August 30, 2018

FOUNDATION INVESTIGATION AND DESIGN REPORT

STRUCTURAL BUNDLE - 11 STRUCTURES ON HIGHWAYS 129, 532, AND 556
HIGHWAY 532 - ACHIGAN CREEK BRIDGE REPLACEMENT, 5.1 KM NORTH OF HIGHWAY 556 (SITE NO. 38S-041)
LAT. 46.789744° ; LONG. -84.054775°
HODGINS AND GAUDETTE TOWNSHIPS, ALGOMA DISTRICT, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5378-11-00 ; WP 151-97-01

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REPORT

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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) has been retained by AECOM on behalf of the Ministry of Transportation, Ontario (MTO) to provide detailed foundation engineering services for the replacement of the Achigan Creek Bridge on Highway 532 (Site No. 38S-041) in the Townships of Gaudette and Hodgins, Algoma District, Ontario.

The purpose of this field investigation is to establish the subsurface conditions at the location of the existing bridge abutments and at the abutments and approach embankments of a proposed temporary modular bridge to be located west of the existing bridge along a temporary detour alignment, by methods of borehole drilling and coring, in-situ testing and laboratory testing on selected soil samples.

This report summarizes the factual results of field and laboratory work (including field investigation procedures, borehole stratigraphy, and geotechnical and analytical laboratory test results) as well as a description of the interpreted soil and groundwater conditions at the Achigan Creek Bridge site.

The Terms of Reference and Scope of Work for the foundation investigation are outlined in MTO's Request for Proposal, dated December 8, 2015. Golder's proposal for foundation engineering services is contained in Section 17.8 of AECOM's Technical Proposal for this assignment.

2.0 PROJECT AND SITE DESCRIPTION

2.1 Project Description

The existing Triple-Double Reinforced Bailey Bridge at the site carries Highway 532 over Achigan Creek in a generally north to south direction. The bridge was constructed as a Triple-Single Chord Reinforced Bailey in 1985 under Contract No. 84-214 and converted to the present configuration in 2012. The bridge underwent a structural assessment in 2015 and was identified as being in good condition with minor deterioration of several elements. However, more significant deterioration of the structural steel coatings and curbs was noted. The current bridge is to be replaced with a new two lane bridge.

2.2 Site Description

The site of the proposed modular bridge replacement is located about 5.1 km north of Highway 556, north of Searchmont, at the boundary between Hodgins Township and Gaudette Township within the Algoma District, Ontario.

The existing structure is a single span, 48.8 m long, Triple-Double Reinforced Bailey Bridge. The structure accommodates a single lane of traffic and is approximately 6.1 m wide. A cantilevered sidewalk is affixed on the west side of the structure. The travelled surface of bridge and the sidewalk is comprised of wooden deck. The bridge is supported on Size 36 timber piles (ten piles per abutment) driven to approximately Elevation 223.4 m.

The Achigan Creek at the location of the existing modular bridge is approximately 20 m wide and flows in a generally northwest to southeast direction. The downstream end of Achigan Creek flows into the Goulais River about 1.5 km southeast of the bridge.

Residential dwellings are located near the bridge on both sides of the creek, particularly at the southwest, northwest and northeast quadrats. Overhead electrical transmission lines run along the highway on the east side of Highway 532 (i.e., about 8 m east of the edge of pavement). However, the overhead lines also cross the highway at several locations south and north of the bridge where the residences are located.

In general, the topography of the area in the immediate vicinity of the bridge is relatively flat to undulating, except for the creek banks which are about 4 m to 5 m high. The presence of a ski resort near Searchmont, located about 2.5 km south of the site, is an indicator of the high relief and rugged topography beyond the site limits. The natural ground surface in the vicinity of the existing bridge varies between about Elevations 238 m and 239 m, and slopes down towards the creek. Despite the presence of several dwellings near the bridge, the site is relatively heavily vegetated, especially near the banks of the Achigan Creek. The vegetation is comprised of grasses, shrubs as well as deciduous and coniferous trees.

3.0 FIELD INVESTIGATION PROCEDURES

3.1 Previous (1981) Investigation

A previous foundation investigation was carried out at the site by MTO's Foundation Design Section in September 1981, following a structural assessment which indicated that the bridge had lost much of its structural integrity and that the adjoining wooden walkway showed signs of severe deterioration. A total of two boreholes (designated as Boreholes 1 and 2) were advanced at the southwest and northeast portion of the bridge, respectively. A Dynamic Cone Penetration Test (DCPT, designated as Borehole 3) was also carried on the northwest side of the bridge. The existing information is summarized in the following report:

- **MTO Geocres No. 41K-041**: "Foundation Investigation Report for Achigan Creek Crossing and Highway 532; W.P. 148-65-00, Site 38S-41; District 18, Sault Ste. Marie" by Engineering Materials Office – Pavement & Foundation Design Section, dated November 4, 1981.

The two boreholes were advanced to depths of about 26.8 m and 26.1 m below existing ground surface, respectively, while the cone was driven to a depth of about 27.5 m. The subsurface conditions encountered in the boreholes consists of a 6.2 m thick deposit of very loose to loose sandy silt and a 2.8 m thick deposit of loose fine sand. These granular deposits are underlain by an extensive cohesive deposit described as a stiff to very stiff "stratified silty clay with alternating layers of silty clay of low plasticity and silty clay of medium plasticity". The boreholes were terminated within the silty clay deposit at depths of about 26.8 m and 26.1 m below the existing ground surface in the respective boreholes. The subsurface conditions encountered during the 1981 field investigation are consistent with the subsurface conditions encountered during the 2017 investigation (described herein).

The approximate locations of the previous boreholes and the DCPT are shown on Drawing 1 along with the boreholes advanced as part of the current investigation (described below). However, the original borehole location and soil strata drawing associated with the 1981 field investigation has also been provided in Appendix A. The original borehole records and geotechnical laboratory test results are also provided in Appendix A.

3.2 Current (2017) Investigation

The recent field work at the Achigan Creek Bridge site was carried out between August 22 and 30, and between September 9 and 12, 2017, during which time a total of eight boreholes were advanced in close proximity to the existing foundation elements and near the abutments and approach embankments of the proposed temporary modular bridge to be located west of the existing bridge along a temporary detour alignment. The borehole locations were selected in consultation with AECOM and a proposed borehole location plan was submitted to MTO Foundations on July 24, 2017. The boreholes were advanced as close as possible to the existing bridge abutments, the new bridge abutments associated with the temporary modular bridge, and along the temporary detour alignment. The approximate locations of the boreholes are summarized as follows:

Approximate Location	Relevant Borehole(s)
Temporary Modular Bridge – South Portion of Temporary Detour Alignment	ACB-01
Achigan Creek Bridge – South Abutment	ACB-02 [1] and ACB-03
Temporary Modular Bridge – South Abutment	ACB-04 [2]
Temporary Modular Bridge – North Abutment	ACB-05
Achigan Creek Bridge – North Abutment	ACB-06 and ACB-07
Temporary Modular Bridge – North Portion of Temporary Detour Alignment	ACB-08

Notes:

1. It was not possible to advance Borehole ACB-02 immediately next to the east side of the existing south bridge abutment since the single lane of traffic along the bridge had to remain open to traffic and the terrain on the east side of the highway was steep and heavily vegetated with large trees.
2. It was not possible to advance Borehole ACB-04 immediately next to the south abutment of the proposed temporary modular bridge due to access restrictions and proximity to the steep and heavily vegetated creek bank slope.

The subsurface soil conditions encountered in the boreholes are shown in detail on the Records of Boreholes in Appendix B. Lists of abbreviations and symbols are also provided in Appendix B to assist in the interpretation of the borehole records. The locations of the as-drilled boreholes are shown in plan on Drawing 1.

All boreholes, except Boreholes ACB-02 and ACB-06 were advanced using a CME-75 track-mounted drill rig, while Boreholes ACB-02 and ACB-06 were advanced using a CME-55 truck-mounted drill rig. The drill rigs were supplied and operated by Landcore Drilling Inc. of Chelmsford, Ontario. Boreholes ACB-01 and ACB-08 were advanced through the overburden using 210 mm outer diameter, continuous flight, hollow-stem augers. The remaining boreholes were advanced through the upper portion of the overburden (i.e., generally through the upper 1.5 m) using 95 mm outer diameter, continuous flight, solid-stem augers or 210 mm outer diameter hollow-stem augers. The rest of the overburden was advanced using ‘NW’ casing with wash boring techniques and also coring using an ‘NQ’ double-tube rock core barrel to penetrate through cobbles and boulders encountered below the cohesive deposit at depths between about 27 m and 30 m below the existing ground surface. Photographs of the recovered cobbles and boulders are provided in Appendix C. Soil samples were generally obtained at intervals of depth of about 0.75 m and 1.5 m, using a 50 mm outer diameter, split-spoon sampler driven by an automatic hammer in accordance with Standard Penetration Test (SPT) procedures (ASTM D1586, *Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils*). Field vane shear tests were carried out in the cohesive deposit for assessment of undrained shear strengths (ASTM D2573, *Standard Test Method for Field Vane Shear Strength Test in Cohesive Soils*) using the MTO Standard ‘N’-size vanes.

The boreholes were advanced to depths ranging between about 15.9 m and 32.5 m below the existing ground surface. In Boreholes ACB-02 to ACB-06 coring methods were used to advance the boreholes below the cohesive deposit due to the presence of cobbles and boulders. A DCPT was carried in Borehole ACB-07 between depths of about 30.6 m and 32.4 m below existing ground surface.

The groundwater conditions and water levels in the boreholes (i.e., generally inside the ‘NW’ casing) were typically observed during drilling operations and measured upon completion of drilling. However, the measured water levels are considered not representative of the groundwater conditions at the site due to introduction of drilling water during wash boring and coring operations. Artesian groundwater conditions were encountered in

Borehole ACB-02 at a depth of about 28.2 m below the existing ground surface; however, flowing artesian groundwater conditions were not observed. All boreholes were backfilled upon completion of drilling/coring in accordance with Ontario Regulation 903 (Wells) (as amended). During a subsequent 2018 field investigation at several culvert sites associated with the Highways 129, 532 and 556 project, the Achigan Creek Bridge site was revisited and a standpipe piezometer was installed at the southwest corner of the bridge (immediately next to Borehole ACB-03) to permit groundwater monitoring at this site. The standpipe piezometer consisted of a 50 mm diameter PVC pipe, with a slotted screen sealed partially in the surficial granular fill and partially within the underlying native granular deposit. The borehole and the annulus surrounding the screen and the solid portion of the piezometer pipe was backfilled with sand. The standpipe piezometer installation details and the water level readings are provided on the Record of Borehole sheet for ACB-03 presented in Appendix B. The standpipe piezometer was decommissioned on August 15, 2018 in accordance with Ontario Regulation 903 (Wells) (as amended).

Prior to commencement of the field work, Golder arranged for the clearance of underground utilities/services. The field work was observed on a full-time basis by a member of Golder's engineering staff who monitored the drilling/coring, in-situ testing and sampling operations, and logged the boreholes in the field. The soil and cobble/boulder core samples were transported to Golder's Mississauga geotechnical laboratory where the samples underwent further visual examination and geotechnical laboratory testing.

Geotechnical classification testing (i.e., water content, Atterberg limits and grain size distribution) was carried out on selected soil samples. In addition, one-dimensional consolidation (i.e., Oedometer) tests were carried out on select samples of the cohesive deposit. The results of the geotechnical laboratory testing are summarized on the borehole records in Appendix B and the details of the geotechnical laboratory testing are provided in Appendix C. All of the laboratory tests were carried out to MTO Laboratory and/or ASTM Standards, as appropriate.

Two soil samples were also collected from Boreholes ACB-04 and ACB-06 for corrosivity testing. The selected soil samples were submitted, under chain-of-custody procedures, to Maxxam Analytics of Mississauga, Ontario (a Standards Council of Canada accredited laboratory) for analysis of a suite of corrosivity parameters including pH, sulphate, sulphide, chloride and resistivity/conductivity.

Temporary benchmarks were established and surveyed near the existing Achigan Creek Bridge by Callon Dietz Inc. prior to the drilling crew mobilizing to site. Upon completion of drilling/coring operations, borehole offsets and corresponding ground surface elevation differences were recorded and tied-in to the surveyed benchmarks to determine the as-drilled borehole locations and ground surface elevations. The borehole survey information, including northing and easting coordinates (presented in the MTM NAD83 Zone 13 and with latitude/longitiude coordinate systems) and the ground surface elevations referenced to Geodetic datum, are provided on the borehole records in Appendix B, presented on Drawing 1, and summarized below.

Approximate Location	Borehole Designation	Coordinates (MTM NAD83 Zone 13)		Ground Surface Elevation	Borehole Depth
		Northing (Latitude)	Easting (Longitude)		
Temporary Modular Bridge – South Portion of Temporary Detour Alignment	ACB-01	5183314.9 m (46.789381°)	300612.5 m (-84.054853°)	238.9 m	15.9 m

Approximate Location	Borehole Designation	Coordinates (MTM NAD83 Zone 13)		Ground Surface Elevation	Borehole Depth
		Northing (Latitude)	Northing (Latitude)		
Achigan Creek Bridge – South Abutment	ACB-02	5183317.1 m (46.789401°)	300617.3 m (-84.054790°)	238.9 m	32.0 m
	ACB-03	5183333.1 m (46.789545°)	300610.5 m (-84.054880°)	238.3 m	32.0 m
Temporary Modular Bridge – South Abutment	ACB-04	5183335.1 m (46.789563°)	300606.0 m (-84.054938°)	238.0 m	32.5 m
Temporary Modular Bridge – North Abutment	ACB-05	5183392.1 m (46.790075°)	300606.9 m (-84.054927°)	237.8 m	32.3 m
Achigan Creek Bridge – North Abutment	ACB-06	5183385.9 m (46.790020°)	300615.1 m (-84.054820°)	238.8 m	32.0 m
	ACB-07	5183380.3 m (46.790020°)	300627.3 m (-84.054660°)	238.2 m	32.4 m [1]
Temporary Modular Bridge – North Portion of Temporary Detour Alignment	ACB-08	5183407.7 m (46.790216°)	300610.7 m (-84.054878°)	238.4 m	15.9 m

Note:
1. Borehole depth includes DCPT carried out between depths of about 30.6 m and 32.4 m below the existing ground surface.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Based on Northern Ontario Engineering Geology Terrain (NOEGTS)[1] mapping, the Achigan Creek Bridge site is located within a valley train/outwash plain consisting primarily of gravelly and sandy soils which "are mainly confined to the larger river valleys and usually occur as flat, terraced landforms" (McQuay, 1980). The granular deposits are variable in thickness and are generally underlain by varved silt and clay to glacial till and bedrock. The valley train is bordered by bedrock knobs.

Based on geological mapping developed by the Ontario Ministry of Northern Development and Mines (MNDM)[2], the site is underlain by bedrock from the gneissic tonalite suite of rocks comprised of tonalite to granodiorite (foliated to gneissic) with minor supracrustal inclusions.

4.2 Soil and Bedrock Conditions

The subsurface soil and groundwater conditions encountered in the boreholes advanced at this site as part of the current foundation investigation, together with the results of the in-situ and geotechnical/analytical laboratory testing, are presented on the Records of Boreholes (provided in Appendix B) and the laboratory test figures/sheets (provided in Appendices C and D). The results of the in-situ field tests (i.e., measured SPT 'N'-values and undrained shear strengths) as presented on the borehole records and in Section 4.2 are uncorrected, and are based on SPT sampling procedures carried out with an automatic hammer and field vane shear test procedures carried out with an MTO 'N'-size vane, respectively.

[1] Ontario Ministry of Natural Resources and Forestry. Northern Ontario Engineering Geology Terrain Study. Ontario Geological Society Electronic Mapping. Map 41KNE, Study Number 91.
[2] Ontario Ministry of Northern Development of Mines. Bedrock Geology of Ontario – East Central Sheet, Ontario Geological Survey – Map 2544.

The stratigraphic boundaries shown on the borehole records and on the interpreted stratigraphic profiles and sections (i.e., Drawings 1 to 3) are inferred from observations of drilling progress, non-continuous sampling, coring, and in-situ testing, and therefore, represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

In general, the subsurface conditions encountered at the Achigan Creek Bridge site consist of granular fill underlain by an upper granular deposit (comprised predominantly of sandy silt to silty sand to sand), underlain by an extensive deposit of clayey silt to silty clay which is varved near the upper portion of deposit and irregularly stratified at depth. The cohesive deposit is in turn underlain by a lower granular deposit with cobbles and boulders.

Detailed descriptions of the subsurface conditions encountered in the boreholes at this site are provided in the following subsections.

4.2.1 Asphalt

An approximately 40 mm thick layer of asphalt was encountered at the ground surface in Borehole ACB-02, which was advanced through the travelled portion of Highway 532 on the south side of the Achigan Creek Bridge.

4.2.2 Sandy Silt to Silty Sand to Sand to Sand and Gravel (Fill)

A granular fill was encountered below the layer of asphalt in Borehole ACB-02 and immediately at the ground surface in the remaining boreholes, except in Borehole ACB-07. The composition of the fill is quite variable, ranging from more fine-grained material (i.e., sandy silt to sand and silt to silty sand) to more coarse-grained material (i.e., sand to gravelly sand to sand and gravel). Trace organics were noted within the fill in Borehole ACB-04. The top of the fill was encountered at elevations between about 238.9 m and 237.8 m, and the overall thickness of the fill varies between approximately 0.7 m and 3.0 m.

The SPT 'N'-values measured within the fill generally range from 6 blows to 24 blows per 0.3 m of penetration, indicating a loose to compact state of compactness. Higher SPT 'N'-values ranging from 38 blows to 52 blows per 0.3 m of penetration, and indicating a dense to very dense state of compactness, were measured within the sand to gravelly sand to sand gravel portion of the fill.

The water content measured on nine samples of the fill ranges between about 4% and 19%.

The results of grain size distribution tests carried out on three samples of the fill recovered from Boreholes ACB-03, ACB-04, and ACB-08 are shown on Figure C1 in Appendix C.

4.2.3 Sandy Silt to Sand and Silt to Silty Sand to Sand (Upper Granular Deposit)

An upper granular deposit comprised predominantly of sandy silt to sand and silt to silty sand to sand was encountered immediately at the ground surface in Borehole ACB-07 and below the fill in the remaining boreholes. A more coarse-grained deposit comprised of sand and gravel was encountered below the sand fill in Borehole ACB-08 advanced on the north side of the creek. In Borehole ACB-03, inclusions/layers of organic silt and peat were encountered within the sand and silt deposit between depths of about 2.6 m and 3.7 m below existing ground surface. Trace organics were also noted within the upper granular deposit encountered in Borehole ACB-05. The top of this deposit was encountered at depths ranging between about 0 m (i.e., at the ground surface in Borehole ACB-07) and 3.0 m below the existing ground surface (between Elevations 238.2 m and 235.0 m), and the thickness of this deposit varies between approximately 0.9 m and 2.6 m.

In general, the SPT 'N'-values measured within the sandy silt to sand and silt to silty sand to sand portion of the upper granular deposit range from 0 blows (weight of hammer) to 9 blows per 0.3 m of penetration, indicating a very loose to loose state of compactness. Two SPT 'N'-values of 31 blows and 39 blows per 0.3 m of penetration were measured within the sand and gravel deposit encountered in Borehole ACB-08, indicating dense state of compactness.

The water contents measured on 15 samples of the upper granular deposit generally range between about 6% and 33%. A water content measured on a sample of the sand and silt deposit recovered from Borehole ACB-03 is about 56%, and the high water content is likely attributed to the presence of organic silt and peat inclusions/layers.

The results of grain size distribution tests carried out on seven samples of the sandy silt to sand and silt to silty sand to sand portion of the upper granular deposit are shown on Figure C2A in Appendix C. The result of a grain size distribution test carried out on a sample of the sand and gravel portion of the upper granular deposit is shown on Figure C2B in Appendix C.

Atterberg limits tests were also carried out on the fines portion of three samples of the upper granular deposit. A test carried out on a sample of a sandy silt recovered from Borehole ACB-02 measured a liquid limit of 25%, a plastic limit of 23%, and a corresponding plasticity index of about 2%. The results of this Atterberg limits test are shown in Figure C3 of Appendix C, and indicate that the fines portion of this material is classified as a silt of low plasticity. The results of Atterberg limits tests carried out on two other samples recovered from Boreholes ACB-03 and ACB-05 indicate that the fines portion of these materials is non-plastic.

A consolidated drained direct shear test was also carried out on samples of the sand and silt to silty sand deposit recovered from Borehole ACB-05. The results are presented on Figure C4.

4.2.4 Clayey Organic Silt

A thin layer of clayey organic sandy silt, was encountered below the sand and silt deposit in Borehole ACB-03. The top of this layer was encountered at a depth of about 4.7 m below existing ground surface, corresponding to Elevation 233.6 m, and is approximately 0.3 m thick.

The SPT 'N'-value measured within this deposit is 2 blows per 0.3 m of penetration, indicating a very soft to soft consistency.

The water content measured on a sample of this deposit is about 44%.

4.2.5 Clayey Silt to Silty Clay (Varved to Irregularly Stratified)

An extensive cohesive deposit comprised of clayey silt to silty clay was encountered below the upper granular deposit in all boreholes, except in Borehole ACB-03, where the cohesive deposit was encountered below the layer of clayey organic silt. The upper portion of the cohesive deposit (above approximately Elevation 228.0 m) is varved (i.e., generally comprised of clayey silt and silty clay laminae). Photographs of the varved cohesive specimens recovered from six Shelby tube samples are shown on Figure C5A in Appendix C. The lower portion of the cohesive deposit (below approximately Elevation 228.0 m) is stratified, but the layers are not oriented or shaped in a regularly repeating pattern as compared to the varved upper portion of the cohesive deposit where the laminae are arranged in horizontal layers parallel to each other. Photographs of the irregularly stratified cohesive specimens recovered from four Shelby tube samples are shown on Figure C5B in Appendix C. The top of this cohesive deposit was encountered at depths ranging between about 2.6 m and 5.0 m (between Elevations 235.6 m and 233.3 m). Boreholes ACB-01 and ACB-08 were terminated within this deposit at a depth of about 15.9 m below existing ground surface, corresponding to Elevations 223.1 m and 222.6 m, respectively. The thickness of the clayey silt to silty clay deposit that was fully penetrated ranges from approximately 22.1 m to 27.4 m.

The SPT ‘N’-values measured within the cohesive deposit generally range between 0 blows (i.e., weight of hammer) and 18 blows per 0.3 m of penetration. In-situ vane tests carried out within the varved upper portion of the deposit (above Elevation 228 m) measured (uncorrected) undrained shear strength ranging from about 38 kPa to 72 kPa, but on average is about 62 kPa. In-situ vane tests carried out within the lower irregularly stratified portion of the deposit measured undrained shear strength ranging from about 67 kPa to 112 kPa, but on average is about 94 kPa. The sensitivity (defined as the quotient between the undisturbed shear strength and the remoulded shear strength) ranges between about 4 and 13, but typically varies from 5 to 8. The higher sensitivities (i.e., 10 or greater) were only recorded in Borehole ACB-04. The in-situ field vanes tests results together with the SPT ‘N’-values indicate that this deposit has a predominantly stiff to very stiff consistency; however, one field vane test measured in Borehole ACB-07 indicates that the cohesive deposit is firm. One SPT ‘N’-value measured near the bottom of the cohesive deposit in Borehole ACB-06 is 109 blows per 0.23 m of penetration. The high blow count can likely be attributed to the presence of a cobble.

The water content measured on 62 samples of this deposit ranges from about 27% to 44% and on average is 37%. A single water content measured on a sample recovered from Borehole ACB-07 is about 3%, but this low water content is likely associated with a sand seam/inclusion encountered within the deposit.

The results of grain size distribution tests carried out on six samples of the clayey silt to silty clay deposit are shown on Figure C6 in Appendix C. Atterberg limits tests were carried out on 39 samples of the clayey silt to silty clay deposit. The tests measured liquid limits between about 24% and 39%, plastic limits between about 18% and 21%, and plasticity indices between about 5% and 13%. The results of the Atterberg limits tests are shown on the plasticity charts on Figures C7A to C7E in Appendix C, and indicate that the material can be generally classified as a mixture of clayey silt of low plasticity and silty clay of intermediate plasticity.

Laboratory consolidation tests were also carried out on two specimens of the clayey silt to silty clay deposit obtained from Shelby tube samples recovered from Boreholes ACB-04 and ACB-05. The preconsolidation stresses was estimated for each specimen from the respective void ratio versus logarithmic pressure plot and from the total work versus pressure plot. Details of the test results are shown on Figures C8 and C9 in Appendix C and the test results are summarized below.

Borehole/ Sample No.	Sample Depth (Elevation)	γ (kN/m^3) (G_s)	σ'_{vo} (kPa)	σ'_p (kPa)	$\sigma'_{vo} - \sigma'_p$ (kPa)	OCR	C_c	C_r	e_o	c_v [1] (cm^2/s)
ACB-04 SA 12	12.7 m (237.9 m)	17.7 (2.74)	140	450	330	3.2	0.52	0.025	1.15	1.7×10^{-2}
ACB-05 SA 11	11.1 m (238.0 m)	17.6 (2.71)	125	275	130	2.2	0.43	0.025	1.16	7.2×10^{-3}

Note:
1. The coefficient of consolidation is based on a stress range between the existing in-situ effective overburden stress and the stress due to an up to about 1.5 m high embankment constructed along the proposed temporary detour alignment. The final stress is estimated to be less than the preconsolidation stress and within the over consolidated stress range.

where: γ is the bulk unit weight in kN/m^3
G_s is the specific gravity
σ'_{vo} is the effective overburden stress in kPa
σ'_p is the preconsolidation stress in kPa
OCR is the overconsolidation ratio
OCR is the overconsolidation ratio
C_c is the compression index
C_r is the recompression index
e_o is the initial void ratio
c_v is the coefficient of consolidation in cm^2/s

4.2.6 Sandy Silt to Silty Sand to Silty Sand and Gravel with Cobbles and Boulders (Lower Granular Deposit)

A lower granular deposit comprised of sandy silt to silty sand to silty sand and gravel was encountered below the clayey silt to silty clay deposit, and sampled with a split-spoon sampler in in Boreholes ACB-02, ACB-06 and ACB-07. In Boreholes ACB-03 to ACB-05, the granular deposit in the lower portion of the boreholes was not sampled with a split-spoon sampler, but is inferred to consist of a deposit of a silty sand, some gravel based on: i) close proximity to the other boreholes advanced at the site to similar depths that were sampled with a split-spoon sampler; ii) difficulties with casing advancement, and; iii) presence of cobbles and/or boulders which were confirmed in six boreholes (not including in Borehole ACB-07 where a DCPT was carried out) by rock coring.

Frequent rock fragments, cobbles, and boulders were encountered within this lower granular deposit. The size of the cobbles and boulders recovered from zones which required rock coring to advance the boreholes were noted to range between about 100 mm and 620 mm. Frequent gravel pieces and rock fragments ranging in size from about 20 mm to 70 mm were also recovered. Photographs of the recovered rock fragments, cobbles, and boulders are shown on Figure C10 in Appendix C. The top of this deposit was encountered at depths ranging between about 27.1 m and 30.0 m below existing ground surface (between Elevations 211.2 m and 208.2 m). Boreholes ACB-02 to ACB-07 were terminated with the lower granular deposit at depths ranging from about 30.6 m to 32.5 m (between Elevations 207.6 m and 205.5 m). In Borehole ACB-07, a DCPT was also carried out between depths ofabout 30.6 m (Elevation 207.6 m) and 32.4 m (Elevation 205.8 m).

The SPT 'N'-values measured within this lower granular deposit were 78 blows for 0.03 m of penetration, 101 blows per 0.3 m of penetration, and 100 blows per 0.15 m of penetration, indicating a very dense state of compactness. These high blow counts can be attributed to the cobbley/bouldery nature of this deposit.

The water content measured on three samples of the lower granular deposit range between about 10% and 23%.

The results of grain size distribution tests carried out on three samples of the lower granular deposit are shown on Figure C11 in Appendix C. Atterberg limits tests were carried out on the fines portion of two samples recovered from Boreholes ACB-02 and ACB-06. The results indicate that the fines portion of this material is non-plastic.

4.3 Groundwater Conditions

The majority of the boreholes were advanced using wash boring techniques which involved the introduction of drilling water. As such, the water level measurements taken upon completion of drilling operations are not considered representative of the groundwater conditions at the site. However, the lower portion of the upper granular deposit, which was typically advanced using hollow- or solid-stem augers, was noted to be wet. Wet soil samples were collected below elevations ranging between about 236.9 m and 234.1 m, and on average below approximately Elevation 235.8 m

As described in Section 3.2, during a subsequent 2018 field investigation at several culvert sites associated with the Highways 129, 532 and 556 project, the Achigan Creek Bridge site was revisited and a standpipe piezometer was installed at the southwest corner of the bridge (immediately next to Borehole ACB-03) to permit groundwater monitoring at the site. Details of the piezometer installation are shown on the Record of Borehole sheet for ACB-03 in Appendix B. The groundwater level was measured daily between August 12 and 15, 2018 at a depth of about 4.5 m below existing ground surface, corresponding to Elevation 233.8 m. The standpipe piezometer was decommissioned on August 15, 2018 in accordance with Ontario Regulation 903 (Wells) (as amended).

It is also noted that artesian groundwater conditions were encountered in Borehole ACB-02 at a depth of about 28.2 m (Elevation 210.7 m), which likely corresponds to the top of the lower granular deposit. Although the groundwater was not observed to be flowing out of the drill casing (i.e., flowing artesian groundwater conditions were not observed) when the lower granular deposit was penetrated, the drillers did note “higher groundwater pressures” making casing advancement more difficult.

The groundwater level at the site is anticipated to fluctuate seasonally in response to changes in precipitation, and should be expected to be higher during wet seasons or during any heavy and/or sustained periods of precipitation. Furthermore, given the presence of a layer of granular fill and/or an upper granular deposit encountered near the ground surface, and considering that the granular deposit is underlain by a cohesive deposit with a relatively low permeability, a perched water table condition may exist within the granular fill/upper granular deposit. The perched water table is also subject to seasonal fluctuations and precipitation events.

The water level measured in the Achigan Creek on November 1, 2017 was at approximately Elevation 234.9 m.

4.4 Analytical Testing of Soil

Two soil samples were selected from Boreholes ACB-04 (advanced near the south abutment of the Achigan Creek Bridge) and ACB-06 (advanced near the north abutment of the Achigan Creek Bridge) and submitted to Maxxam Analytics of Mississauga, Ontario for corrosivity testing. The analytical laboratory test results are provided on the Certificate of Analysis presented in Appendix D, and summarized below.

Borehole Designation	Sample No.	Average Approx. Sample Depth (m)	Average Approx. Sample Elevation (m)	Material Type	Resistivity (ohm·cm)	Conductivity (µohm/cm)	pH	Chloride (Cl) Content (ppm or µg/g)	Sulphate (SO_4) Content (ppm or µg/g)
ACB-04 [1]	SA 4	2.6 m	235.7	Silt and Sand	7,300	135	6.5	58	<20 [2]
ACB-06 [1]	SA 3	2.6 m	236.2	Sand	7,200	139	5.0	70	<20 [2]

Notes:
1. It is noted that corrosivity results associated with soil samples recovered from boreholes that were advanced at other sites associated with this project are also presented on the Certificates of Analysis.
2. The sulphate concentration is below the reportable detection limit of 20 µg/g.

It is noted that the sulphide content measured on the soil samples recovered from Boreholes ACB-04 and ACB-06 was also analyzed and is approximately 0.69 µg/g and 0.60 µg/g, respectively.

5.0 CLOSURE

The field work for this investigation was supervised by Mr. Jeremy Lebow, B.A.Sc. and Ms. Amelia Jewison, B.A.Sc. The Foundation Investigation Report was prepared by Ms. Alysha Kobylinski, B.A.Sc., and reviewed by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer with Golder. Mr. Paul Dittrich, P.Eng., a Principal and a MTO Foundations Designated Contact for Golder, conducted an independent quality control review of the report.

Report Signature Page

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PART B

FOUNDATION DESIGN REPORT
STRUCTURAL BUNDLE – 11 STRUCTURES ON HIGHWAYS 129, 532 AND 556
HIGHWAY 532 – ACHIGAN CREEK BRIDGE REPLACEMENT, 5.1 KM NORTH OF HIGHWAY 556 (SITE NO. 38S-041)
LAT. 46.789744° ; LONG. -84.054775°
HODGINS AND GAUDETTE TOWNSHIPS, ALGOMA DISTRICT, ONTARIO
MINISTRY OF TRANSPORTATION, ONTARIO
GWP 5378-11-00 ; WP 151-97-01

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

This section of the report provides foundation design recommendations for the replacement of the Achigan Creek Bridge on Highway 532, and construction of a temporary modular bridge (to the west of the replacement bridge) to accommodate traffic during construction of the permanent structure. These recommendations are based on interpretation of the factual data obtained from the boreholes advanced during the field investigation. The discussion and recommendations presented are intended to provide the designers with sufficient information to assess the feasible foundation alternatives and carry out the design of the bridge foundations. The foundation investigation report, discussion and recommendations are intended for the use of MTO and its designers and shall not be used or relied upon for any other purpose or by any other parties, including the construction or design-build contractor.

Contractors must make their own interpretation based on the factual data presented in the Foundation Investigation Report (Part A of this report). Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project and for which special provisions may be required in the Contract Documents. Those requiring information on aspects of construction must make their own interpretation of the factual information provided as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

6.1 General

The existing Achigan Creek Bridge was originally constructed circa 1985 and upgraded to its current configuration in 2012 and carries Highway 532 over Achigan Creek in a generally north to south direction. The structure consists of a single span, 48.8 m long, 6.1 m wide, Triple-Double Reinforced Bailey Bridge which accommodates a single lane of traffic. A cantilevered sidewalk is affixed on the west side of the structure. The travelled surface of bridge and the sidewalk is comprised of a lumber plank deck. The bridge is supported on Size 36 timber piles (ten piles per abutment) driven to approximately Elevation 223.4 m.

The bridge underwent a structural assessment in 2015 and was identified as being in good condition with minor deterioration of several elements. However, more significant deterioration of the structural steel coatings and curbs was noted. The current bridge is to be replaced with a new bridge accommodating two lanes of traffic. The new bridge is expected to consist of a single 39 m long span with a 9.5 m wide deck and integral abutments founded on driven steel H-piles. It is understood that a grade raise of no more than 150 mm will be required at the abutments.

Furthermore, in order to accommodate construction of the new permanent bridge, it is understood that a temporary detour alignment, including a temporary modular bridge, will be constructed approximately 15 m west of the existing bridge. The temporary modular bridge is expected to be about 52 m long and be located between approximately Stations 10+184 and 10+236. Up to about 1.2 m and 0.8 m of new fill will be required along the south and north portions of the temporary detour (approach embankment) alignment, respectively. The temporary detour alignment is shown in plan on Drawing 1. It is also understood that the temporary bridge will likely remain in operation for about one year before the new Achigan Creek replacement bridge is complete opened to traffic.

6.2 Consequence and Site Understanding Classification

In accordance with Section 6.5 of the 2014 *Canadian Highway Bridge Design Code* (CHBDC 2014) and its *Commentary*, the proposed bridge and foundation system is expected to carry relatively low traffic volumes, however, its performance may have potential impacts on other transportation corridors, hence having a "typical consequence level" associated with exceeding limits states design. In addition, given the typical project-specific foundation investigation carried out at this site (as presented in Part A of the report), in comparison to the degree of site understanding in Section 6.5 of *CHBDC (2014)*, the level of confidence for design is considered to be a "typical degree of site and prediction model understanding." Accordingly, the appropriate corresponding ULS and SLS consequence factor, Ψ, and geotechnical resistance factors, ϕ_{gu} and ϕ_{gs}, from Tables 6.1 and 6.2 of the *CHBDC* have been used for design.

6.3 Foundation Options – Achigan Creek Bridge Replacement and Temporary Modular Bridge

Based on the proposed configuration of the replacement bridge and the temporary modular bridge, and the subsurface conditions encountered at the site, both shallow and deep foundation options have been considered for support of both structures. Details of the foundation options are outlined below.

- **SHALLOW FOUNDATIONS**
  - **Spread/Strip Footings**
    - **Replacement Bridge:** Shallow foundations comprised of spread or strip footings and founded on native subgrade are considered feasible for support of the new abutments, although this foundation type will preclude the use of integral abutments. Furthermore, due to the large loads imposed by the replacement structure and the presence of a generally very loose to loose upper granular deposit underlain by an extensive cohesive deposit, the geotechnical resistances (in particular at Serviceability Limit State) will be insufficient for a conventional bridge deck design. Ground improvement/settlement mitigation measures would have to be implemented in order to make this option viable from a foundations point of view, but this approach would result in higher construction costs and would impact the overall construction schedule. The shallow foundation option is not considered to be the preferred alternative for the replacement structure.
    - **Temporary Modular Bridge:** Shallow foundations comprised of spread/strip footings or bearing pads/plates sitting on top of timber cribbing at each corner of the bridge and founded on native subgrade are also considered feasible for support of the temporary structure. Although the structural loads associated with a temporary modular bridge are much smaller compared to a conventional bridge, the relatively weak subsurface conditions encountered at the site will result in low geotechnical resistances as well as post-construction settlement of the foundation elements supporting the bridge. From a foundations perspective, shallow foundations are not considered the preferred alternative; however, if the temporary modular bridge can accommodate the estimated settlement and/or can be maintained (re-leveled) during the construction works, shallow foundations will be more economical as compared to deep foundations.

- **DEEP FOUNDATIONS**
  - **Driven Steel H-Piles/Tube Piles**
    - **Replacement Bridge:** Steel H-piles or tube piles driven into the very dense, lower granular deposit below the thick cohesive stratum are considered feasible for support of the new abutments, although given that H-piles are a lower displacement pile they likely offer constructability/drivability advantages over tube piles for the conditions at this site. The end-bearing piles would be up to about 30 m long. Driven piles would permit design of conventional and semi-integral abutments (for H-piles and tube piles) or integral abutments (generally for H-piles). Consideration could also be given to friction piles founded within the extensive cohesive stratum, which would result in shorter piles. However, given the relatively large loads imposed by the new structure, the use of shorter friction piles may not be practical as a result of the significantly lower geotechnical resistance available for these piles.
    - **Temporary Modular Bridge:** friction piles comprised of steel H-pile or tube piles driven into the extensive cohesive stratum are considered feasible for support of the temporary modular bridge. This foundation alternative would offer higher geotechnical resistances compared to a shallow foundation option and a lower cost then deep piles driven into the very dense lower granular deposit.
  - **Drilled Shafts (Caissons)**
    - **Replacement Bridge:** Drilled shafts (caissons) founded within the very dense lower granular deposit, resulting in approximately 30 m long drilled shafts, are considered feasible for the support of the new abutments; however, this option would preclude an integral abutment design. Moreover, given the high likelihood of encountering artesian groundwater conditions when penetrating the lower granular deposit, the use of drilled shafts will carry risks associated with maintaining a stable and undisturbed base during construction. In addition, drilled shafts would be more expensive than shallow foundations and driven pile foundations at this site. As such, this particular deep foundation option is not considered to be the preferred alternative at this site and is not discussed further herein.
    - **Temporary Modular Bridge:** Given the relatively low structural loads imposed by the structure and considering that the structure will remain in operation for up to about two years, the use of drilled shafts to support the temporary modular bridge is economically unwarranted. In addition, based on the constructability issues outlined above, drilled shafts are not the preferred foundation alternative and are not discussed further herein.

