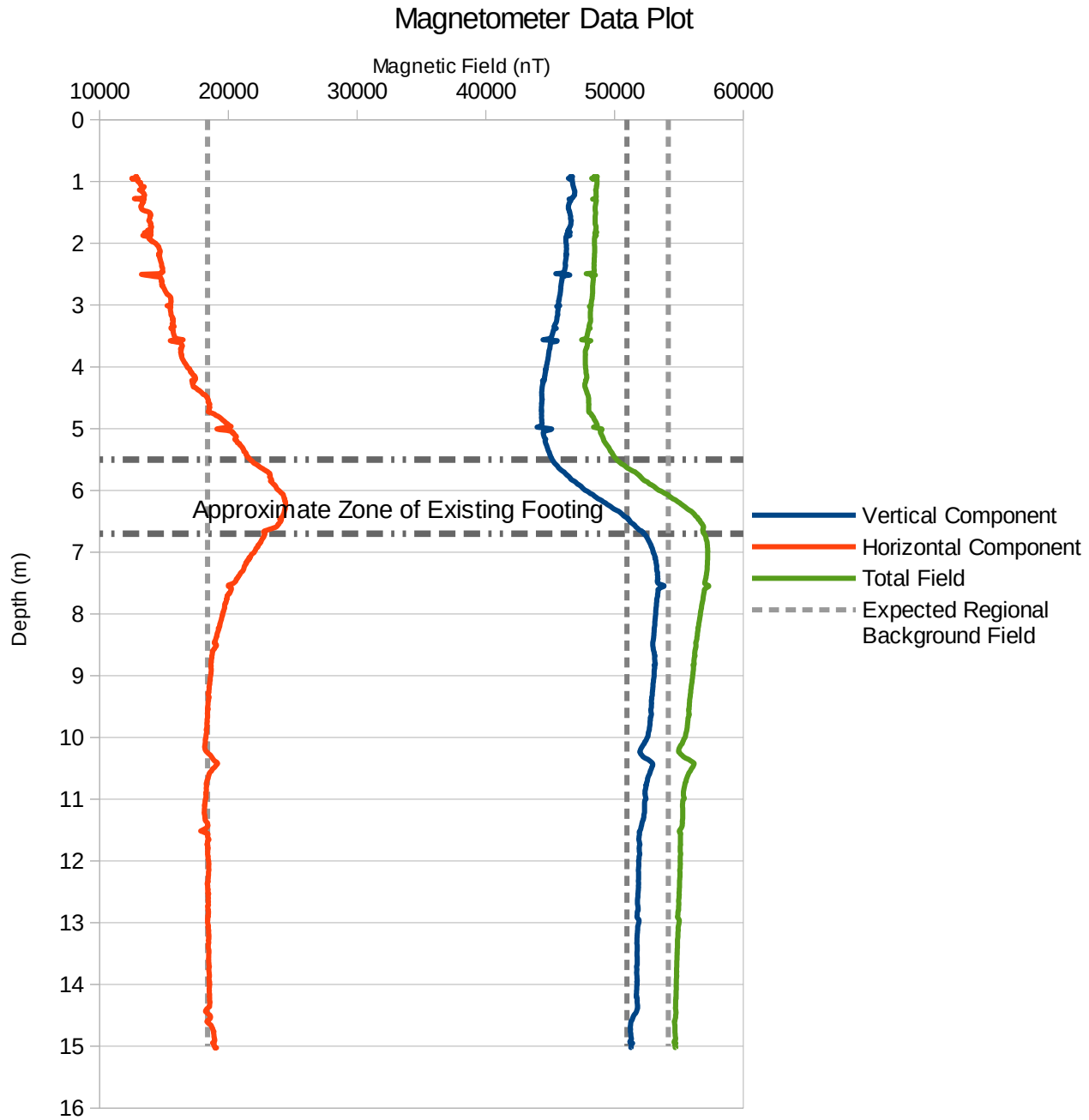


Figure 1: Down-hole magnetometer data plot



Seismic Downhole Test

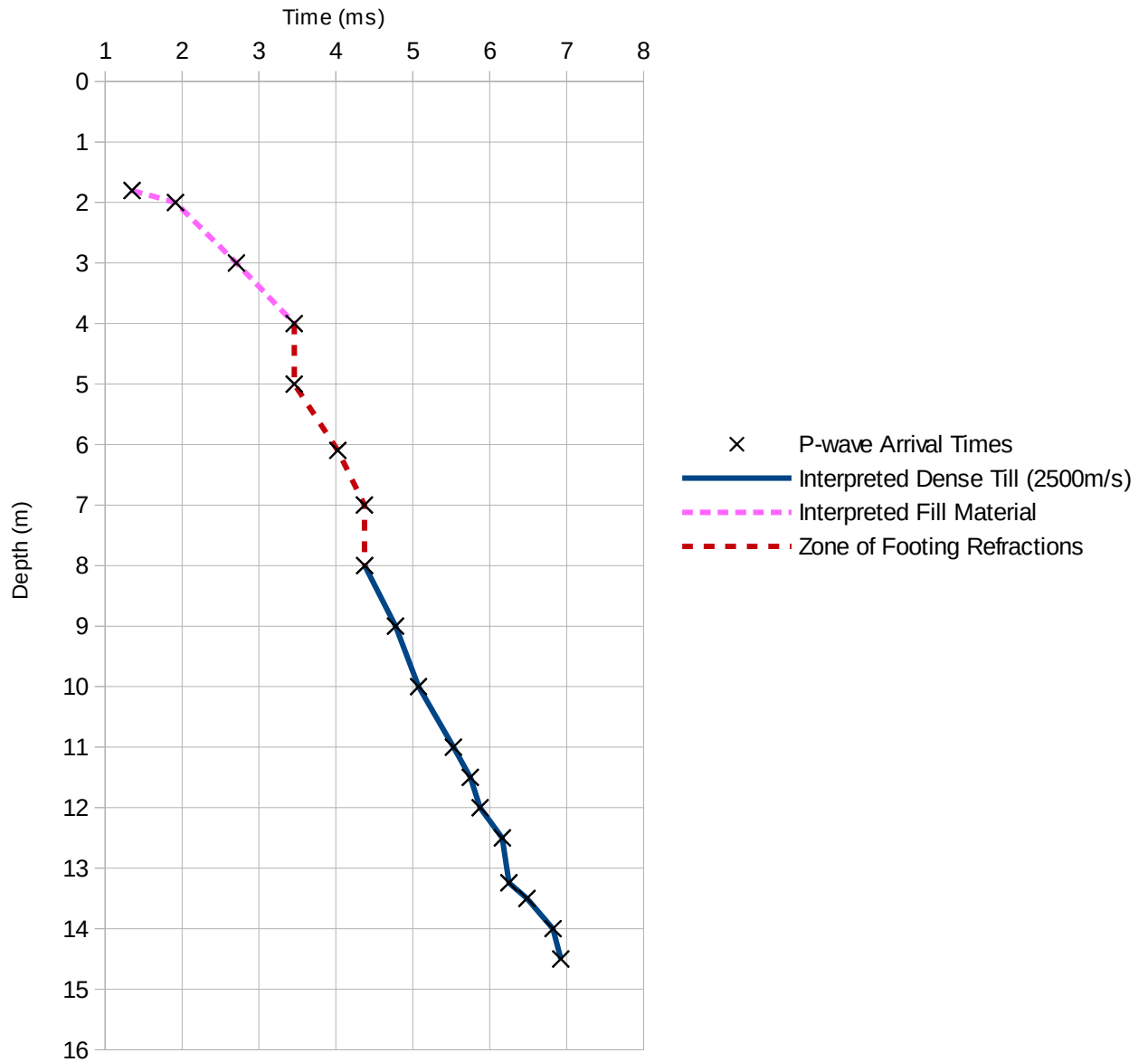
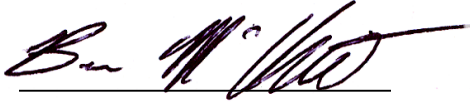


Figure 2: Down-hole seismic time-distance plot

The seismic and magnetometer measurements do not suggest that there are timber or steel piles present. It is noted that the test results are representative of the area surrounding the borehole. Piles offset from the borehole by distances greater than approximately 2 m to 3 m may not be detected. The borehole was located within the interior of the 'U' shaped footing to be as close as feasible to potential piles.

This report has been prepared by Ben McClement, P.Eng. and reviewed by Milan Situm, P.Geo.



Ben McClement, P.Eng.