A more comprehensive summary of the advantages, disadvantages and risks for each foundation option, from a geotechnical/foundations perspective, for the Achigan Creek replacement bridge and for the temporary modular bridge is presented in Table 1A and Table 1B, respectively, following the text of this report. The key challenges and considerations for the various foundation options are also discussed in greater detail within Sections 6.3.1 and 6.3.2.

Based on the above considerations, end-bearing steel H-piles driven into the lower granular deposit are considered the most feasible and practical, from a geotechnical/foundations perspective, for support of the replacement structure. As mentioned above, driven steel H-piles would permit integral abutment design, which is the preferred structural design, and are therefore considered advantageous from this perspective. Furthermore, steel H-piles are expected to be more economical and subject to fewer construction and constructability challenges than drilled shafts or tube piles. In terms of the temporary modular bridge, friction steel H-piles/pipe piles driven into the extensive cohesive deposit are considered the preferred option from a geotechnical/foundations perspective.

However, shallow foundations are more economical, and if the structural designer and MTO can accept the settlement and associated maintenance issues during construction, this foundation alternative can be considered.

6.3.1 Shallow Foundations – Spread/Strip Footings

Consideration could be given to founding strip/spread footings 2 m below the exiting ground surface (i.e., below the frost penetration depth as outlined in Section 6.4) on the granular fill or upper granular deposit.

The factored ultimate and serviceability geotechnical resistances that may be used for the design of strip/spread footing at the abutments are provided below.

Foundation Element	Footing Width	Founding Elevation	Founding Soils	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 25 mm of Settlement
Achigan Creek Replacement Bridge					
South Abutment	3 m strip footing	~235.7 m	Loose to compact silt and sand to silty sand fill, underlain by very loose to loose sandy silt to silt and sand, and stiff to very stiff varved clayey silt to silty clay [1]	175 kPa	55 kPa [1]
North Abutment	3 m strip footing	~235.9 m	Very loose to loose silt and sand to sand, underlain by stiff to very stiff varved clayey silt to silty clay	175 kPa	65 kPa
Temporary Modular Bridge					
South Abutment	2 m strip footing	~235.6 m	Compact silty sand fill underlain by loose sand and stiff varved clayey silt to silty clay	175 kPa	95 kPa
	1 m by 2 m footing at each corner[2]			200 kPa	185 kPa
North Abutment	2 m strip footing	~235.8 m	Very loose to loose silt and sand to silty sand underlain by stiff varved clayey silt to silty clay	175 kPa	70 kPa
	1 m by 2 m footing at each corner[2]			200 kPa	110 kPa

Notes:
1. In addition to an approximately 0.3 m thick layer of clayey organic silt encountered below the upper granular deposit in Borehole ACB-03, the upper granular deposit itself includes inclusions/layers of organic silt and peat. Consequently, the presence of these weak/soft organic inclusions/layers could result in additional settlements below the footprint of the strip footing. In order to eliminate the risk of additional settlement as a result of the organics, sub-excavation on the order of 5.0 m below the existing ground surface would be required.
2. Consideration can also be given to constructing timber cribbing instead of concrete footings – see “Key Challenges and Considerations” section below for more details.

Given the low geotechnical resistances associated with strip footings founded at the location of the replacement bridge, ground improvement measures would be required to achieve adequate geotechnical resistances for design. Consequently, the use of shallow foundations for the replacement bridge are not discussed further.

Key Challenges and Considerations (Temporary Modular Bridge)

- As noted above, consideration can be given to constructing timber cribbing to support the temporary modular bridge. The timber cribbing should be constructed in accordance with Ontario Provincial Standard Specification (OPSS).PROV 918 (*Modular Bridge Structures*). More details pertaining to the construction of the timber cribbing are presented in Section 6.11.5. Furthermore, it is noted that the <u>geotechnical resistances should be confirmed once the exact dimensions and locations of the timber cribbing (relative to the adjacent crest of slope) are confirmed</u>.
- Depending on the selected dimension of the footings/timber cribbing and the load imposed by the temporary modular bridge, the structure may settle more than 25 mm. In this case, maintenance (e.g., shimming between the bridge and the footings or bearing pads/plates sitting on top of timber cribbing) during construction may be required to ensure that the travelled surface of the bridge is maintained at an appropriate elevation. The potential requirement for shimming while the temporary modular bridge is in use should be included in the Contract Documents.
- Consideration can be given to founding the footings/timber cribbing above the frost penetration depth. However, given the presence of fine-grained soils (i.e., silty soils which are highly frost susceptible) and a potentially high water table due to perched water conditions, the frost susceptible soils would have to be sub-excavated from the footprint of the footings and replaced with granular fill such as OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II material. <u>The higher founding elevations and corresponding geotechnical resistances would have to be confirmed</u>.

6.3.2 Deep Foundations – Driven Steel H-Piles/Tube Piles

Driven steel H-piles founded below the cohesive deposit within the very dense lower granular deposit (i.e., end-bearing piles) are considered the preferred foundation alternative for support of the integral abutments of the replacement bridge. Shorter piles (H-piles or tube piles) founded within the cohesive deposit and relying mostly on shaft resistance (i.e., friction piles) can be considered as well; however, significantly more piles would be required to achieve the desired axial capacity, making this alternative not economical.

The shorter friction piles can however be considered for supporting the much lighter temporary modular bridge.

The factored ultimate and serviceability geotechnical resistances that may be used for the design of steel HP 310x110 piles and 324 mm (12 ¾ in) diameter steel tube piles having a minimum wall thickness of 9.5 mm (3/8 in) are presented below.

Foundation Element	Approximate Pile Length [1]	Estimated Pile Tip Elevation [2]	Founding Stratum	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)
Achigan Creek Bridge Replacement					
South Abutment	27.0 m	209.0 m	Very dense lower granular deposit with cobbles and boulders	1,600 kN	-- [3]
	20 m	216.0 m	Stiff to very stiff clayey silt to silty clay	500 kN	-- [4]

Foundation Element	Approximate Pile Length [1]	Estimated Pile Tip Elevation [2]	Founding Stratum	Factored Ultimate Geotechnical Resistance	Factored Serviceability Geotechnical Resistance (for 25 mm of settlement)
North Abutment	26.5 m to 29 m	209.5 m to 207.0 m	Very dense lower granular deposit with cobbles and boulders	1,600 kN	-- [3]
	20 m	216.0 m	Stiff to very stiff clayey silt to silty clay	500 kN	-- [4]
Temporary Modular Bridge					
South Abutment	27.0 m	209.0 m	Very dense lower granular deposit with cobbles and boulders	1,600 kN	-- [3]
	15 m	221.0 m	Stiff to very stiff clayey silt to silty clay	400 kN	-- [4]
North Abutment	27.0 m	209.0 m	Very dense lower granular deposit with cobbles and boulders	1,600 kN	-- [3]
	15 m	221.0 m	Stiff to very stiff clayey silt to silty clay	400 kN	-- [4]

Notes:
1. It is assumed that the piles will extend down from the underside of the abutment wall at about Elevation 236.0 m.
2. Given the presence of cobbles and boulders encountered within the lower granular deposit, the piles can hang-up on these obstruction and the pile tip elevations may be variable at both bridge abutments.
3. For piles driven to refusal into the lower granular deposit, the factored geotechnical serviceability resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and, as such, the SLS condition does not apply.
4. For friction piles driven into the stiff to very stiff clayey silt to silty clay deposit, the factored geotechnical serviceability resistance for 25 mm of settlement will be greater than the factored ultimate geotechnical resistance and, as such, the SLS condition does not apply.

Key Challenges and Considerations

- Piles should be installed in accordance with OPSS.PROV 903 (*Deep Foundations*).
- Given the presence of timber piles at the abutments of the existing Achigan Creek Bridge (Size 36 timber piles driven vertically to approximately Elevation 223.2 m – as shown on Sheet 23 from Contract No. 84-412 package, dated August 1983), care must be taken during pile driving operations to avoid hitting the existing piles which could result in misalignment and/or damage of the new piles. The bridge design should also consider the location of the new abutments with respect to the existing timber piles. Consideration should be given to cutting-off the upper portion of the existing timber piles (i.e., below the base of the excavation proposed at each abutment) instead of attempting to extract the piles, which could result in remoulding of the cohesive deposit or creation of voids in the vicinity of the new piles. Although this operation is not expected to a have a significant impact on the axial capacity of the end-bearing piles, it could adversely impact the lateral capacity of the piles and/or the axial capacity of friction piles, if selected.
- The lower granular deposit contains cobbles and boulders. These obstructions can result in damage to the piles during pile driving operations and appropriate measures will need to be implemented to protect the piles.

For steel H-piles driven into the lower granular deposit, the pile tips should be reinforced with driving shoes or standard pile points (refer to Section 6.11.5 for more details).

- As noted above, given the presence of cobbles and boulders encountered within the lower granular deposit, the piles can hang-up on these obstructions and as such, the pile tip elevations may be variable at both bridge abutments.
- Given the potential of encountering artesian groundwater conditions (as noted in Borehole ACB-02 at a depth of about 28.2 m below existing ground surface, corresponding to Elevation 210.8 m) during pile driving into the lower granular deposit, a seepage control system/sand filter comprised of a concrete sand drainage blanket wrapped in a geotextile and containing collector pipes is recommended to control migration of fines that may be brought up along the piles during and following the pile driving operations (refer to Section 6.11.7 for more details).
- Tube piles are generally not considered sufficiently flexible to be used in an integral abutment configuration, but this detail should be confirmed by the structural engineer.
- Given the need for fill placement in the immediate vicinity of the temporary modular bridge, the friction piles will experience downdrag and drag loads. The drag loads will need to be considered in the assessment of the pile's structural capacity (refer to Section 6.6 for more details).
- Given the presence of residential dwellings in the vicinity of the proposed site, consideration should be given to vibration monitoring at the location of the residences during pile driving operations (refer to Section 6.11.7 for more details).

6.4 Frost Protection

Spread/strip footings and pile caps for deep foundation elements (i.e., H-piles or tube piles) should be provided with a minimum 2.0 m of soil cover for frost protection as per OPSD 3090.100 (*Frost Depths for Northern Ontario*), as measured vertically from ground surface and perpendicular to the face of the abutment slope to the edge of the underside of the footing or pile cap. Timber cribbing, if selected as the preferred foundation alternative for the temporary modular bridge, is also recommended to be founded below the frost penetration depth (refer to Section 6.11.5 for more details).

If adequate soil cover cannot be provided for the footing or pile cap, rigid polystyrene insulation could be installed to compensate for the lack of soil cover and provide protection from frost penetration. As a guideline for design, 25 mm of rigid polystyrene insulation can be assumed to provide a 300 mm reduction in soil cover. For unheated structures, such as bridge abutments exposed to the elements, the insulation should be placed along the bridge abutment wall and extend outwards horizontally from the foundation element (spread/strip footing or pile cap). The lateral extent (horizontal distance) of the polystyrene from the foundation is dependent on the thickness of the insulation and the amount of soil cover provided above the base of the abutment wall that is exposed (i.e., front of the abutment wall). However, in general, the total length of insulation extending vertically along the abutment wall (below the finished grade) and extending horizontally outward from the abutment wall should equal to the anticipated frost penetration depth at this site. Assuming a 75 mm thick rigid polystyrene insulation and assuming the front of the abutment wall will extend about 1 m below the grade, the insulation should extend 1.1 m outward from the front of the abutment wall. It is noted that insulation along the base of the abutment wall and spread/strip footing or pile cap is not required as long as 2 m of conventional soil cover is provided.

The insulation extending horizontally from the abutment wall should be sloped downwards to provide positive drainage away from abutment wall. In addition, a minimum 75 mm thick uncompacted levelling layer consisting of OPSS.PROV 1010 Granular ‘A’ material or concrete fine aggregate meeting the gradation requirements specified in OPSS.PROV 1002 (*Aggregates – Concrete*) should be provided below the base of the insulation. Care must also be taken during fill placement above the insulation to avoid damaging the rigid polystyrene boards. The overlying fill should be free of angular gravel/rock fragments.

Alternatively, for timber cribbing supporting the temporary modular bridge, consideration could be given to removing all frost susceptible materials below the footprint of the cribbing and replacing with non-susceptible granular fill in order to reduce the depth of embedment. However, if the depth of embedment for the footings or timber cribbing is less than 2.0 m, the factored ultimate and serviceability geotechnical resistances would have to be re-evaluated.

6.5 Resistance to Lateral Forces/Sliding

Resistance to lateral forces/sliding resistance between the concrete spread/strip footings and the existing granular fill or the upper granular deposit at the abutments should be calculated in accordance with Section 6.10.5 of the *CHBDC* (2014). The unfactored coefficient of friction (tan φ’) between pre-cast and cast-in-place concrete spread/strip footings and the subgrade may be taken as follows:

Interface Materials	Unfactored Coefficient of Friction (tan δ)
Pre-cast concrete footing on existing granular fill or upper granular deposit	0.25
Cast-in-place concrete footing on existing granular fill or upper granular deposit	0.35

6.6 Resistance to Lateral Loads for Driven Piles

The design of piles subjected to lateral loads should take into account such factors as the batter of the pile (if any), the relative rigidity of the pile to the surrounding soil, the fixity condition at the head of the pile (i.e., at the pile cap level), the structural capacity of the pile to withstand bending moments, the soil resistance that can be mobilized, the tolerable lateral deflections at the head of the pile and group effects. For a longer, more flexible pile, the maximum yield moment of the pile may be reached prior to mobilization of the lateral geotechnical resistance. For design purposes, both the structural and geotechnical resistances should be evaluated to establish the governing case.

Lateral loading could be resisted fully or partially by the use of battered piles, where possible.

Where ground conditions are generally competent and the lateral loads applied to piles are relatively small such that the maximum lateral pile deflections will be relatively small, the resistance to lateral loading in front of a single pile can be estimated using subgrade reaction theory where the coefficient of horizontal subgrade reaction, k_h (kPa/m), is based on the following equations (*CFEM*, 1992 as referenced in the *Commentary of the CHBDC, 2014*):

For non-cohesive soils:

$$k_h = \frac{n_h z}{B}$$

where: n_h = coefficient related to soil density (kPa/m)
z = depth (m)
B = pile/drilled shaft diameter or width (m)

For cohesive soils:

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

where: s_u = undrained shear strength of the soil (kPa)
B = pile/drilled shaft diameter or width (m)

However, it is noted that the response of a pile to lateral loads is highly non-linear and methods that assume linear behaviour (such as subgrade reaction theory) are only appropriate where the maximum pile deflections are less than about 1% of the pile diameter, and in scenarios where the loading is static (i.e., no cyclic loading) and where the pile material is linear (CFEM, 2006). In scenarios where these conditions are not satisfied, the non-linear lateral behaviour of the soil should be considered by developing soil reaction versus pile deflection (i.e., P-y) curves.

The values of n_h (Terzaghi, 1955 and Reese, 1975) and s_u to be incorporated into the calculations of the coefficient of horizontal subgrade reaction (k_h) within the native overburden, to be used for the structural analysis of the piles at this site (for both the Achigan Creek replacement bridge and the temporary modular bridge) are summarized below.

Foundation Element	Soil Unit	Elevation	n_h	s_u
Abutments	Existing fill and/or upper granular deposit at abutments of temporary modular bridge (above groundwater table)	238.0 m to 235.6 m	5,000 kPa/m	--
	Generally very loose to loose upper granular deposit at abutments of temporary modular bridge (below groundwater table)	236.0 m to 234.4 m	3,000 kPa/m	--
	Loose sand inside CSP at the abutments of new Achigan Creek Bridge (below groundwater table)	235.8 m to 232.8 m	1,500 kPa/m	--
	Firm to very stiff varved clayey silt to silty clay	235.6 m to 233.0 m	--	65 kPa
	Firm to very stiff varved clayey silt to silty clay	233 m to 232.0 m	--	65 kPa to 50 kPa (refer to Figure 1)
	Firm to very stiff varved clayey silt to silty clay	232.0 m to 228.0m	--	50 kPa
	Stiff to very stiff clayey silt to silty clay (irregularly stratified)	228.0 m to 227.0 m	--	50 kPa to 100 kPa (refer to Figure 1)
	Stiff to very stiff clayey silt to silty clay (irregularly stratified)	227.0 m to 210.2 m	--	100 kPa

Foundation Element	Soil Unit	Elevation	n_h	s_u
	Very dense sandy silt to silty sand with cobbles and boulders	210.2 m to 206.1 m	35,000 kPa/m	--

For a single H-pile and tube pile the estimated factored ultimate geotechnical resistance and factored serviceability geotechnical resistances for 10 mm of factored horizontal deflection at the abutments are presented below. These values are based on analyses carried out using the commercially available program LPILE Plus (Version 5.0), developed by Ensoft Inc.

Foundation Element	Deep Foundation Unit	Axial Load Applied at the Top of Pile	Factored Ultimate Lateral Geotechnical Resistance	Factored Serviceability Geotechnical Resistance for 10 mm of Lateral Deflection
Achigan Creek Replacement Bridge				
Abutments	HP 310 x 110 pile (end-bearing pile; ~30 m long)	1,600 kN	175 kN [1,3]	25 kN [1,3]
	324 mm dia. tube pile; hollow (end-bearing pile; ~30 m long)	1,600 kN	40 kN [1,3]	30 kN [1,3]
	324 mm dia. tube pile; filled with concrete (end-bearing pile; ~30 m long)	1,600 kN	85 kN [1,3]	35 kN [1,3]
Temporary Modular Bridge				
Abutments	HP 310 x 110 pile (friction pile; 15 m long)	400 kN	255 kN [2, 3]	30 kN [2, 3]
	HP 310 x 110 pile (end-bearing pile; 27 m long)	1,600 kN	255 kN [2, 3]	30 kN [2, 3]
	324 mm dia. tube pile; hollow (friction pile; 15 m long)	400 kN	55 kN [3]	20 kN [3]
	324 mm dia. tube pile; hollow (end-bearing pile; 27 m long)	400 kN	55 kN [3]	20 kN [3]
	324 mm dia. tube pile; filled with concrete (friction pile; 15 m long)	400 kN	65 kN [3]	25 kN [3]
	324 mm dia. tube pile; filled with concrete (end-bearing pile; 27 m long)	1,600 kN	65 kN [3]	25 kN [3]

Notes:
1. Analysis assume that the steel H-piles at the abutments of the replacement bridge are oriented for weak axis bending.
2. Analysis assume that the steel H-piles at the abutments of the temporary modular bridge are oriented for strong axis bending.
3. Analyses assume a free-head condition at the abutments.

Based on the above, both the structural and geotechnical resistances of the piles should be evaluated to establish the governing case at ultimate limit state (ULS). At serviceability limit state (SLS), the horizontal resistance of the piles will be controlled by deflections, and the horizontal resistance of the piles should be calculated based on the coefficient of horizontal subgrade reaction (k_h) of the soil as discussed above. The SLS resistance should correspond to a factored horizontal deflection of 10 mm at the underside of the pile cap for units supporting the abutments (see Section C6.11.2.2.2 of the *Commentary to the CHBDC* (2014)).

Group action for lateral loading should also be evaluated by reducing the coefficient of horizontal subgrade reaction either in the direction of loading or perpendicular to the direction of loading by relevant group pile efficiency factors as outlined in Section C6.11.3.4 of the *Commentary to the CHBDC* (2014).

6.7 Downdrag and Drag Loads

As a result of the loading from the new approach embankments along the temporary detour alignment, elastic compression of the upper granular deposit and long-term consolidation settlement of the underlying cohesive deposit will occur. The difference in the vertical movement between the thick overburden (i.e., from the immediate settlement of the upper granular deposit and consolidation settlement of the thick cohesive deposit) and the piles (i.e., from the elastic deformation of the piles under the load from the temporary modular bridge and from the punching of the piles into the soil deposit below the pile tip) will likely result in the development of negative skin friction on the piles and downdrag. Consequently, if the piles for the temporary modular bridge are selected as the preferred foundation option and are installed prior to the construction of the approach embankments, drag loads will need to be considered in the assessment of the pile's structural capacity.

Analyses to estimate drag loads and geotechnical resistances for the recommended pile foundation option at the abutments was carried out in accordance with Section 6.11.4.10 of the *Commentary to the CHBDC* (2014) using the method proposed by Briaud and Tucker (1994). It is noted that the method used to assess the deformation of the piles and the associated drag loads is dependent on a number of factors including the pile length, foundation conditions at the pile tip, the unfactored dead load on the pile and the anticipated settlement profile of the foundation soils. If any of these factors are different from those assumed in the analysis, the estimated drag loads and pile capacities need to be reassessed.

The calculated drag loads at the neutral plane for both friction and end-bearing piles associated with the temporary modular bridge are as follows:

Pile Type	Pile Length	Estimated Depth from Ground Surface to Neutral Plane	Calculated Drag Load at Neutral Plane
Steel HP 310x110 pile	15 m	7.0 m	300 kN
	27 m	10.0 m	400 kN
324 mm diameter steel tube pile	15 m	6.5 m	225 kN
	27 m	10.0 m	375 kN

6.8 Lateral Earth Pressures

The lateral earth pressures acting on the abutment walls of the permanent bridge will depend on the type and method of placement of the backfill material, the nature of the soils behind the backfill, the magnitude of surcharge including construction loadings, the freedom of lateral movement of the structure, and the drainage conditions behind the walls.

The following recommendations are made concerning the design of abutment walls. These design recommendations and parameters assume level backfill and ground surface behind the walls. Where there is sloping ground behind the walls, the coefficient of lateral earth pressure must be adjusted to account for the slope.

- Select, free draining granular fill meeting the specifications of OPSS.PROV 1010 (*Aggregates*) Granular 'A' or Granular 'B' Type II should be used as backfill behind the walls. Longitudinal drains and weep holes should be installed to provide positive drainage of the granular backfill. Compaction (including type of equipment, target densities, etc.) should be carried out in accordance with OPSS.PROV 501 (*Compacting*). Other aspects of the granular backfill requirements with respect to sub-drains and frost taper should be in accordance with OPSD 3101.150 (*Walls, Abutment, Backfill, Minimum Granular Requirement*), OPSD 3121.150 (*Walls, Retaining, Backfill, Minimum Granular Requirement*), and OPSD 3190.100 (*Walls, Retaining and Abutment, Wall Drain*).

- A minimum compaction surcharge of 12 kPa should be included in the lateral earth pressures for the structural design of the wall stem, in accordance with *CHBDC* (2014) Section 6.12.3 and Figure 6.6. Care must be taken during the compaction operation not to overstress the abutment walls, with limitations required on heavy construction equipment and requirements for the use of hand-operated compaction equipment per OPSS.PROV 501 (*Compacting*). Other surcharge loadings should be accounted for in the design, as required.

- For restrained walls, granular fill should be placed in a zone with the width equal to at least 2.0 m behind the back of the wall (in accordance with Figure C6.20(a) of the *Commentary* to the *CHBDC, 2014*). For unrestrained walls, fill should be placed within the wedge shaped zone defined by a line drawn at 1.5 horizontal to 1 vertical (1.5H:1V) extending up and back from the rear face of the footing or base of wall (in accordance with Figure C6.20(b) of the *Commentary* to the *CHBDC, 2014*). The pressures are based on the proposed embankment fill material and the following parameters (unfactored) may be used:

Fill Type	Soil Unit Weight	Coefficients of Static Lateral Earth Pressure	
		At-Rest, K_o	Active, K_a
Granular 'A'	22 kN/m^3	0.43	0.27
Granular 'B' Type II	21 kN/m^3	0.43	0.27

Where the wall support does not allow lateral yielding, at-rest earth pressures should be assumed for the geotechnical design. Where the wall support allows lateral yielding of the stem, active earth pressures should be used in the geotechnical design of the wall structure(s). The movement required to allow active pressures to develop within the backfill, and thereby assume an unrestrained structure for design, should be calculated in accordance with Section C6.12.1 and Table C6.6 of the Commentary to the *CHBDC, 2014*.

6.9 Seismic Considerations

6.9.1 Site Class Classification

The ground conditions for seismic site characterization were established based on the results of the borehole investigation carried out at the site. Given the anticipated foundation levels (i.e., within the upper granular deposit), the site may be classified as Site Class D in accordance with Table 4.1 of the CHBDC (2014), in the absence of in-situ measured shear wave velocities. Geophysics testing (e.g., Vertical Seismic Profiling), if carried out, could provide a more favourable site classification for seismic site response.

6.9.2 Seismic Hazard and Seismic Performance Zone

Based on the location of the Achigan Creek Bridge (Latitude: 46.789744° ; Longitude: -84.054775°), the following reference Site Class C spectral acceleration values were obtained for a return period of 2,475 years (i.e., 2% exceedance in 50 years) based on the 5th generation seismic hazard maps published by the Geological Survey of Canada (GSC):

Spectral Acceleration, Sa(T [1]) [2]
Sa (0.2) = 0.064
Sa (1.0) = 0.029

Notes:
1. T is the period in seconds.
2. The spectral acceleration is given in units of g (9.81 m/s^2).

Based on the spectral acceleration values presented above, and in accordance with Table 4.10 of the *CHBDC* (2014), this site should be considered to be located in Seismic Performance Category 1. In accordance with Section 4.4.5.1 of the *CHBDC* (2014), “bridges in Seismic Category 1 need not be analyzed for seismic loads, regardless of their importance and geometry.”

6.9.3 Liquefaction Assessment

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

The liquefaction susceptibility of the granular fill and upper granular native soils encountered at the site was evaluated by comparing the estimated penetration resistance below which liquefaction could occur with the available penetration resistance. Liquefaction is predicted to occur when the available penetration resistance is less than the resistance required.

The methodology used to assess liquefaction potential at the site is consistent with that presented in the *Commentary to the CHBDC, 2014*. It involves comparing the cyclic shear stresses applied to the soil by the design earthquake, represented as the cyclic stress ratio (CSR), to the cyclic shear strength, represented as the cyclic resistance ratio (CRR) provided by the soil.

The liquefaction analysis was carried out using in-situ testing data collected at the borehole locations. The design groundwater level was assumed to be 1 m below the existing ground surface (worst case scenario, assuming a perched water table). The CRR with depth was calculated at each borehole location using the parameter, $(N_1)_{60cs}$, that is based on the SPT “N”-value obtained in the field and corrected for overburden stress, rod length during sampling, hammer energy efficiencies, and fines content.

The results of the liquefaction assessment indicate that the granular fill and upper granular deposit (comprised of silts and sands) encountered at the site are not considered to be potentially liquefiable during the 2,475-year design earthquake.

6.10 Fill Placement Along Temporary Detour Alignment

As outlined in Section 6.1, it is understood that fill placement up to about 1.2 m and 0.8 m high will be required along the temporary detour alignment on the south side and north side of the Achigan Creek, respectively in the approach area to the temporary detour structure. As such, both settlement and static global stability analyses were carried out for the temporary detour alignment.

It is also understood that the existing highway grade along the south and north bridge approaches is to remain essentially the same as part of the replacement works (i.e., grade raise in the immediate vicinity of the abutments will not exceed 150 mm). As such, the approach embankments are not expected to experience any appreciable settlement. However, static global stability of the natural valley slopes next to the abutments was evaluated based on the temporary/short-term condition and permanent/long-term condition.

6.10.1 Global Slope Stability

Limit equilibrium slope stability analyses were performed using the commercially available program Slide (Version 6.0), developed by Rocscience Inc., employing the Morgenstern-Price method of analysis. For all analyses, the Factors of Safety of numerous potential failure surfaces were computed in order to establish the minimum Factor of Safety. The Factor of Safety is defined as the ratio of forces tending to resist failure to driving forces tending to cause failure. Minimum target Factors of Safety of 1.3 and 1.5 have been used for evaluating the approach embankment and adjacent natural valley slopes for the temporary/short-term and permanent/long-term conditions, respectively, as per Table 6.2 of *CHBDC* (2014).

The simplified stratigraphy together with the foundation engineering parameters employed for the fills/soils encountered at the site are provided in Table 2.

For the non-cohesive soils present at the site, the effective stress parameters employed in the analysis were estimated from the results of a laboratory drained direct shear test as well as from empirical correlations based on the results of the in-situ Standard Penetration Tests (SPT). The correlations proposed by Peck et al (1974) and U.S. Navy (1986) were employed and the results were adjusted by engineering judgment based on precedent experience in similar soil conditions.

For the cohesive deposits, total stress parameters were employed in the analyses of the short-term, undrained conditions (i.e., temporary conditions). The total stress parameters (i.e., average mobilized undrained shear strength – s_u) for the cohesive soils were estimated from the in-situ field vane tests, correlations with the SPT results and other laboratory test data (i.e., natural water content), where appropriate. Effective stress parameters were also assigned to the cohesive deposits to evaluate the stability based on long-term, drained conditions (i.e., permanent conditions). The effective stress parameters (i.e., effective friction angle (ϕ') for the cohesive deposits were estimated from empirical correlations based on the plasticity index. The correlations proposed by Mitchell (1993), Kulhawy and Mayne (1990), and Ladd et al. (1977) were employed and the results were adjusted using engineering judgment based on precedent experience in similar (i.e., varved and layered) soil conditions.

For the purpose of the stability analyses, the groundwater level was assumed to be at approximately Elevation 236 m on the south side of the creek and at approximately Elevation 237 m on the north side of the creek. In terms of the embankment and surface geometry in the vicinity of the bridge approach embankments/abutments, the topographical survey provided by AECOM has been utilized to generate the existing ground surface line along Highway 532, the temporary detour alignment, and the river bed.

The stability analyses indicate that the front slopes at the abutments of the Achigan Creek replacement bridge will have a Factor of Safety greater than 1.3 and 1.5 during the temporary/short-term and permanent/long-term conditions, respectively, for deep-seated, global failure surfaces of the slopes that would impact the operation of the highway (refer to Figures 2A and 2B). The Factors of Safety at the south abutment and north abutment during the temporary/short-term condition are 1.5 and 1.6, respectively. The Factors of Safety associated with both the temporary/short-term and permanent/long-term conditions are the same at the south abutment (i.e., 1.5) given that the potential failure surfaces associated with the minimum Factors of Safety are confined to the upper granular deposit and do not extend into the underlying clayey silt to silty clay deposit. At the north abutment, the top of the clayey silt to silty clay deposit is encountered at a higher elevation as compared at the south abutment where the cohesive deposit is encountered much deeper, and the Factor of Safety at the north abutment during the permanent/long-term condition is 1.5.

The stability analyses indicate that front slopes at the abutments of the temporary modular bridge will have a Factor of Safety greater than 1.3 (i.e., 1.7 at the south abutment and 1.4 at the north abutment) during the temporary/short-term condition for deep-seated, global failure surfaces of the slopes that would impact the operation of the temporary detour highway (refer to Figure 3). Given that the temporary modular bridge will only be in operation while the new Achigan Creek replacement bridge is being constructed, it is considered that the minimum Factor of Safety associated with the permanent/long-term condition does not have to be satisfied at the location of temporary modular bridge.

6.10.2 Settlement

Settlement analyses along the temporary detour alignment were carried out using the commercially available program Settle^{3D} (Version 4.0), developed by Rocscience Inc.

The sources of settlement are considered to include immediate settlement of the granular soils (short-term), and primary time-dependent consolidation of the cohesive deposit (using Terzaghi's one-dimensional consolidation theory long-term). Secondary time dependent (creep) consolidation of the cohesive deposits (long-term) is anticipated to be negligible given that the cohesive deposit is over-consolidated.

The simplified stratigraphy together with the deformation and time-rate consolidation parameters, where applicable, employed for the different soil types encountered at the site are summarized in Table 1. The parameters associated with the extensive cohesive deposit encountered at the site are presented on Figure 1.

The immediate compression of the granular deposits were modelled by estimating an elastic modulus of deformation based on the SPT "N"-values and using correlations proposed by Bowles (1984) and Kulhawy and Mayne (1990). These estimated values were compared with the typical range of expected values for similar soil types, as outlined in Section C6.9.3.6 of the *Commentary to the CHBDC* (2014) and adjusted, if necessary.

The consolidation settlement of the cohesive deposit was assessed using the results of the laboratory consolidation testing, where appropriate, and in-situ field vane tests to estimate the stress history and deformation parameters for the cohesive deposits. In addition, the results of the laboratory index tests were employed to further assess deformation parameters (i.e., compression and recompression indices) using empirical correlations proposed in literature by Azzouz et al. (1976), Koppula (1986), Kulhawy and Mayne (1990), Nishida (1956) and

Terzaghi and Peck (1967). The correlation by Koppula (1986) relating the natural water content (w_n) and liquid limit (w_L) to the compression index (i.e., $C_c = 0.009 \cdot w_n + 0.005 \cdot w_L$) is considered to be the most relevant based on our experience with similar soils in Northern Ontario. The recompression index of clayey soils encountered in Northern Ontario typically varies between about 5% and 10% of the compression index. Based on the consolidation test results, a value of 5%, or $C_r = (1/20) \cdot C_c$ was selected.

The coefficient of consolidation, c_v (cm^2/s), required in the time-rate settlement analysis was established using the results of the laboratory consolidation tests and also estimated from the U.S. Navy (1986) correlation with liquid limit assuming over-consolidated soils.

The results from the settlement assessment along the temporary detour alignment are summarized as follows:

Location	***Unfactored Settlement*** [1]				***Factored Settlement*** [1]			
	$\delta_{Immediate}$	$\delta_{Primary}$	$\delta_{Secondary}$	δ_{Total} [2]	$\delta_{Immediate}$	$\delta_{Primary}$	$\delta_{Secondary}$	δ_{Total} [2]
Approaches	*10 mm*	*5 mm*	*~0 mm*	*15 mm*	*~15 mm*	*~5 mm*	*~0 mm*	*~20 mm*

Notes:

1. The estimated magnitudes of settlement correspond to the end of the first year following construction of the temporary modular bridge.
2. The total settlement (δ_{total}) is defined as the sum of the immediate settlement ($\delta_{immediate}$) due to elastic compression of the non-cohesive deposits as well as primary ($\delta_{primary}$) and secondary ($\delta_{secondary}$) settlements due to time-dependent consolidation of the cohesive deposits.

The estimated magnitudes of settlement generally represent the maximum amount of settlement that will be experienced along the temporary detour alignment. However, given the variable existing ground surface along the proposed temporary detour alignment, and considering that in places there will be relatively insignificant fill placement or even minor earth cuts to achieve the roadway profile, the settlement will be negligible in places. Regardless, the estimated settlements along the temporary detour embankments over the two-year operational life are minor (i.e., less than 25 mm) and as such, settlement mitigation measures are not required along the temporary detour alignment; however, minor maintenance of the roadway may be required during operation of the temporary detour alignment.

6.11 Construction Considerations

This section identifies key construction considerations that may impact the design and construction of the Achigan Creek replacement bridge and the temporary modular bridge.

6.11.1 Open-Cut Excavations

All excavations at the abutments of the Achigan Creek replacement bridge and the temporary modular bridge must be carried out in accordance with Ontario Regulation 213 (Ontario Occupation Health and Safety Act for Construction Projects), as amended, and OPSS 902 (*Excavating and Backfilling – Structures)*.

The soils to be excavated can be classified according to OHSA as follows (assuming the groundwater level is below the foundation subgrade level):

- Existing granular fill – Type 3; and,
- Generally very loose to loose sandy silt to silt and sand to silty sand to sand – Type 4.

Temporary excavations (i.e., those open for a relatively short period of time) should be made with side slopes no steeper than 1H:1V and 3H:1V in Type 3 and Type 4 soils, respectively. However, if water inflow is observed, flatter slopes and dewatering measures may need to be implemented. Temporary excavations should be observed by a qualified geotechnical engineer and reviewed during construction to confirm that the soil and groundwater conditions encountered are as anticipated in this report. If unexpected conditions are encountered, the geotechnical engineer should review the excavation plan based on the conditions encountered at that time.

6.11.2 Fill Placement Along Temporary Detour Alignment

Placement of granular fill (Granular 'B' Type I or Type II) above the water table for construction of temporary embankments along the temporary detour alignment should be carried out in accordance with the requirements as outlined in OPSS.PROV 206 (*Grading*). The granular fill should be placed and compacted in accordance with OPSS.PROV 501 (*Compacting*). Inspection and field testing should be carried out by a qualified personnel during construction to confirm that appropriate materials are used and that adequate levels of compaction are being achieved. Side slopes for the granular fill roadway embankment should be no steeper than 2H:1V.

6.11.3 Control of Groundwater and Surface Water

As described in Section 4.3, the majority of the boreholes were advanced using wash-boring techniques which involved the introduction of drilling water, and the water level measurements taken upon completion of drilling operations are not considered representative of the groundwater conditions at the site. However, the lower portion of the upper granular deposit, which was typically advanced using hollow-stem or solid-stem augers, was noted to be wet. Wet soil samples were collected below elevations ranging between about 236.9 m and 234.1 m, and on average below approximately Elevation 235.8 m. Furthermore, groundwater level measurements were taken between August 12 and 15, 2018 in a standpipe piezometer installed at the west corner of the south bridge abutment. The groundwater level in the piezometer was measured at approximately Elevation 233.8 m. Therefore, based on the assumption that the footings or pile caps will be founded at approximately Elevation 236.0 m, it is anticipated that no significant groundwater control measures will be required. However, given the potential for encountering a perched water table, some form of groundwater control will be required. If the water level is at or slightly above to the founding elevation, it is assumed that a sump pump system will be adequate, but if the water level is near the ground surface at the time of construction, a more extensive dewatering system and/or a groundwater cut-off system may need to be implemented. In this case, if construction water pumping volumes are anticipated to exceed 50 m^3/day, an Environmental Activity Section Registry (EASR) will be required as per the relatively recent changes to the Environmental Protection Act by the Ontario Ministry of Environment.

Surface water should be directed away from the excavation areas to prevent ponding of water that could result in disturbance/loosening of the foundation subgrade.

6.11.4 Subgrade Protection

The overburden soils exposed at the founding level of shallow foundations, if selected as the preferred option at the location of the temporary modular bridge, will be likely susceptible to disturbance from construction traffic and/or ponded water. To limit the effect of this disturbance, a concrete working slab should be placed on the subgrade if the concrete footings are not placed within four hours after preparation, inspection and approval of the subgrade. The minimum thickness of the concrete working slab should be 100 mm and the concrete should have a 28-day compressive strength of not less than 20 MPa. A sample Non-Standard Special Provision to address this requirement is included in Appendix E.

6.11.5 Timber Cribbing Construction

Timber cribbing, if selected as the preferred foundation option for the temporary modular bridge, should be constructed in accordance with OPSS.PROV 918 (*Modular Bridge Structures*). The timber cribbing should be founded 2 m below the existing ground surface on a 300 mm thick OPPS.PROV 1010 Granular' A' or Granular 'B' Type II pad compacted to a minimum of 95% of the material's Standard Proctor maximum dry density in accordance with OPSS.PROV 501 (*Compacting*). If the timber cribbing is not founded below the frost penetration depth of 2 m, the timber cribbing and the structure itself may experience significant deformation as a result of freeze/thaw cycles, particularly considering that the temporary modular bridge is expected to be in use for one year. The effects of frost heave on timber crib foundations could be reduced by sub-excavation and replacement of the upper granular soils below the cribs with non-frost susceptible materials, however, if the cribs are founded at a depth less than 2 m below the final adjacent ground surface, the geotechnical resistances provided in Section 6.3.1 will have to be revised.