APPENDIX E

Figure E1: Geotechnical Model (East Abutment)

Figure E2: Geotechnical Model (West Abutment)

Figure E3: Lateral Deflection for HP 310x110 -Strong Axis

Figure E4: Lateral Deflection for HP 310x110 -Weak Axis

Figure E5: p-y Curves for HP 310x110 – East Abutment

Figure E6: p-y Curves for HP 310x110 – West Abutment

Slope Stability Evaluation (West abutment):

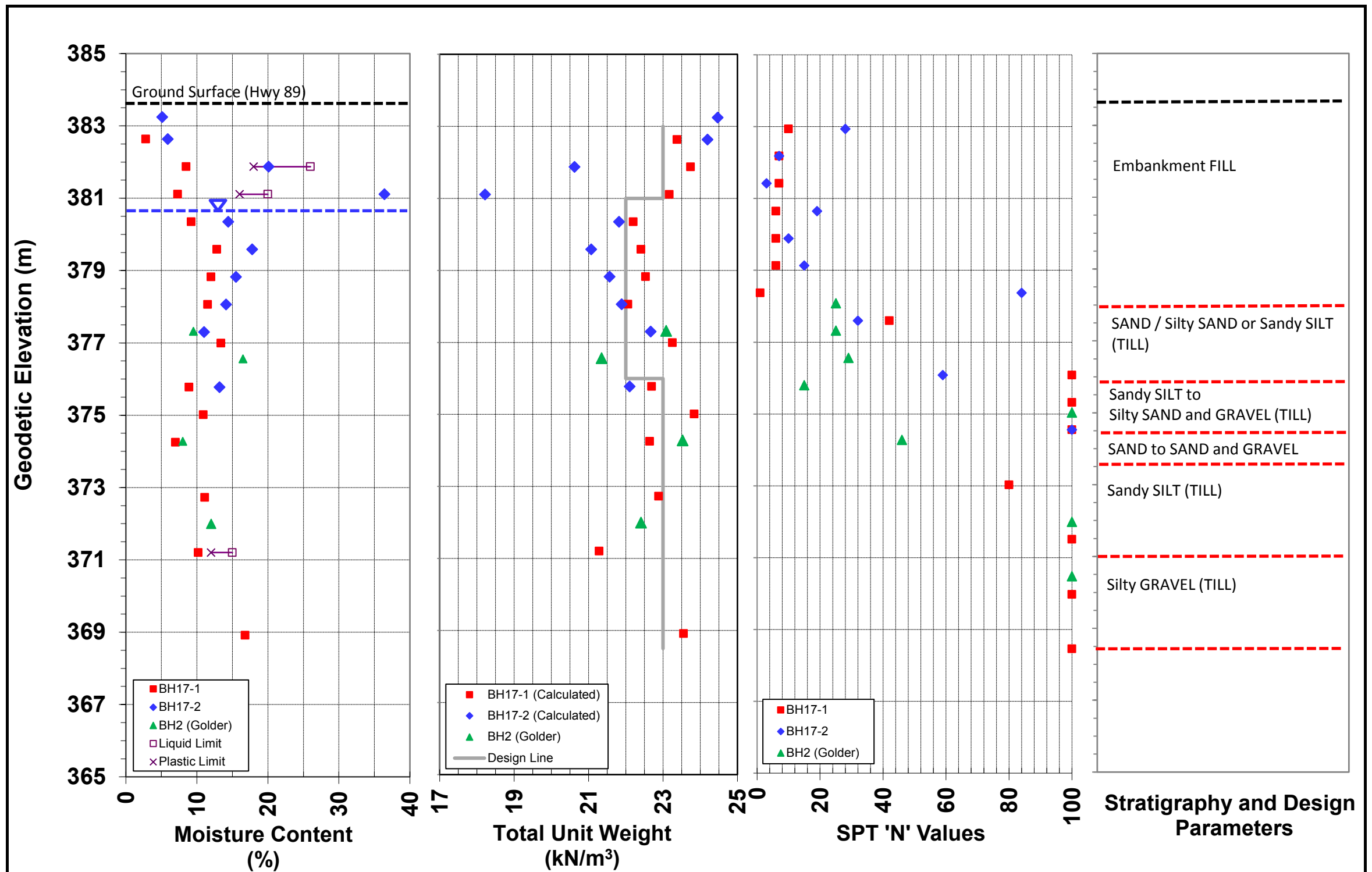
Figure E7: Static Slope Stability Analysis (Temporary Condition – 2H:1V sideslope)

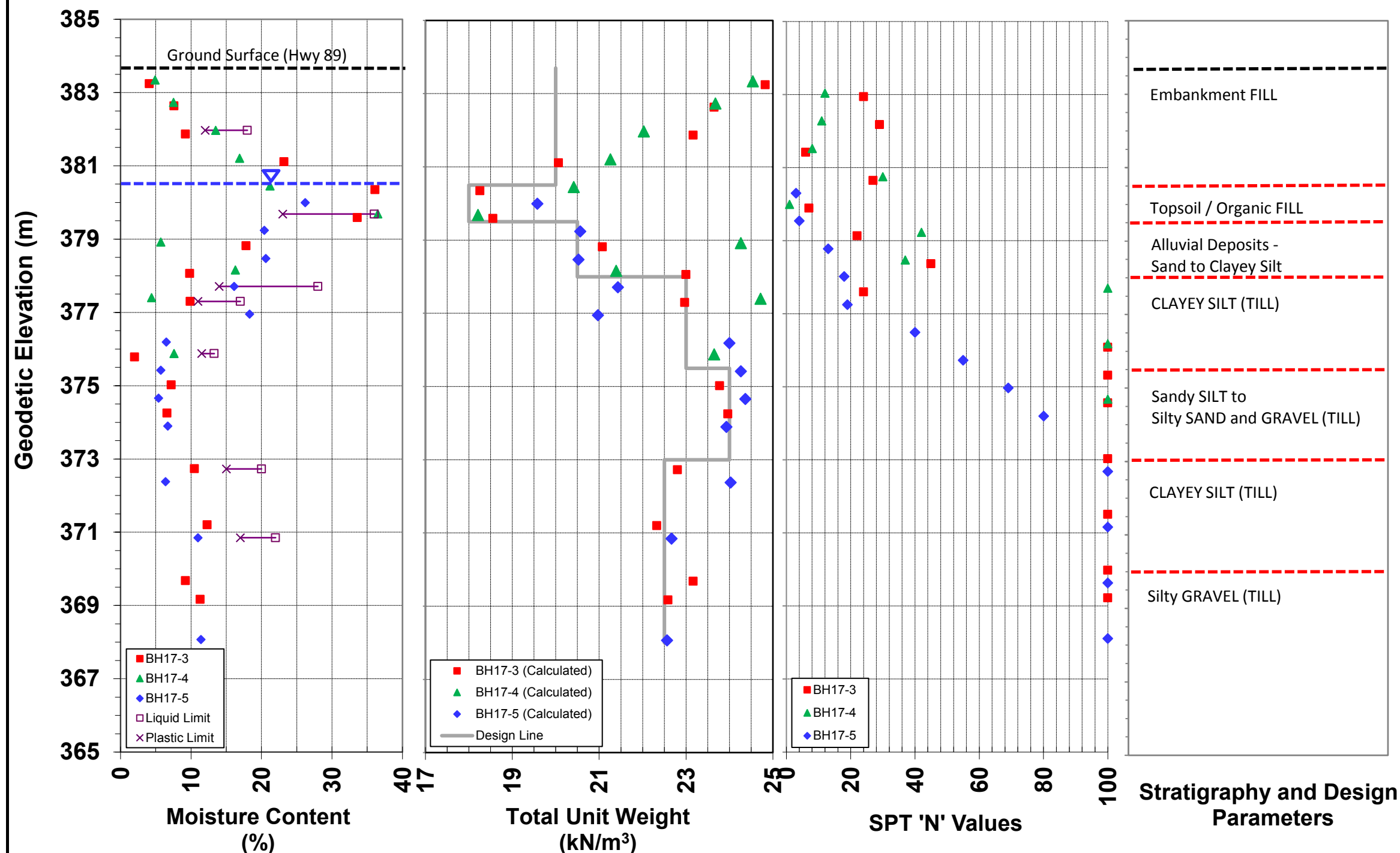
Figure E8: Static Slope Stability Analysis (Final Condition)

Figure E9: Static Slope Stability Analysis ((Temporary Condition – 1.5H:1V
sideslope)

Tables E-1: Load vs Lateral Deflection of HP310x110 Piles at Various Depths – East
Abutment

Tables E-2: Load vs Lateral Deflection of HP310x110 Piles at Various Depths – West
Abutment





LPile Results - Deformed Shapes of Piles

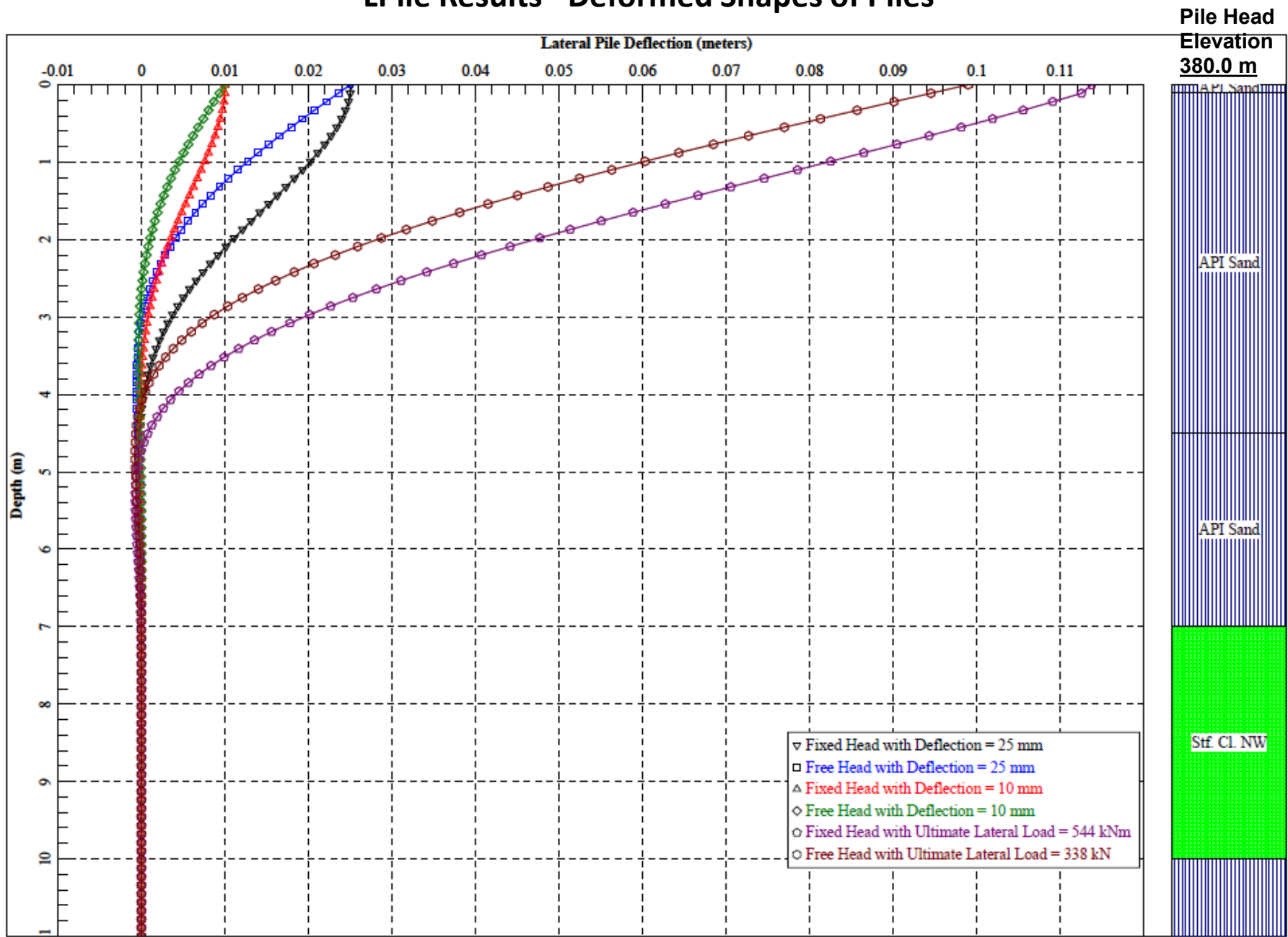


Figure E3
Lateral Deflection of H-Pile 310×110 - Strong Axis

LPile Results - Deformed Shapes of Piles

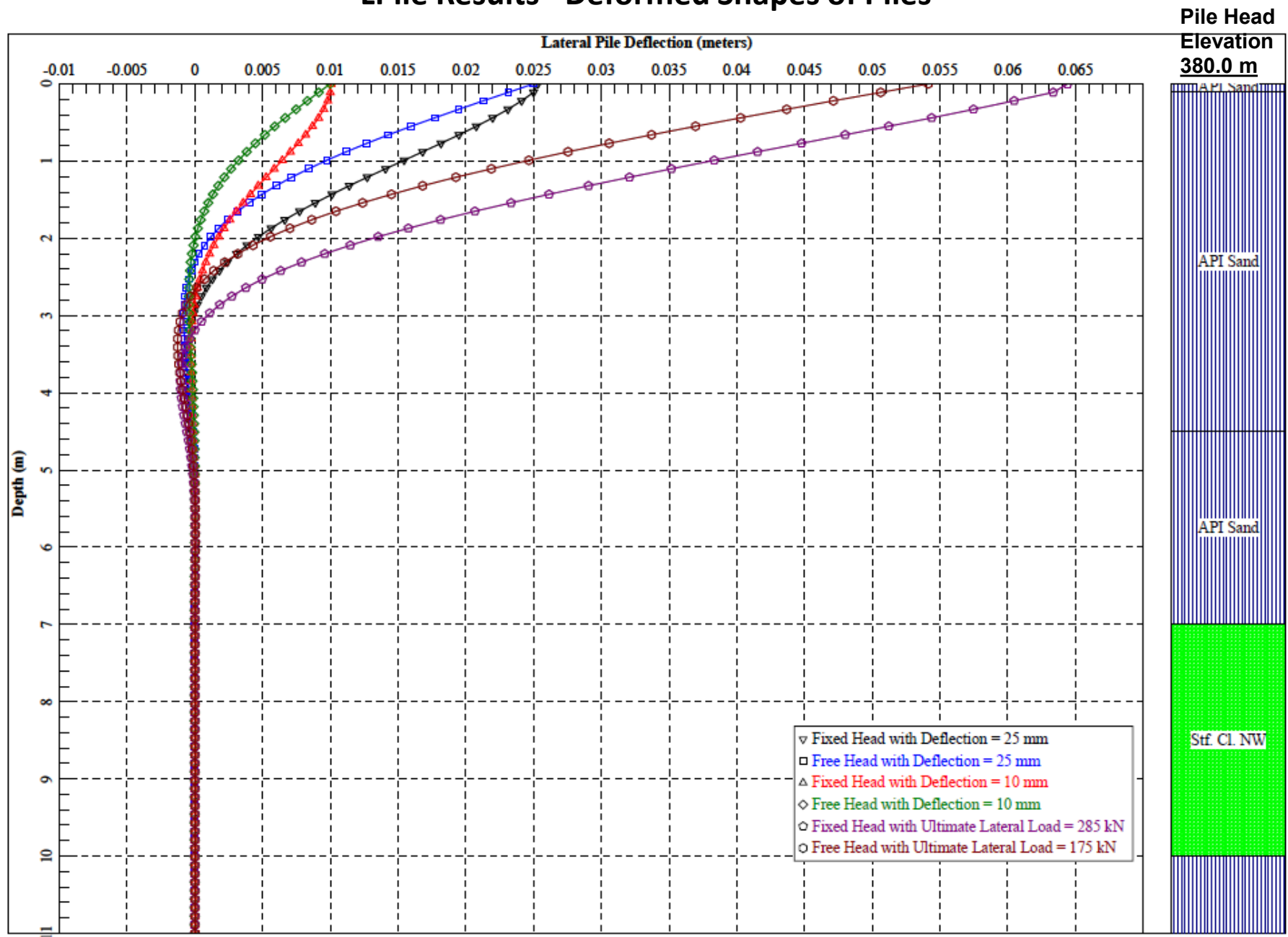


Figure E4