The timber cribbing should be filled with a durable, very strong (Grade R5) rock fill. The bearing pad/plate resting on the rock fill/timber cribbing should be suitably sized and a levelling layer of Granular 'B' Type II should be placed between the rock fill and the pad/plate to provide for an even load distribution and avoid point contact/stress concentrations from the rock fill on the pad.

6.11.6 Obstructions During Pile Driving

As described in Section 4.2.6, the lower granular deposit encountered below the clayey silt to silty clay deposit contains a significant amount of crushed rock fragments, cobbles and boulders which are generally comprised of strong to very strong granite or basalt rock. It is anticipated these obstructions will affect the installation of steel H-piles as the piles may hang-up on the cobbles and boulders.

The steel H-piles should be reinforced with Type I driving shoes in accordance with OPSD 3000.100 (*Foundation Piles – Steel H-Pile Driving Shoe*), for protection during pile driving. Pile installation and driving shoes should also satisfy OPSS.PROV 903 (*Deep Foundations*) requirements.

A sample Non-Standard Special Provision to address obstructions is included in Appendix E.

6.11.7 Control of Fines Migration

As a result of potential artesian groundwater conditions encountered at the site (as noted in Borehole ACB-02), a seepage control system/sand filter comprised of a concrete sand drainage blanket wrapped in a geotextile and containing collector pipes is recommended to control migration of fines that may be brought up along the piles driven into the lower granular deposit due to water flow under artesian pressure, during and following the pile driving operations.

The drainage blanket should consist of a minimum 0.5 m thick layer of concrete fine aggregate meeting the gradation requirements of OPSS.PRVO 1002 (*Aggregates – Concrete*). The concrete fine aggregate should extend a minimum of 0.5 m horizontally beyond each of the piles. Appropriate drainage from under the pile cap should be provided for the granular blanket, such as by using a 100 mm perforated subdrain in accordance with OPSS.PROV 405 (*Pipe Subdrains*) wrapped in a knitted sock geotextile and draining to an adjacent ditch. The geotextile surrounding the drainage blanket should consist of a non-woven, Class 1 geotextile with filtration opening size (FOS) as specified in the Contract Documents in accordance with OPSS.PROV 1860 (*Geotextiles*).

6.11.8 Vibration Monitoring During Pile Driving

If driven steel piles are adopted at this site, it is recommended that a pre-condition survey and monitoring of vibrations be conducted at the surrounding building locations during construction to defend against potential damage claims by the home owners. A maximum peak particle velocity (PPV) threshold of 25 mm/s is generally considered applicable for residential buildings and should be specified as a limit in the contract. A sample Non-Standard Special Provision has been provided in Appendix E.

6.11.9 Erosion Protection

Provisions should be made for scour and erosion protection to be constructed along the front slopes adjacent to the north and south bridge abutments of the Achigan Creek replacement bridge. The requirements for, and design of, the erosion protection measures (i.e., size, thickness and extent(s)) should be assessed by a hydraulic design engineer.

6.11.10 Analytical Testing of Construction Material

The results of analytical tests carried out on two samples from the upper granular deposit recovered from Boreholes ACB-04 and ACB-06 are summarized in Section 4.4 and on the Certificates of Analysis in Appendix D.

The analytical test results were compared to CSA A23.1 Table 3 (*Additional requirements for concrete subjected to sulphate attack*) to assess the potential severity of sulphate attack on concrete during its service life. The sulphate concentrations measured on the soil samples are less than 0.002%, which is below the moderate degree of exposure (i.e., below the class S-3 exposure limits). Therefore, based on the two soil samples tested, when the designer is selecting the exposure class for concrete elements, the effects of sulphates from within the non-cohesive deposit in contact with the concrete elements of the foundation and any portion of the proposed structure constructed below the ground surface may not need to be considered. However, if the proposed structure is expected to be exposed to de-icing salt or other solutions, consideration should be given by the designer to designing the concrete structure for a “C” type exposure class as defined by CSA A23.1 Table 1.

The analytical test results of the soil samples were also compared to Table 7.1 (Relative Effect of Resistivity on Corrosion Potential/Aggressiveness (from NCHRP 1978)), as presented in the Federal Highway Administration/National Highway Institute Publication No. FHWA-NHI-14-007 (Federal Highway Administration, 2015), to assess the relative level of corrosion potential on buried steel in contact with soil. The resistivity values measured on the granular soil samples were 7,200 ohm·cm and 7,300 ohm·cm, indicating a “mildly corrosive” potential.

It is also noted that the measured pH level measured on a sample recovered from Borehole ACB-06 is below 6, suggesting the presence of acidic soils.

Ultimately, it is the designer’s decision to determine the appropriate exposure class and to ensure that all aspects of CSA A23.1 Section 4.1.1 (Durability Requirements) are satisfied.

7.0 CLOSURE

This Foundation Design Report was prepared by Mr. Tomasz Zalucki, P.Eng., a geotechnical engineer with Golder. Mr. Paul Dittrich, P.Eng., a MTO Foundations Designated Contact and Principal for Golder, conducted an independent quality control review of this report.

Report Signature Page

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ASTM D1586	Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils
ASTM D2573	Standard Test Method for Field Vane Shear Strength Test in Cohesive Soils

Canadian Standards Association (CSA):

CAN/CSA A23.1-14	Concrete Materials and Methods of Concrete Construction

Commercial Software:

LPLE Plus (Version 5.0) by Ensoft Inc.

Settle (Version 4.0) by Rocscience Inc.

Slide (Version 6.0) by Rocscience Inc.

Ontario Occupational Health and Safety Act:

Ontario Regulation 213	Construction Projects (as amended)

Ontario Provincial Standard Specifications (OPSS), Construction:

OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 405	Construction Specification for Pipe Subdrains
OPSS.PROV 501	Construction Specification for Compacting
OPSS 902	Construction Specification for Excavating and Backfilling - Structures
OPSS.PROV 903	Construction Specification for Deep Foundations
OPSS.PROV 918	Construction Specification for Modular Bridge Structures for Temporary Installations

Ontario Provincial Standard Specifications (OPSS), Materials:

OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1860	Material Specification for Geotextiles

Ontario Provincial Standard Drawings (OPSD):

OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe
OPSD 3090.100	Foundation, Frost Depths for Northern Ontario
OPSD 3101.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 3190.100	Walls, Retaining and Abutment, Wall Drain

Ontario Regulations:

R.R.O 1990, Regulation 903	Wells, under Ontario Water Resources Act, R.S.O. 1990, c. O.40

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DRAWINGS

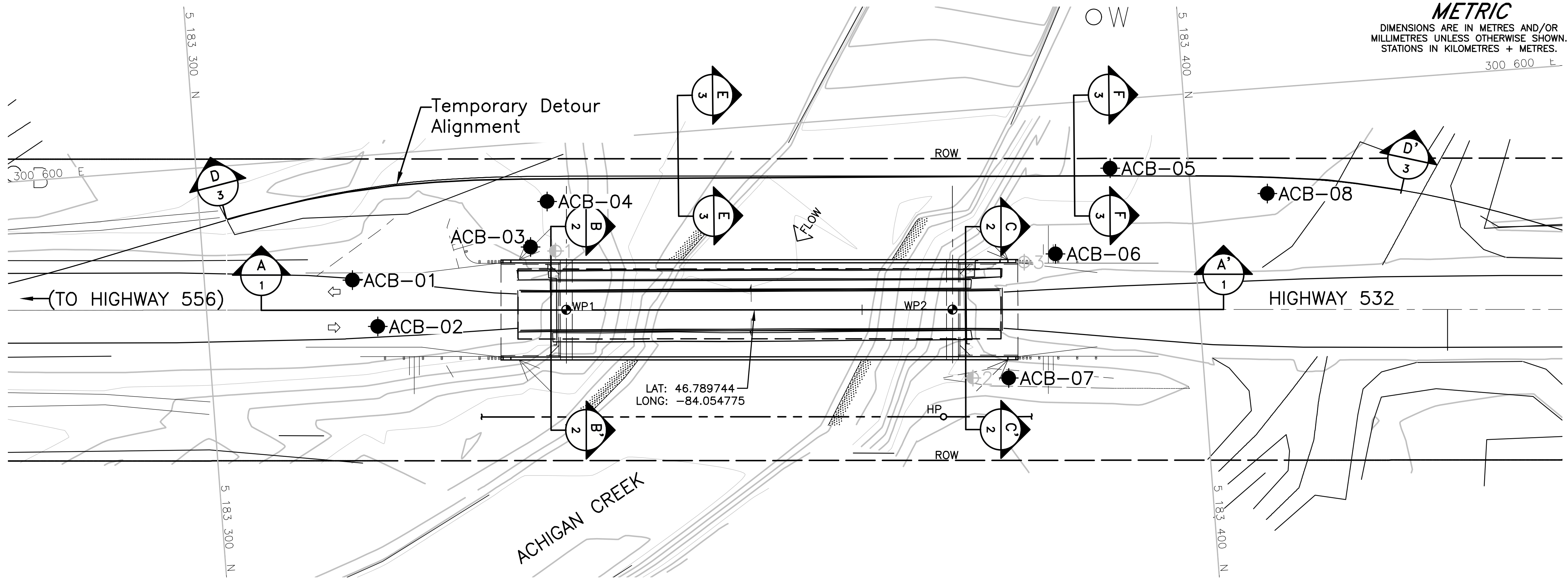
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 151-97-01

HIGHWAY 532
ACHIGAN CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

Golder Associates



PLAN
SCALE
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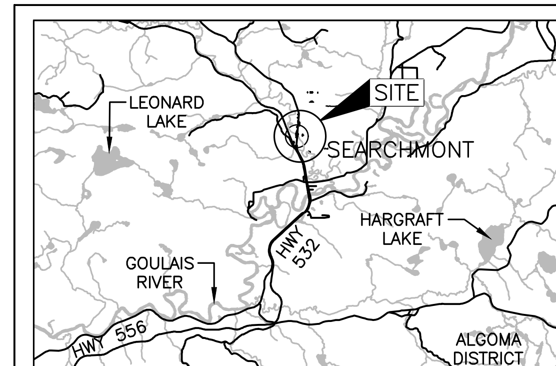


KEY PLAN
SCALE
2 0 2 4 km

LEGEND

- Borehole – Current Investigation
- Borehole – Previous Investigation (MTO Geocres No. 41K-041)
- DCPT – Previous Investigation (MTO Geocres No. 41K-041)
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- Piezometer
- WL in piezometer, measured on August 15, 2018

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 13)

No.	ELEVATION	NORTHING	EASTING
1	237.8	5183335.6	300611.1
2	237.8	5183376.7	300627.0
3	238.3	5183382.7	300615.7
ACB-01	238.9	5183314.9	300612.5
ACB-02	238.9	5183317.1	300617.3
ACB-03	238.3	5183333.1	300610.5
ACB-04	238.0	5183335.1	300606.0
ACB-05	237.8	5183392.1	300606.9
ACB-06	238.8	5183385.9	300615.1
ACB-07	238.2	5183380.3	300627.3
ACB-08	238.4	5183407.7	300610.7

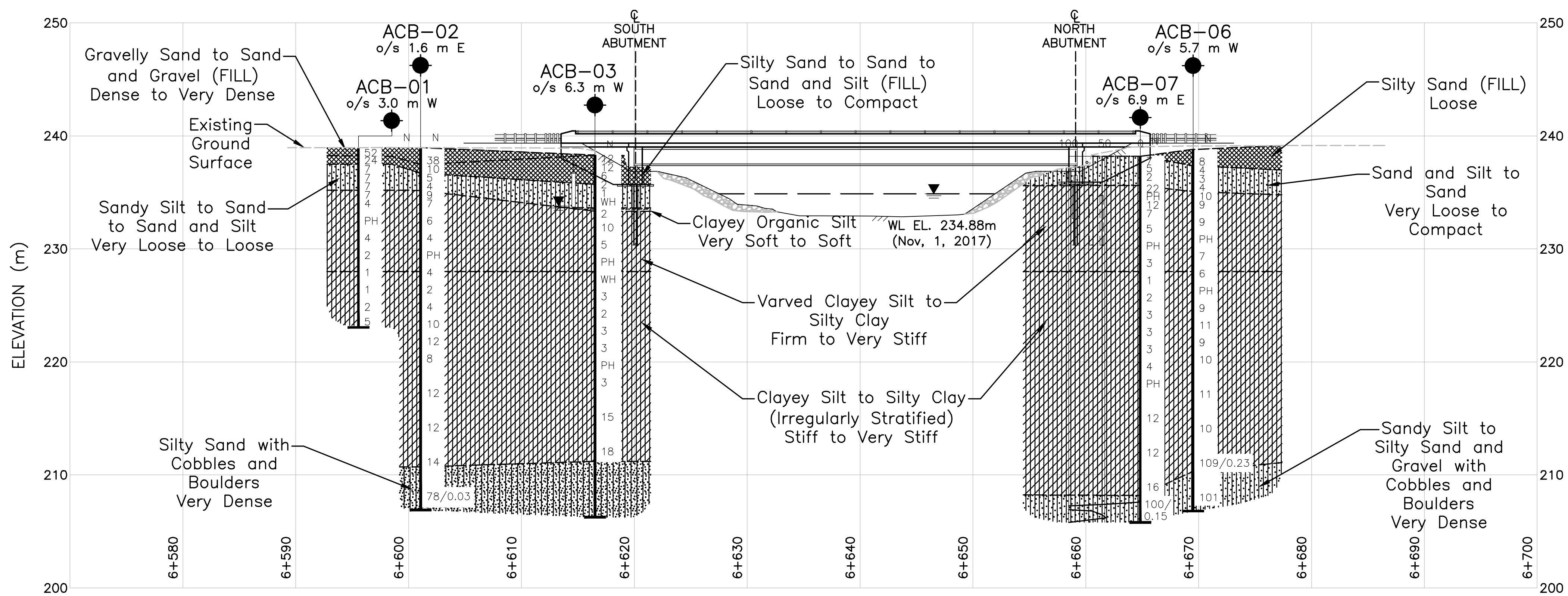
NOTES

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

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

REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. 60546679-S1.dwg and GWP 5378-11-00 Achigan Creek Bridge Detour Alignment.dwg, received on August 27, 2018.



A–A' 1 CENTRELINE PROFILE
ACHIGAN CREEK BRIDGE
SCALE
5 0 5 10 m

LICENSED PROFESSIONAL ENGINEER J.P. DITTRICH August 29, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T.ZALUCKI 100128614 August 29, 2018 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	

Geocres No. 41K-108

HWY. 532		PROJECT NO. 1670846	DIST. ALGOMA
SUBM'D. AK	CHKD.	DATE: 8/29/2018	SITE: 38S-041
DRAWN: TB	CHKD. TZ	APPD. JPD	DWG. 1

PLOT DATE: August 30, 2018

MINISTRY OF TRANSPORTATION, ONTARIO

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

CONT No.
WP No. 151–97–01

HIGHWAY 532
ACHIGAN CREEK BRIDGE
SOIL STRATA

SHEET

Golder Associates

LEGEND

Symbol	Description
●	Borehole – Current Investigation
N	Standard Penetration Test Value
16	Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
	Piezometer
▼	WL in piezometer, measured on August 15, 2018

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 13)			
No.	ELEVATION	NORTHING	EASTING
ACB–01	238.9	5183314.9	300612.5
ACB–02	238.9	5183317.1	300617.3
ACB–03	238.3	5183333.1	300610.5
ACB–06	238.8	5183385.9	300615.1
ACB–07	238.2	5183380.3	300627.3

NOTES

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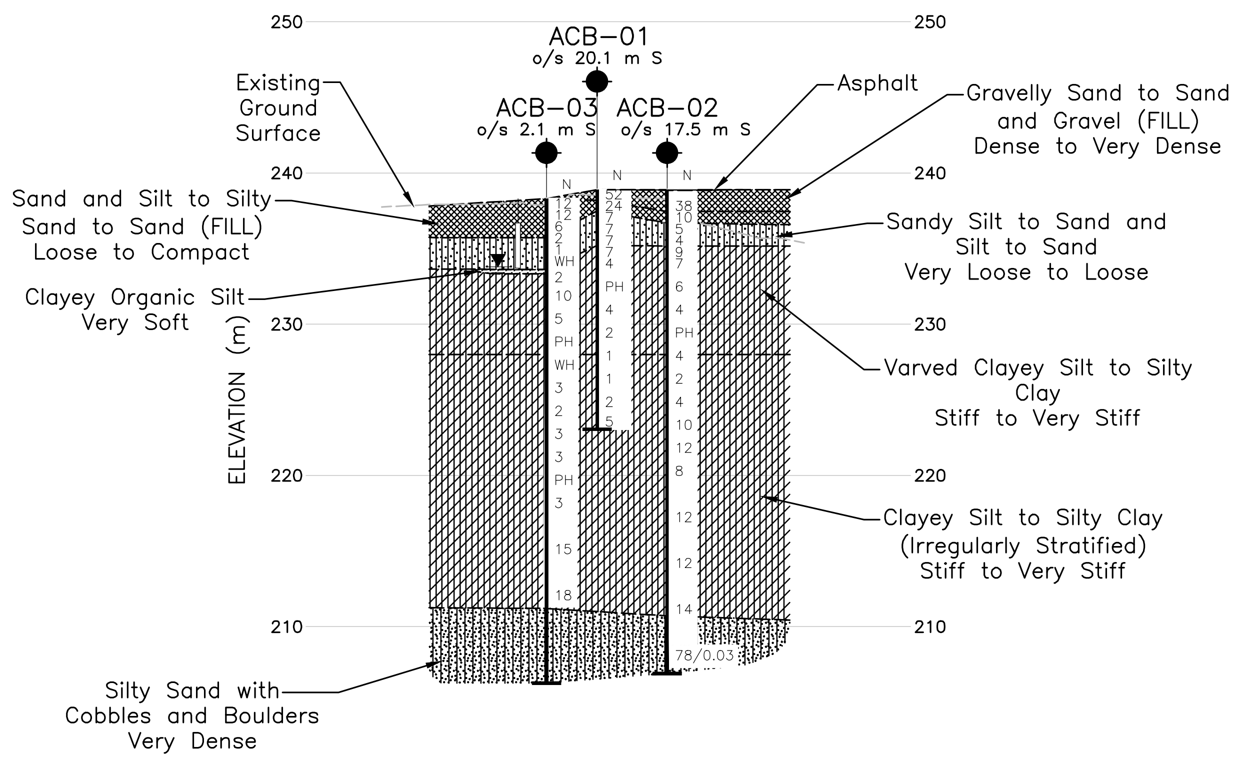
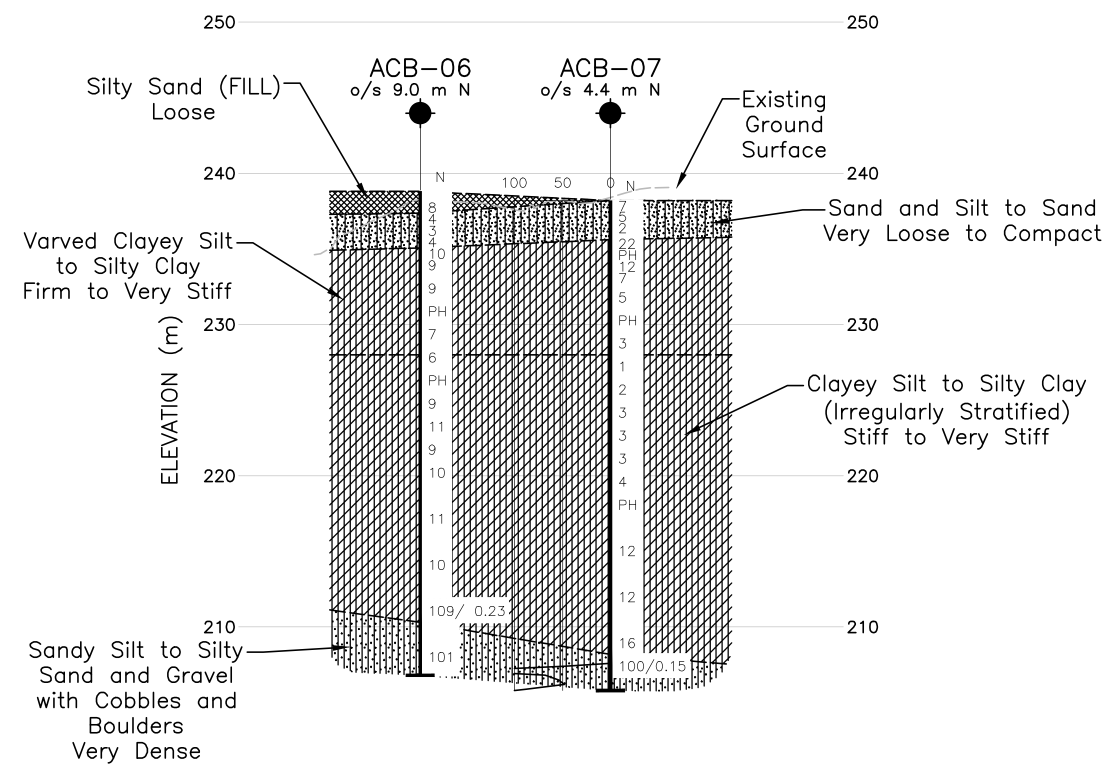
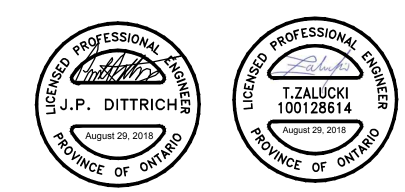


B-B' 1 SOUTH ABUTMENT CROSS–SECTION
ACHIGAN CREEK BRIDGE
SCALE 5 0 5 10 m



C-C' 1 NORTH ABUTMENT CROSS–SECTION
ACHIGAN CREEK BRIDGE
SCALE 5 0 5 10 m



NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 41K–108

HWY. 532		PROJECT NO. 1670846	DIST. ALGOMA
SUBM'D. AK	CHKD.	DATE: 8/29/2018	SITE: 38S–041
DRAWN: TB	CHKD. TZ	APPD. JPD	DWG. 2

PLOT DATE: August 30, 2018

MINISTRY OF TRANSPORTATION, ONTARIO

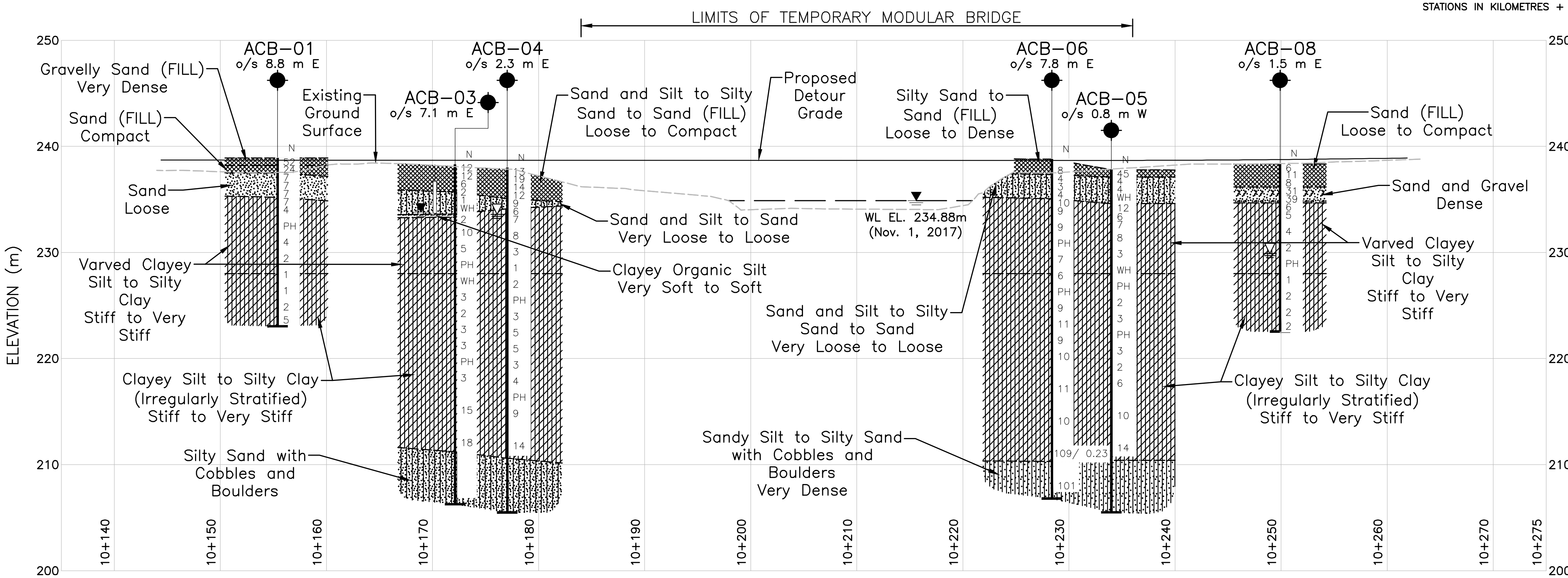
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES.

CONT No.
WP No. 151-97-01

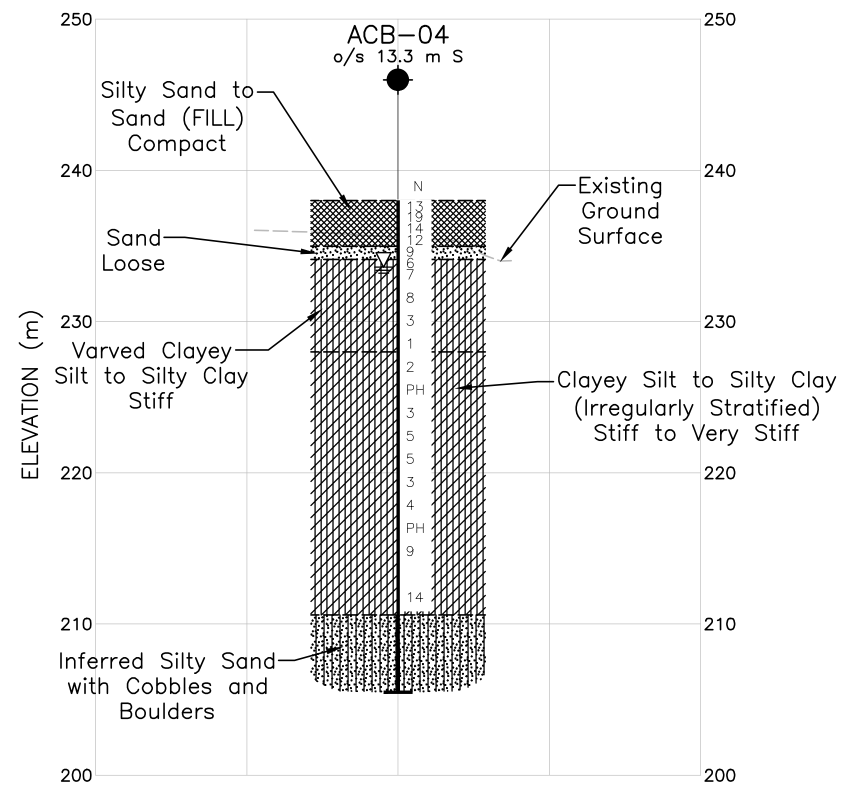
HIGHWAY 532
TEMPORARY MODULAR BRIDGE
SOIL STRATA

SHEET

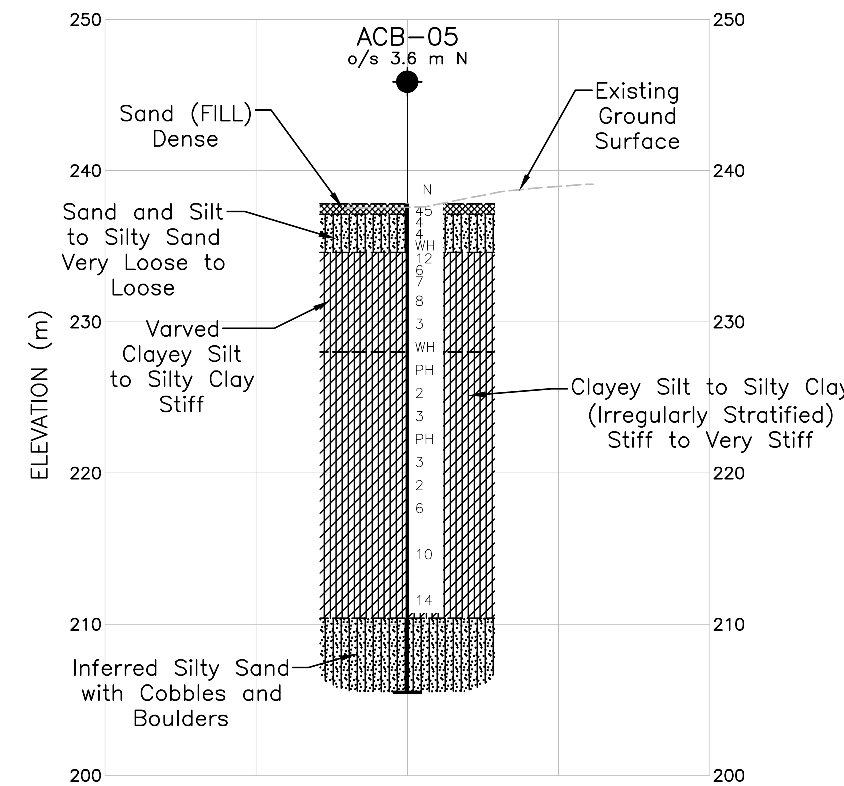
Golder Associates



D-D' 1 CENTRELINE PROFILE
TEMPORARY MODULAR BRIDGE
SCALE 5 0 5 10 m



E-E' 1 SOUTH ABUTMENT CROSS-SECTION
TEMPORARY MODULAR BRIDGE
SCALE 5 0 5 10 m



F-F' 1 NORTH ABUTMENT CROSS-SECTION
TEMPORARY MODULAR BRIDGE
SCALE 5 0 5 10 m

LEGEND

- Borehole – Current Investigation
- N Standard Penetration Test Value
- 16 Blows/0.3m unless otherwise stated (Std. Pen. Test, 475 j/blow)
- WL upon completion of drilling
- Piezometer
- WL in piezometer, measured on August 15, 2018

BOREHOLE CO-ORDINATES (MTM NAD83 ZONE 13)

No.	ELEVATION	NORTHING	EASTING
ACB-01	238.9	5183314.9	300612.5
ACB-03	238.3	5183333.1	300610.5
ACB-04	238.0	5183335.1	300606.0
ACB-05	237.8	5183392.1	300606.9
ACB-06	238.8	5183385.9	300615.1
ACB-08	238.4	5183407.7	300610.7

NOTES

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REFERENCE

Base plans provided in digital format by AECOM, drawing file nos. 60546679-S2.dwg and GWP 5378-11-00 Achigan Creek Detour Alignment.dwg, received on August 27, 2018.

LICENSED PROFESSIONAL ENGINEER J.P. DITTRICH August 29, 2018 PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER T. ZALUCKI 100128614 August 29, 2018 PROVINCE OF ONTARIO

NO.	DATE	BY	REVISION
.	.	.	.

Geocres No. 41K-108

HWY. 532	PROJECT NO. 1670846		DIST. ALGOMA
SUBM'D. AK	CHKD.	DATE: 8/29/2018	SITE: 38S-041
DRAWN: TB	CHKD. TZ	APPD. JPD	DWG. 3

PLOT DATE: August 30, 2018

TABLES

TABLE 1A – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES (ACHIGAN CREEK BRIDGE REPLACEMENT BRIDGE)

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread/strip footings founded on the upper granular deposit	• Feasible, but low geotechnical resistances (not recommended from a foundations perspective).	• Conventional construction techniques. • Generation of less noise and vibration during construction compared to deep foundation alternatives, especially driven steel piles.	• Does not allow for integral abutment construction. • Requirement for excavations up to about 2.0 m deep for frost protection. • Low geotechnical resistances.	• Lower relative cost than deep foundations.	• As a result of low geotechnical resistances, ground improvement / settlement mitigation measures would have to be implemented in order to make this option viable from a foundations point of view.
End-Bearing Piles: steel H-piles (HP 310x110) driven into the lower granular deposit (end-bearing piles)	• Feasible and preferred from a foundations perspective. ***Note***: H-piles are a lower displacement pile and likely offer constructability and driveability advantages over tube piles, as such, tube piles driven to the lower granular deposit are not recommended.	• Conventional construction methods for H-pile foundations. • Allows for integral abutment design.	• Requirement for excavations up to about 2.0 m deep for frost protection of pile caps. • Long piles (about 30 m) – requirement for at least one splice. • Requires driving shoes due to presence of cobbles/boulders within the lower granular deposit. • As a result of potential artesian groundwater conditions, a seepage control system/sand filter is required at each abutment. • Noise nuisance from pile driving hammer to nearby residents. • Requires pre-condition survey and monitoring of vibrations at surrounding building locations during pile driving.	• Higher relative cost than spread/strip footings and friction piles. • Lower relative cost than drilled shafts (caissons). • Additional cost for driving shoes or standard pile points. • Additional cost for a seepage control system/sand filter at each abutment. • Additional cost for a pre-condition survey and vibration monitoring.	• Risk of piles hitting existing timber piles which could result in misalignment and/or damage of the new piles. • Risk of H-piles hanging up above the design pile tip elevation, or of damage to the piles, due to cobbles and boulders present within the lower granular deposit. • Risk of migration of fines along the piles due to water flow under artesian pressure, during and following the pile driving operations.

TABLE 1A – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES (ACHIGAN CREEK BRIDGE REPLACEMENT BRIDGE)

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Friction Piles: steel H-piles (HP 310x110) or tube piles (324 mm outer diameter) driven into the cohesive deposit	• Feasible, but the use of shorter friction piles may not be practical as a result of the lower geotechnical resistance available for these piles.	• Conventional construction methods for H-pile / tube pile foundations. • Allows for integral abutment design, but tube piles mat not be feasible for integral abutment design.	• Requirement for excavations up to about 2.0 m deep for frost protection of pile caps. • Lower geotechnical resistance compared to end-bearing piles, and therefore, a larger number of (shorter) piles is required. • Requires pre-condition survey and monitoring of vibrations at surrounding building locations during pile driving.	• Higher relative cost than spread/strip footings. • Cost for friction piles may be very similar to the end-bearing piles option since piles are shorter, but a larger number is required. • Additional cost for a pre-condition survey and vibration monitoring.	• Lower geotechnical resistance associated with friction piles. • Low risk of pile group settlement since piles are not end-bearing.
Drilled shafts founded within the lower granular deposit (caissons)	• Feasible, but less economical than driven steel piles and relatively high risk of not achieving adequate geotechnical resistances due to potential artesian groundwater conditions with the lower granular deposit which may disturb the base of the caisson.	• Conventional construction methods for drilled shaft foundations. • Offers higher geotechnical resistance compared to driven steel piles, requiring fewer foundation elements. • Drilled shafts can be affixed to underside of the bridge deck to reduce the amount of excavation.	• Precludes use of integral abutments. • Long drilled shafts (about 30 m long). • Temporary or potentially permanent liners will be required, plus special measures such as use of bentonite or polymer drilling fluid to counterbalance groundwater pressures and minimize disturbance at the founding level of the drilled shaft. • Generation of soil cuttings during drilled shaft advancement. • Potential for noise or vibration nuisance to nearby residents during liner installation and/or removal.	• Higher relative cost than spread/strip footings and driven steel piles. • Additional cost associated with the use of temporary or permanent liners and special drilling fluids.	• High risk of caisson refusal on cobbles and boulders. • Given the high likelihood of encountering artesian groundwater conditions when penetrating the lower granular deposit, the use of drilled shafts will carry high risks associated with maintaining a stable and undisturbed base during construction. • May not be possible to inspect the base of the drilled shaft due to length of foundation element and need for bentonite or polymer drilling mud inside the liners.

TABLE 1B – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES (TEMPORARY MODULAR BRIDGE)

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
Spread/strip footings founded on the upper granular deposit	• Feasible	• Conventional construction techniques. • Generation of less noise and vibration during construction compared to deep foundation alternatives, especially driven steel piles.	• Requirement for excavations up to about 2.0 m for frost protection. • Relatively low geotechnical resistances.	• Lower relative cost than deep foundations. • Additional cost associated with decommissioning the temporary foundation system upon opening the new Achigan Creek replacement bridge.	• Depending on the selected dimension of the footings and the load imposed by the bridge, the structure may settle more than 25 mm and maintenance (e.g. shimming) during construction may be required.
Bearing pads/plates sitting on top of timber cribbing at each corner of the bridge and founded on the upper granular deposit	• Feasible	• Conventional construction techniques. • Quick installation procedure. • Concrete is not required to construct this foundation system. • Generation of less noise and vibration during construction compared to deep foundation alternatives, especially driven steel piles.	• Requirement for excavations up to about 2.0 m deep for frost protection. • Good quality/very strong rock fill required to fill the timber cribbing. • Less robust system compared to concrete spread/strip footings and driven steel piles. • Relatively low geotechnical resistances.	• Lower relative cost than spread/strip footings and deep foundations. • Additional cost associated with importing rock fill to fill the timber cribbing. • Additional cost associated with decommissioning the temporary foundation system upon opening the new Achigan Creek replacement bridge.	• Depending on the selected dimension of the timber cribbing, the quality of the rock fill, and the load imposed by the bridge, the structure may settle more than 25 mm and maintenance (e.g., shimming) during construction may be required.
Friction Piles: steel H-piles (HP 310x110) or tube piles (324 mm outer diameter) driven into the cohesive deposit	• Feasible and preferred from a foundations perspective	• Conventional construction methods for H-pile and tube pile foundations. • Relatively short piles (15 m long) – splicing is not required. • Higher geotechnical resistances compared to the shallow foundation option.	• Requirement for excavations up to about 2.0 m deep for frost protection of pile caps. • Friction piles will experience downdrag and drag loads. • Noise nuisance to nearby residents. • Requires pre-condition survey and monitoring of vibrations at surrounding building locations during pile driving.	• Higher relative cost than spread/strip footings. • Lower relative cost than drilled shafts (caissons). • Additional cost associated with decommissioning the temporary foundation system (i.e., extracting piles or cutting-off upper portion of piles) upon opening the new Achigan Creek replacement bridge. • Additional cost for a pre-condition survey and vibration monitoring.	• Low risk of piles not achieving desired geotechnical resistances; piles can be driven deeper if higher resistances are required. • Drag loads need to be considered in the assessment of the pile's structural capacity.

TABLE 1B – COMPARISON OF POTENTIAL FOUNDATION ALTERNATIVES (TEMPORARY MODULAR BRIDGE)

Foundation Option	Feasibility	Advantages	Disadvantages	Relative Costs	Risk / Consequences
End-Bearing Piles: steel H-piles (HP 310x110) driven into the lower granular deposit (end-bearing piles)	• Feasible, but given low structural loads imposed by the temporary modular bridge, the use of long, end-bearing piles may not be warranted.	• Conventional construction methods for H-pile foundations.	• Requirement for excavations up to about 2.0 m deep for frost protection of pile caps. • Long piles (about 30 m) – requirement for at least one splice. • Requires driving shoes due to presence of cobbles/boulders within the lower granular deposit. • As a result of potential artesian groundwater conditions, a seepage control system/sand filter is required at each abutment. • Noise nuisance from pile driving hammer to nearby residents. • Requires pre-condition survey and monitoring of vibrations at surrounding building locations during pile driving.	• Higher relative cost than spread/strip footings and friction piles. • Additional cost for driving shoes or standard pile points. • Additional cost for a seepage control system/sand filter at each abutment. • Additional cost for a pre-condition survey and vibration monitoring.	• Risk of H-piles hanging up above the design pile tip elevation, or of damage to the piles, due to cobbles and boulders present within the lower granular deposit. • Risk of migration of fines along the piles due to water flow under artesian pressure, during and following the pile driving operations.
Drilled shafts (caissons)	• Given the relatively low structural loads imposed by the temporary modular bridge and considering that the structure is expected to remain in operation for only up to about two years, the use of drilled shafts to support the temporary modular bridge is economically unwarranted.				

TABLE 2 – SUMMARY OF FOUNDATION ENGINEERING PARAMETERS

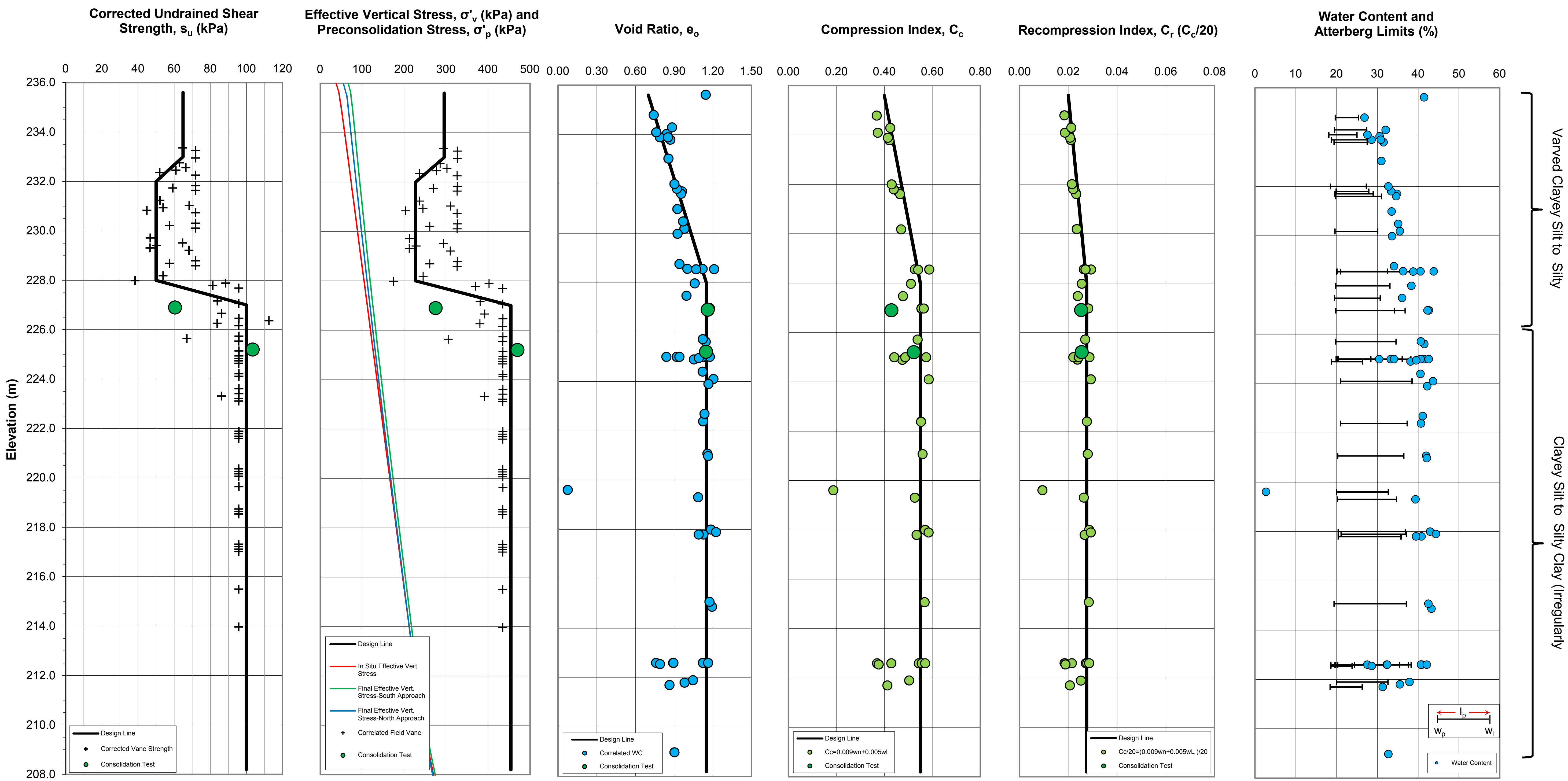
Foundation Investigation Area (Relevant Boreholes)	Stratigraphic Unit	Top Elevation (m)	Thickness (m)	γ (kN/m^3)	ϕ' (°)	c' (kPa)	s_u (kPa)	σ_p' (kPa)	e_o	C_c	C_r	m_v (kPa^{-1})	E' (MPa)	c_v (cm^2/s)
Achigan Creek Bridge and Temporary Modular Bridge (ACB-01 to ACB-08)	New Granular Fill	~238.7	Up to about 1.2 along the detour alignment	21	35	0	--	--	--	--	--	--	--	--
	Sandy Silt to Silty Sand to Sand to Sand and Gravel (Fill)	238.9 – 237.8	0.7 – 3.0	19	30	0	--	--	--	--	--	--	10	--
	Sandy Silt to Silt and Sand to Silty Sand to Sand (Upper Granular Deposit)	238.2 – 235.0	0.9 – 2.6	19	34 [2]	0	--	--	--	--	--	--	5	--
	Sand and Gravel (Upper Granular Deposit)	~236.2	~1.5	21	35	0	--	--	--	--	--	--	35	--
	Varved Clayey Silt to Silty Clay	235.6 – 233.3	5.3 – 7.6	17.5	26 – 28	0	50 – 65 [3]	225 – 295 [3]	0.70 – 1.15 [3]	0.40 – 0.55 [2]	0.020 – 0.0275 [2]	1.5×10^{-4}	--	1.2×10^{-2}
	Clayey Silt to Silty Clay (Irregularly Stratified)	~228.0	16.8 – 19.8	17.5	28	0	50 – 100 [3]	225 – 455 [3]	1.15 [3]	0.55 [3]	0.0275 [2]	1.5×10^{-4}	--	1.2×10^{-2}
	Sandy Silt to Silty Sand to Silty Sand and Gravel with Cobbles and Boulders (Lower Granular Deposit)	211.2 – 208.2 [1]	2.4 – 5.1 [1]	22	36	0	--	--	--	--	--	--	100	--

Notes:
1. The lower granular deposits was not fully penetrated; Boreholes ACB-02 to ACB-07 were terminated within the lower granular deposit.
2. The effective friction angle was based on the results of a laboratory consolidated drained direct shear test carried out on a sample of sand and silt to silty sand (upper granular deposit) recovered from Borehole ACB-05 (refer to Figure C4 in Appendix C).
3. Complete plots of the parameters (i.e., undrained shear strength (s_u), preconsolidation stress (σ_p'), void ratio (e_o), compression index (C_c) and recompression index (C_r)) versus elevation for the cohesive deposit are presented on Figure 1

FIGURES

SUMMARY PLOT OF ENGINEERING PARAMETERS FOR COHESIVE DEPOSIT
Highway 532 - Achigan Creek Bridge

FIGURE 1



C:\Users\TZalucki\Desktop\Sault Ste Marie\Reports\Achigan Creek\Analysis\[Figure 1-Engineering Parameters of Cohesive Deposits.xlsx]FIG 1-Achigan Creek Bridge

Date: August 2018
Project No: 1670846

Prepared By: TZ
Checked By: JPD

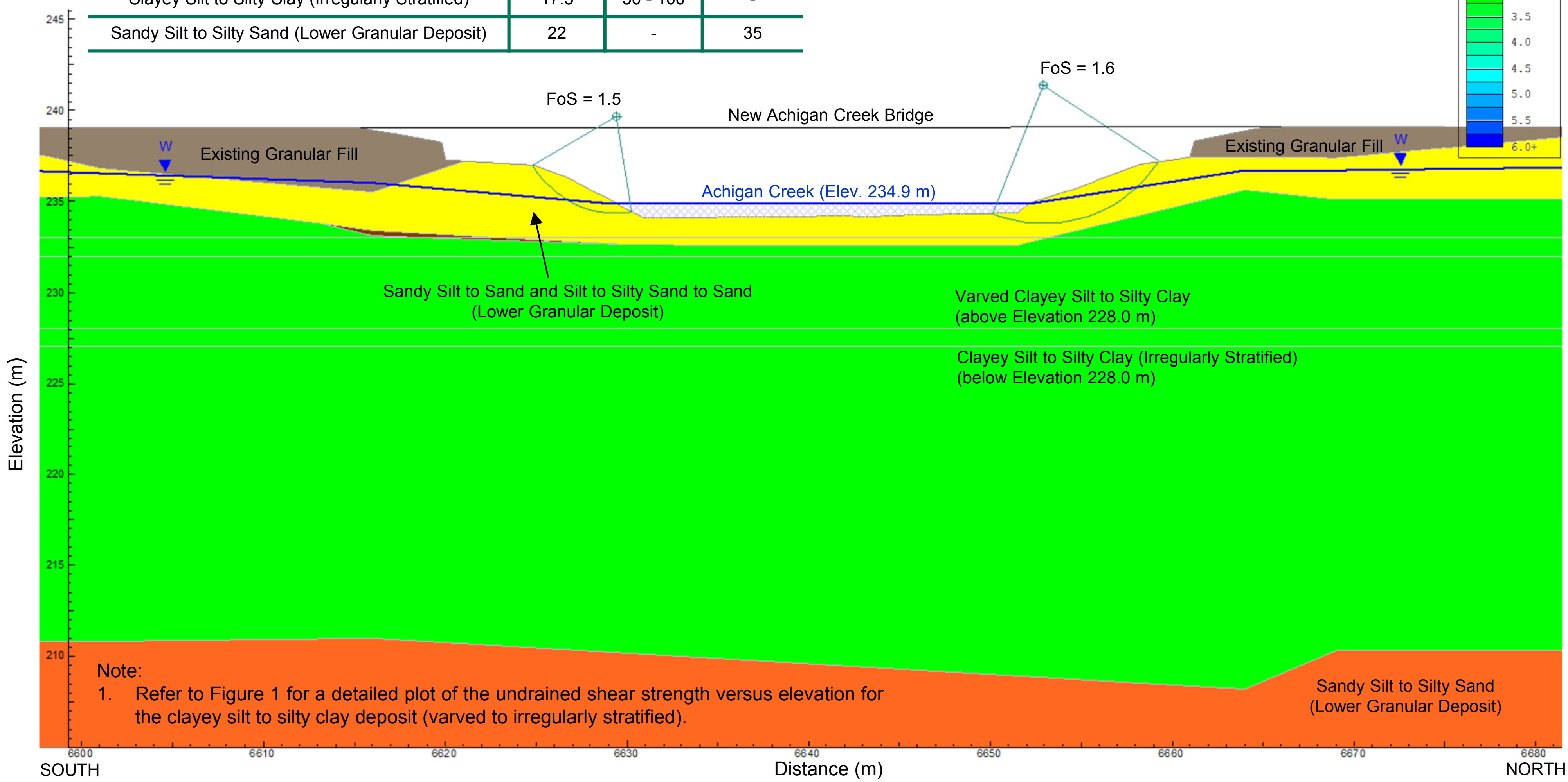
Golder Associates

Golder Associates Ltd.

Highway 532 – Achigan Creek Bridge Replacement (Site No. 38S-041)
Global Slope Stability (Temporary/Short-Term Condition)
Achigan Creek Replacement Bridge

Figure 2A

Material Name	γ (kN/m³)	s_u (kPa)	ϕ' (degrees)
Existing Granular Fill	19	-	30
Sandy Silt to Sand and Silt to Silty Sand to Sand (Upper Granular Deposit)	19	-	34
Varved Clayey Silt to Silty Clay	17.5	50 - 65	-
Clayey Silt to Silty Clay (Irregularly Stratified)	17.5	50 - 100	-
Sandy Silt to Silty Sand (Lower Granular Deposit)	22	-	35



Date: August 30, 2018

Project No: 1670846

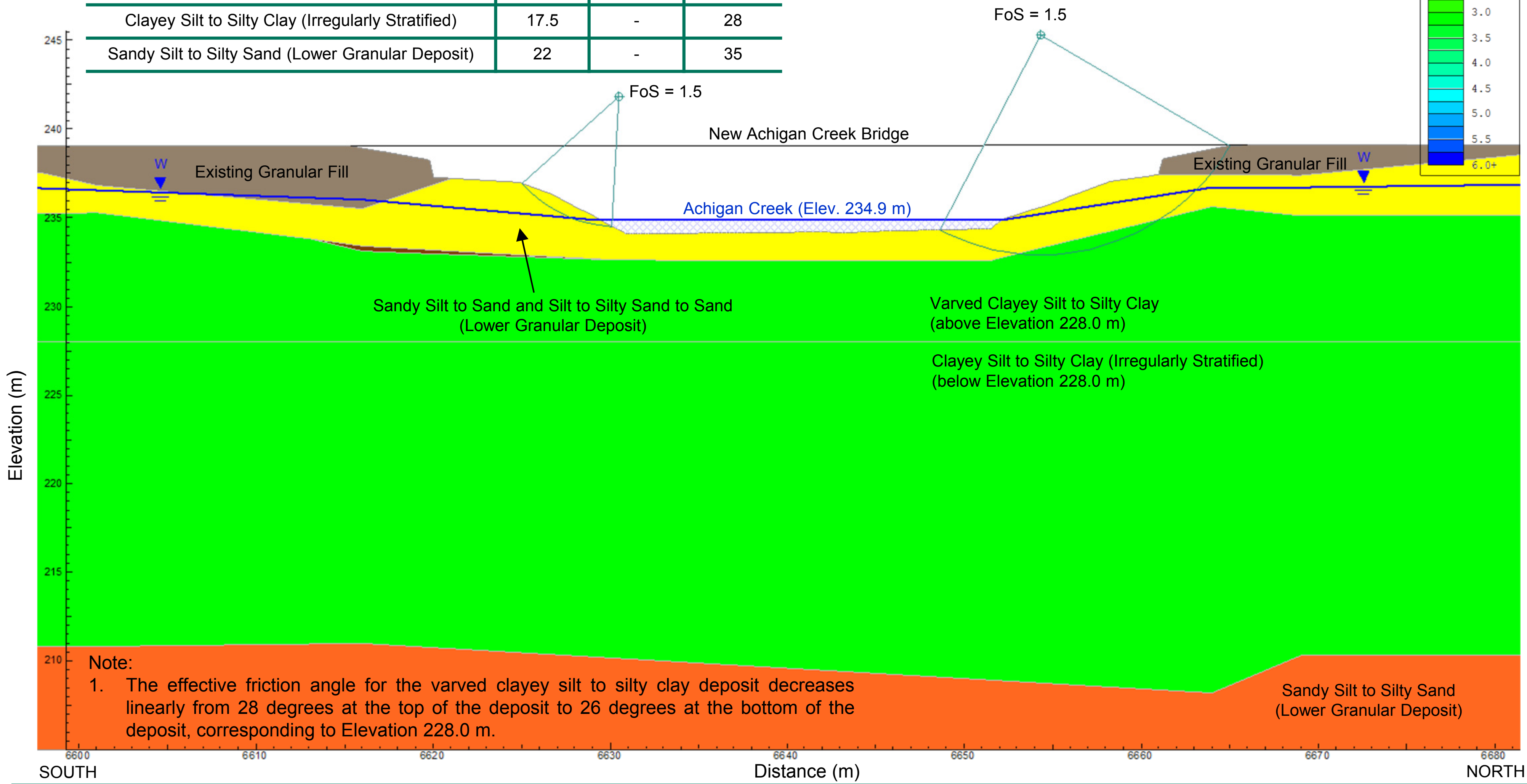
Analysis By: TZ **Reviewed By:** JPD

Golder Associates

Highway 532 – Achigan Creek Bridge Replacement (Site No. 38S-041)
Global Slope Stability (Permanent/Long-Term Condition)
Achigan Creek Replacement Bridge

Figure 2B

Material Name	γ (kN/m³)	s_u (kPa)	φ' (degrees)
Existing Granular Fill	19	-	30
Sandy Silt to Sand and Silt to Silty Sand to Sand (Upper Granular Deposit)	19	-	34
Varved Clayey Silt to Silty Clay	17.5	-	28 - 26
Clayey Silt to Silty Clay (Irregularly Stratified)	17.5	-	28
Sandy Silt to Silty Sand (Lower Granular Deposit)	22	-	35



Date: August 30, 2018

Project No: 1670846

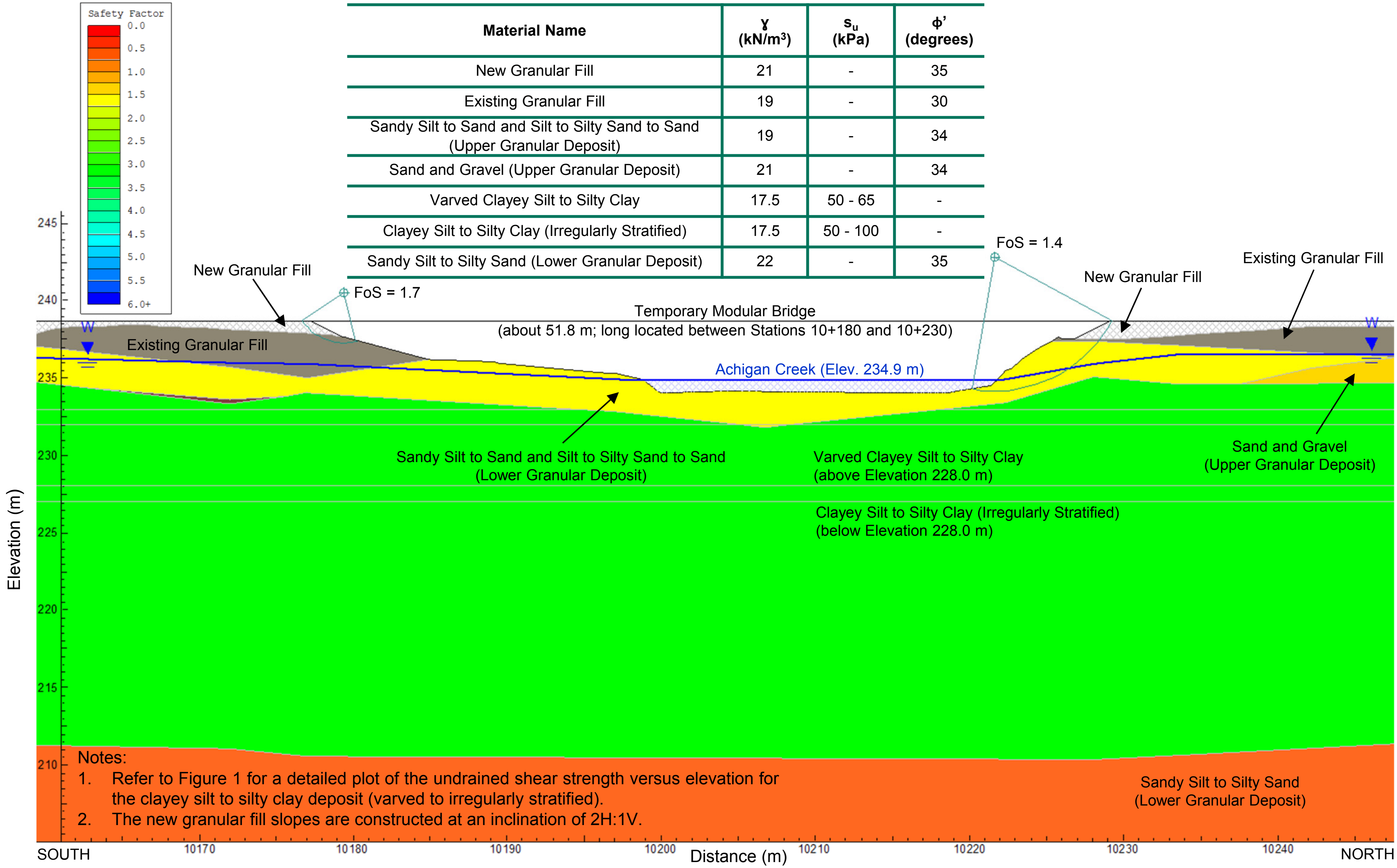
Analysis By: TZ **Reviewed By:** JPD

Golder Associates

Highway 532 – Achigan Creek Bridge Replacement (Site No. 38S-041)
Global Slope Stability (Temporary/Short-Term Condition)
Temporary Modular Bridge

Figure 3

Material Name	γ (kN/m³)	s_u (kPa)	ϕ' (degrees)
New Granular Fill	21	-	35
Existing Granular Fill	19	-	30
Sandy Silt to Sand and Silt to Silty Sand to Sand (Upper Granular Deposit)	19	-	34
Sand and Gravel (Upper Granular Deposit)	21	-	34
Varved Clayey Silt to Silty Clay	17.5	50 - 65	-
Clayey Silt to Silty Clay (Irregularly Stratified)	17.5	50 - 100	-
Sandy Silt to Silty Sand (Lower Granular Deposit)	22	-	35



Notes:

1. Refer to Figure 1 for a detailed plot of the undrained shear strength versus elevation for the clayey silt to silty clay deposit (varved to irregularly stratified).
2. The new granular fill slopes are constructed at an inclination of 2H:1V.

Date: August 30, 2018

Project No: 1670846

Analysis By: TZ **Reviewed By:** JPD

Golder Associates

APPENDIX A

Previous Borehole Investigation (MTO Geocres No. 41K-041)

MINISTRY OF TRANSPORTATION AND COMMUNICATIONS, ONTARIO

METRIC

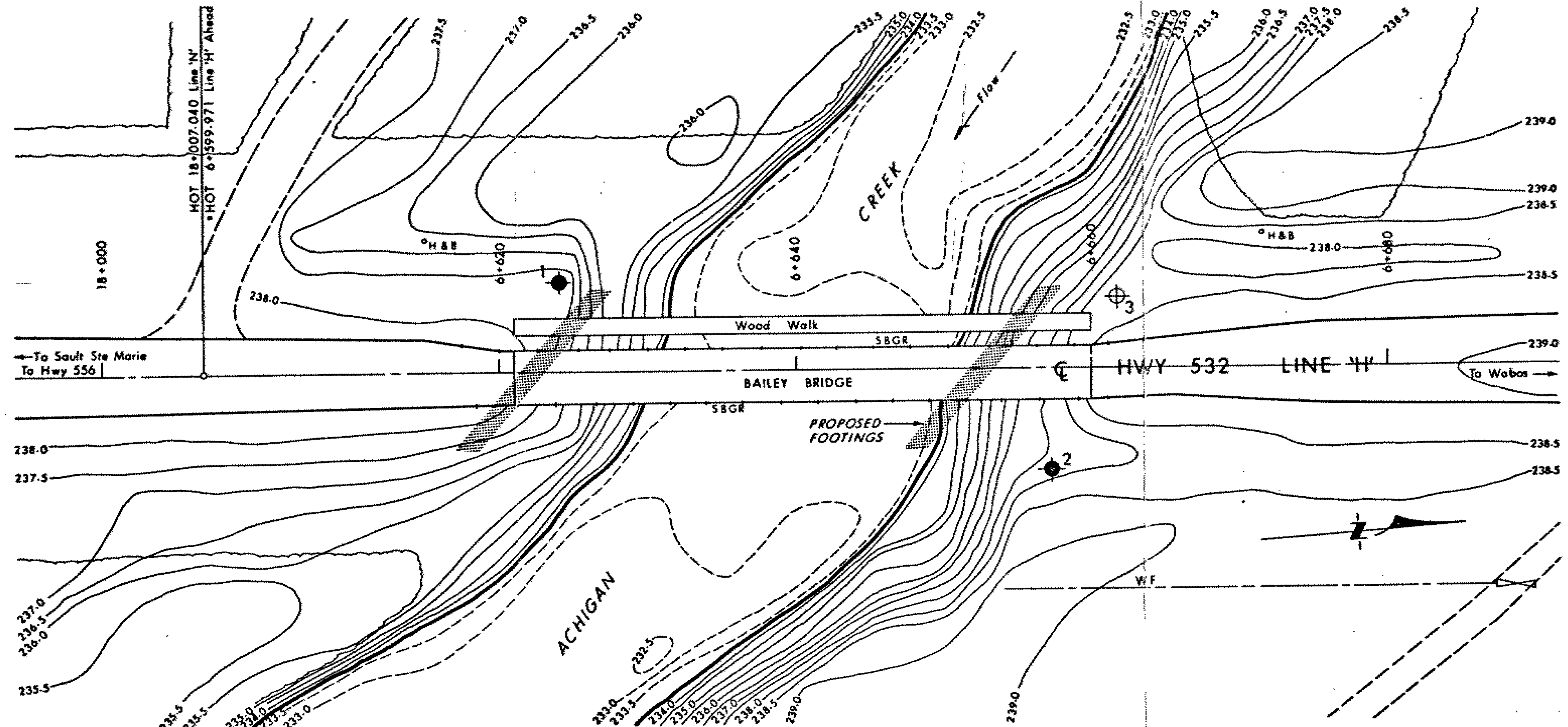
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES + METRES.

CONT No
WP No 148-65-00

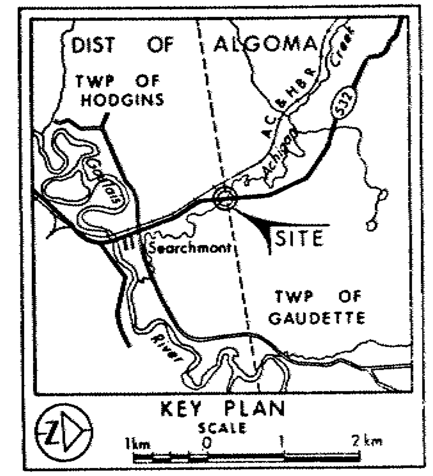
ACHIGAN CREEK BRIDGE

BORE HOLE LOCATIONS & SOIL STRATA

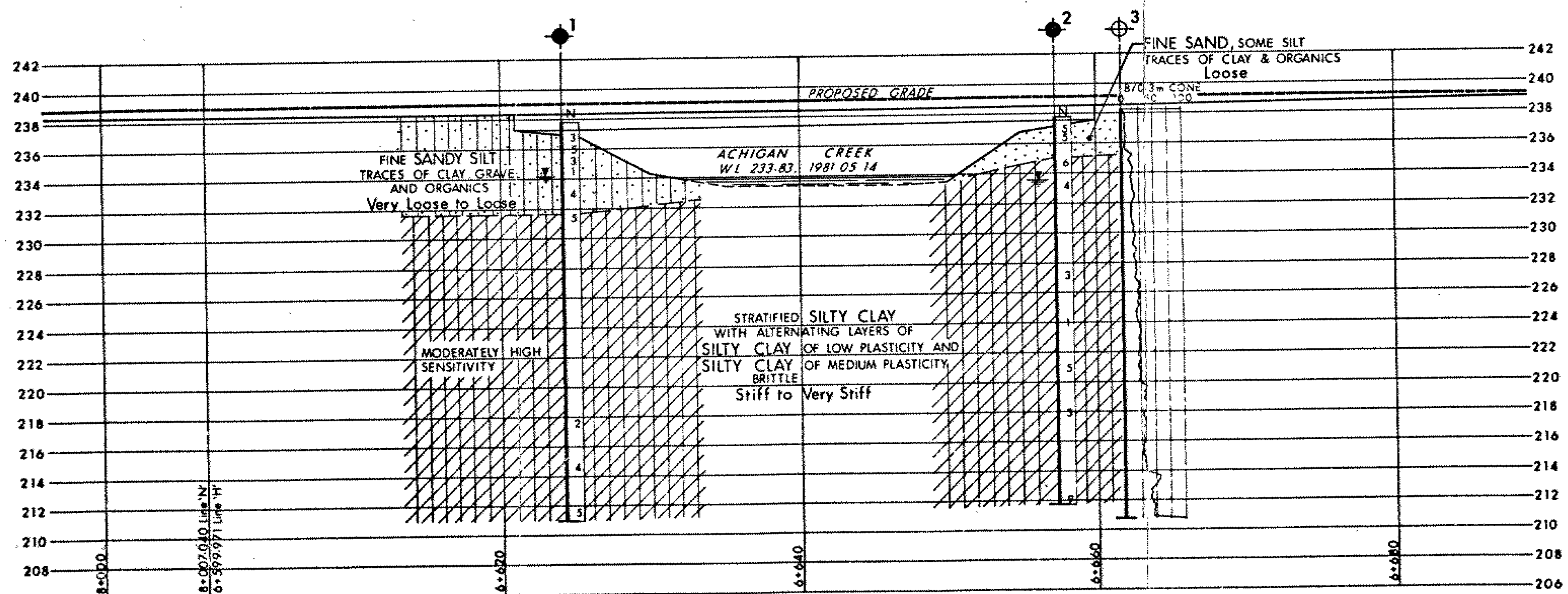
SHEET



PLAN
SCALE
4m 2 0 4m



KEY PLAN
SCALE
1km 0 1 2km



℄ PROFILE HWY 532 LINE 'H'
SCALE
4m 2 0 4m

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test (Cone)
- Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation 1981 09

No	ELEVATION	STATION	OFFSET
1	237.8	6+624.1	6.1m Lt
2	237.8	6+657.2	6.9m Rt
3	238.3	6+661.7	4.8m Lt

=NOTE=
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No 41K-41

HWY No 532			DIST 18
SUBM'D N.S.	CHECKED	DATE 1981 10 27	SITE 38S-41
DRAWN	CHECKED	APPROVED	DWG 1486500-A

REF No E-8002-1, 1981 06

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	>200
	VERY SOFT	*SOFT*	*FIRM*	*STIFF*	*VERY STIFF*	*HARD*

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3 m)	0 - 5	5 - 10	10 - 30	30 - 50	>50
	VERY LOOSE	*LOOSE*	*COMPACT*	*DENSE*	*VERY DENSE*

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

JOINTING AND BEDDING:

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	>3m
JOINTING	*VERY CLOSE*	*CLOSE*	*MOD. CLOSE*	*WIDE*	*VERY WIDE*
BEDDING	*VERY THIN*	*THIN*	*MEDIUM*	*THICK*	*VERY THICK*

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_p	%	PLASTIC LIMIT
w_s	%	SHRINKAGE LIMIT
I_p	%	PLASTICITY INDEX $= w_L - w_p$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_p}{I_p}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_p}$
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 1 METRIC

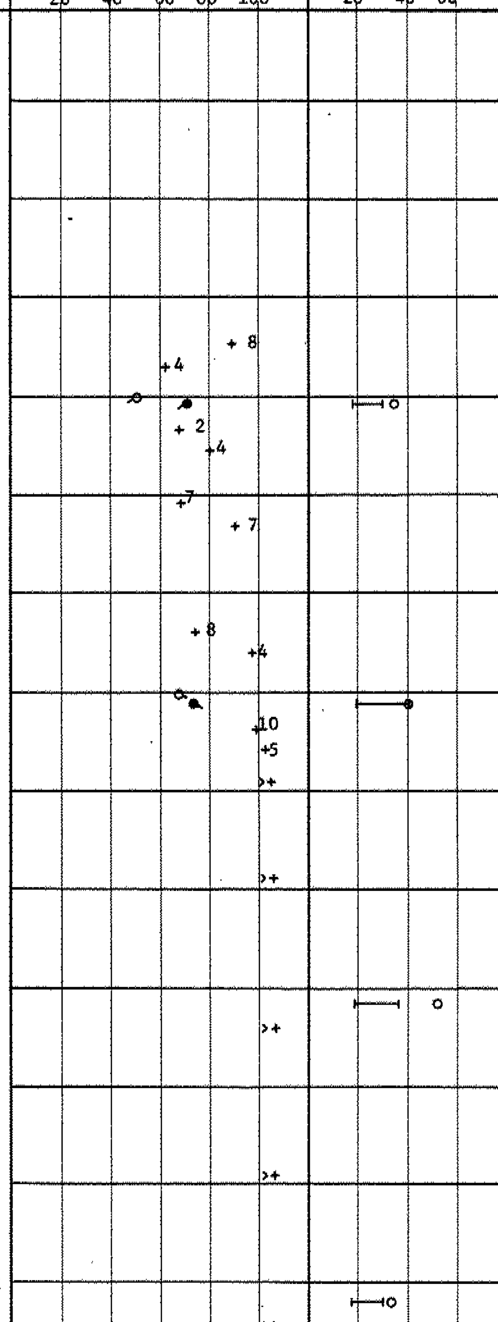
W P 148-65-00 LOCATION Sta. 6+624.1; o/s 6.1 m Lt. of ℄ Highway 532 ORIGINATED BY N. S.

DIST 18 HWY 532 BOREHOLE TYPE Hollow Stem Continuous Flight Augers COMPILED BY N. S.

DATUM Geodetic DATE 81 09 05 CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	'N' VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
237.8	Ground Surface										
0.0	Fine sandy silt with traces of clay and gravel and organics	1	SS	3							
		2	SS	5		236		4	39	48	9
		3	SS	3	*						
	Very loose to loose	4	SS	1				2	40	52	6
					▽	234					
	Brown	5	SS	4							
231.6						232					
6.2	Stratified silty clay with alternating layers of silty clay of low plasticity and silty clay of medium plasticity	6	SS	5							
		7	TW	PH		230	18.5	0	0	60	40
		8	TW	PH		228					
	Moderately high sensitivity					226					
	stiff / very stiff										
		9	TW	PH		224	17.7	0	0	54	46
	Brittle					222					
	Grey	10	TW	PH		220					
		11	SS	2		218					
						216					
		12	SS	4		214					
211.0		13	SS	5		212					
26.8	End of Borehole										
	*Water level obtained on 81 09 06										

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100)
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE (20 40 60 80 100)
PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L — WATER CONTENT (%) (20 40 60)



$+^3$, $\times^5$: Numbers refer to Sensitivity

15 -◇- 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

Ministry of Transportation and Communications
Ontario

RECORD OF BOREHOLE No 2

METRIC

W P 148-65-00
LOCATION Sta. 6+657.2; o/s 6.9 m Rt. of ℄ Highway 532
ORIGINATED BY N. S.

DIST 18 HWY 532
BOREHOLE TYPE Hollow Stem Continuous Flight Augers
COMPILED BY N. S.