Lateral Deflection of H-Pile 310×110 - Weak Axis

L-Pile Results - P-Y curves

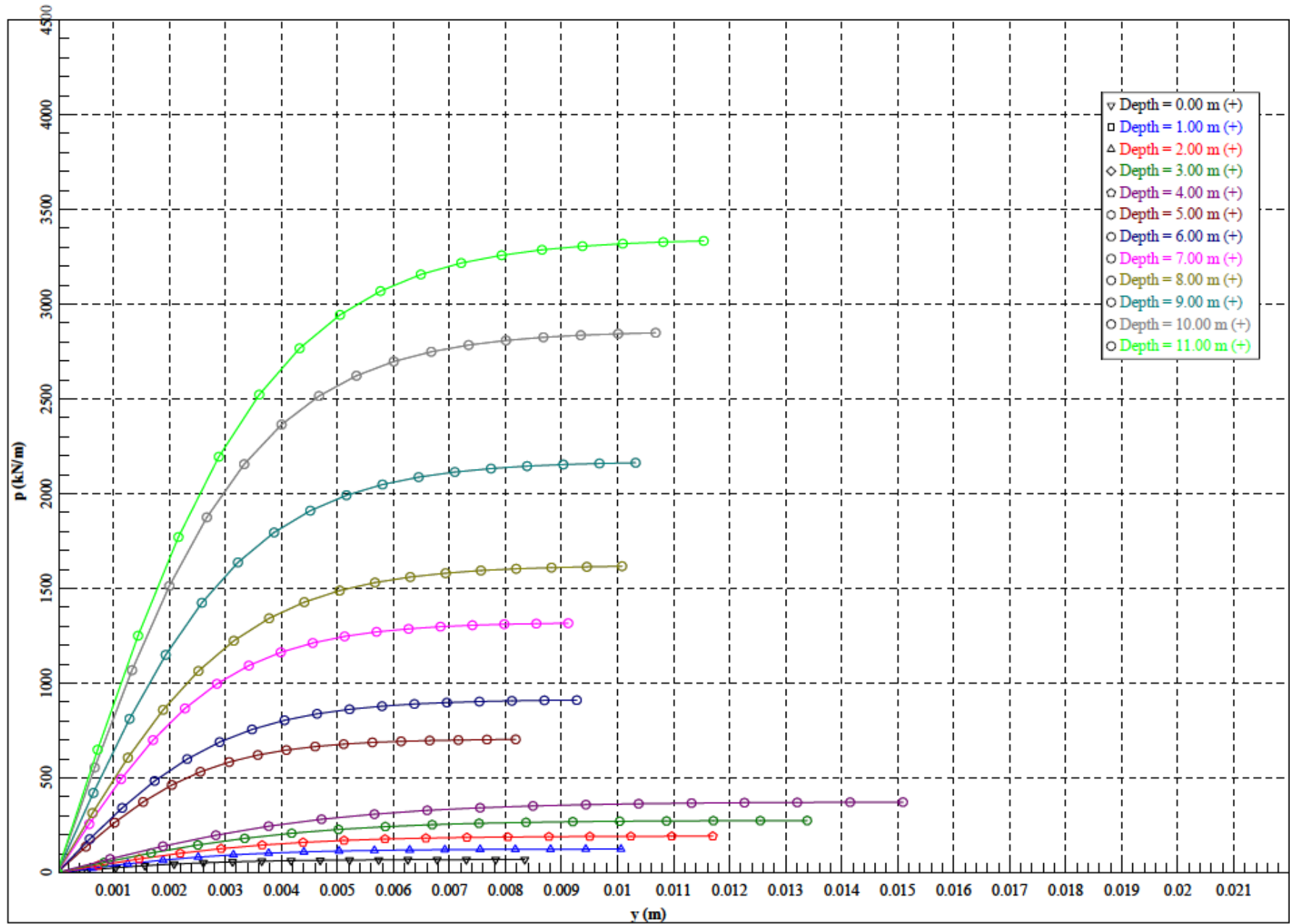


Figure E5
P-Y Curves of H-Pile 310x110 - East Abutment

L-Pile Results - P-Y Curves

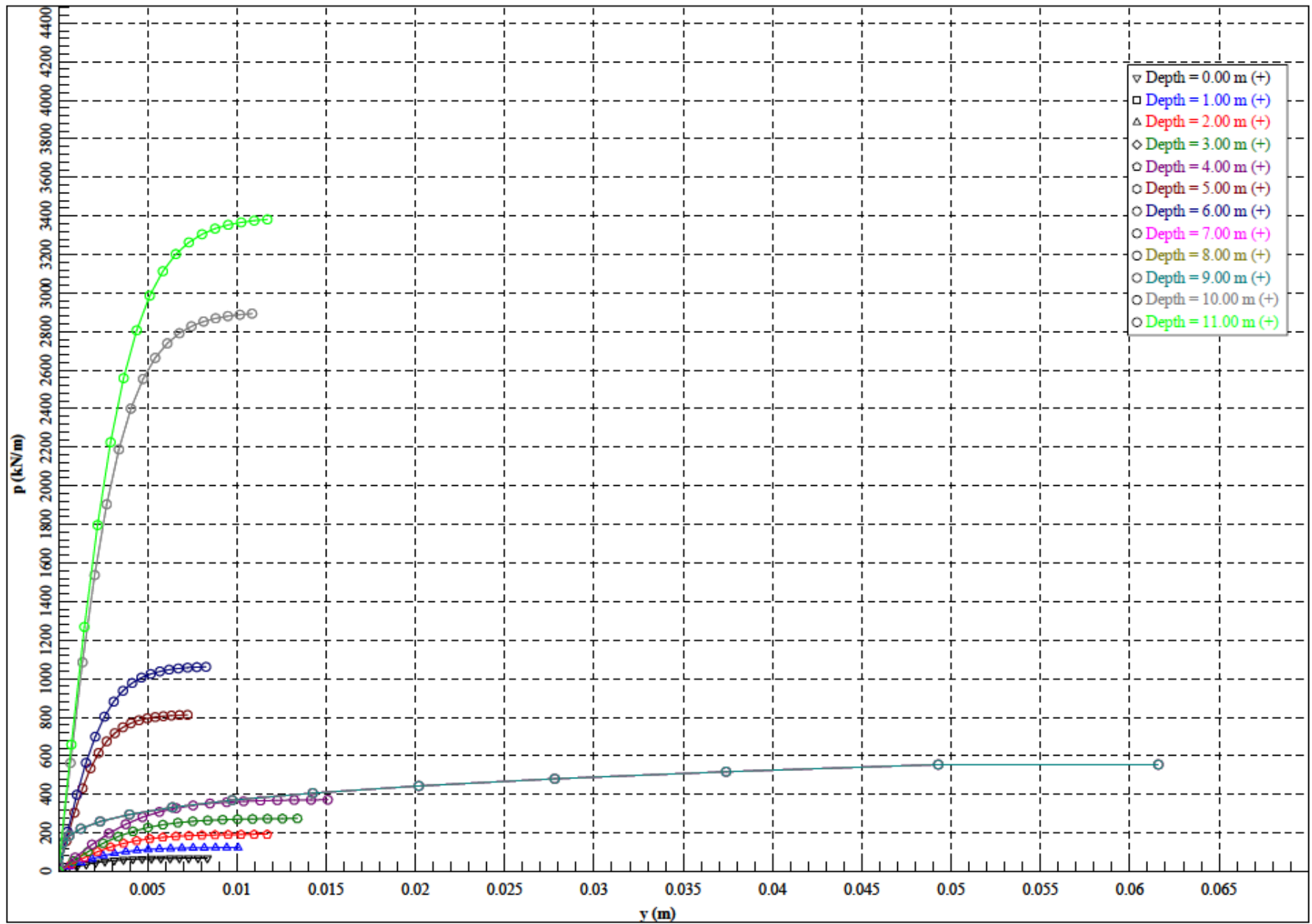
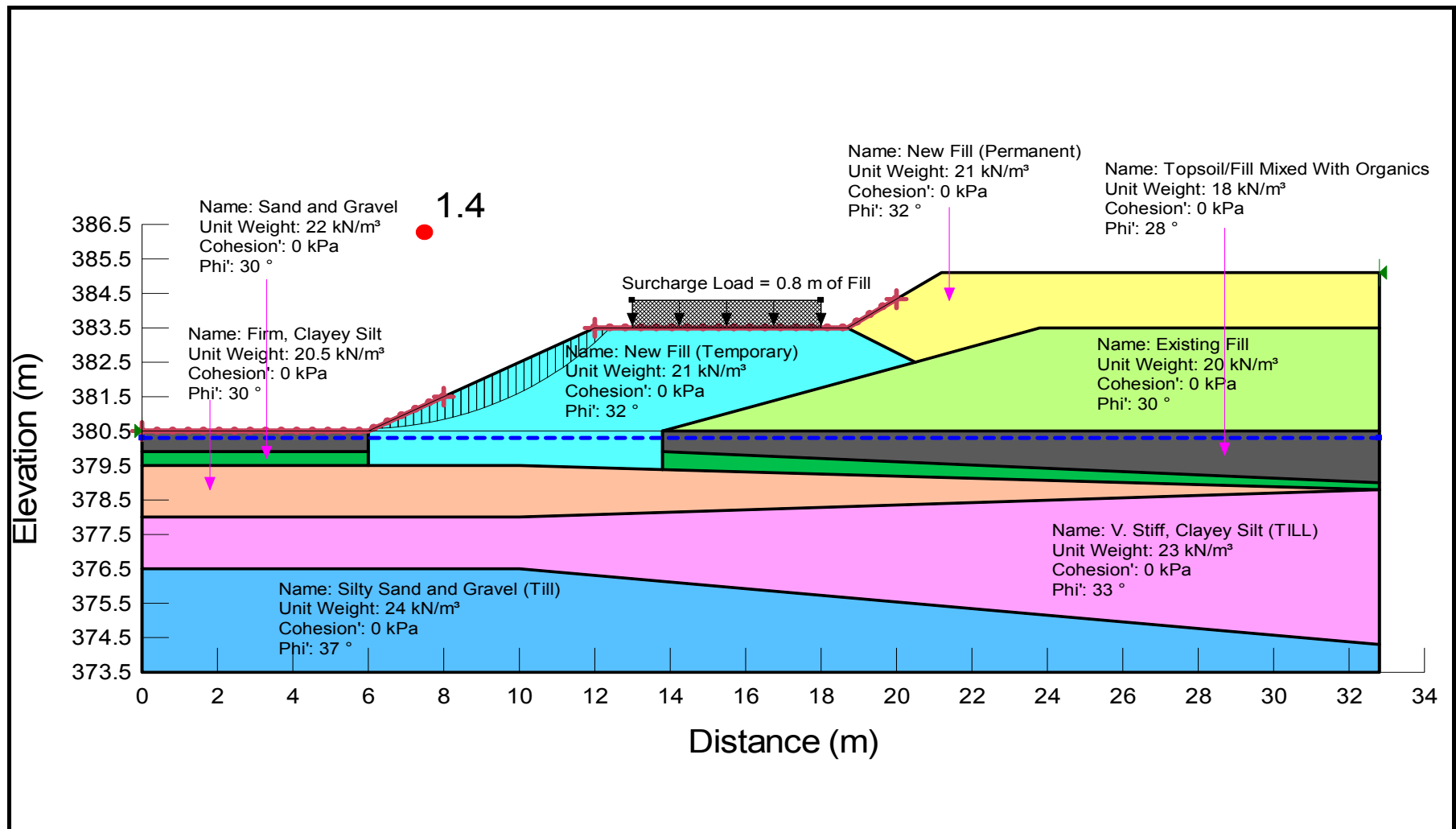