DATUM Geodetic
DATE 81 09 06
CHECKED BY [illegible]

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
237.8	Ground Surface
0.0	Fine sand with some silt and traces of clay and organics. Loose. Brown
235.0 / 2.8	Stratified silty clay with alternating layers of silty clay of low plasticity and silty clay of medium plasticity. Very stiff. Brittle. Grey
211.7 / 26.1	End of Borehole

*Water level obtained on 81 09 06

SAMPLES

NUMBER	TYPE	'N' VALUES	UNIT WEIGHT γ kN/m³	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5		0 77 15 8
2	SS	5		
3	SS	6		
4	SS	4		
5	TW	PH	18.6	0 1 54 45
6	SS	3		
7	SS	1		
8	SS	5		
9	SS	3		
10	TW	PH	18.0	0 1 50 49
11	SS	9		

ELEVATION SCALE: 236, 234, 232, 230, 228, 226, 224, 222, 220, 218, 216, 214, 212

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100

PLASTIC LIMIT W_p — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

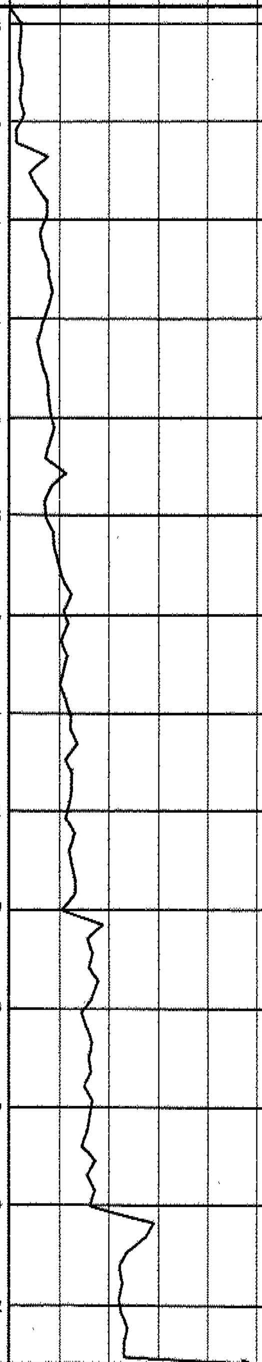
$+^3$, $\times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 3 METRIC

W P 148-65-00 LOCATION Sta. 6+661.7; o/s 4.8 m Lt. of ℄ Highway 532 ORIGINATED BY N. S.
DIST 18 HWY 532 BOREHOLE TYPE Dynamic Cone Penetration Test COMPILED BY N. S.
DATUM Geodetic DATE 81 09 06 CHECKED BY

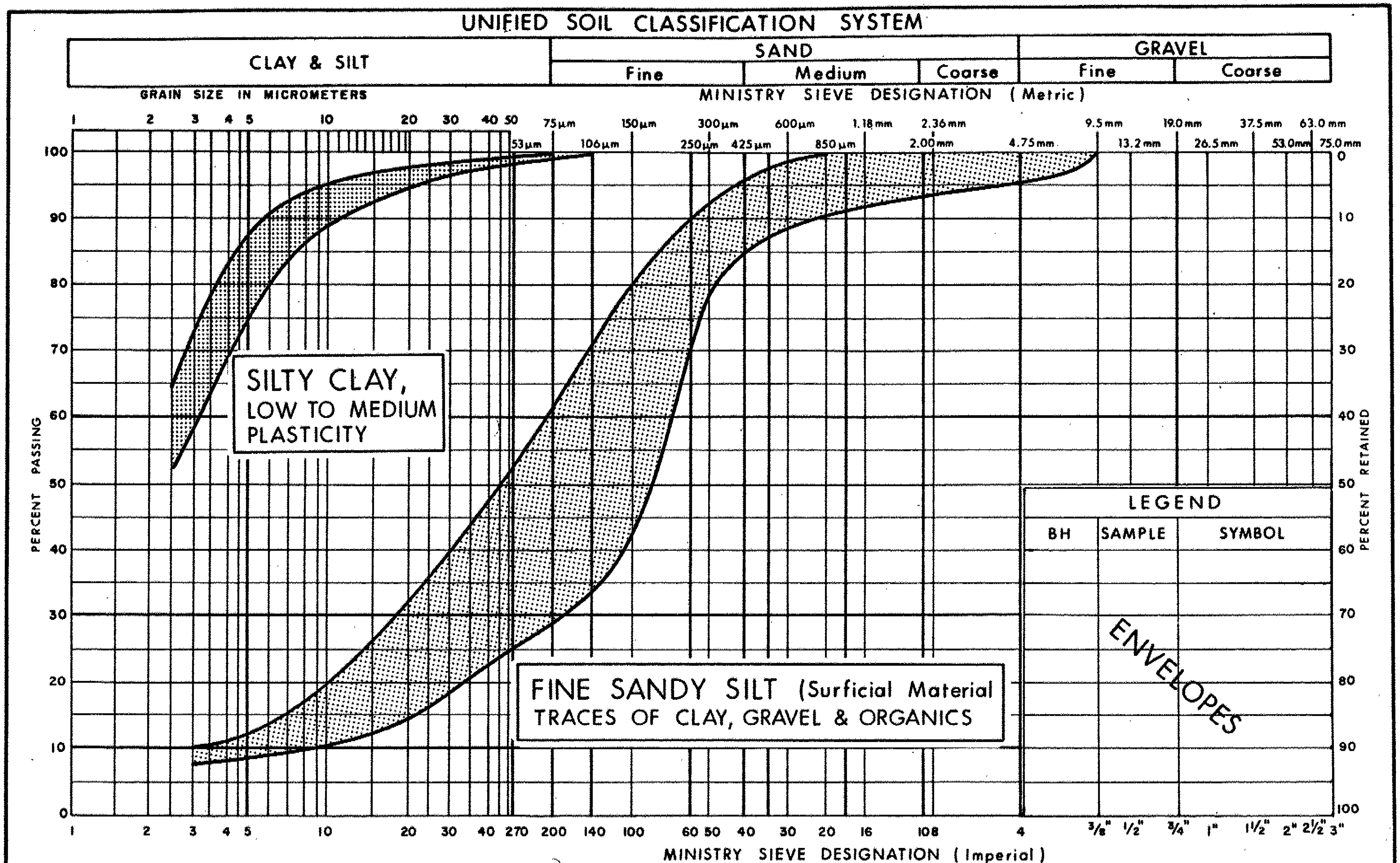
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE	PLASTIC LIMIT W_p / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%)	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES						
238.3	Ground Surface										
0.0							238, 236, 234, 232, 230, 228, 226, 224, 222, 220, 218, 216, 214, 212				
210.8 / 27.5	End of Cone Test							100/76 mm			

$+^3, \times^5$: Numbers refer to Sensitivity

15 ⌖ 5 (%) STRAIN AT FAILURE (20 above, 10 below)

OFFICE REPORT ON SOIL EXPLORATION

78 12 M



Ministry of Transportation and Communications

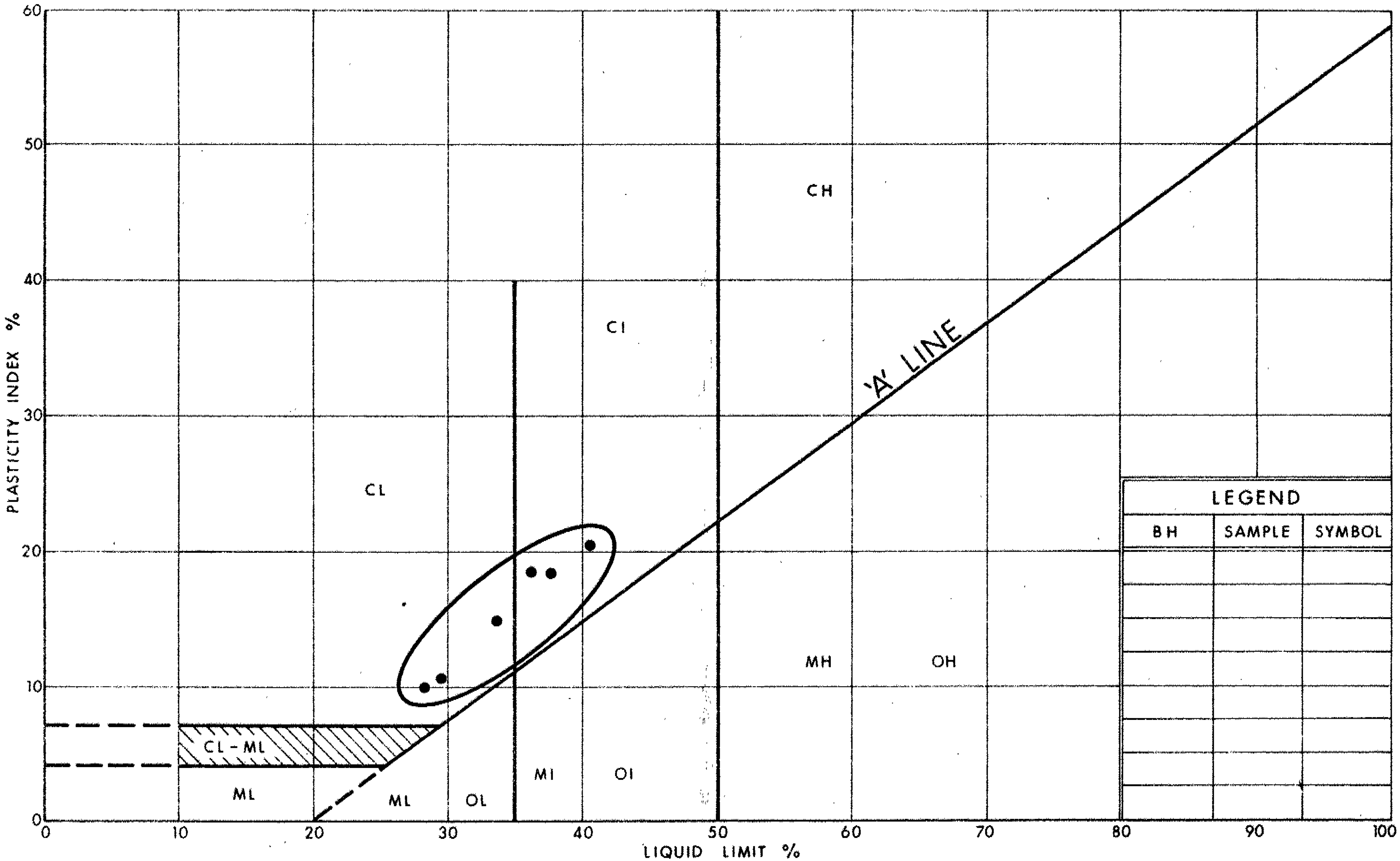
Ontario

GRAIN SIZE DISTRIBUTION

FIG No 1

W P 148-65-00

Oct 75, FF-S-21



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PLASTICITY CHART

SILTY CLAY, OF LOW TO MEDIUM PLASTICITY

FIG No 2

W P 148 - 65 - 00

APPENDIX B

Records of Borehole Sheets

LIST OF SYMBOLS

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

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c'	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p'	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes: 1 $\tau = c' + \sigma' \tan \phi'$

2 shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Compactness Condition	N Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u kPa	c_u, s_u psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or With (cohesive)	Sand and Gravel Silty Clay with sand / Clayey Silt with sand

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-01

SHEET 1 OF 2 **METRIC**

PROJECT 1670846
W.P. 151-97-01 LOCATION N 5183314.9; E 300612.5 MTM NAD 83 ZONE 13 (LAT. 46.789381; LONG. -84.054853) ORIGINATED BY JL
DIST ALGOMA HWY 532 BOREHOLE TYPE 210 mm O.D. Continuous Flight Hollow Stem Augers COMPILED BY AK
DATUM Geodetic DATE August 22, 2017 CHECKED BY TZ

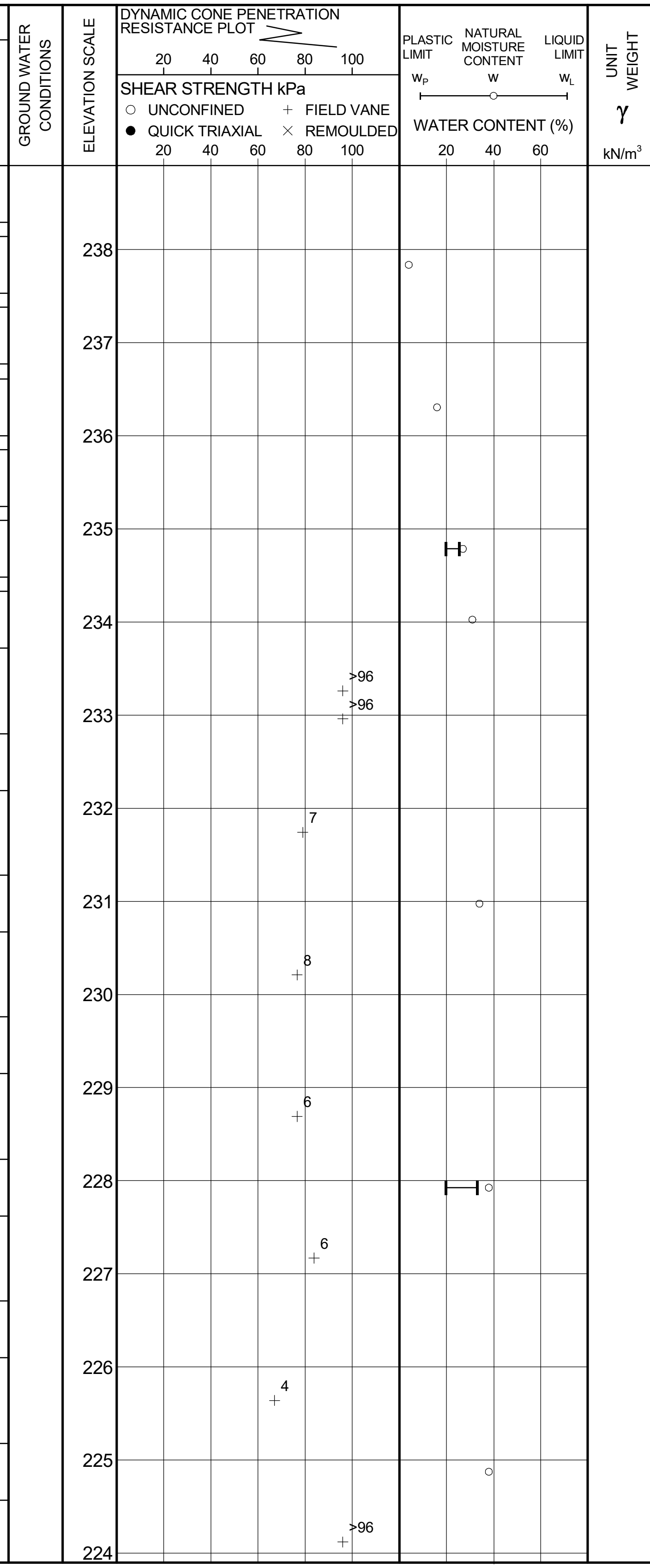
SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
238.9	GROUND SURFACE
0.0	Gravelly sand, trace silt (FILL) Very dense Brown Moist
238.2 / 0.7	Sand, trace to some gravel, trace silt (FILL) Compact Brown Moist
237.5 / 1.4	SAND, some silt, trace gravel, trace clay Loose Brown Moist to wet - Wet below a depth of about 3.0 m
235.2 / 3.7	Varved CLAYEY SILT to SILTY CLAY, trace sand Stiff to very stiff Grey Wet
228.0 / 10.9	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES
1	SS	52
2	SS	24
3	SS	7
4	SS	7
5	SS	7
6	SS	7
7	SS	4
8	TO	PH
9	SS	4
10	SS	2
11	SS	1
12	SS	1
13	SS	2

REMARKS & GRAIN SIZE DISTRIBUTION (%) (Sample 4): GR 4, SA 76, SI 19, CL 1



Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-01

SHEET 2 OF 2 **METRIC**

PROJECT 1670846

W.P. 151-97-01 LOCATION N 5183314.9; E 300612.5 MTM NAD 83 ZONE 13 (LAT. 46.789381; LONG. -84.054853) ORIGINATED BY JL

DIST ALGOMA HWY 532 BOREHOLE TYPE 210 mm O.D. Continuous Flight Hollow Stem Augers COMPILED BY AK

DATUM Geodetic DATE August 22, 2017 CHECKED BY TZ

ELEV DEPTH	SOIL PROFILE DESCRIPTION	STRAT PLOT	SAMPLES NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
223.1			14	SS	5						
15.9	END OF BOREHOLE NOTE: 1. Borehole dry upon completion of drilling, prior to auger removal.										

$+^3, \times^3$: Numbers refer to Sensitivity $\bigcirc^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-02

SHEET 1 OF 3 **METRIC**

PROJECT	1670846		
W.P.	151-97-01	LOCATION	N 5183317.1; E 300617.3 MTM NAD 83 ZONE 13 (LAT. 46.789401; LONG. -84.054790)
DIST	ALGOMA HWY 532	BOREHOLE TYPE	95 mm O.D. Solid Stem Augers; Wash Boring; NQ Coring
DATUM	Geodetic	DATE	September 11 and 12, 2017
ORIGINATED BY	AJ	COMPILED BY	AK
CHECKED BY	TZ		

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
238.9	GROUND SURFACE
0.0	ASPHALT (about 40 mm thick)
0.0	Sand and gravel, trace silt (FILL) Dense Brown Moist
237.5 / 1.4	Silty sand, trace clay (FILL) Compact Brown Moist
236.7 / 2.2	Sandy SILT of slight plasticity, trace clay Very loose to loose Brown Moist
235.2 / 3.7	- An approximately 0.2 m thick layer of wet gravel, some sand encountered at a depth of about 3.5 m Varved CLAYEY SILT to SILTY CLAY, trace gravel, some sand Stiff to very stiff Grey Wet
228.0 / 10.9	CLAYEY SILT to SILTY CLAY, trace gravel, trace sand, irregularly stratified Stiff to very stiff Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	38	
2	SS	10	
3	SS	5	0 23 75 2
4A / 4B	SS	4	
5	SS	9	3 1 83 13
6	SS	7	
7	SS	6	
8	SS	4	
9	TO	PH	
10	SS	4	
11	SS	2	
12	SS	4	

Field vane sensitivity values: 9 (approx. elev. 233.3); >96 (approx. elev. 231.8); >96 (approx. elev. 230.3); >96 (approx. elev. 228.8); >96 (approx. elev. 225.7); >96 (approx. elev. 224.2)

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100
SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100
PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L
WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m^3
ELEVATION SCALE 238 – 224

Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-02

METRIC

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183317.1; E 300617.3 MTM NAD 83 ZONE 13 (LAT. 46.789401; LONG. -84.054790)

ORIGINATED BY AJ

DIST ALGOMA HWY 532

BOREHOLE TYPE 95 mm O.D. Solid Stem Augers; Wash Boring; NQ Coring

COMPILED BY AK

DATUM Geodetic

DATE September 11 and 12, 2017

CHECKED BY TZ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)
	--- CONTINUED FROM PREVIOUS PAGE ---						
	CLAYEY SILT to SILTY CLAY, trace gravel, trace sand, irregularly stratified Stiff to very stiff Grey Wet	13	SS	10			
					223		
					222		
		14	SS	12			
					221		
		15	SS	8			
					220		
						+ >96 (field vane)	
					219		
					218		
		16	SS	12			
					217		
					216		
					215		
		17	SS	12			
					214		
					213		
					212		
		18	SS	14			
					211		
210.7 28.2	SILTY SAND, some gravel, trace clay, with cobbles and boulders Very dense Grey Wet						
		-	RC	-	210		
					209		

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

GTA-MTO 001 \\GOLDER.GDS\GAL\MISSISSAUGA\SIM\CLIENTS\MTO\SAULT_STE_MARIE\02_DATA\GINT\SAULT_STE_MARIE.GPJ GAL-GTA.GDT 8-28-18

RECORD OF BOREHOLE No ACB-02

SHEET 3 OF 3 **METRIC**

PROJECT 1670846
W.P. 151-97-01
LOCATION N 5183317.1; E 300617.3 MTM NAD 83 ZONE 13 (LAT. 46.789401; LONG. -84.054790)
ORIGINATED BY AJ
DIST ALGOMA HWY 532
BOREHOLE TYPE 95 mm O.D. Solid Stem Augers; Wash Boring; NQ Coring
COMPILED BY AK
DATUM Geodetic
DATE September 11 and 12, 2017
CHECKED BY TZ

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	SILTY SAND, some gravel, trace clay, with cobbles and boulders Very dense Grey Wet		-	RC	-						
			19	SS	78/0.03				○	Non-Plastic	15 61 23 1
							208				
206.9							207				
32.0	CASING AND SPLIT-SPOON REFUSAL END OF BOREHOLE NOTES: 1. Artesian groundwater conditions encountered below a depth of about 28.2 m (Elev. 210.8 m) during casing advancement. 2. The cored depth intervals and particle sizes of recovered cobbles/boulders are summarized as follows: Depth (m) Recovered 28.7 - 30.5 620mm; 110mm 100mm; 50mm to 70mm rock fragments/ gravel pieces										

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No ACB-03

SHEET 1 OF 3 **METRIC**

PROJECT 1670846
W.P. 151-97-01
DIST ALGOMA HWY 532
DATUM Geodetic
LOCATION N 5183333.1; E 300610.5 MTM NAD 83 ZONE 13 (LAT. 46.789545; LONG. -84.054880)
BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring; NQ Coring
DATE August 23 and 24, 2017
ORIGINATED BY JL
COMPILED BY AK
CHECKED BY TZ

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	FIELD VANE (kPa, sensitivity)	WATER CONTENT	UNIT WEIGHT kN/m³	GR	SA	SI	CL
238.3	GROUND SURFACE										
0.0	Sand and silt, trace to some gravel, trace clay, trace organics (FILL) Loose to compact Brown to grey Moist to wet	1	SS	12							
		2	SS	12							
		3	SS	6		○		11	58	30	1
235.7 2.6	- Wet below a depth of about 2.3 m	4A 4B	SS	2		○					
	SAND and SILT, trace clay, trace organics Very loose Brown to black Wet - Inclusions/layers of organic silt and peat encountered between depths of about 2.6 m and 3.7 m	5	SS	1		○					
		6	SS	WH		○	Non-Plastic	0	53	46	1
233.6	CLAYEY ORGANIC SILT Very soft to soft Grey to black Moist	7A 7B 7C	SS	2		○ ○					
233.3 5.0	Varved CLAYEY SILT to SILTY CLAY, trace sand Stiff Grey Wet				+ (9)						
		8	SS	10		├─┤ ○					
					+ (6)						
		9	SS	5		├─┤ ○					
					+ (6)						
		10	TO	PH							
228.0 10.3	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet				+ (9)						
		11	SS	WH							
					+ (9)						
		12	SS	3		○					
					+ (>96)						
		13	SS	2		├─┤ ○					
					+ (6)						

Elevation scale: 238 to 224. Groundwater level shown at about elev. 234.

Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-03 SHEET 2 OF 3 **METRIC**

PROJECT 1670846

W.P. 151-97-01 LOCATION N 5183333.1; E 300610.5 MTM NAD 83 ZONE 13 (LAT. 46.789545; LONG. -84.054880) ORIGINATED BY JL

DIST ALGOMA HWY 532 BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring; NQ Coring COMPILED BY AK

DATUM Geodetic DATE August 23 and 24, 2017 CHECKED BY TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (Field Vane)	WATER CONTENT
	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet	14	SS	3	223		
					222	>96	
		15	SS	3	221		
						>96	
		16	TO	PH	220		
					219	>96	
		17	SS	3	218		
						>96	
					217		
					216		
		18	SS	15	215		
					214		
					213		
		19	SS	18	212		
211.2 27.1	Inferred SILTY SAND, some gravel, with cobbles and boulders				211		
					210		
		-	RC	-			
			RC	-	209		
			RC	-			

Column headings: SOIL PROFILE (ELEV DEPTH, DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE; DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No ACB-03

SHEET 3 OF 3 **METRIC**

PROJECT 1670846
W.P. 151-97-01
LOCATION N 5183333.1; E 300610.5 MTM NAD 83 ZONE 13 (LAT. 46.789545; LONG. -84.054880)
ORIGINATED BY JL
DIST ALGOMA HWY 532
BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring; NQ Coring
COMPILED BY AK
DATUM Geodetic
DATE August 23 and 24, 2017
CHECKED BY TZ

ELEV DEPTH	DESCRIPTION (SOIL PROFILE)	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Inferred SILTY SAND, some gravel, with cobbles and boulders			RC	-		208				
				RC	-		207				
206.3 32.0	END OF BOREHOLE										

NOTES:

1. Water level measured in casing at a depth of about 10.3 m below ground surface (Elev. 227.8 m) on August 24, 2017.

2. The cored depth intervals and particle sizes of recovered cobbles / boulders are summarized as follows:

Depth (m)	Recovered
28.3 - 29.0	130mm; 20mm to 70mm rock fragments/ gravel pieces
29.0 - 29.9	440mm
29.9 - 30.6	40mm to 70mm rock fragments/ gravel pieces
30.6 - 32.0	20mm to 60mm rock fragments/ gravel pieces

3. A borehole was advanced on August 12, 2018 to a depth of about 4.6 m below ground surface immediately next to Borehole ACB-03 in order to install a standpipe piezometer.

4. Water level measurments in standpipe piezometer:

Date	Depth (m)	Elev. (m)
12/08/18	4.5	233.8
13/08/18	4.5	233.8
14/08/18	4.5	233.8
15/08/18	4.5	233.8

5. The standpipe piezometer was decommissioned on August 15, 2018 in accordance with Ontario Regulation 903 (as amended).

$+^3, \times^3$: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

Foundation Design

RECORD OF BOREHOLE No ACB-04

METRIC

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183335.1; E 300606.0 MTM NAD 83 ZONE 13 (LAT. 46.789563; LONG. -84.054938)

ORIGINATED BY JL

DIST ALGOMA HWY 532

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring; NQ Coring

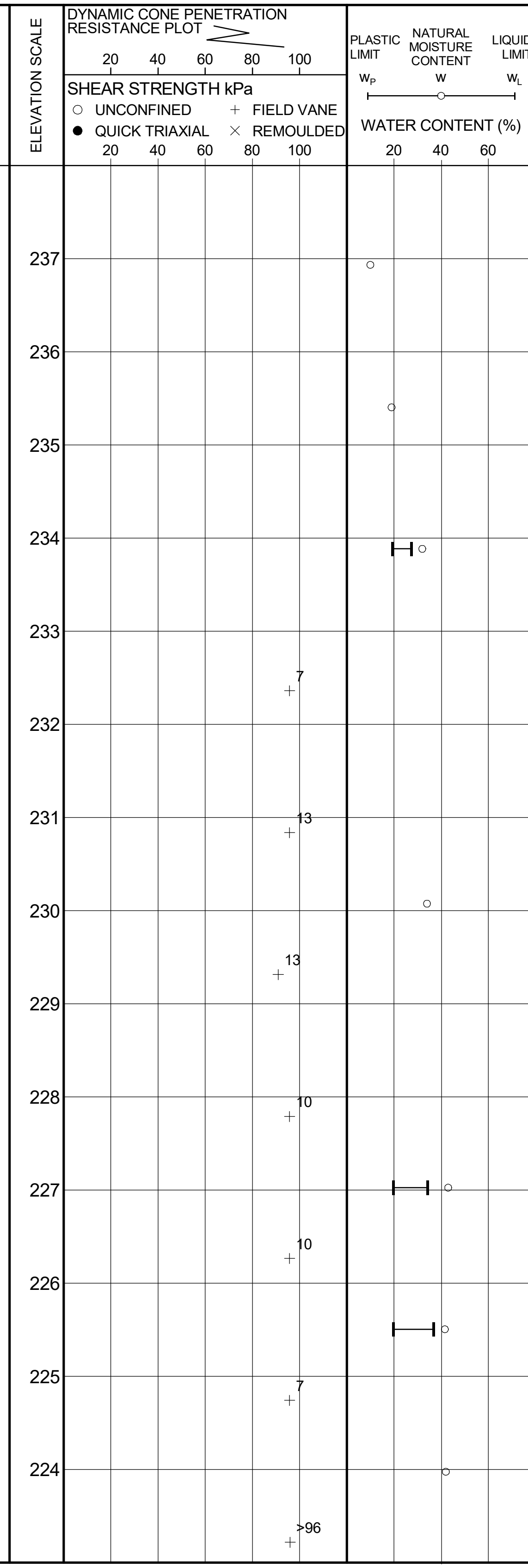
COMPILED BY AK

DATUM Geodetic

DATE August 24 and 25, 2017

CHECKED BY TZ

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
238.0	GROUND SURFACE						
0.0	Silty sand, some gravel, trace clay to sand, trace to some gravel, trace silt, trace clay (FILL) Compact Brown Moist	1	SS	13			
		2	SS	19			
		3	SS	14			
	- Trace organics encountered above a depth of about 2.1 m	4	SS	12			16 56 27 1
235.0 / 3.0	SAND, some silt, trace gravel Loose Brown Moist	5	SS	9			
234.1 / 3.9	Varved CLAYEY SILT to SILTY CLAY, trace sand Stiff Grey Wet	6	SS	6	▽		
		7	SS	7			
		8	SS	8			
		9	SS	3			
		10	SS	1			
228.0 / 10.0	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet	11	SS	2			
		12	TO	PH		17.7	(C)
		13	SS	3			



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

RECORD OF BOREHOLE No ACB-04

METRIC

PROJECT	1670846
W.P.	151-97-01
LOCATION	N 5183335.1; E 300606.0 MTM NAD 83 ZONE 13 (LAT. 46.789563; LONG. -84.054938)
ORIGINATED BY	JL
DIST	ALGOMA
HWY	532
BOREHOLE TYPE	210 mm O.D. Hollow Stem Augers; Wash Boring; NQ Coring
COMPILED BY	AK
DATUM	Geodetic
DATE	August 24 and 25, 2017
CHECKED BY	TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa (Field Vane)	WATER CONTENT (%)
	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet	14	SS	5			w_P–w
					222	>96	
		15	SS	5	221		
					220	>96	
		16	SS	3			w_P–w
					219	>96	
		17	SS	4	218		
					217	>96	
		18	TO	PH			w_P–w_L
					216	>96	
		19	SS	9	215		
					214	>96	
					213		
		20	SS	14	212		w_P–w
					211		
210.6 27.4	Inferred SILTY SAND, some gravel, with cobbles and boulders				210		
		-	RC	-	209		
			RC	-			

Continued Next Page

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

RECORD OF BOREHOLE No ACB-04

SHEET 3 OF 3 **METRIC**

PROJECT 1670846
W.P. 151-97-01
LOCATION N 5183335.1; E 300606.0 MTM NAD 83 ZONE 13 (LAT. 46.789563; LONG. -84.054938)
ORIGINATED BY JL
DIST ALGOMA HWY 532
BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring; NQ Coring
COMPILED BY AK
DATUM Geodetic
DATE August 24 and 25, 2017
CHECKED BY TZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Inferred SILTY SAND, some gravel, with cobbles and boulders			RC	-		207				
205.5				RC	-		206				
32.5	END OF BOREHOLE NOTES: 1. Water level measured in casing at a depth of about 4.4 m below ground surface (Elev. 233.6 m) on August 25, 2017. 2. The cored length intervals and particle sizes of recovered cobbles/boulders are summarized as follows: Depth (m) Recovered 28.7 - 29.5 130mm; 30mm to 45mm rock fragments/ gravel pieces 29.5 - 31.0 20mm to 70mm rock fragments/ gravel pieces 31.0 - 32.5 45mm to 60mm rock fragments/ gravel pieces										

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

RECORD OF BOREHOLE No ACB-05

SHEET 1 OF 3 **METRIC**

PROJECT 1670846
W.P. 151-97-01
LOCATION N 5183392.1; E 300606.9 MTM NAD 83 ZONE 13 (LAT. 46.790075; LONG. -84.054927)
ORIGINATED BY JL
DIST ALGOMA HWY 532
BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring:; NQ Coring
COMPILED BY AK
DATUM Geodetic
DATE August 28 and 29, 2017
CHECKED BY TZ

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	Field vane (kPa) / Sensitivity	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	UNIT WEIGHT γ kN/m³
237.8	GROUND SURFACE						
0.0	Sand, some gravel, trace to some silt (FILL) Dense Brown Moist	1	SS	45			
237.1 / 0.7	SAND and SILT, trace clay to SILTY SAND, trace clay, trace organics Very loose to loose Brown to grey Wet	2	SS	4			
		3	SS	4		0 75 23 2	
	- Grey below a depth of about 2.3 m	4	SS	WH		0 61 36 3	Non-Plastic
234.6 / 3.2	Varved CLAYEY SILT to SILTY CLAY, trace sand Stiff Grey Wet	5A / 5B	SS	12			
		6	SS	6			
		7	SS	7			
					+ (6)		
		8	SS	8			
					+ (6)		
		9	SS	3			
					+ (5)		
		10	WH	WH			
228.0 / 9.8	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet				+ (7)		
		11	TO	PH		(C)	17.6
					+ (5)		
		12	SS	2			
					+ (>96)		
		13	SS	3			
					+ (>96)		

Continued Next Page

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

RECORD OF BOREHOLE No ACB-05

SHEET 2 OF 3 **METRIC**

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183392.1; E 300606.9 MTM NAD 83 ZONE 13 (LAT. 46.790075; LONG. -84.054927)

ORIGINATED BY JL

DIST ALGOMA HWY 532

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring:; NQ Coring

COMPILED BY AK

DATUM Geodetic

DATE August 28 and 29, 2017

CHECKED BY TZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) / SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20, 40, 60, 80, 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) (20, 40, 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet		14	TO	PH		222				
								+ >96			
			15	SS	3		221				
							220	+ >96			
			16	SS	2		219				
								+ >96			
			17	SS	6		218				
							217	+ >96			
							216				
			18	SS	10		215				
							214				
							213				
			19	SS	14		212				
							211				
210.4 27.4	- Casing grinding at a depth of about 27.4 m										
	Inferred SILTY SAND, some gravel, with cobbles and boulders						210				
			-	RC	-		209				
				RC	-		208				

Continued Next Page

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

RECORD OF BOREHOLE No ACB-05

SHEET 3 OF 3

METRIC

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183392.1; E 300606.9 MTM NAD 83 ZONE 13 (LAT. 46.790075; LONG. -84.054927)

ORIGINATED BY JL

DIST ALGOMA HWY 532

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring:; NQ Coring

COMPILED BY AK

DATUM Geodetic

DATE August 28 and 29, 2017

CHECKED BY TZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Inferred SILTY SAND, some gravel, with cobbles and boulders			RC	-						
							207				
				RC	-						
							206				
205.5 32.3	END OF BOREHOLE NOTE: 1. The cored depth intervals and particle sizes of recovered rock fragments are summarized as follows: Depth (m) Recovered 28.7 - 32.3 20mm to 70mm rock fragments/ gravel pieces										

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-06

SHEET 1 OF 3

METRIC

PROJECT	1670846		
W.P.	151-97-01	LOCATION	N 5183385.9; E 300615.1 MTM NAD 83 ZONE 13 (LAT. 46.790020; LONG. -84.054820)
DIST	ALGOMA HWY 532	BOREHOLE TYPE	95 mm O.D. Solid Stem Augers; Wash Boring; NQ Coring
DATUM	Geodetic	DATE	September 9 and 10, 2017
ORIGINATED BY	AJ	COMPILED BY	AK
CHECKED BY	TZ		

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
238.8	GROUND SURFACE
0.0	Silty sand, trace to some gravel, trace organics (FILL) Loose Brown Moist
237.4 1.4	SAND, some silt, trace gravel, trace clay Very loose to loose Brown Moist to wet - Wet below a depth of about 3.0 m
235.1 3.7	Varved CLAYEY SILT to SILTY CLAY, trace sand Stiff to very stiff Grey Wet
228.0 10.8	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	8	
2	SS	4	
3	SS	3	0 78 19 3
4	SS	4	
5	SS	10	0 0 84 16
6	SS	9	
7	SS	9	
8	TO	PH	
9	SS	7	
10	SS	6	
11	TO	PH	
12	SS	9	

GROUND WATER CONDITIONS | ELEVATION SCALE: 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED; 20 40 60 80 100

Field vane: >96 (at approx. elev. 231.6), >96 (230.1), >96 (228.6), >96 (227.1), >96 (225.6)

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No ACB-06

SHEET 2 OF 3

METRIC

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183385.9; E 300615.1 MTM NAD 83 ZONE 13 (LAT. 46.790020; LONG. -84.054820)

ORIGINATED BY AJ

DIST ALGOMA HWY 532

BOREHOLE TYPE 95 mm O.D. Solid Stem Augers; Wash Boring; NQ Coring

COMPILED BY AK

DATUM Geodetic

DATE September 9 and 10, 2017

CHECKED BY TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet	13	SS	11	223	
		14	SS	9	222	
					221	
		15	SS	10	220	
					219	
					218	
		16	SS	11	217	0 13 64 23
					216	
					215	
		17	SS	10	214	
					213	
					212	
		18	SS	109/0.23	211	0 0 85 15
210.3 28.5	Sandy SILT, some gravel, with cobbles and boulders Very dense Grey Wet				210	
		-	RC	-	209	

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; GROUND WATER CONDITIONS; STRAT PLOT

Continued Next Page

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

$\bigcirc^{3\%}$ STRAIN AT FAILURE

GOLDER

RECORD OF BOREHOLE No ACB-06 SHEET 3 OF 3 METRIC

PROJECT 1670846
W.P. 151-97-01
LOCATION N 5183385.9; E 300615.1 MTM NAD 83 ZONE 13 (LAT. 46.790020; LONG. -84.054820)
ORIGINATED BY AJ
DIST ALGOMA HWY 532
BOREHOLE TYPE 95 mm O.D. Solid Stem Augers; Wash Boring; NQ Coring
COMPILED BY AK
DATUM Geodetic
DATE September 9 and 10, 2017
CHECKED BY TZ

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED) (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	--- CONTINUED FROM PREVIOUS PAGE ---										
	Sandy SILT, some gravel, with cobbles and boulders Very dense Grey Wet		-	RC	-						
			19	SS	101		208			Non-Plastic	0 22 78 0
	- Casing grinding between depths of about 31.1 m to 31.8 m						207				
206.8 32.0	CASING AND SPLIT-SPOON REFUSAL END OF BOREHOLE NOTES: 1. The cored depth intervals and particle sizes of recovered cobbles/boulders are summarized as follows: Depth (m) Recovered 29.3 - 30.5 100mm; 340mm; 20mm to 50mm rock fragments/ gravel pieces										

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

RECORD OF BOREHOLE No ACB-07

SHEET 1 OF 3 **METRIC**

PROJECT 1670846			
W.P. 151-97-01	LOCATION N 5183380.3; E 300627.3 MTM NAD 83 ZONE 13 (LAT. 46.790020; LONG. -84.054660)		ORIGINATED BY JL
DIST ALGOMA HWY 532	BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring		COMPILED BY AK
DATUM Geodetic	DATE August 26 and 27, 2017		CHECKED BY TZ

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
238.2	GROUND SURFACE
0.0	SAND and SILT, trace gravel, trace clay Very loose to compact Brown Moist to wet - Wet below a depth of about 1.5 m - Sand and gravel layer encountered below a depth of about 2.5 m
235.6 2.6	Varved CLAYEY SILT to SILTY CLAY, trace sand Firm to stiff Grey Wet
228.0 10.2	CLAYEY SILT to SILTY CLAY, trace gravel, trace sand, irregularly stratified Stiff to very stiff Grey Wet

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	5	0 67 31 2
3	SS	2	
4A 4B 4C	SS	22	
5	TO	PH	
6	SS	12	2 3 80 15
7	SS	7	
8	SS	5	0 1 76 23
9	TO	PH	
10	SS	3	
11	SS	1	
12	SS	2	
13	SS	3	

GROUND WATER CONDITIONS

ELEVATION SCALE: 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225, 224

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × REMOULDED: 20 40 60 80 100

Field vane sensitivity values (top to bottom): 5, 6, 7, 5, >96, >96, >96

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

GOLDER

RECORD OF BOREHOLE No ACB-07

SHEET 2 OF 3

METRIC

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183380.3; E 300627.3 MTM NAD 83 ZONE 13 (LAT. 46.790020; LONG. -84.054660)

ORIGINATED BY JL

DIST ALGOMA HWY 532

BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring

COMPILED BY AK

DATUM Geodetic

DATE August 26 and 27, 2017

CHECKED BY TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT to SILTY CLAY, trace gravel, trace sand, irregularly stratified Stiff to very stiff Grey Wet						223				
			14	SS	3						
							222	+ >96			
			15	SS	3		221				
								+ >96			
			16	SS	4		220				
							219				
			17	TO	PH		218				
							217	+ >96			
							216				
			18	SS	12		215				
							214				
							213				
			19	SS	12		212				
							211				
							210				
			20	SS	16		209				
208.2											

Continued Next Page

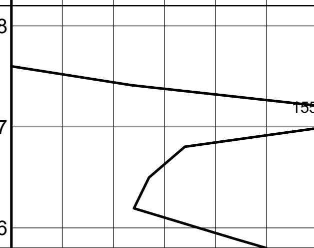
+³, ×³: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No ACB-07 SHEET 3 OF 3 METRIC

PROJECT 1670846
W.P. 151-97-01
LOCATION N 5183380.3; E 300627.3 MTM NAD 83 ZONE 13 (LAT. 46.790020; LONG. -84.054660)
ORIGINATED BY JL
DIST ALGOMA HWY 532
BOREHOLE TYPE 210 mm O.D. Hollow Stem Augers; Wash Boring
COMPILED BY AK
DATUM Geodetic
DATE August 26 and 27, 2017
CHECKED BY TZ

--- CONTINUED FROM PREVIOUS PAGE ---

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
30.0	SILTY SAND and GRAVEL, trace clay Very dense Grey Wet						208						
207.6	END OF BOREHOLE Dynamic Core Penetration Test (DCPT)		21	SS	100/0.15		207	○		41	30	27	2
205.8 32.4	END OF DCPT						206						



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

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-08

SHEET 1 OF 2

METRIC

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183407.7; E 300610.7 MTM NAD 83 ZONE 13 (LAT. 46.790216; LONG. -84.054878)

ORIGINATED BY JL

DIST ALGOMA HWY 532

BOREHOLE TYPE 210 mm O.D. Continuous Flight, Hollow Stem Augers

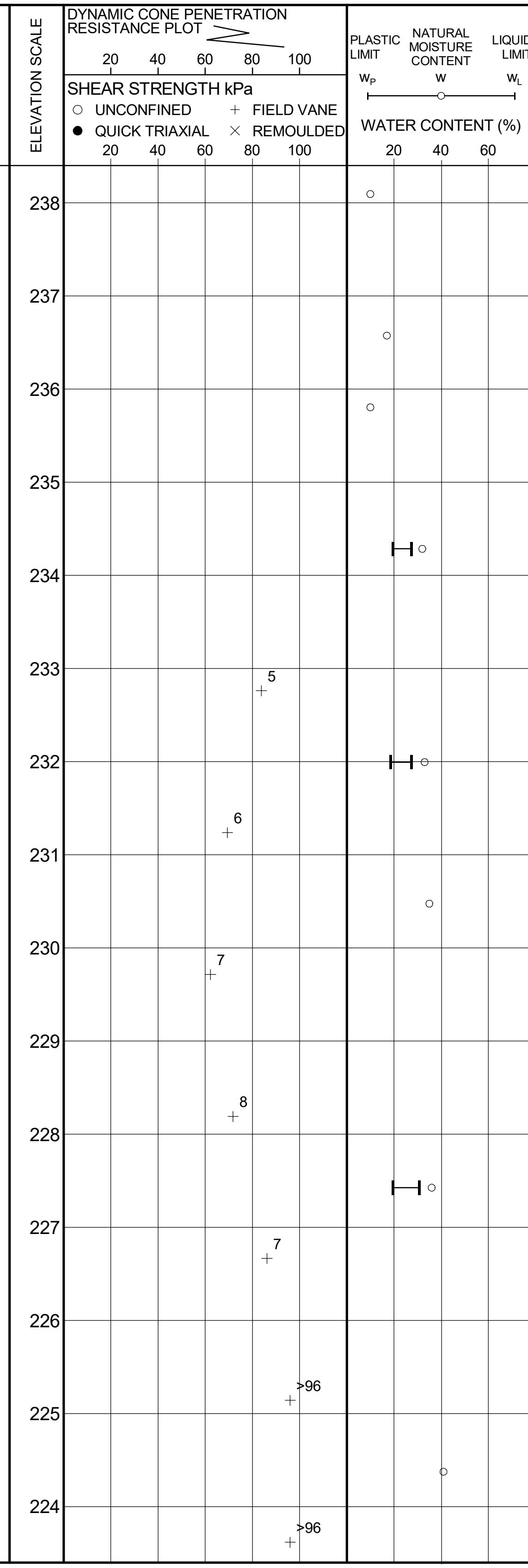
COMPILED BY AK

DATUM Geodetic

DATE August 29 and 30, 2017

CHECKED BY TZ

ELEV DEPTH	SOIL PROFILE – DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
238.4	GROUND SURFACE					
0.0	Sand, trace to some gravel, trace to some silt (FILL) Loose to compact Brown Moist to wet	1	SS	6		12 76 12 0
		2	SS	11		
	- Wet below a depth of about 1.5 m	3	SS	8		
236.2 2.2	SAND and GRAVEL, trace to some silt, trace clay Dense Brown to grey Wet	4	SS	31		51 42 6 1
		5	SS	39		
234.7 3.7	Varved CLAYEY SILT to SILTY CLAY, trace sand Stiff Grey Wet	6	SS	6		
		7	SS	5		
		8	SS	4		
		9	SS	2	▽ (approx. El. 230)	
		10	TO	PH		
228.0 10.4	CLAYEY SILT to SILTY CLAY, trace sand, irregularly stratified Stiff to very stiff Grey Wet	11	SS	1		
		12	SS	2		
		13	SS	2		



UNIT WEIGHT γ kN/m³

Continued Next Page

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

GOLDER

Foundation Design

RECORD OF BOREHOLE No ACB-08

SHEET 2 OF 2

METRIC

PROJECT 1670846

W.P. 151-97-01

LOCATION N 5183407.7; E 300610.7 MTM NAD 83 ZONE 13 (LAT. 46.790216; LONG. -84.054878)

ORIGINATED BY JL

DIST ALGOMA HWY 532

BOREHOLE TYPE 210 mm O.D. Continuous Flight, Hollow Stem Augers

COMPILED BY AK

DATUM Geodetic

DATE August 29 and 30, 2017

CHECKED BY TZ

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (20 40 60)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION --- CONTINUED FROM PREVIOUS PAGE ---	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
222.6			14	SS	2		223				
15.9	END OF BOREHOLE NOTE: 1. Water level measured in casing at a depth of about 8.5 m below ground surface (Elev. 229.9 m) on August 30, 2017.										