Figure E6
P-Y Curves of H-Pile 310x110 - West Abutment

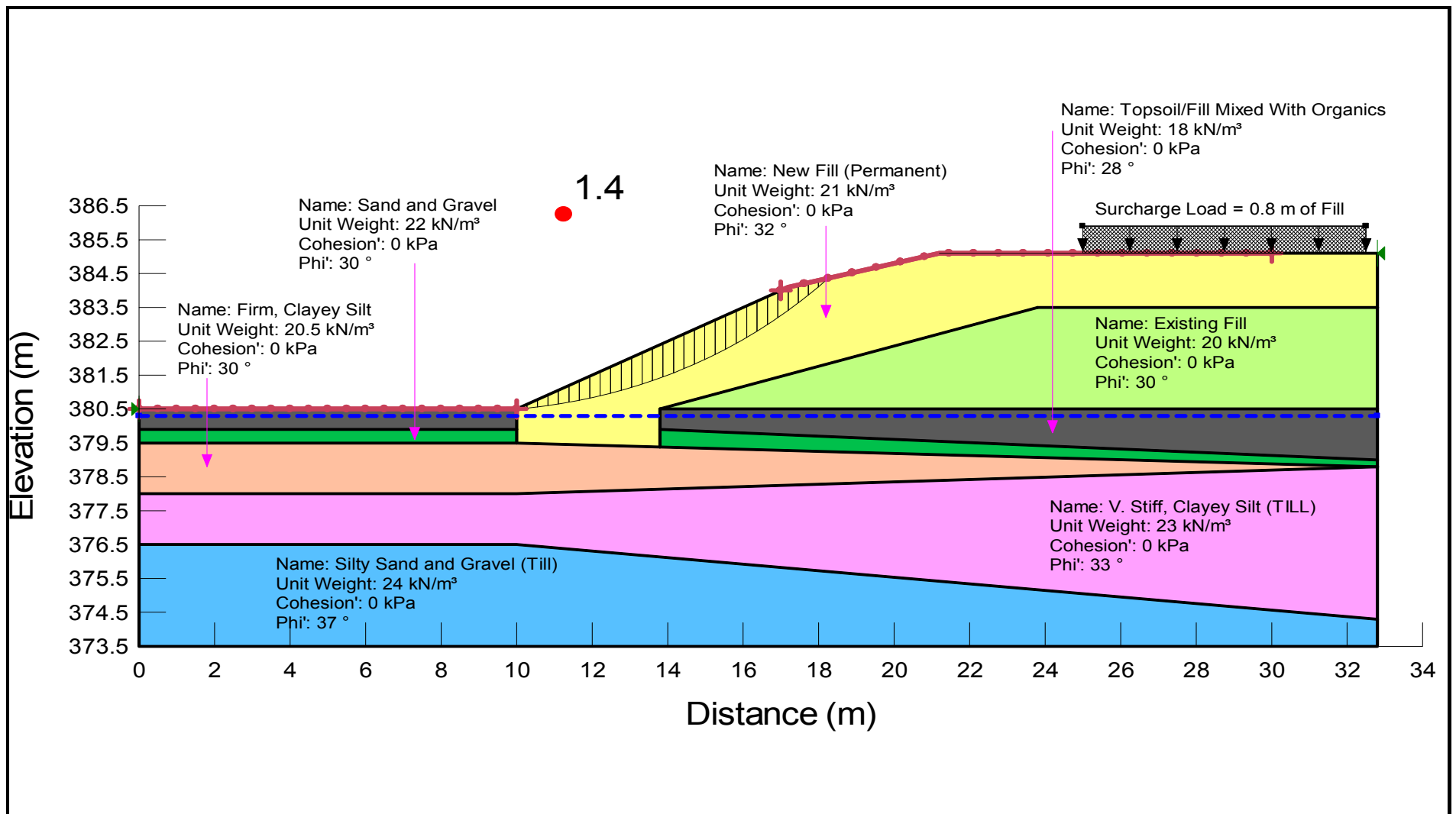


STATIC SLOPE STABILITY ANALYSIS (Temporary Condition - 2H:1V Sideslope) Highway 89 Bridge at South Saugeen River