+3, ×3: Numbers refer to Sensitivity

○3% STRAIN AT FAILURE

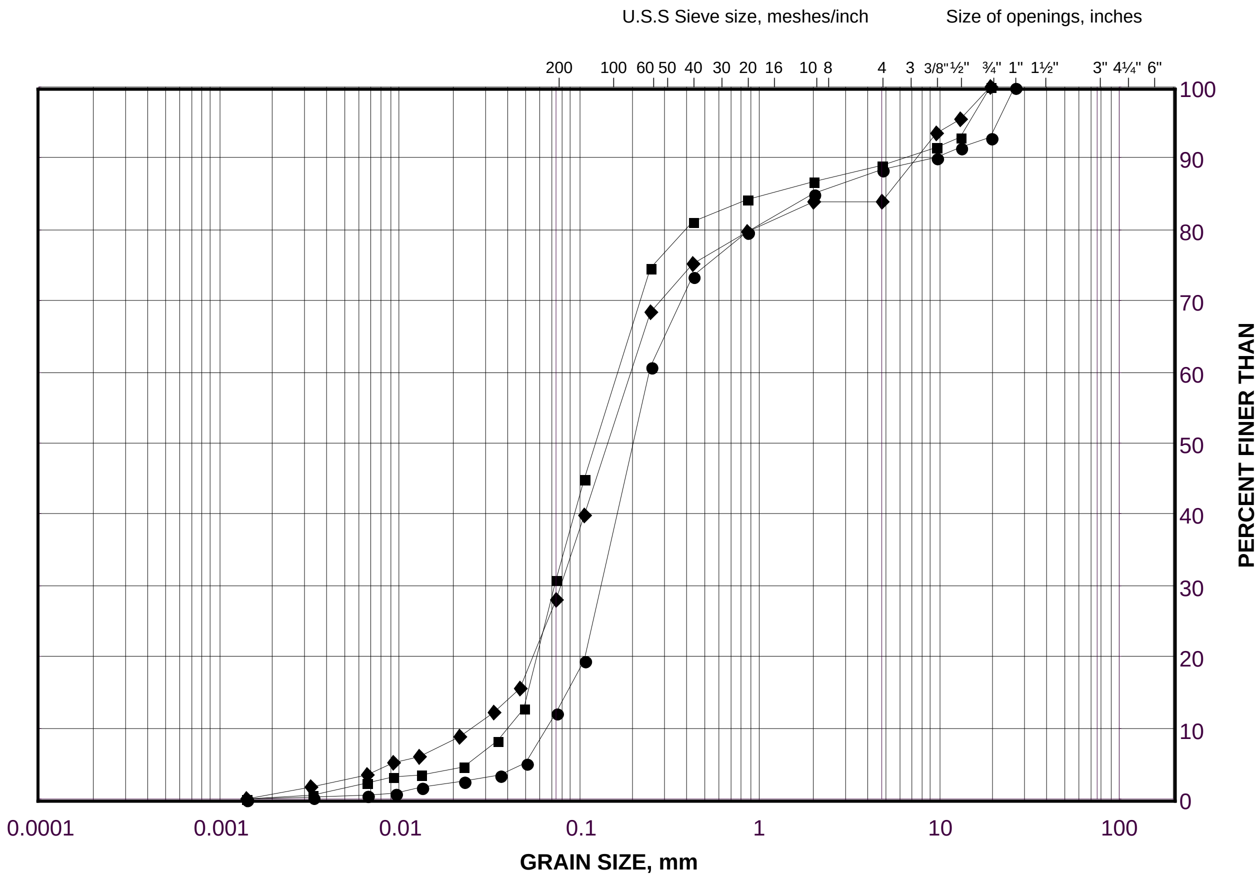
APPENDIX C

Geotechnical Laboratory Test Results

GRAIN SIZE DISTRIBUTION

Silt and Sand to Silty Sand to Sand (Fill)

FIGURE C1



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ACB-08	1	238.1
■	ACB-03	3	236.5
◆	ACB-04	4	235.4

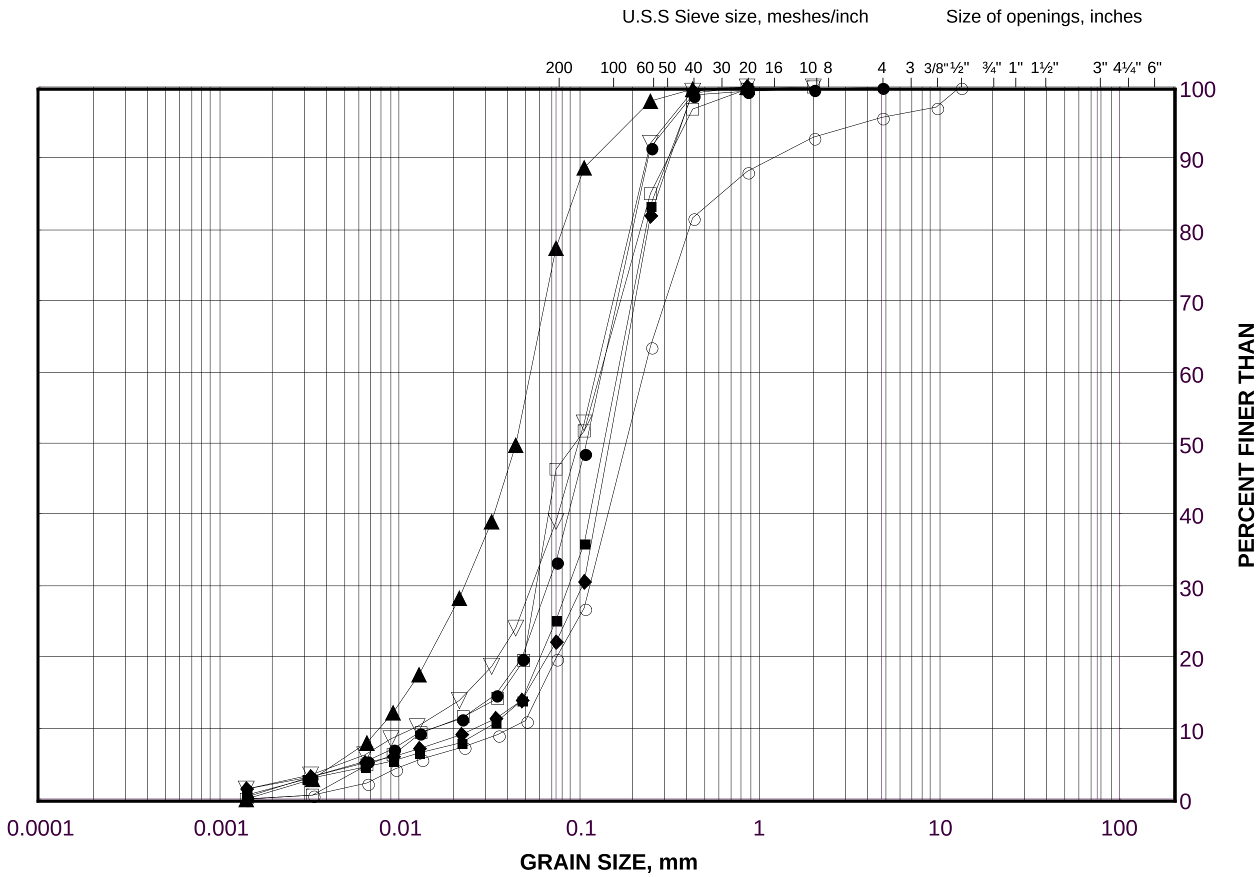
Project Number: 1670846

Checked By: TZ

Golder Associates

Date: 11-Jun-18

GRAIN SIZE DISTRIBUTION Sandy Silt to Sand and Silt to Silty Sand to Sand (Upper Granular Deposit)	FIGURE C2A



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ACB-07	2	237.1
■	ACB-05	3	236.0
◆	ACB-06	3	236.2
▲	ACB-02	3	236.3
▽	ACB-05	4	235.2
❍	ACB-01	4	236.3
☐	ACB-03	6	234.2

Project Number: 1670846

Checked By: TZ

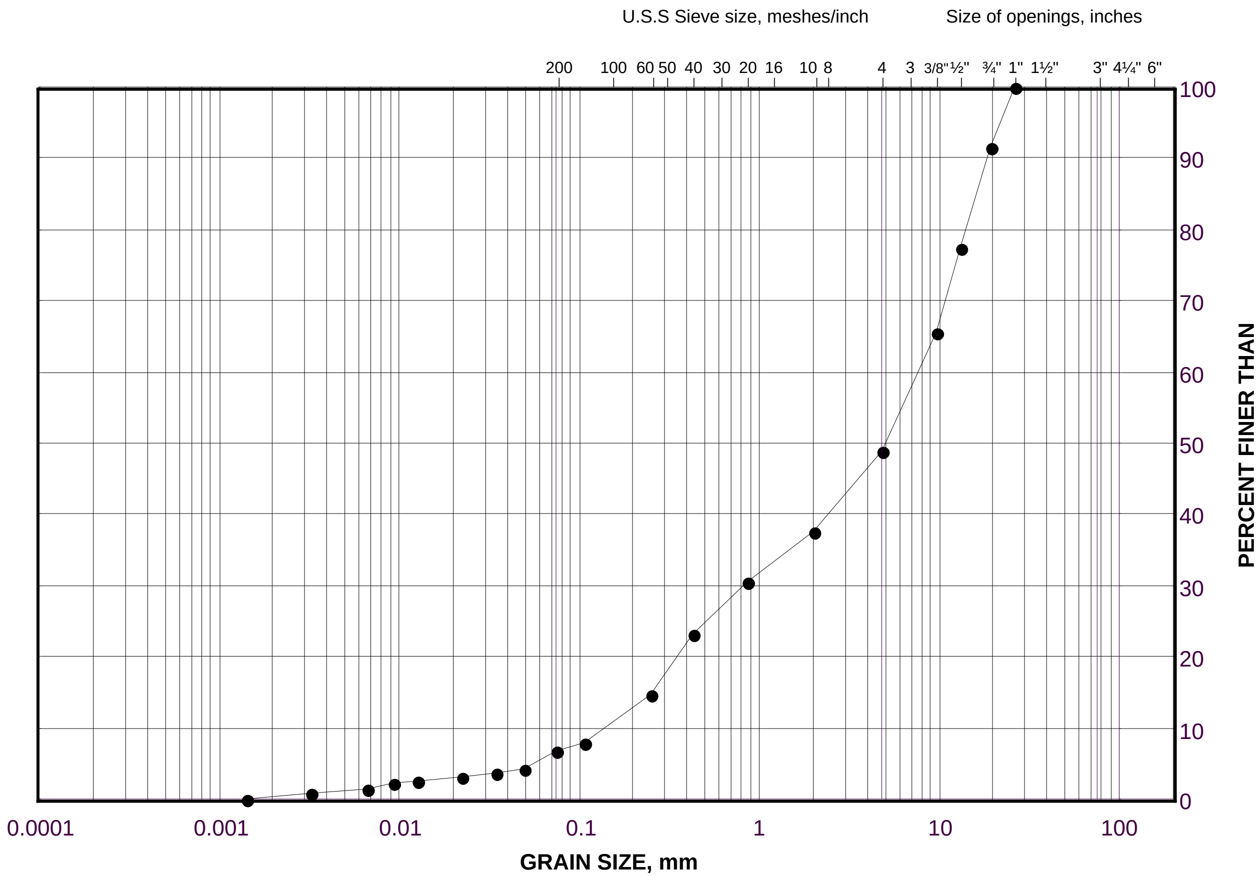
Golder Associates

Date: 11-Jun-18

GRAIN SIZE DISTRIBUTION

Sand and Gravel (Upper Granular Deposit)

FIGURE C2B



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	**COBBLE**
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ACB-08	4	235.8

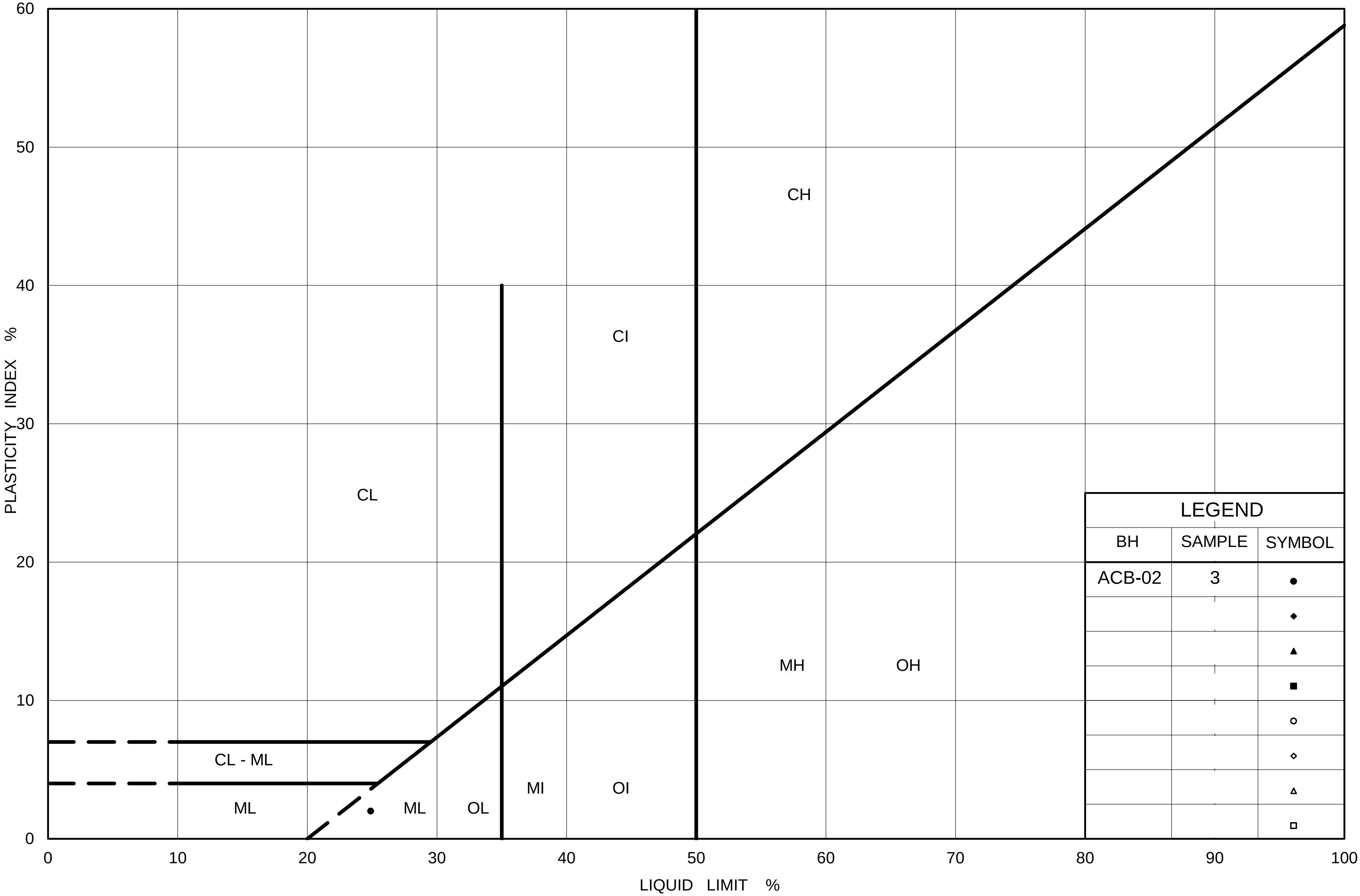
Project Number: 1670846

Checked By: TZ

Golder Associates

Date: 11-Jun-18

Oct 75, FF-S-21



Ministry of Transportation

Ontario

PLASTICITY CHART
Sandy Silt of Slight Plasticity (Fines Portion)

Figure No. C3

Project No. 1670846

Checked By: TZ

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 1 OF 3	FIGURE C4A		
TEST STAGE	A	B	C
BOREHOLE NUMBER		ACB-05	
SAMPLE		2, 3 and 4	
SAMPLE DEPTH, (m)		-	
SAMPLE HEIGHT, (mm)	27.41	27.44	27.51
SAMPLE LENGTH, (mm)	60.00	60.00	60.00
WATER CONTENT, BEFORE TEST, (%)	25.01	25.01	25.01
NORMAL (CONSOLIDATION) STRESS, (kPa)	25	50	100
WATER CONTENT, AFTER TEST, (%)	20.07	19.34	19.55
DISPLACEMENT RATE, mm/min	0.012	0.012	0.012
TIME TO FAILURE, hours	2.4	4.8	6.2
PEAK SHEAR STRESS[1], (kPa)	22.7	46.8	90.0
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	1.7	3.5	4.5
DRY DENSITY, initial, Mg/m^3	1.553	1.521	1.521
WET DENSITY, initial, Mg/m^3	1.942	1.902	1.901

TEST NOTES:

1 In the absence of a peak, the shear stress reported is at 10 percent relative horizontal displacement (ASTM D3080).

2 Direct Shear Tests carried out under submerged conditions.

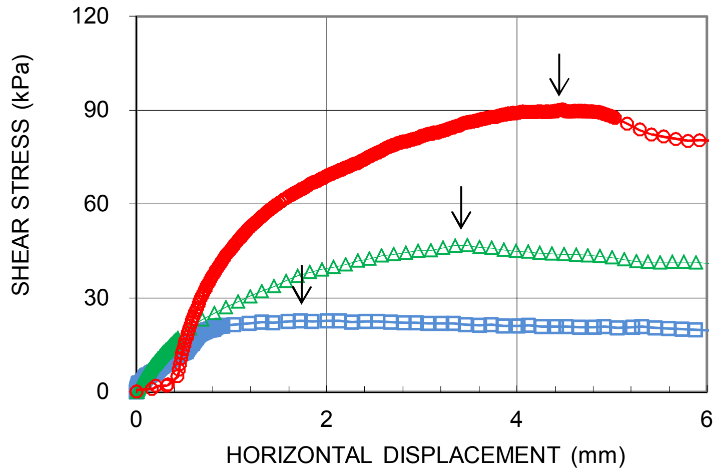
Date: 6/21/2018 Prepared By: LH

Project No. 1670846 **Golder Associates Ltd.** Checked By: TZ

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 2 OF 3	FIGURE C4B

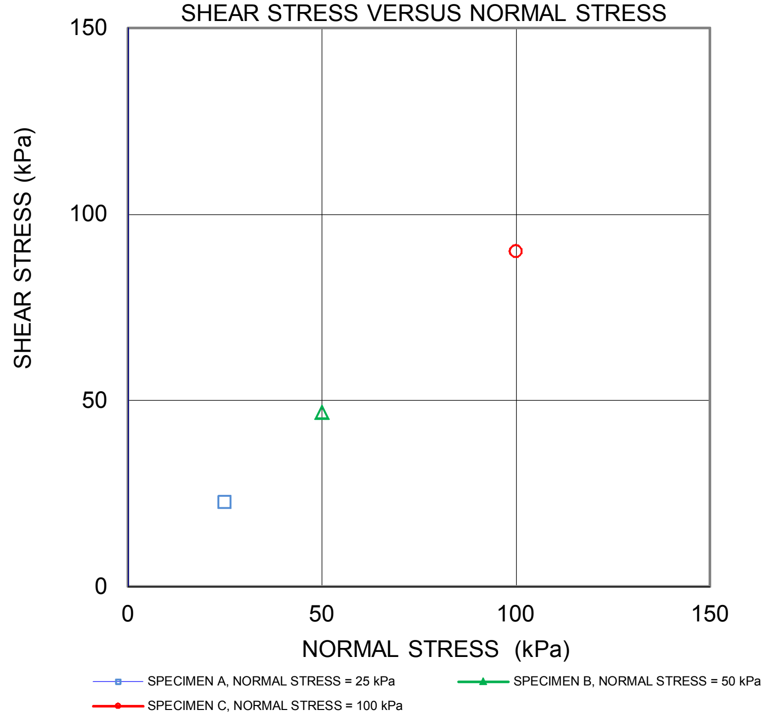
BH ACB-05 SA Nos. 2, 3, and 4

SHEAR STRESS VERSUS HORIZONTAL DISPLACEMENT



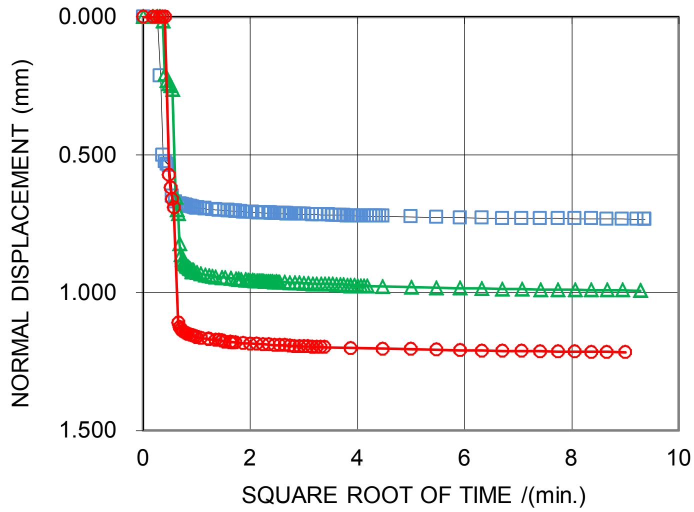
BH ACB-05 SA Nos. 2, 3, and 4

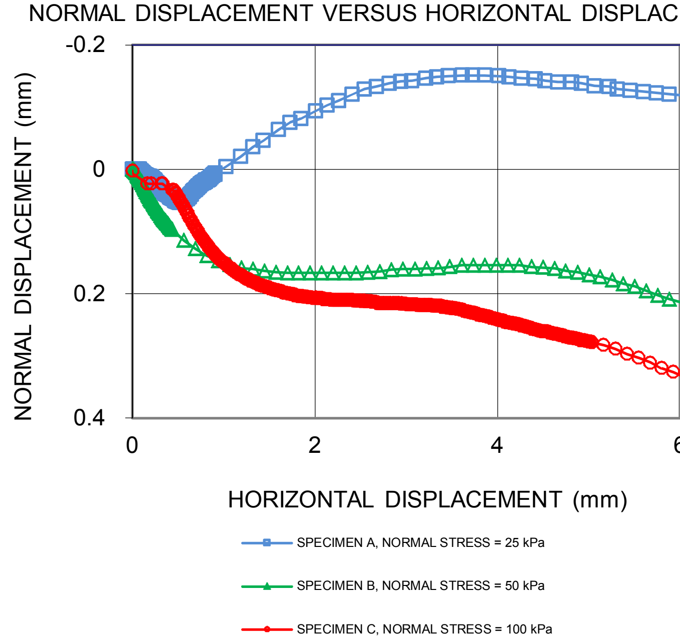
SHEAR STRESS VERSUS NORMAL STRESS



Date:	6/21/2018		Prepared By:	LH
Project No.	1670846	**Golder Associates Ltd.**	Checked By:	TZ

CONSOLIDATED DRAINED DIRECT SHEAR TEST SHEET 3 OF 3	FIGURE C4C

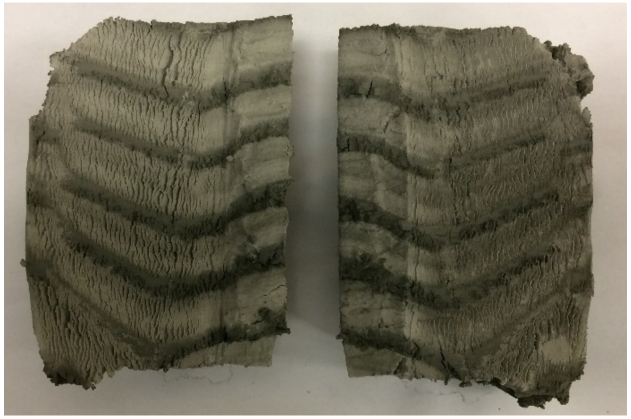




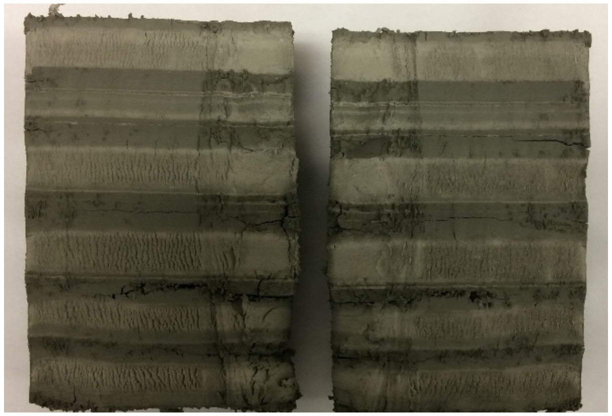
Date:	6/21/2018		Prepared By:	LH
Project No.	1670846	**Golder Associates Ltd.**	Checked By:	TZ

Varved Clayey Silt to Silty Clay

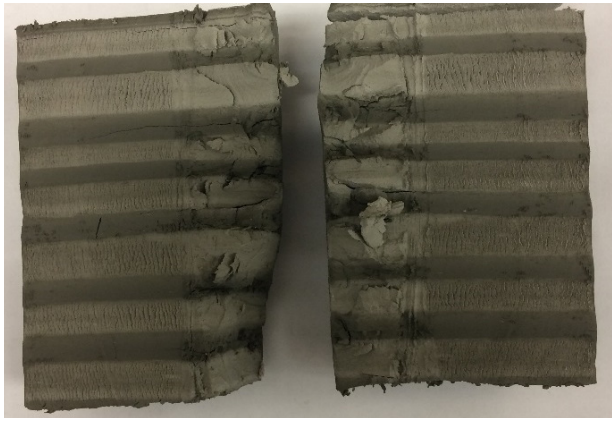
Figure C5A



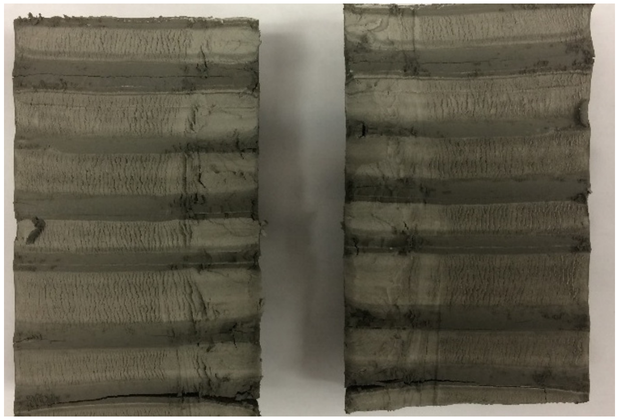
Photograph 1: Soil sample from Borehole ACB-01
Sample 8



Photograph 2: Soil sample from Borehole ACB-02
Sample 9

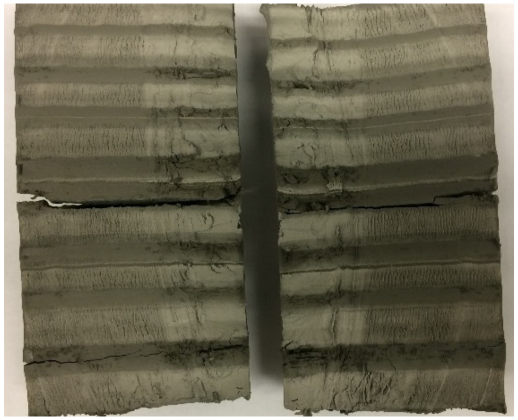


Photograph 3: Soil sample from Borehole ACB-03
Sample 10



Photograph 4: Soil sample from Borehole ACB-06
Sample 8

Photograph 5: Soil sample from Borehole ACB-07
Sample 9



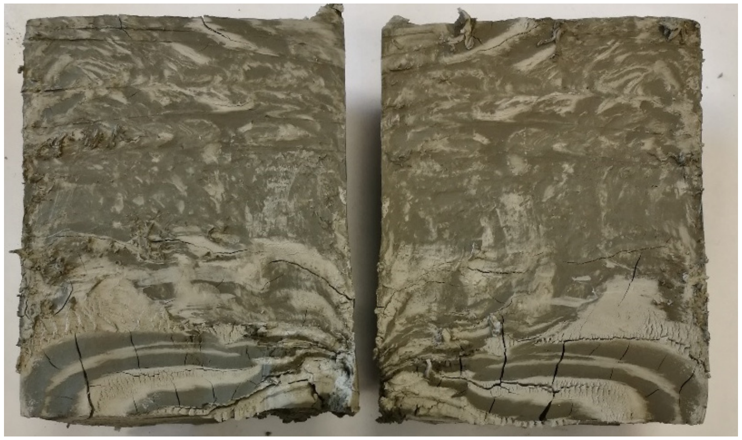
Photograph 6: Soil sample from Borehole ACB-08
Sample 10

Notes:

1. The dark laminae represent silty clay of intermediate plasticity, while the lighter laminae represent clayey silt of low plasticity and/or silt.
2. The soil samples were extracted from Shelby tubes and partially dried to illustrate the distinctions between the various laminae.

Date: June 22, 2018
Project No: 1670846

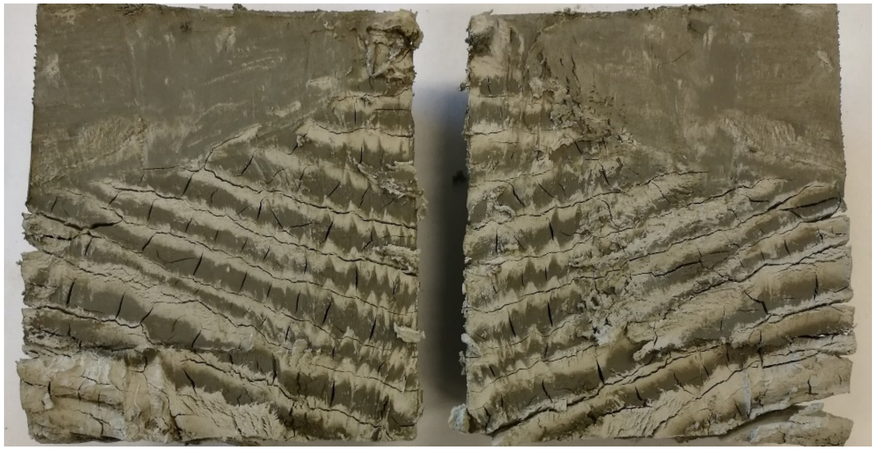
Golder Associates



Photograph 1: Soil sample from Borehole ACB-04 Sample 12

Photograph 2: Soil sample from Borehole ACB-04 Sample 18

Photograph 3: Soil sample from Borehole ACB-05 Sample 11



Photograph 4: Soil sample from Borehole ACB-05 Sample 14

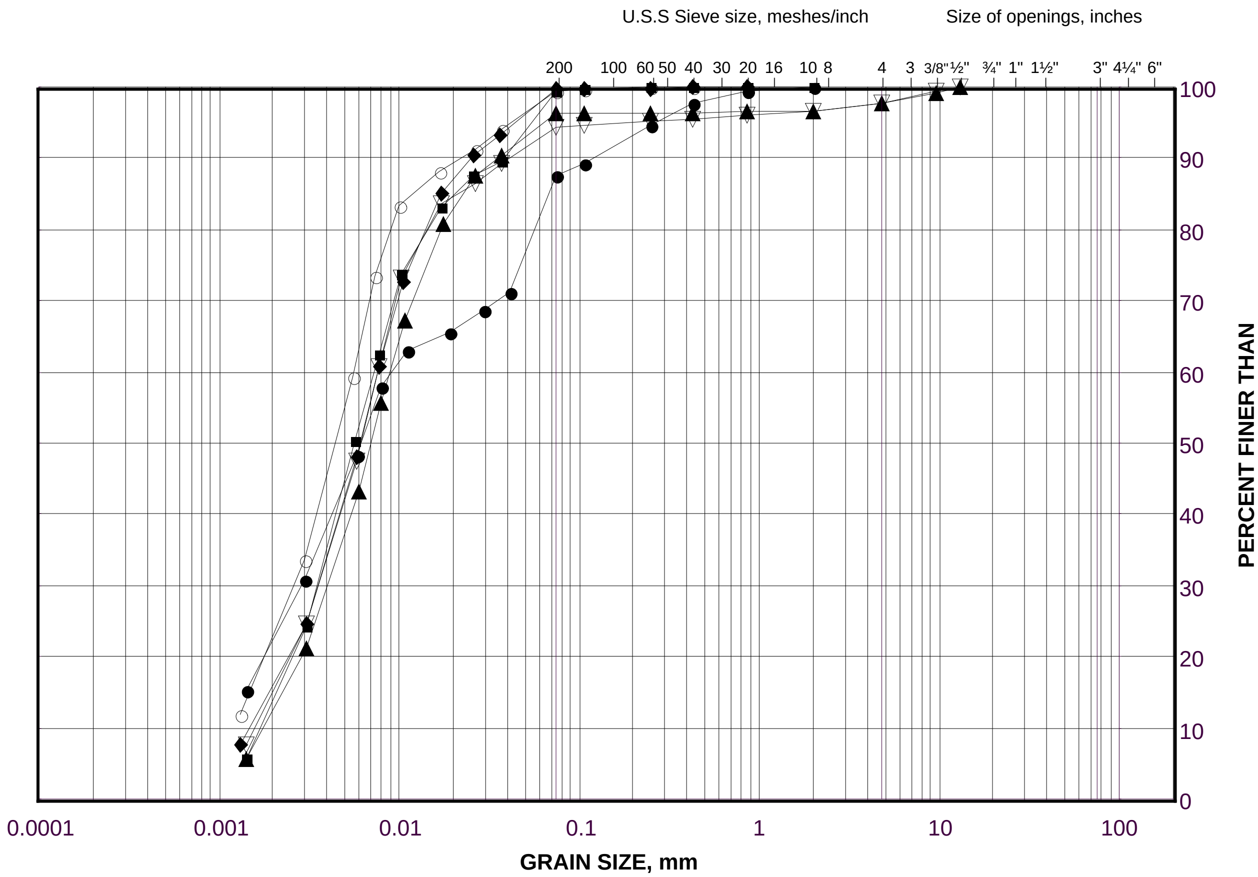
Notes:

1. The dark layers represent silty clay of intermediate plasticity, while the lighter layers represent clayey silt of low plasticity and/or silt.
2. The soil samples were extracted from Shelby tubes and partially dried to illustrate the distinctions between the various layers.