Figure E7

Project No. 165001038

GWP No. 3093-12-00

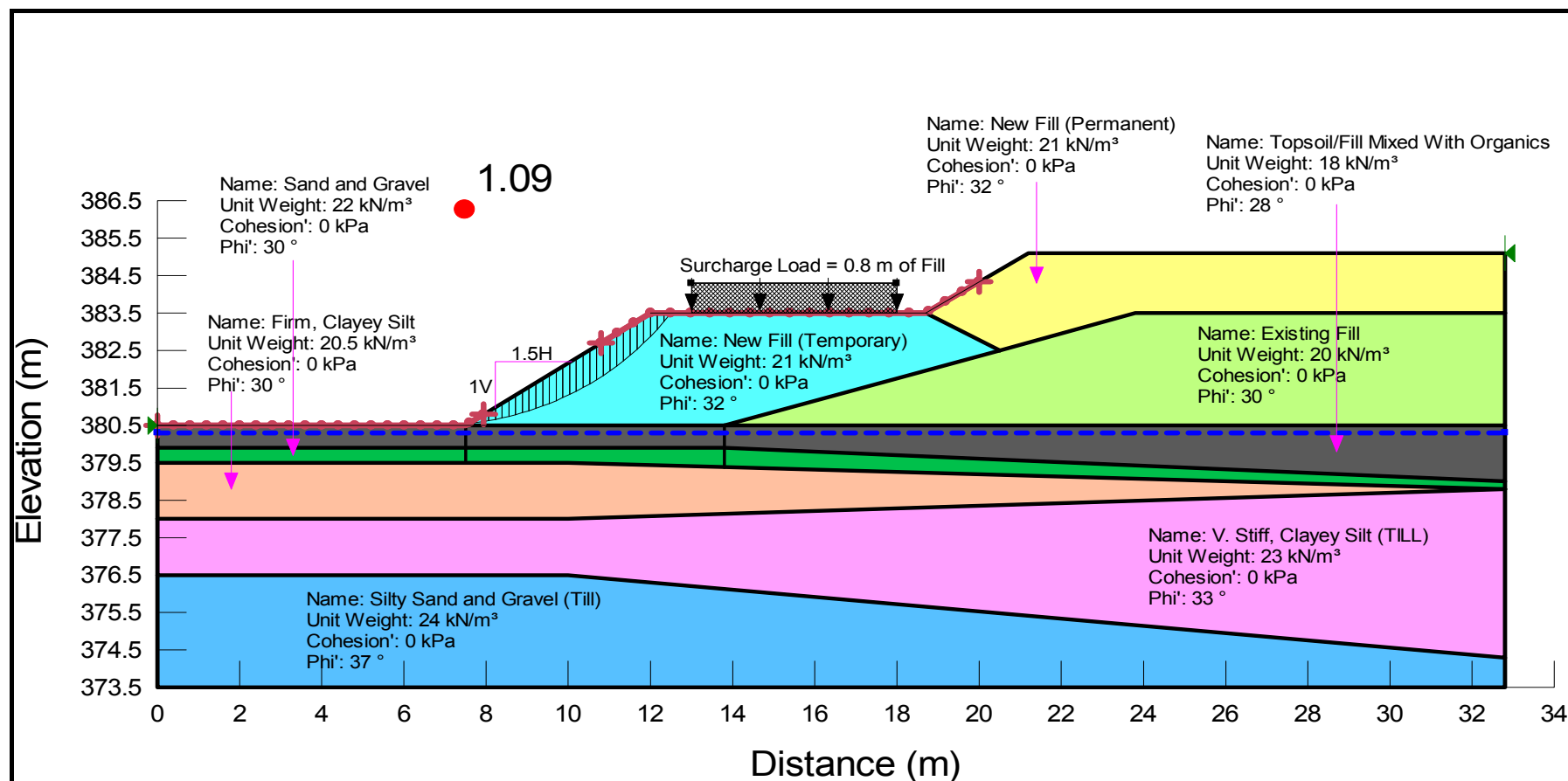


STATIC SLOPE STABILITY ANALYSIS
 (Final Condition)
 Highway 89 Bridge at South Saugeen River

Figure E8

Project No. 165001038

GWP No. 3093-12-00



STATIC SLOPE STABILITY ANALYSIS (Temporary Condition - 1.5H:1V Sideslope) Highway 89 Bridge at South Saugeen River

Figure E9

Project No. 165001038

GWP No. 3093-12-00

Table E-1
Load Intensity p (kN/m) vs Lateral Deflection y (m) of HP 310x110 Piles
at Various Depths below Pile Head - East Abutment

Depth Below Pile Head (m)	Curve Points																	
		1	2	3	4	5	6	7	8									
0.0	Y	0.0000	0.0005	0.0010	0.0016	0.0021	0.0026	0.0031	0.0037	0.0042	0.0047	0.0052	0.0057	0.0063	0.0068	0.0073	0.0078	0.0084
	P	0.0	13.3	25.7	36.4	45.1	51.8	56.9	60.5	63.1	64.9	66.1	67.0	67.6	68.0	68.2	68.4	68.5
1.0	Y	0.0000	0.0006	0.0013	0.0019	0.0025	0.0031	0.0038	0.0044	0.0050	0.0057	0.0063	0.0069	0.0076	0.0082	0.0088	0.0094	0.0101
	P	0.0	24.1	46.5	65.8	81.6	93.8	102.9	109.4	114.1	117.4	119.6	121.1	122.2	122.9	123.4	123.7	123.9
2.0	Y	0.0000	0.0007	0.0015	0.0022	0.0029	0.0037	0.0044	0.0051	0.0059	0.0066	0.0073	0.0080	0.0088	0.0095	0.0102	0.0110	0.0117
	P	0.0	37.4	72.1	102.1	126.5	145.4	159.5	169.6	176.9	181.9	185.4	187.8	189.4	190.5	191.3	191.8	192.1
3.0	Y	0.0000	0.0008	0.0017	0.0025	0.0034	0.0042	0.0050	0.0059	0.0067	0.0075	0.0084	0.0092	0.0100	0.0109	0.0117	0.0126	0.0134
	P	0.0	53.5	103.1	146.0	180.9	207.9	228.0	242.6	252.9	260.2	265.1	268.5	270.9	272.5	273.5	274.3	274.8
4.0	Y	0.0000	0.0009	0.0019	0.0028	0.0038	0.0047	0.0057	0.0066	0.0076	0.0085	0.0094	0.0104	0.0113	0.0123	0.0132	0.0142	0.0151
	P	0.0	72.4	139.5	197.5	244.8	281.4	308.6	328.3	342.3	352.1	358.8	363.4	366.6	368.7	370.2	371.2	371.8
5.0	Y	0.0000	0.0005	0.0010	0.0015	0.0021	0.0026	0.0031	0.0036	0.0041	0.0046	0.0051	0.0056	0.0061	0.0067	0.0072	0.0077	0.0082
	P	0.0	136.7	263.6	373.3	462.6	531.7	583.2	620.5	646.9	665.4	678.1	686.8	692.8	696.8	699.6	701.5	702.7
6.0	Y	0.0000	0.0006	0.0012	0.0017	0.0023	0.0029	0.0035	0.0041	0.0046	0.0052	0.0058	0.0064	0.0070	0.0075	0.0081	0.0087	0.0093
	P	0.0	177.1	341.4	483.5	599.1	688.7	755.4	803.7	837.9	861.8	878.3	889.6	897.3	902.6	906.1	908.5	910.2
7.0	Y	0.0000	0.0006	0.0011	0.0017	0.0023	0.0029	0.0034	0.0040	0.0046	0.0051	0.0057	0.0063	0.0068	0.0074	0.0080	0.0086	0.0091
	P	0.0	256.1	493.7	699.2	866.4	995.9	1092.4	1162.3	1211.7	1246.3	1270.1	1286.5	1297.7	1305.3	1310.4	1313.9	1316.3
8.0	Y	0.0000	0.0006	0.0013	0.0019	0.0025	0.0032	0.0038	0.0044	0.0050	0.0057	0.0063	0.0069	0.0076	0.0082	0.0088	0.0095	0.0101
	P	0.0	314.6	606.4	858.8	1064.2	1223.3	1341.8	1427.5	1488.3	1530.7	1560.1	1580.2	1593.9	1603.2	1609.5	1613.8	1616.7
9.0	Y	0.0000	0.0006	0.0013	0.0019	0.0026	0.0032	0.0039	0.0045	0.0052	0.0058	0.0065	0.0071	0.0077	0.0084	0.0090	0.0097	0.0103
	P	0.0	420.9	811.3	1148.9	1423.8	1636.6	1795.1	1909.9	1991.2	2047.9	2087.2	2114.0	2132.4	2144.9	2153.3	2159.1	2163.0
10.0	Y	0.0000	0.0007	0.0013	0.0020	0.0027	0.0033	0.0040	0.0047	0.0053	0.0060	0.0067	0.0073	0.0080	0.0087	0.0093	0.0100	0.0107
	P	0.0	554.3	1068.4	1513.0	1874.9	2155.2	2364.0	2515.1	2622.1	2696.9	2748.5	2783.9	2808.1	2824.5	2835.7	2843.2	2848.4
11.0	Y	0.0000	0.0007	0.0014	0.0022	0.0029	0.0036	0.0043	0.0051	0.0058	0.0065	0.0072	0.0079	0.0087	0.0094	0.0101	0.0108	0.0115
	P	0.0	648.6	1250.3	1770.7	2194.2	2522.2	2766.5	2943.4	3068.7	3156.2	3216.6	3258.0	3286.3	3305.5	3318.6	3327.4	3333.4

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The Table E-1 presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m) of a HP 310x110 pile for the east abutment. The p-y points can be used for the structural design of the pile in response to lateral loads.