GRAIN SIZE DISTRIBUTION

Clayey Silt to Silty Clay

FIGURE C6



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ACB-06	16	217.2
■	ACB-06	18	211.2
◆	ACB-06	5	234.7
▲	ACB-02	5	234.8
▽	ACB-07	6	234.1
❍	ACB-07	8	231.8

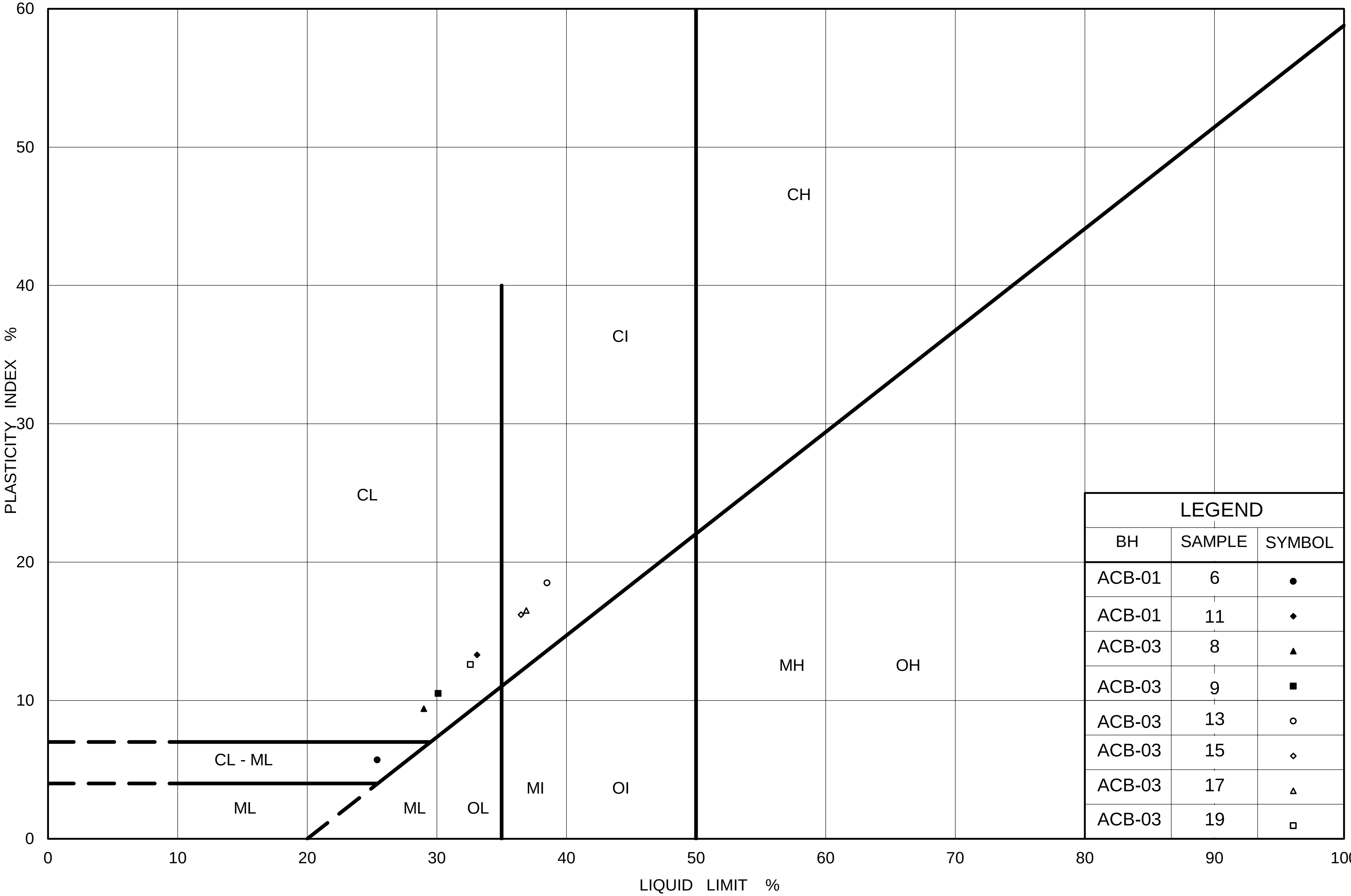
Project Number: 1670846

Checked By: TZ

Golder Associates

Date: 13-Apr-18

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
ACB-01	6	●
ACB-01	11	◆
ACB-03	8	▲
ACB-03	9	■
ACB-03	13	○
ACB-03	15	◇
ACB-03	17	△
ACB-03	19	□

Ministry of Transportation

Ontario

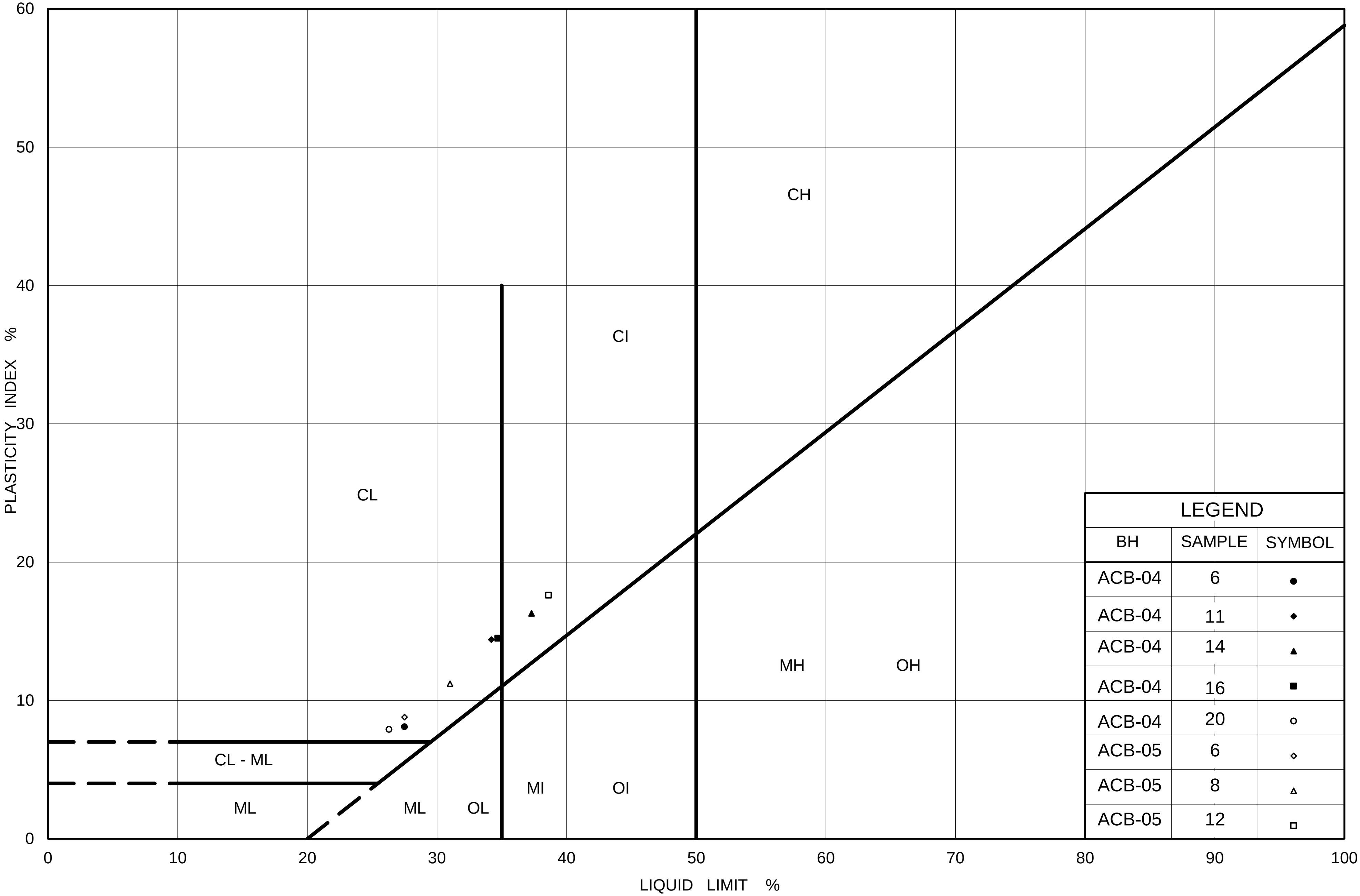
PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. C7A

Project No. 1670846

Checked By: TZ

Oct 75, FF-S-21



Ministry of Transportation

Ontario

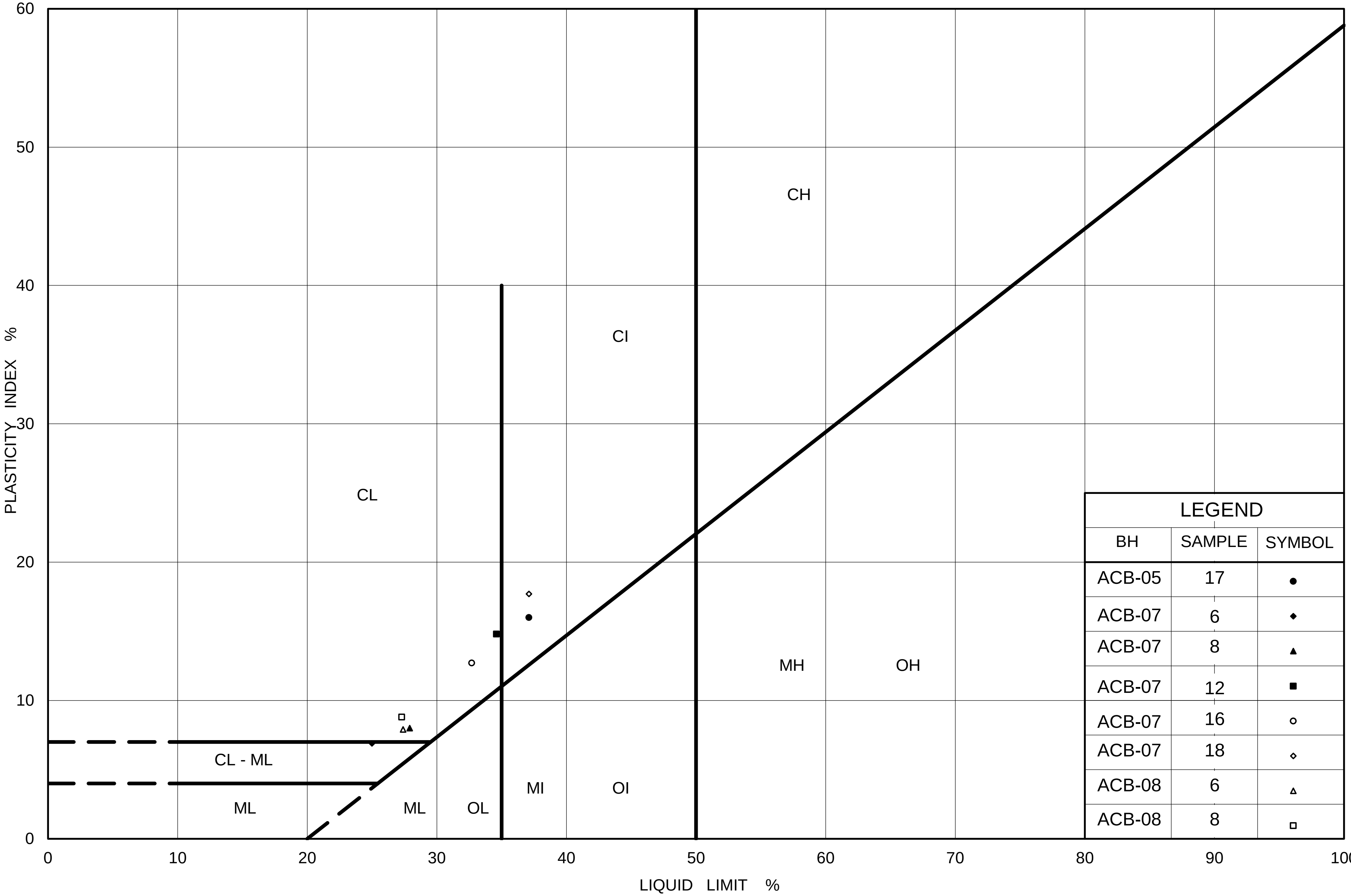
PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. C7B

Project No. 1670846

Checked By: TZ

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
ACB-05	17	●
ACB-07	6	◆
ACB-07	8	▲
ACB-07	12	■
ACB-07	16	○
ACB-07	18	◇
ACB-08	6	△
ACB-08	8	□

Ministry of Transportation

Ontario

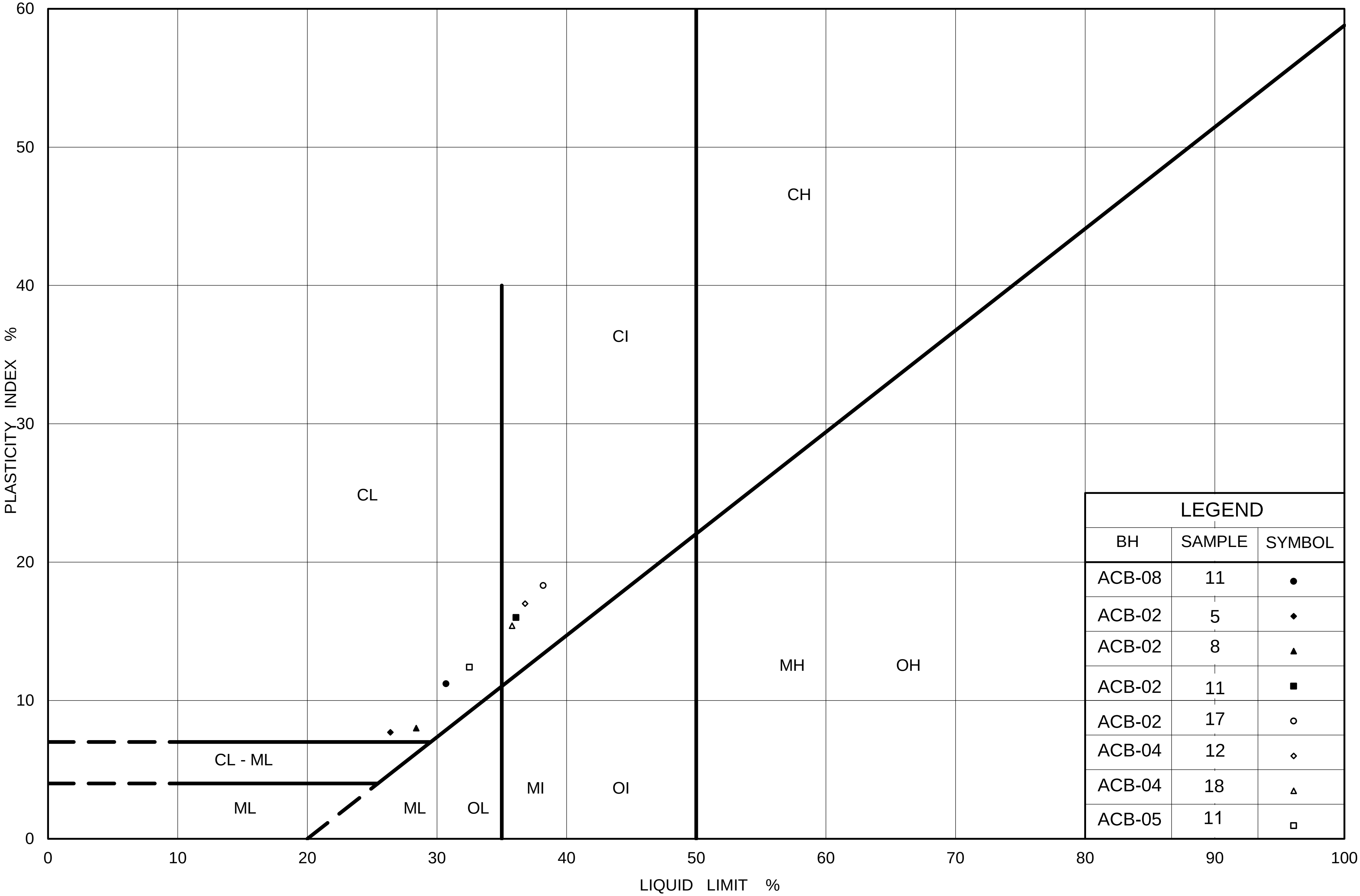
PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. C7C

Project No. 1670846

Checked By: TZ

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
ACB-08	11	●
ACB-02	5	◆
ACB-02	8	▲
ACB-02	11	■
ACB-02	17	○
ACB-04	12	◇
ACB-04	18	△
ACB-05	11	□

Ministry of Transportation

Ontario

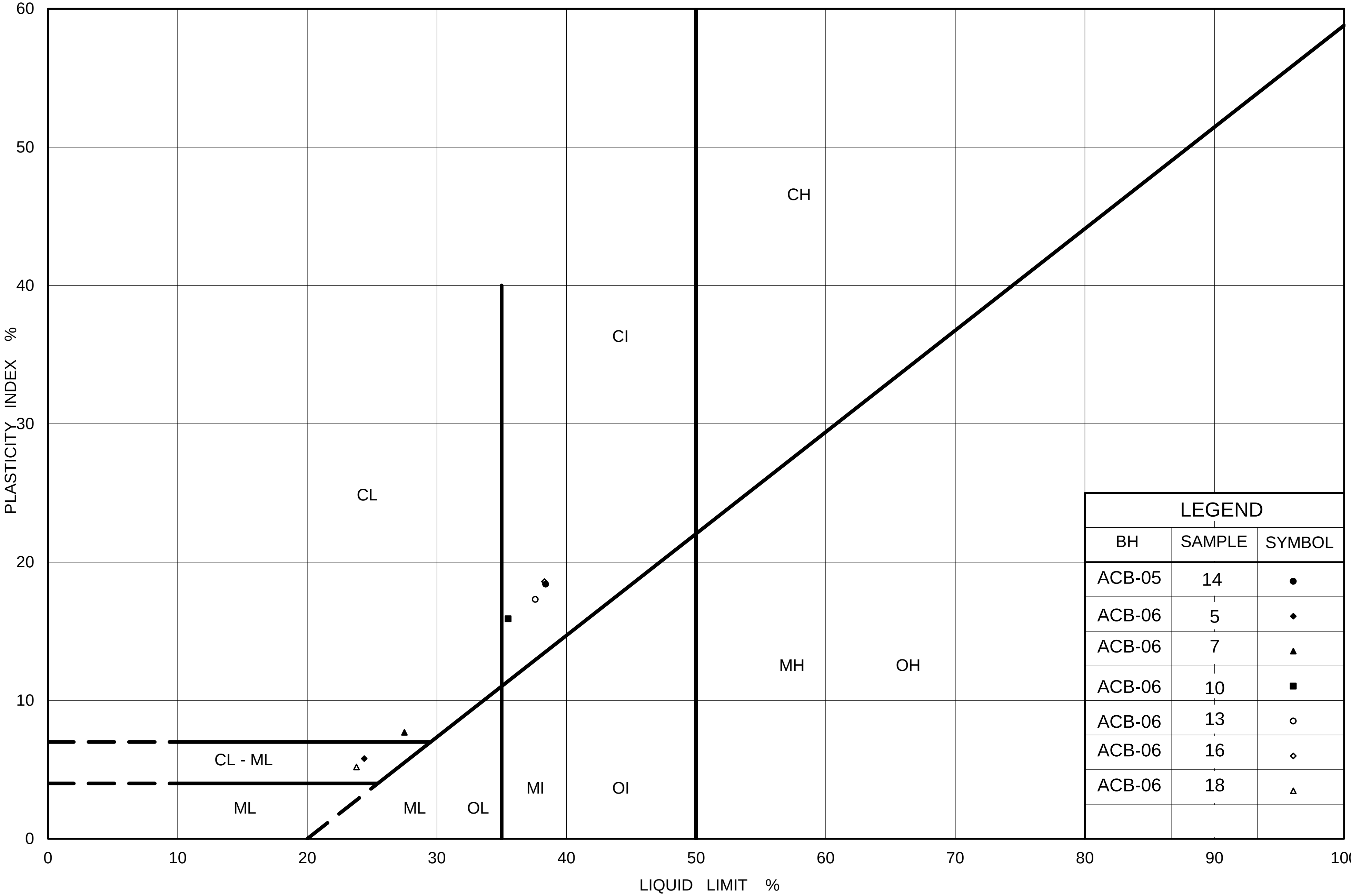
PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. C7D

Project No. 1670846

Checked By: TZ

Oct 75, FF-S-21



LEGEND		
BH	SAMPLE	SYMBOL
ACB-05	14	●
ACB-06	5	◆
ACB-06	7	▲
ACB-06	10	■
ACB-06	13	○
ACB-06	16	◇
ACB-06	18	△

Ministry of Transportation

Ontario

PLASTICITY CHART
Clayey Silt to Silty Clay

Figure No. C7E

Project No. 1670846

Checked By: TZ

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE C8A

SAMPLE IDENTIFICATION

Project Number	1670846	Sample Number	12
Borehole Number	ACB-04	Sample Depth, m	12.65-12.73

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	8		
Date Started	09/25/2017		
Date Completed	10/11/2017		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.90	Unit Weight, kN/m^3	17.73
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	12.52
Area, cm^2	31.50	Specific Gravity, measured	2.74
Volume, cm^3	59.94	Solids Height, cm	0.887
Water Content, %	41.61	Volume of Solids, cm^3	27.94
Wet Mass, g	108.40	Volume of Voids, cm^3	32.01
Dry Mass, g	76.55	Degree of Saturation, %	99.5

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.903	1.146	1.903				
6.36	1.902	1.144	1.903				
11.14	1.902	1.144	1.902	83	9.24E-03	2.20E-05	1.99E-08
21.23	1.898	1.140	1.900	60	1.28E-02	1.87E-04	2.34E-07
40.61	1.893	1.134	1.896	54	1.41E-02	1.46E-04	2.02E-07
79.44	1.878	1.117	1.885	79	9.54E-03	2.00E-04	1.87E-07
123.31	1.866	1.104	1.872	86	8.64E-03	1.45E-04	1.23E-07
40.53	1.877	1.116	1.871				
21.23	1.879	1.119	1.878				
60.12	1.869	1.107	1.874	34	2.19E-02	1.41E-04	3.02E-07
123.33	1.864	1.102	1.866	22	3.36E-02	4.16E-05	1.37E-07
157.10	1.858	1.095	1.861	43	1.71E-02	9.49E-05	1.59E-07
312.55	1.829	1.062	1.844	38	1.90E-02	9.67E-05	1.80E-07
623.50	1.759	0.983	1.794	113	6.04E-03	1.19E-04	7.07E-08
1245.54	1.608	0.813	1.683	129	4.66E-03	1.27E-04	5.79E-08
2488.87	1.517	0.710	1.562	98	5.28E-03	3.88E-05	2.01E-08
623.50	1.529	0.724	1.523				
123.38	1.560	0.759	1.545				
40.53	1.578	0.779	1.569				
11.24	1.603	0.807	1.590				

Notes:

Consolidation loading and unloading schedule assigned by the client.

c_v and k are approximate only and based on t_{90} estimated from the Square Root of Time Method (ASTMD2435/2435M).

Specimen swelled under a stress of 6.36 kPa.

SAMPLE DIMENSIONS AND PROPERTIES - FINAL

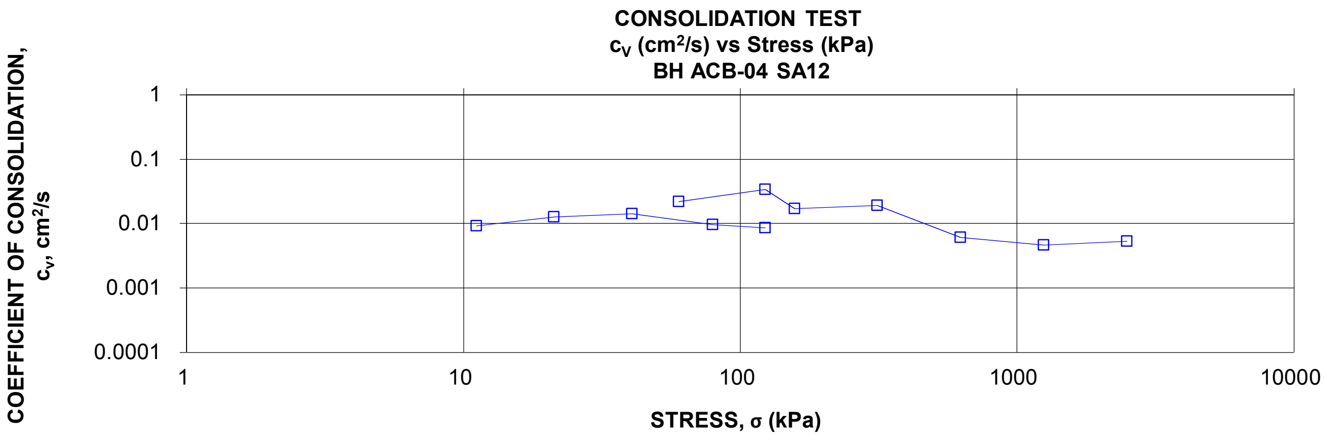
Sample Height, cm	1.60	Unit Weight, kN/m^3	19.43
Sample Diameter, cm	6.33	Dry Unit Weight, kN/m^3	14.87
Area, cm^2	31.50	Specific Gravity, measured	2.74
Volume, cm^3	50.48	Solids Height, cm	0.887
Water Content, %	30.62	Volume of Solids, cm^3	27.94
Wet Mass, g	99.99	Volume of Voids, cm^3	22.54
Dry Mass, g	76.55		

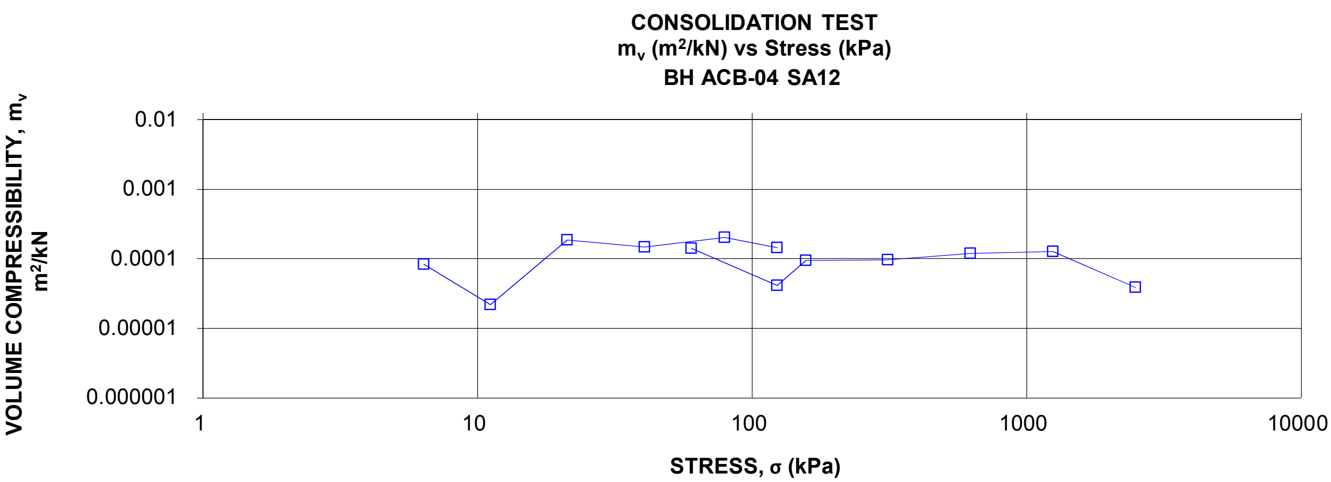
Prepared By: LH

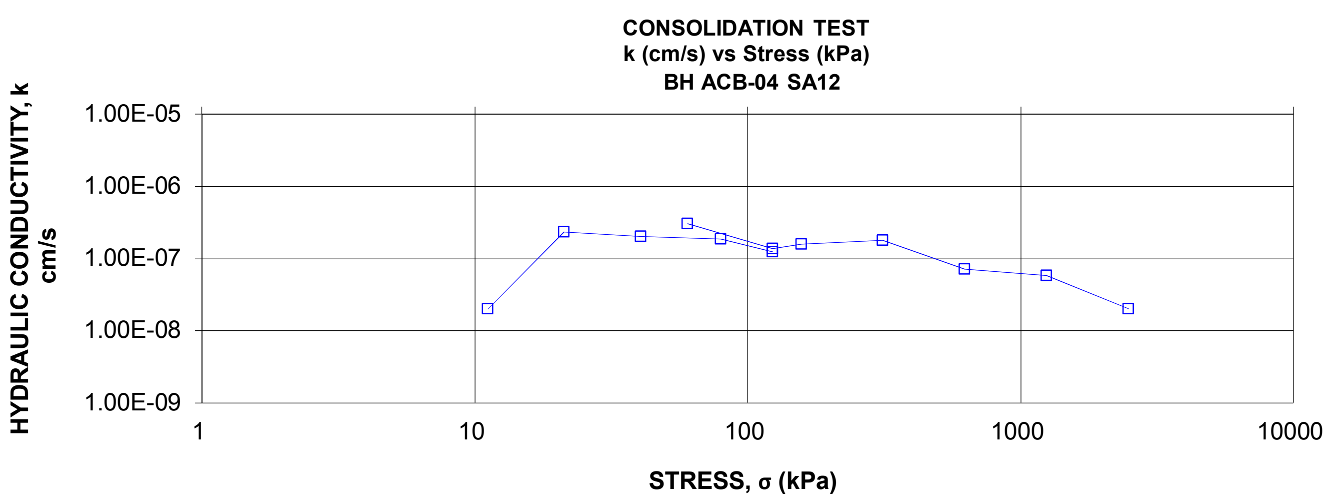
Golder Associates Ltd.

Checked By: TZ

CONSOLIDATION TEST SUMMARY	FIGURE C8B







Project No. 1670846

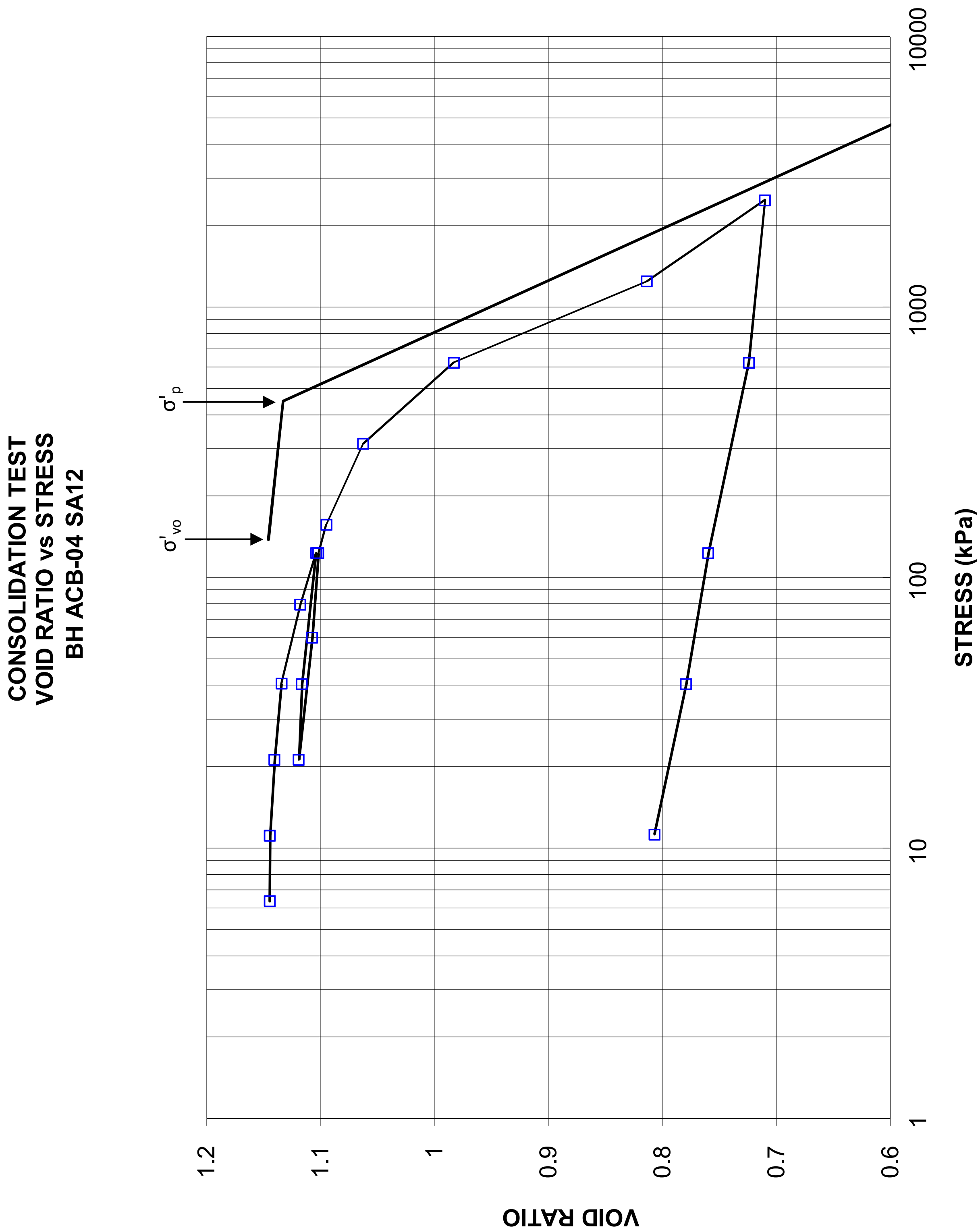
Prepared By: LH

Golder Associates Ltd.

Checked By: TZ

CONSOLIDATION TEST
VOID RATIO VS LOG STRESS

FIGURE C8C



Project No. 1670846

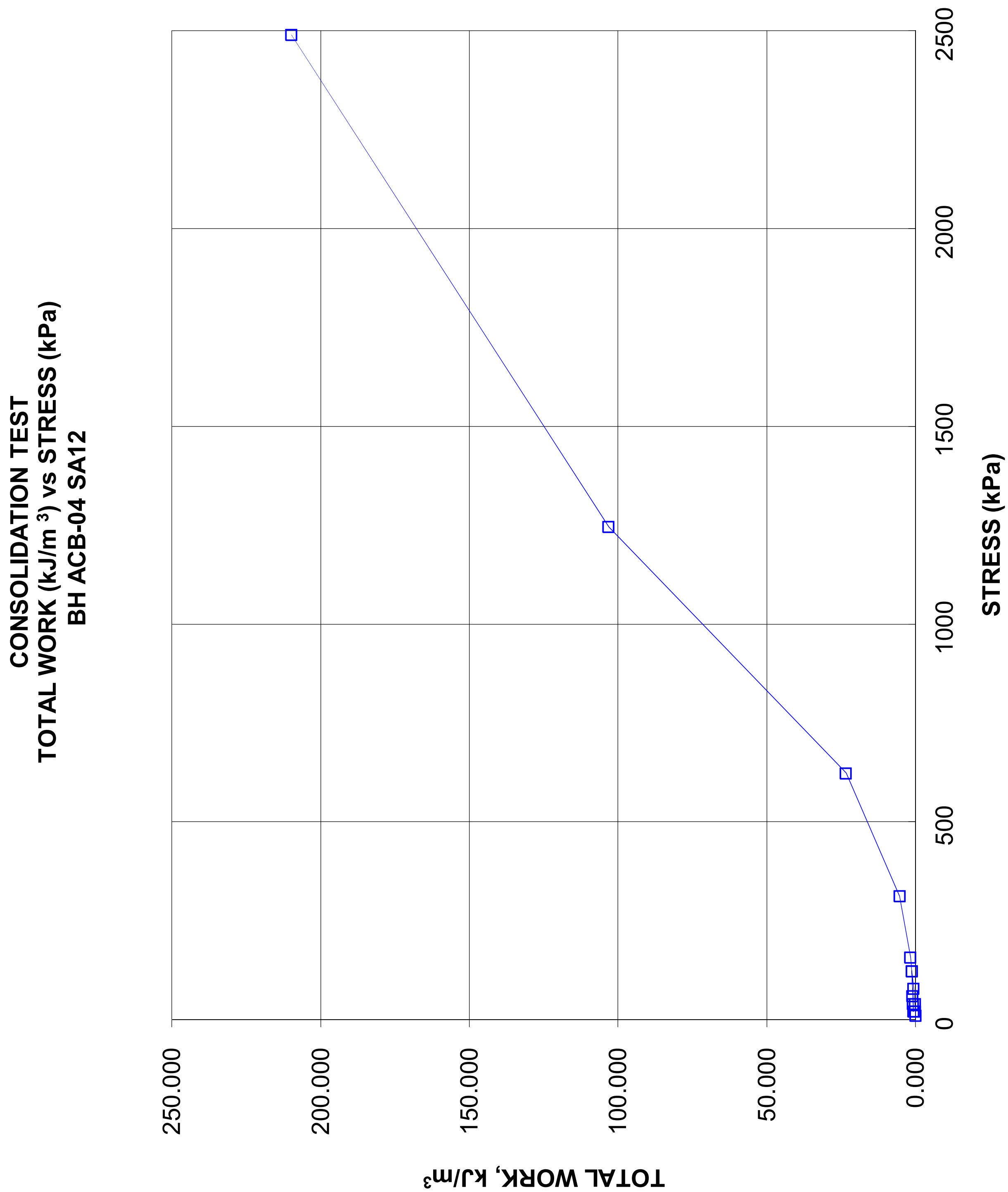
Prepared By: LH

Golder Associates Ltd.

Checked By: TZ

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C8D



Project No.1670746

Prepared By: LH

Golder Associates Ltd.