Table E-2
Load Intensity p (kN/m) vs Lateral Deflection y (m) of HP 310x110 Piles
at Various Depths below Pile Head - West Abutment

Depth Below Pile Head (m)	Curve Points																	
		1	2	3	4	5	6	7	8									
0.0	Y	0.0000	0.0005	0.0010	0.0016	0.0021	0.0026	0.0031	0.0037	0.0042	0.0047	0.0052	0.0057	0.0063	0.0068	0.0073	0.0078	0.0084
	P	0.0	13.3	25.7	36.4	45.1	51.8	56.9	60.5	63.1	64.9	66.1	67.0	67.6	68.0	68.2	68.4	68.5
1.0	Y	0.0000	0.0006	0.0013	0.0019	0.0025	0.0031	0.0038	0.0044	0.0050	0.0057	0.0063	0.0069	0.0076	0.0082	0.0088	0.0094	0.0101
	P	0.0	24.1	46.5	65.8	81.6	93.8	102.9	109.4	114.1	117.4	119.6	121.1	122.2	122.9	123.4	123.7	123.9
2.0	Y	0.0000	0.0007	0.0015	0.0022	0.0029	0.0037	0.0044	0.0051	0.0059	0.0066	0.0073	0.0080	0.0088	0.0095	0.0102	0.0110	0.0117
	P	0.0	37.4	72.1	102.1	126.5	145.4	159.5	169.6	176.9	181.9	185.4	187.8	189.4	190.5	191.3	191.8	192.1
3.0	Y	0.0000	0.0008	0.0017	0.0025	0.0034	0.0042	0.0050	0.0059	0.0067	0.0075	0.0084	0.0092	0.0100	0.0109	0.0117	0.0126	0.0134
	P	0.0	53.5	103.1	146.0	180.9	207.9	228.0	242.6	252.9	260.2	265.1	268.5	270.9	272.5	273.5	274.3	274.8
4.0	Y	0.0000	0.0009	0.0019	0.0028	0.0038	0.0047	0.0057	0.0066	0.0076	0.0085	0.0094	0.0104	0.0113	0.0123	0.0132	0.0142	0.0151
	P	0.0	72.4	139.5	197.5	244.8	281.4	308.6	328.3	342.3	352.1	358.8	363.4	366.6	368.7	370.2	371.2	371.8
5.0	Y	0.0000	0.0005	0.0009	0.0014	0.0018	0.0023	0.0027	0.0032	0.0036	0.0041	0.0045	0.0050	0.0054	0.0059	0.0063	0.0068	0.0072
	P	0.0	158.1	304.7	431.5	534.7	614.6	674.1	717.2	747.7	769.0	783.8	793.9	800.8	805.4	808.6	810.8	812.2
6.0	Y	0.0000	0.0005	0.0010	0.0016	0.0021	0.0026	0.0031	0.0036	0.0041	0.0047	0.0052	0.0057	0.0062	0.0067	0.0072	0.0078	0.0083
	P	0.0	206.5	398.1	563.8	698.7	803.1	880.9	937.2	977.1	1004.9	1024.2	1037.4	1046.4	1052.5	1056.7	1059.5	1061.4
7.0	Y	0.0000	0.0000	0.0000	0.0001	0.0002	0.0006	0.0013	0.0023	0.0040	0.0064	0.0097	0.0143	0.0202	0.0278	0.0374	0.0493	0.0616
	P	0.0	37.0	73.9	110.9	147.8	184.8	221.8	258.7	295.7	332.6	369.6	406.6	443.5	480.5	517.4	554.4	554.4
8.0	Y	0.0000	0.0000	0.0000	0.0001	0.0002	0.0006	0.0013	0.0023	0.0040	0.0064	0.0097	0.0143	0.0202	0.0278	0.0374	0.0493	0.0616
	P	0.0	37.0	73.9	110.9	147.8	184.8	221.8	258.7	295.7	332.6	369.6	406.6	443.5	480.5	517.4	554.4	554.4
9.0	Y	0.0000	0.0000	0.0000	0.0001	0.0002	0.0006	0.0013	0.0023	0.0040	0.0064	0.0097	0.0143	0.0202	0.0278	0.0374	0.0493	0.0616
	P	0.0	37.0	73.9	110.9	147.8	184.8	221.8	258.7	295.7	332.6	369.6	406.6	443.5	480.5	517.4	554.4	554.4
10.0	Y	0.0000	0.0007	0.0014	0.0020	0.0027	0.0034	0.0041	0.0048	0.0054	0.0061	0.0068	0.0075	0.0081	0.0088	0.0095	0.0102	0.0109
	P	0.0	563.0	1085.1	1536.8	1904.3	2189.0	2401.1	2554.5	2663.3	2739.2	2791.6	2827.6	2852.1	2868.8	2880.2	2887.8	2893.0
11.0	Y	0.0000	0.0007	0.0015	0.0022	0.0029	0.0037	0.0044	0.0051	0.0059	0.0066	0.0073	0.0081	0.0088	0.0095	0.0102	0.0110	0.0117
	P	0.0	658.0	1268.4	1796.3	2226.0	2558.7	2806.6	2986.0	3113.1	3201.9	3263.2	3305.2	3333.9	3353.4	3366.7	3375.6	3381.7

The response of a pile to lateral loads is a nonlinear relationship. The p-y geotechnical approach was used to estimate the anticipated deformation of a pile within the soil medium. The p-y curves represent the load-deformation characteristics of elastic-plastic springs with a non-linear response within the elastic range. These non-linear elastic-plastic springs provide a more realistic representation or modeling of the soil pressure response against the face of the pile. The Table E-2 presents the Load Intensity per unit length of pile p (kN/m) vs Lateral Deflection y (m) of a HP 310x110 pile for the west abutment. The p-y points can be used for the structural design of the pile in response to lateral loads.

APPENDIX F

2015 National Building Code Seismic Hazard Calculation

2015 National Building Code Seismic Hazard Calculation

INFORMATION: Eastern Canada English (613) 995-5548 français (613) 995-0600 Facsimile (613) 992-8836
Western Canada English (250) 363-6500 Facsimile (250) 363-6565

June 19, 2017

Site: 43.9724 N, 80.7891 W User File Reference: Highway 89 Bridge over South Saugeen River

Requested by: Abraham Mineneh, Stantec

National Building Code ground motions: 2% probability of exceedance in 50 years (0.000404 per annum)

Sa(0.05)	Sa(0.1)	Sa(0.2)	Sa(0.3)	Sa(0.5)	Sa(1.0)	Sa(2.0)	Sa(5.0)	Sa(10.0)	PGA (g)	PGV (m/s)
0.067	0.093	0.091	0.079	0.066	0.041	0.021	0.0051	0.0023	0.053	0.053

Notes. Spectral ($S_a(T)$, where T is the period in seconds) and peak ground acceleration (PGA) values are given in units of g (9.81 m/s^2). Peak ground velocity is given in m/s . Values are for "firm ground" (NBCC 2015 Site Class C, average shear wave velocity 450 m/s). NBCC2015 and CSAS6-14 values are specified in **bold** font. Three additional periods are provided - their use is discussed in the NBCC2015 Commentary. Only 2 significant figures are to be used. *These values have been interpolated from a 10-km-spaced grid of points. Depending on the gradient of the nearby points, values at this location calculated directly from the hazard program may vary. More than 95 percent of interpolated values are within 2 percent of the directly calculated values.*

Ground motions for other probabilities:

Probability of exceedance per annum	0.010	0.0021	0.001
Probability of exceedance in 50 years	40%	10%	5%
Sa(0.05)	0.0081	0.026	0.040
Sa(0.1)	0.013	0.038	0.059
Sa(0.2)	0.015	0.041	0.060
Sa(0.3)	0.014	0.036	0.052
Sa(0.5)	0.011	0.031	0.044
Sa(1.0)	0.0054	0.018	0.027
Sa(2.0)	0.0023	0.0085	0.014
Sa(5.0)	0.0005	0.0019	0.0033
Sa(10.0)	0.0004	0.0009	0.0014
PGA	0.0072	0.022	0.033
PGV	0.0063	0.022	0.034

References

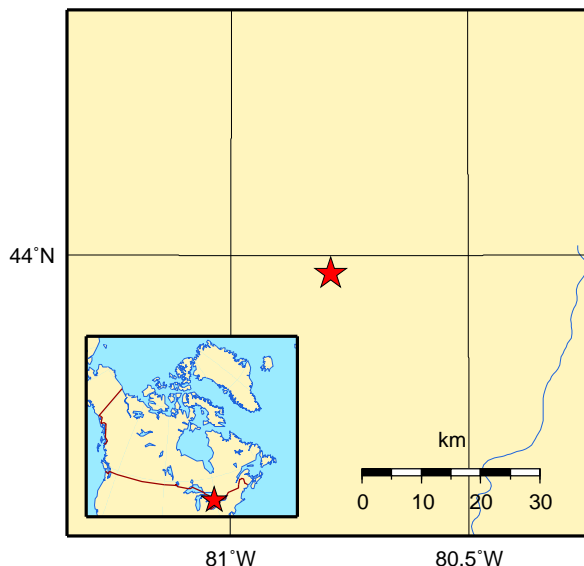
National Building Code of Canada 2015 NRCC no. 56190;
Appendix C: Table C-3, Seismic Design Data for Selected Locations in Canada

User's Guide - NBC 2015, Structural Commentaries NRCC no. xxxxxx (in preparation)
Commentary J: Design for Seismic Effects

Geological Survey of Canada Open File 7893 Fifth Generation Seismic Hazard Model for Canada: Grid values of mean hazard to be used with the 2015 National Building Code of Canada

See the websites www.EarthquakesCanada.ca and www.nationalcodes.ca for more information

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APPENDIX G

Sample NSSP – High-Strain Dynamic Testing, Deep Foundations

HIGH-STRAIN DYNAMIC TESTING, DEEP FOUNDATIONS – Item No.

Special Provision

Amendment to OPSS 903, April 2016

903.02 REFERENCES

Section 903.02 of OPSS 903 is amended by the addition of the following under **ASTM International**:

D 4945-17 Standard Test Method for High-Strain Dynamic Testing of Deep Foundations

903.03 DEFINITIONS

Section 903.03 of OPSS 903 is amended by the addition of the following:

High Strain Dynamic Testing means a method of evaluating the subsequent strain and acceleration response and integrity of deep foundations (herein referred to as piles) after applying an impact force, and measuring the performance of the drive assembly system. It is a form of load testing and involves the instrumenting and application of dynamic loads to a pile.

903.04 DESIGN AND SUBMISSION REQUIREMENTS

903.04.02 Submission Requirements

Subsection 903.04.02 of OPSS 903 is amended by the addition of the following clause:

903.04.02.07 High-Strain Dynamic Testing

Prior to commencing high-strain dynamic testing, calibration certificates of all equipment used shall be submitted to the Contract Administrator. All equipment used shall be in good working condition, and shall have been calibrated within the last 2 years according to ASTM D 4945. Equipment set-up may be completed by trained Piling Contractor personnel; however, testing shall be performed under the direction of a Professional Engineer with at least 5 years of experience in high-strain dynamic testing and holding a proficiency rating at the Intermediate level or better for Dynamic Measurement and Analysis Proficiency Test as administered by the Pile Driving Contractors Association (PDCA). After December 31, 2020, the Engineer shall be required to hold a proficiency rating level of Advanced or better.

A preliminary report on the test results and its analysis shall be submitted to the Contract Administrator on the same day of the testing. The analysis shall be based on a closed-form solution (Case Method or approved equivalent) or signal-matching analyses (CAPWAP or approved equivalent). As a minimum, the preliminary report shall include:

- a) Pile ultimate resistance in axial compression, and pile integrity.
- b) Calculated driving stresses.
- c) Transferred energy and hammer efficiency at the time of the test.

A final report shall be submitted to the Contract Administrator within 5 Calendar Days of the field testing. The final report and or supporting test forms shall be prepared in accordance to ASTM D4945 and shall include, but not be limited to, the following:

- a) Results of pile ultimate resistance in axial compression (including a summary of the toe and shaft resistances), and pile integrity based on signal-matching analyses (CAPWAP or approved equivalent), hammer performance and comparisons with any applicable static load test.
- b) Discussion and recommendations for soil setup/relaxation, and/or revised pile installation criteria.
- c) An appendix shall be included containing the following documents:
 - i. Pile installation record
 - ii. Reference subsurface information (borehole record)
 - iii. Pile location drawing
 - iv. Initial calibration check by the test computer unit
 - v. Test set up geometry

The report shall be signed and sealed by two Professional Engineers of the testing company, one of whom shall be identified as MTO's designated contact and one of whom shall have the required experience in high-strain dynamic testing and hold the required certificate of PDCA Proficiency Test.

903.07

CONSTRUCTION

903.07.02.07

Monitoring Driven Piles

903.07.02.07.03

Driving to a Specified Ultimate Resistance

903.07.02.07.03.01

General

Clause 903.07.02.07.03.01 of OPSS 903 is deleted in its entirety and replaced with the following:

When piles are specified to be driven to a specified ultimate resistance, the specified ultimate resistance shall be validated using high-strain dynamic testing at the end of initial drive (EOID). If the specified ultimate resistance is not achieved, retap/restrike should be conducted after sufficient time has passed to allow soil setup. The soil setup requirements are specified elsewhere in the Contract.

The results of the high-strain dynamic tests shall be submitted to the Contract Administrator who shall, in collaboration with the independent testing company, verify that the specified ultimate resistance has been achieved.

903.07.02.07.04 Wave Equation Analysis

Clause 903.07.02.07.04 is deleted in its entirety and replaced with the following:

903.07.02.07.04 Wave Equation Analysis and High-Strain Dynamic Testing

903.07.02.07.04 .01 Wave Equation Analysis

Prior to mobilizing piling equipment to the site, a WEAP analysis must be performed by the Piling Contractor to demonstrate the potential for the proposed piling equipment to activate the specified ultimate resistance as stipulated by the Contract.

When requested by the Contract Administrator, all equipment, material, and personnel shall be supplied to conduct the wave equation analysis procedure. The Quality Verification Engineer shall review the results of the analysis.

903.07.02.07.04 .02 High-Strain Dynamic Testing

An independent testing company with no corporate affiliation with the Contractor shall be employed to perform the high-strain dynamic testing. The independent testing company shall be RAQs qualified (Specialty: Geotechnical (Structures and Embankments – Medium or High Complexity)).

High-strain dynamic tests shall be performed by a Professional Engineer employed by the independent testing company. The Engineer shall have documented evidence of training and experience in foundation engineering and wave equation analyses, and have a certificate of proficiency (intermediate level or better) in the PDCA Dynamic Measurement and Analysis Proficiency Test.

High-strain dynamic testing shall be performed using the Pile Driving Analyzer, or approved equivalent, for the determination of pile ultimate resistance, establishment of pile installation criteria, assessment of pile integrity, monitoring of hammer/drive system performance and driving stresses, as specified in the Contract Documents. The method and equipment for testing and its reporting shall be according to ASTM D 4945-17.

The location, sequencing and scheduling of the individual pile testing shall be proposed by the Contractor based on the purpose of the testing, and shall be submitted to the Contract Administrator for approval.

High-strain dynamic testing shall be carried out at the end of initial driving on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents.

Additional high strain dynamic testing (i.e. restrike testing) shall be carried out during the retapping of piles, as specified in the Retapping Tests on Piles clause. Restrike testing shall be performed on a minimum of 10% of piles in each pile group, rounded up, but no fewer than 2 piles; or as specified in the Contract Documents.

Restrike testing shall be carried out no sooner than 24 hours after installation of the individual pile or at a time specified in the Contract Documents. If the hammer needs to be warmed up prior to performing a restrike, it shall not be warmed up by striking the intended test pile.

903.10 BASIS OF PAYMENT

Section 903.10 of OPSS 903 is amended by the addition of the following subsection:

903.10.04 High-Strain Dynamic Testing, Deep Foundations - Item

Payment for the above tender item shall be full compensation for all labour, Equipment and Material to do the work.

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APPENDIX H

Sample NSSP – Obstructions

Obstructions —Item No.

Special Provision

1.0 SCOPE

The work required for excavations and the installation of piles and temporary roadway protection systems shall include consideration for encountering cobbles and/or boulders and the existing concrete bridge foundations at the site.

2.0 CONSTRUCTION

Cobbles, boulders and concrete foundations may obstruct excavation activities and the installation of piles and temporary roadway protection systems. Impacts associated with these obstructions could include, but are not limited to, the following:

- Pre-drilling and/or coring through the existing bridge foundations would be required both to permit installation of the piles and corrugated steel pipe (CSP) liners for the new bridge abutments.
- Additional effort will be required to excavate material containing cobbles and boulders.
- The installation of sheet piles or driven soldier piles could be obstructed and the piles could be damaged as a result of encountering obstructions. Multiple attempts at driving sheet piles or soldier piles should be anticipated.
- The installation of driven piles could be obstructed by cobbles and boulders prior to the piles reaching the pile tip elevation specified on the Contract Drawings. Pre-drilling to achieve the required pile penetrations should be anticipated.
- The installation of drilled soldier piles could be obstructed. Multiple attempts at installing drilled soldier piles should be anticipated unless the obstructions are broken-up or removed prior to pile driving.
- The contractor will require appropriate excavation equipment and construction methods to penetrate or remove cobbles and boulders.
- The removal of cobbles and boulders from excavations may lead to undermining of materials in the sidewalls of excavations. The contractor would need to implement measures to prevent instability of the undermined materials.

3.0 BASIS OF PAYMENT

Payment at the Contract price for the appropriate tender items associated with excavations, pile installation and temporary roadway protection systems shall include full compensation for all labour, Equipment and Material to do the work.