Checked By: TZ

CONSOLIDATION TEST SUMMARY

ASTM D2435/D2435M

FIGURE C9A

SAMPLE IDENTIFICATION

Project Number	1670846	Sample Number	11
Borehole Number	ACB-05	Sample Depth, m	11.03-11.13

TEST CONDITIONS

Test Type	Laboratory Standard	Load Duration, hr	24
Oedometer Number	6		
Date Started	09/25/2017		
Date Completed	10/11/2017		

SAMPLE DIMENSIONS AND PROPERTIES - INITIAL

Sample Height, cm	1.89	Unit Weight, kN/m^3	17.55
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	12.30
Area, cm^2	31.60	Specific Gravity, measured	2.71
Volume, cm^3	59.69	Solids Height, cm	0.875
Water Content, %	42.68	Volume of Solids, cm^3	27.63
Wet Mass, g	106.85	Volume of Voids, cm^3	32.06
Dry Mass, g	74.89	Degree of Saturation, %	99.7

TEST COMPUTATIONS

Stress kPa	Corr. Height cm	Void Ratio	Average Height cm	t_{90} sec	cv. cm^2/s	mv m^2/kN	k cm/s
0.00	1.889	1.160	1.889				
5.85	1.887	1.158	1.888				
10.70	1.882	1.152	1.885	79	9.53E-03	5.46E-04	5.10E-07
20.47	1.875	1.144	1.879	147	5.09E-03	3.79E-04	1.89E-07
39.89	1.868	1.136	1.872	135	5.50E-03	1.91E-04	1.03E-07
78.74	1.853	1.119	1.861	231	3.18E-03	2.04E-04	6.36E-08
117.30	1.824	1.086	1.846	936	7.72E-04	3.01E-04	2.28E-08
39.86	1.829	1.091	1.827				
20.47	1.834	1.097	1.832				
59.18	1.831	1.094	1.833	22	3.24E-02	4.10E-05	1.30E-07
117.21	1.818	1.079	1.825	34	2.08E-02	1.19E-04	2.41E-07
156.07	1.799	1.057	1.809	97	7.15E-03	2.59E-04	1.81E-07
311.03	1.744	0.994	1.772	109	6.10E-03	1.88E-04	1.12E-07
620.91	1.673	0.913	1.709	126	4.91E-03	1.21E-04	5.84E-08
1240.45	1.578	0.804	1.626	118	4.75E-03	8.12E-05	3.78E-08
2480.16	1.485	0.698	1.532	173	2.87E-03	3.97E-05	1.12E-08
620.91	1.509	0.725	1.497				
117.78	1.544	0.766	1.527				
39.86	1.564	0.788	1.554				
10.70	1.590	0.818	1.577				

Note:
Consolidation loading and unloading schedule assigned by the client.
cv and k are approximate only based on t_{90} estimated from Square Root of Time Method (ASTMD2435/2435M)
Specimen swelled under 5.85 kPa

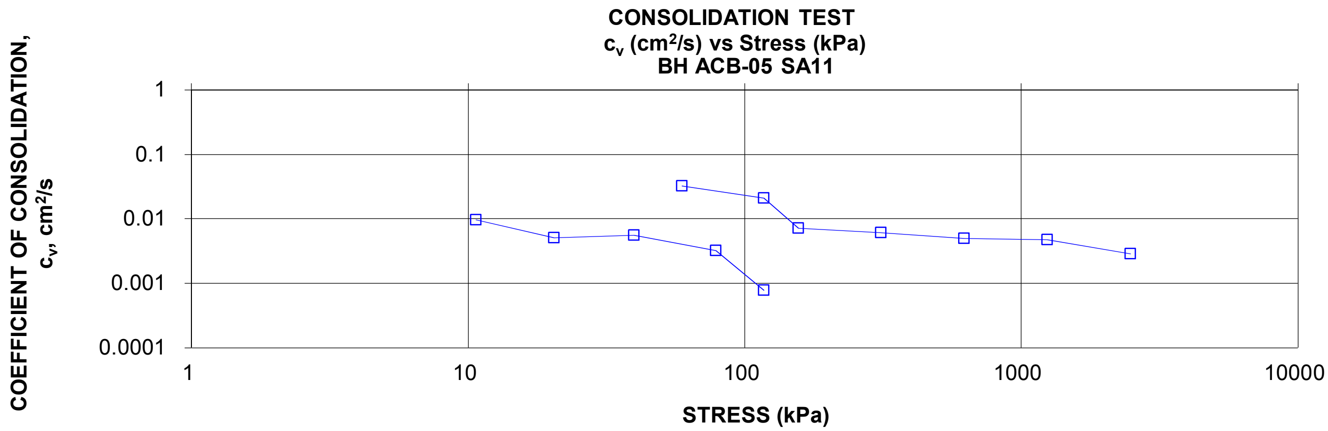
SAMPLE DIMENSIONS AND PROPERTIES - FINAL

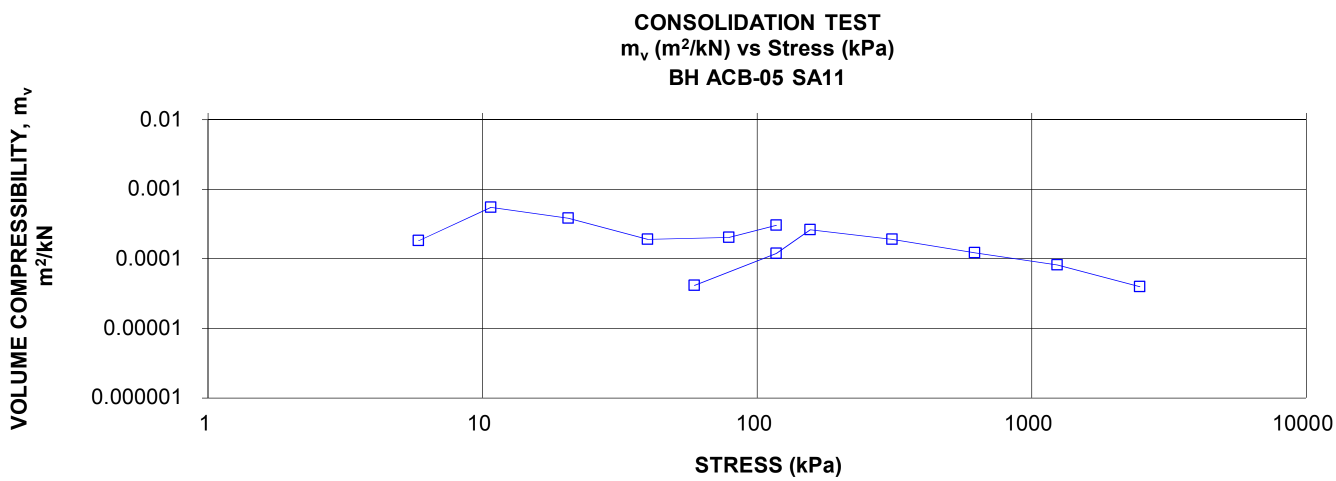
Sample Height, cm	1.59	Unit Weight, kN/m^3	19.22
Sample Diameter, cm	6.34	Dry Unit Weight, kN/m^3	14.62
Area, cm^2	31.60	Specific Gravity, measured	2.71
Volume, cm^3	50.24	Solids Height, cm	0.875
Water Content, %	31.46	Volume of Solids, cm^3	27.63
Wet Mass, g	98.45	Volume of Voids, cm^3	22.61
Dry Mass, g	74.89		

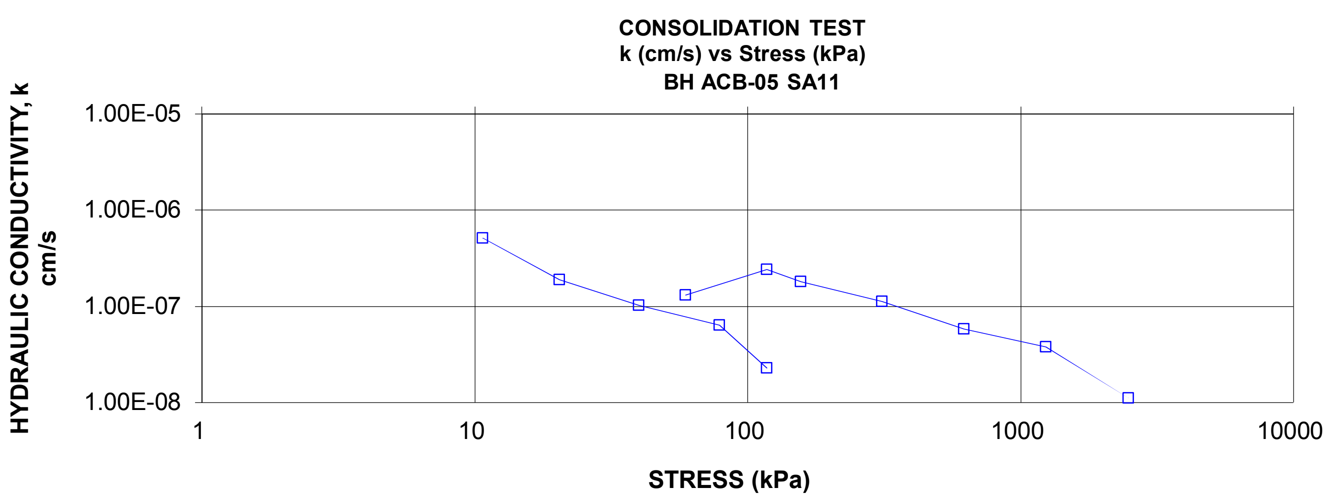
Prepared By: LH **Golder Associates Ltd.** Checked By: TZ

CONSOLIDATION TEST SUMMARY

FIGURE C9B







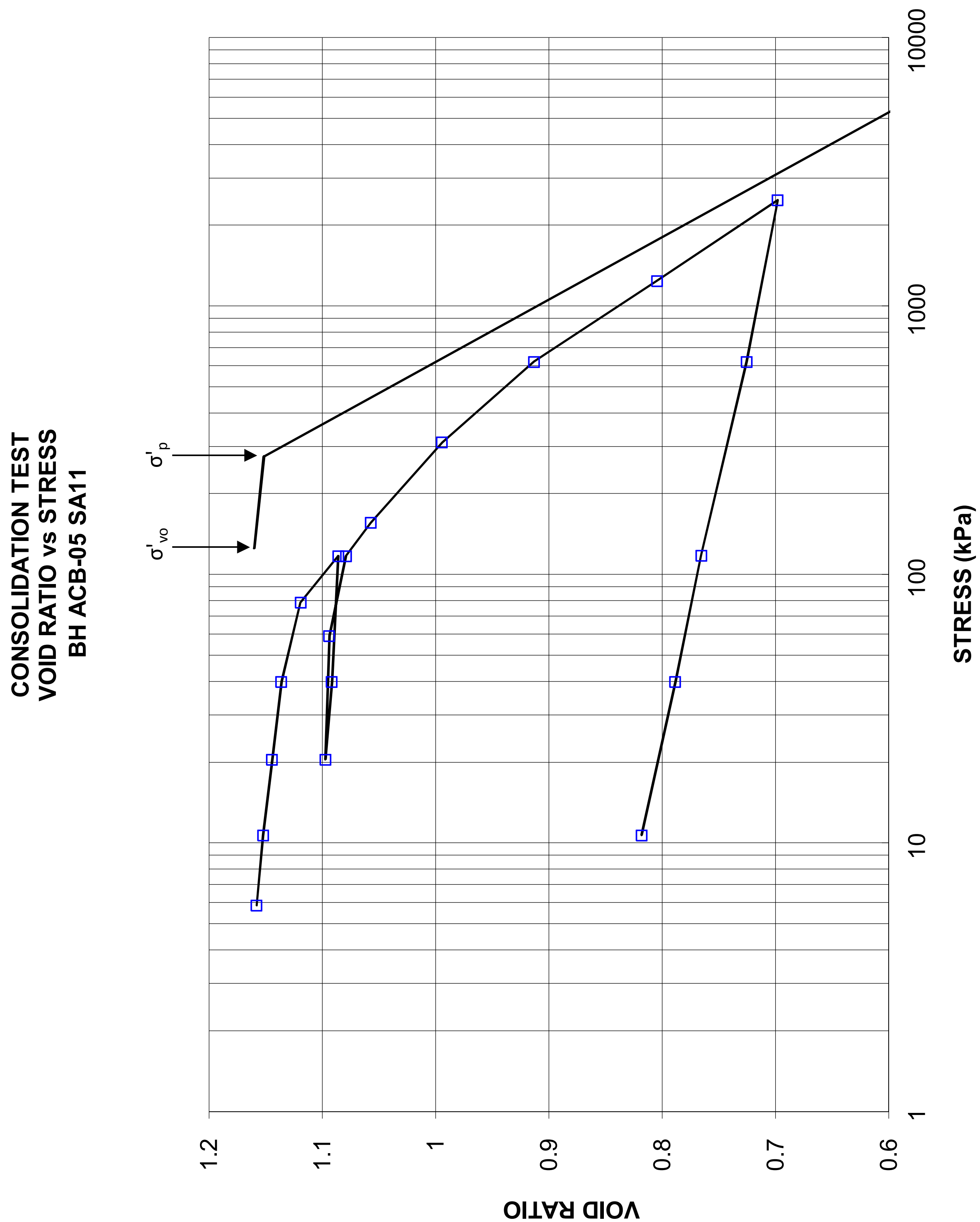
Project No. 1670846

Prepared By: LH

Golder Associates Ltd.

Checked By: TZ

CONSOLIDATION TEST VOID RATIO VS LOG STRESS	FIGURE C9C



Project No. 1670846

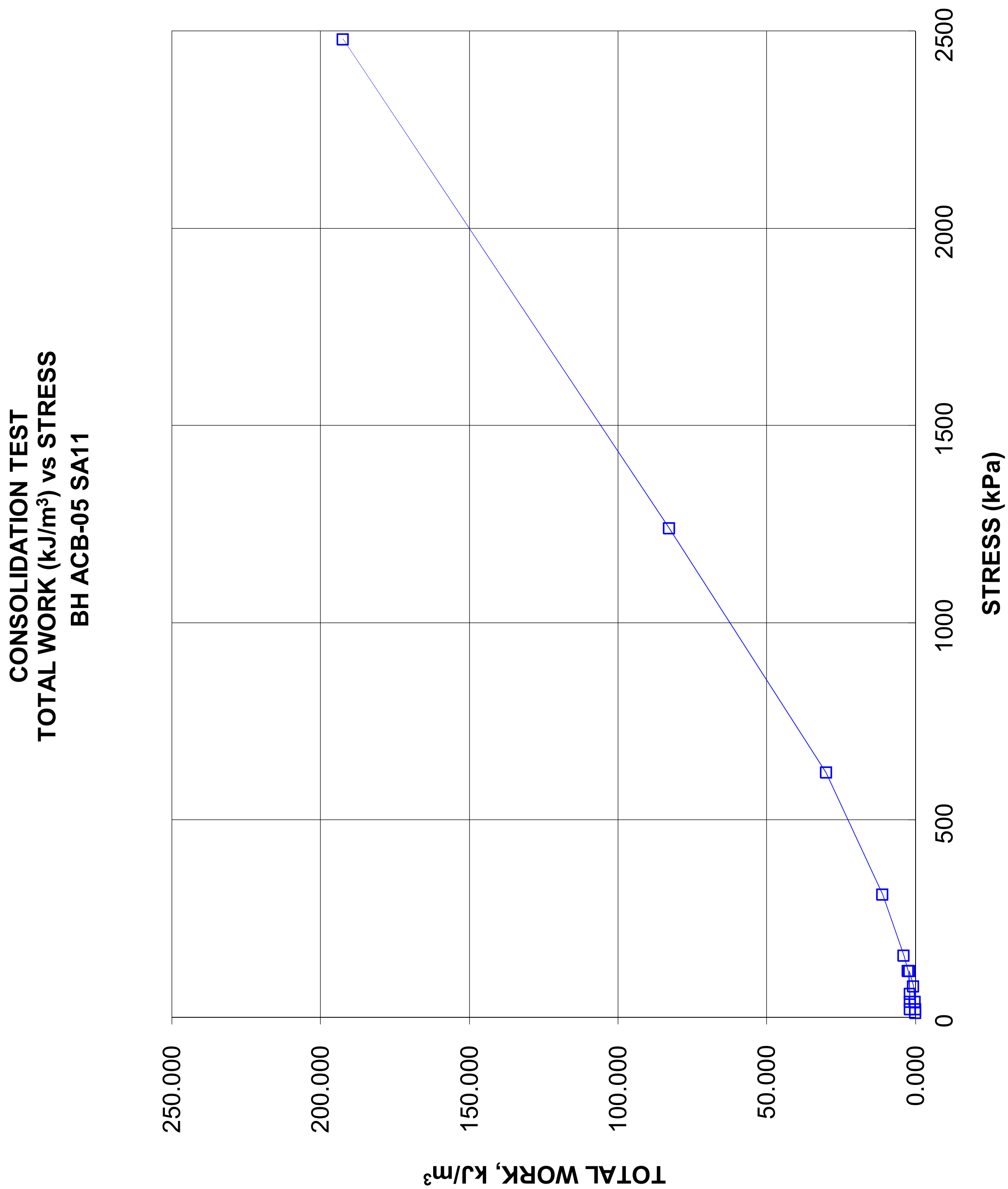
Prepared By: LH

Golder Associates Ltd.

Checked By: TZ

CONSOLIDATION TEST
TOTAL WORK VS STRESS

FIGURE C9D



Project No.1670746

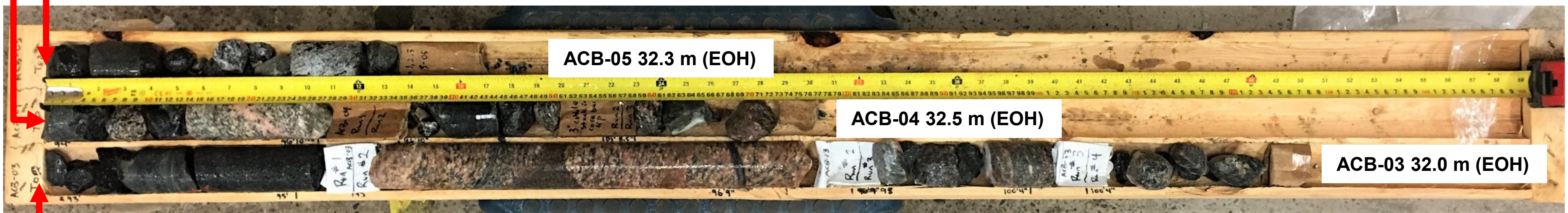
Prepared By: LH

Golder Associates Ltd.

Checked By: TZ

Borehole ACB-04: Cobbles and boulders cored between 28.7 m and 32.5 m

Borehole ACB-05: Cobbles and boulders cored between 28.7 m and 32.3 m



Borehole ACB-03: Cobbles and boulders cored between 28.3 m and 32.0 m

Borehole ACB-06: Cobbles and boulders cored between 29.3 m and 30.5 m

Borehole ACB-02: Cobbles and boulders cored between 28.2 m and 30.5 m

Scale: 0 m — 0.40 m; 0 ft — 1.0 ft — 2.0 ft

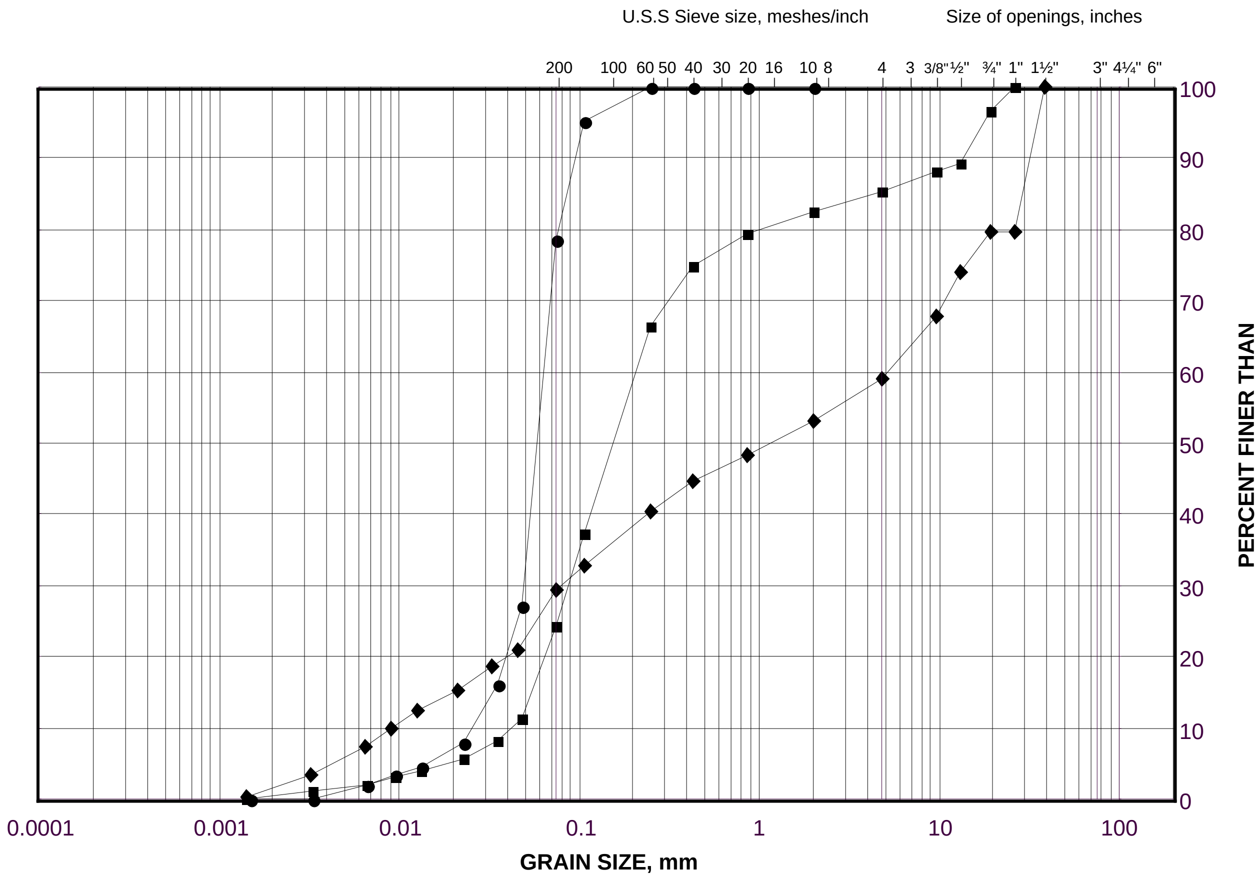
NOTE:
'EOH' represents End of Borehole.

PROJECT	**Highway 532 – Achigan Creek Bridge Replacement, 5.1 km North of Highway 556 (Site No. 38S-041) Gaudette and Hodgins Townships, Algoma District, Ontario**				
TITLE	**COBBLES AND BOULDERS CORE PHOTOGRAPHS BOREHOLES ACB-02 TO ACB-06**				
Golder Associates	PROJECT No. 1670846			FILE No. ----	
	DESIGN	AK	20180422	SCALE NTS	VER. 1.
	CADD	--		**FIGURE C10**	
	CHECK	ACK	20180516		
	REVIEW	TZ	20180622		

REVISION DATE: June 11, 2018 BY: AK Project: 1670846

GRAIN SIZE DISTRIBUTION
Sandy Silt to Silty Sand to Silty Sand and Gravel
(Lower Granular Deposit)

FIGURE C11



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	**SAND SIZE**			**GRAVEL SIZE**		**SIZE**

LEGEND

SYMBOL	Borehole	SAMPLE	ELEVATION(m)
●	ACB-06	19	208.0
■	ACB-02	19	208.3
◆	ACB-07	21	207.7

Project Number: 1670846
Checked By: TZ

Golder Associates

Date: 11-Jun-18

APPENDIX D

Analytical Laboratory Test Results

Maxxam
A Bureau Veritas Group Company

Your Project #: 1670846
Your C.O.C. #: 628368-01-01

Attention:Darcy Hansen
Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/09/20
Report #: R4722990
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7J9789
Received: 2017/09/13, 11:39

Sample Matrix: Soil
Samples Received: 8

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	8	N/A	2017/09/18	CAM SOP-00463	EPA 325.2 m
Conductivity	8	N/A	2017/09/18	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	8	2017/09/15	2017/09/15	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	8	2017/09/14	2017/09/18	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	8	N/A	2017/09/18	CAM SOP-00464	EPA 375.4 m
Sulphide (from Campobello) (1)	8	N/A	N/A		

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) This test was performed by Campo to Burnaby Subcontract

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam
A Bureau Veritas Group Company

Your Project #: 1670846
Your C.O.C. #: 628368-01-01

Attention:Darcy Hansen

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/09/20
Report #: R4722990
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7J9789
Received: 2017/09/13, 11:39

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Job #: B7J9789
Report Date: 2017/09/20

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

RESULTS OF ANALYSES OF SOIL

Maxxam ID		FCS510	FCS510	FCS511	FCS512	FCS513	FCS514		
Sampling Date		2017/08/23	2017/08/23	2017/09/07	2017/09/06	2017/07/16	2017/07/11		
COC Number		628368-01-01	628368-01-01	628368-01-01	628368-01-01	628368-01-01	628368-01-01		
	UNITS	**ACB-03 SA4**	**ACB-03 SA4 Lab-Dup**	**ACC1-03 SA2**	**ACCS-03 SA2**	**MRB-04 SA3**	**MRB-03 SA5**	**RDL**	**QC Batch**
Calculated Parameters									
Resistivity	ohm-cm	7300		15000	4100	5900	2400		5165355
Inorganics									
Soluble (20:1) Chloride (Cl)	ug/g	55	58	24	130	58	260	20	5167700
Conductivity	umho/cm	137	133	69	246	169	424	2	5167946
Available (CaCl2) pH	pH	6.48		6.20	5.13	5.62	5.77		5165977
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	64	22	29	<20	20	5167702

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

Maxxam ID		FCS515	FCS516	FCS517		
Sampling Date		2017/08/23	2017/07/29	2017/08/02		
COC Number		628368-01-01	628368-01-01	628368-01-01		
	UNITS	**DCC-01 SA2**	**MCC-03 SA1**	**WRC-01 SA3**	**RDL**	**QC Batch**
Calculated Parameters						
Resistivity	ohm-cm	2200	24000	43000		5165355
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	190	<20	<20	20	5167700
Conductivity	umho/cm	450	41	23	2	5167946
Available (CaCl2) pH	pH	8.18	6.90	6.62		5165977
Soluble (20:1) Sulphate (SO4)	ug/g	<20	<20	24	20	5167702

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch

Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B7J9789
Report Date: 2017/09/20

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

TEST SUMMARY

Maxxam ID: FCS510
Sample ID: ACB-03 SA4
Matrix: Soil

Collected: 2017/08/23
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

Maxxam ID: FCS510 Dup
Sample ID: ACB-03 SA4
Matrix: Soil

Collected: 2017/08/23
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine

Maxxam ID: FCS511
Sample ID: ACC1-03 SA2
Matrix: Soil

Collected: 2017/09/07
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

Maxxam ID: FCS512
Sample ID: ACCS-03 SA2
Matrix: Soil

Collected: 2017/09/06
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

Maxxam ID: FCS513
Sample ID: MRB-04 SA3
Matrix: Soil

Collected: 2017/07/16
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine

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Maxxam Job #: B7J9789
Report Date: 2017/09/20

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

TEST SUMMARY

Maxxam ID: FCS513
Sample ID: MRB-04 SA3
Matrix: Soil

Collected: 2017/07/16
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

Maxxam ID: FCS514
Sample ID: MRB-03 SA5
Matrix: Soil

Collected: 2017/07/11
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

Maxxam ID: FCS515
Sample ID: DCC-01 SA2
Matrix: Soil

Collected: 2017/08/23
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

Maxxam ID: FCS516
Sample ID: MCC-03 SA1
Matrix: Soil

Collected: 2017/07/29
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

Maxxam Job #: B7J9789
Report Date: 2017/09/20

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

TEST SUMMARY

Maxxam ID: FCS517
Sample ID: WRC-01 SA3
Matrix: Soil

Collected: 2017/08/02
Shipped:
Received: 2017/09/13

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5167700	N/A	2017/09/18	Deonarine Ramnarine
Conductivity	AT	5167946	N/A	2017/09/18	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5165977	2017/09/15	2017/09/15	Tahir Ahmed
Resistivity of Soil		5165355	2017/09/18	2017/09/18	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5167702	N/A	2017/09/18	Deonarine Ramnarine
Sulphide (from Campobello)	SPEC	5170216	N/A	2017/09/19	Lims Auto Schedule Runner

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GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	5.7°C

Custody seal was present and intact.

Sample FCS513 [MRB-04 SA3] : Sample submitted and analyzed past the recommended hold time for pH, Chloride, Sulphate and Conductivity/Resistivity analysis.

Sample FCS514 [MRB-03 SA5] : Sample submitted and analyzed past the recommended hold time for pH, Chloride, Sulphate and Conductivity/Resistivity analysis.

Sample FCS517 [WRC-01 SA3] : Sample submitted and analyzed past the recommended hold time for pH, Chloride, Sulphate and Conductivity/Resistivity analysis.

Results relate only to the items tested.

Maxxam
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Maxxam Job #: B7J9789
Report Date: 2017/09/20

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5165977	Available (CaCl2) pH	2017/09/15			99	97 - 103			0.11	N/A
5167700	Soluble (20:1) Chloride (Cl)	2017/09/18	NC	70 - 130	104	70 - 130	<20	ug/g	5.5	35
5167702	Soluble (20:1) Sulphate (SO4)	2017/09/18	124	70 - 130	107	70 - 130	<20	ug/g	NC	35
5167946	Conductivity	2017/09/18			101	90 - 110	<2	umho/cm	3.2	10

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

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VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Cristina Carriere

Cristina Carriere, Scientific Service Specialist

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

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13-Sep-17 11:39
Ema Gitej
B7J9789
KES ENV-689

Page of

Bottle Order #: 628368
Project Manager: Ema Gitej
C#628368-01-01

INVOICE TO:		REPORT TO:		PROJECT INFORMATION:	
Company Name:	#1326 Golder Associates Ltd	Company Name:		Quotation #:	B70916
Attention:	Accounts Payable	Attention:	Darcy Hansen	P.O. #:	
Address:	6925 Century Ave Suite 100	Address:		Project:	~~1671122~~ 1670846
	Mississauga ON L5N 7K2			Project Name:	
Tel:	(905) 567-4444 x Fax: (905) 567-6561 x	Tel:	(905) 567-4444 x2064 Fax:	Site #:	
Email:	AP_CustomerService@golder.com	Email:	Darcy_Hansen@golder.com	Sampled By:	

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153 (2011): ☐ Table 1 ☐ Res/Park ☐ Medium/Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/Other ☐ For RSC ☐ Table ___

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw ☐ Reg 558 ☐ Storm Sewer Bylaw ☐ MISA Municipality ___ ☐ PWQO ☐ Other ___

Special Instructions

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☐
(will be applied if Rush TAT is not specified)
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

Job Specific Rush TAT (if applies to entire submission)
Date Required: ___ Time Required: ___ ☐
Rush Confirmation Number: ___ *(call lab for #)*

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Chloride & SO4 (20:1 extract)	Conductivity/Resistivity	pH CaCl2 EXTRACT	Sulphide (Maxxam BC)							# of Bottles	Comments
1		ACB-03 SA4	Aug 23, 2017	—			1	1	1	1							2	
2		ACC1-03 SA2	sept 7, 2017	—			1	1	1	1							2	
3		ACC2-03 SA2	sept 6, 2017	—			1	1	1	1							2	
4		MRB-04 SA3	Jul 16, 2017	—			1	1	1	1							2	
5		MRB-03 SA5	Jul 11, 2017	—			1	1	1	1							2	
6		PCC-01 SA2	Aug 23, 2017	—			1	1	1	1							2	
7		MCC-03 SA1	Jul 29 2017	—			1	1	1	1							2	
8		WRC-01 SA3	Aug 2, 2017	—			1	1	1	1							2	
9																		
10																		

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
/ Darcy Hansen	17/09/13	10 am	/ USA MUNSTER	2017/09/13	11:39	

Laboratory Use Only

Time Sensitive	Temperature (°C) on Recei	Custody Seal	Yes	No
	5 / 5 / 7	Present	✓	
		Intact	✓	

White: Maxxam Yellow: Client

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO MAXXAM'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.MAXXAM.CA/TERMS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT HTTP://MAXXAM.CA/WP-CONTENT/UPLOADS/ONTARIO-COC.PDF.

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

Maxxam Analytics International Corporation o/a Maxxam Analytics

Your Project #: 1670846
Your C.O.C. #: 628368-02-01

Attention:Darcy Hansen

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/10/23
Report #: R4798069
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7L2287
Received: 2017/09/27, 12:13

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Reference
Chloride (20:1 extract)	2	N/A	2017/10/03	CAM SOP-00463	EPA 325.2 m
Conductivity	2	N/A	2017/10/02	CAM SOP-00414	OMOE E3530 v1 m
pH CaCl2 EXTRACT	2	2017/09/29	2017/09/29	CAM SOP-00413	EPA 9045 D m
Resistivity of Soil	2	2017/09/27	2017/10/02	CAM SOP-00414	SM 22 2510 m
Sulphate (20:1 Extract)	2	N/A	2017/10/03	CAM SOP-00464	EPA 375.4 m
Sulphide (from Campobello) (1)	2	N/A	N/A		

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.
(1) This test was performed by Campo to Burnaby Subcontract

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Your Project #: 1670846
Your C.O.C. #: 628368-02-01

Attention:Darcy Hansen

Golder Associates Ltd
Mississauga - Standing Offer
6925 Century Ave
Suite 100
Mississauga, ON
CANADA L5N 7K2

Report Date: 2017/10/23
Report #: R4798069
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B7L2287
Received: 2017/09/27, 12:13

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Ema Gitej, Senior Project Manager
Email: EGitej@maxxam.ca
Phone# (905)817-5829
==
Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

Maxxam Job #: B7L2287
Report Date: 2017/10/23

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

RESULTS OF ANALYSES OF SOIL

Maxxam ID		FFD202	FFD203	FFD203		
Sampling Date		2017/08/26	2017/09/09	2017/09/09		
COC Number		628368-02-01	628368-02-01	628368-02-01		
	UNITS	DCC-04 SA-2	ACB-06 SA-3	ACB-06 SA-3 Lab-Dup	RDL	QC Batch
Calculated Parameters						
Resistivity	ohm-cm	5100	7200			5185712
Inorganics						
Soluble (20:1) Chloride (Cl)	ug/g	<20	70	69	20	5191890
Conductivity	umho/cm	198	139	131	2	5191368
Available (CaCl2) pH	pH	8.03	4.97			5188854
Soluble (20:1) Sulphate (SO4)	ug/g	39	<20	<20	20	5191917

RDL = Reportable Detection Limit
QC Batch = Quality Control Batch
Lab-Dup = Laboratory Initiated Duplicate

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Maxxam
A Bureau Veritas Group Company

Maxxam Job #: B7L2287
Report Date: 2017/10/23

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

TEST SUMMARY

Maxxam ID: FFD202
Sample ID: DCC-04 SA-2
Matrix: Soil

Collected: 2017/08/26
Shipped:
Received: 2017/09/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5191890	N/A	2017/10/03	Alina Dobreanu
Conductivity	AT	5191368	N/A	2017/10/02	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5188854	2017/09/29	2017/09/29	Tahir Anwar
Resistivity of Soil		5185712	2017/10/02	2017/10/02	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5191917	N/A	2017/10/03	Alina Dobreanu
Sulphide (from Campobello)	SPEC	5223606	N/A		Ema Gitej

Maxxam ID: FFD203
Sample ID: ACB-06 SA-3
Matrix: Soil

Collected: 2017/09/09
Shipped:
Received: 2017/09/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5191890	N/A	2017/10/03	Alina Dobreanu
Conductivity	AT	5191368	N/A	2017/10/02	Neil Dassanayake
pH CaCl2 EXTRACT	AT	5188854	2017/09/29	2017/09/29	Tahir Anwar
Resistivity of Soil		5185712	2017/10/02	2017/10/02	Automated Statchk
Sulphate (20:1 Extract)	KONE/EC	5191917	N/A	2017/10/03	Alina Dobreanu
Sulphide (from Campobello)	SPEC	5223606	N/A		Ema Gitej

Maxxam ID: FFD203 Dup
Sample ID: ACB-06 SA-3
Matrix: Soil

Collected: 2017/09/09
Shipped:
Received: 2017/09/27

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Chloride (20:1 extract)	KONE/EC	5191890	N/A	2017/10/03	Alina Dobreanu
Conductivity	AT	5191368	N/A	2017/10/02	Neil Dassanayake
Sulphate (20:1 Extract)	KONE/EC	5191917	N/A	2017/10/03	Alina Dobreanu

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GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	1.7°C

Results relate only to the items tested.

Maxxam Job #: B7L2287
Report Date: 2017/10/23

QUALITY ASSURANCE REPORT

Golder Associates Ltd
Client Project #: 1670846
Sampler Initials: DH

			Matrix Spike		SPIKED BLANK		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
5188854	Available (CaCl2) pH	2017/09/29			100	97 - 103			0.80	N/A
5191368	Conductivity	2017/10/02			98	90 - 110	<2	umho/cm	5.7	10
5191890	Soluble (20:1) Chloride (Cl)	2017/10/03	NC	70 - 130	108	70 - 130	<20	ug/g	0.87	35
5191917	Soluble (20:1) Sulphate (SO4)	2017/10/03	102	70 - 130	104	70 - 130	<20	ug/g	NC	35

N/A = Not Applicable

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Matrix Spike): The recovery in the matrix spike was not calculated. The relative difference between the concentration in the parent sample and the spike amount was too small to permit a reliable recovery calculation (matrix spike concentration was less than the native sample concentration)

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

Maxxam Analytics International Corporation o/a Maxxam Analytics 6740 Campobello Road, Mississauga, Ontario, L5N 2L8 Tel: (905) 817-5700 Toll-Free: 800-563-6266 Fax: (905) 817-5777 www.maxxam.ca

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Ema Gitej, Senior Project Manager

Ewa Pranjic, M.Sc., C.Chem, Scientific Specialist

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

IMMEDIATE TEST

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6740 Campobello Road, Mississauga, Ontario Canada L5N 2L8 Tel:(905) 817-5700 Toll-free:800-563-6266 Fax:(905) 817-5777 www.maxxam.ca

CHAIN OF CUSTODY RECORD

Page 1 of 1

INVOICE TO:	
Company Name:	#1326 Golder Associates Ltd
Attention:	Accounts Payable
Address:	6925 Century Ave Suite 100
	Mississauga ON L5N 7K2
Tel:	(905) 567-4444 x Fax: (905) 567-6561 x
Email:	AP_CustomerService@golder.com

REPORT TO:	
Company Name:	
Attention:	Darcy Hansen
Address:	
Tel:	(905) 567-4444 x2064 Fax:
Email:	Darcy_Hansen@golder.com

PROJECT INFORMATION:	
Quotation #:	~~[illegible]~~
P.O. #:	
Project:	~~[illegible]~~ 1670846
Project Name:	
Site #:	
Sampled By:	

Laboratory Use Only:	
Maxxam Job #:	Bottle Order #: 628368
COC #: C#628368-02-01	Project Manager: Ema Gitej

MOE REGULATED DRINKING WATER OR WATER INTENDED FOR HUMAN CONSUMPTION MUST BE SUBMITTED ON THE MAXXAM DRINKING WATER CHAIN OF CUSTODY

Regulation 153 (2011): ☐ Table 1 ☐ Res/Park ☐ Medium/Fine ☐ Table 2 ☐ Ind/Comm ☐ Coarse ☐ Table 3 ☐ Agri/Other ☐ For RSC ☐ Table ___

Other Regulations: ☐ CCME ☐ Sanitary Sewer Bylaw ☐ Reg 558 ☐ Storm Sewer Bylaw ☐ MISA Municipality ___ ☐ PWQO ☐ Other ___

Special Instructions:

Include Criteria on Certificate of Analysis (Y/N)? ___

ANALYSIS REQUESTED (PLEASE BE SPECIFIC)

Turnaround Time (TAT) Required:
Please provide advance notice for rush projects

Regular (Standard) TAT: ☑
(will be applied if Rush TAT is not specified)
Standard TAT = 5-7 Working days for most tests.
Please note: Standard TAT for certain tests such as BOD and Dioxins/Furans are > 5 days - contact your Project Manager for details.

Job Specific Rush TAT (if applies to entire submission)
Date Required: ___ Time Required: ___
Rush Confirmation Number: ___ *(call lab for #)*

	Sample Barcode Label	Sample (Location) Identification	Date Sampled	Time Sampled	Matrix	Field Filtered (please circle): Metals / Hg / Cr VI	Chloride & SO4 (20:1 extract)	Conductivity/Resistivity	pH CaCl2 EXTRACT	Sulphide (Maxxam BC)						# of Bottles	Comments
1	~~[illegible]~~	DCL-04 S.A.2	26/08/17	—	SOIL		1	1	1	1						2	
2		ACB-06 S.A.3	09/09/17	—	SOIL		1	1	1	1						2	
3																	
4																	
5																	
6																	
7																	
8																	
9																	
10																	

27-Sep-17 12:13
Ema Gitej
B7L2287
SEL ENV-1117

* RELINQUISHED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	RECEIVED BY: (Signature/Print)	Date: (YY/MM/DD)	Time	# jars used and not submitted
/DARCY HANSEN	17/09/27	9:15	GURPR[illegible]	20H/09/27	12:13	

Laboratory Use Only: Time Sensitive | Temperature (°C) on Recei 2, 1, 2 | Custody Seal Present Intact: Yes / No

* UNLESS OTHERWISE AGREED TO IN WRITING, WORK SUBMITTED ON THIS CHAIN OF CUSTODY IS SUBJECT TO MAXXAM'S STANDARD TERMS AND CONDITIONS. SIGNING OF THIS CHAIN OF CUSTODY DOCUMENT IS ACKNOWLEDGMENT AND ACCEPTANCE OF OUR TERMS WHICH ARE AVAILABLE FOR VIEWING AT WWW.MAXXAM.CA/TERMS.

* IT IS THE RESPONSIBILITY OF THE RELINQUISHER TO ENSURE THE ACCURACY OF THE CHAIN OF CUSTODY RECORD. AN INCOMPLETE CHAIN OF CUSTODY MAY RESULT IN ANALYTICAL TAT DELAYS.

** SAMPLE CONTAINER, PRESERVATION, HOLD TIME AND PACKAGE INFORMATION CAN BE VIEWED AT HTTP://MAXXAM.CA/WP-CONTENT/UPLOADS/ONTARIO-COC.PDF.

SAMPLES MUST BE KEPT COOL (< 10° C) FROM TIME OF SAMPLING UNTIL DELIVERY TO MAXXAM

White: Maxxa Yellow: Client

Maxxam Analytics International Corporation o/a Maxxam Analytics

Your Project #: MB7J9789
Site Location: 1670846
Your C.O.C. #: B7J9789-M058-01-01

Attention:EMA GITEJ

MAXXAM ANALYTICS
CAMPOBELLO
6740 CAMPOBELLO ROAD
MISSISSAUGA, ON
CANADA L5N 2L8

Report Date: 2017/09/18
Report #: R2445858
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B780085
Received: 2017/09/16, 12:10

Sample Matrix: Soil
Samples Received: 8

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Moisture	8	2017/09/18	2017/09/18	BBY8SOP-00017	BCMOE BCLM Dec2000 m
Sulphide in Soil	8	2017/09/18	2017/09/18	BBY6SOP-00006	SM 22 4500 S2- D m

Remarks:

Maxxam Analytics' laboratories are accredited to ISO/IEC 17025:2005 for specific parameters on scopes of accreditation. Unless otherwise noted, procedures used by Maxxam are based upon recognized Provincial, Federal or US method compendia such as CCME, MDDELCC, EPA, APHA.

All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

Maxxam Analytics' liability is limited to the actual cost of the requested analyses, unless otherwise agreed in writing. There is no other warranty expressed or implied. Maxxam has been retained to provide analysis of samples provided by the Client using the testing methodology referenced in this report. Interpretation and use of test results are the sole responsibility of the Client and are not within the scope of services provided by Maxxam, unless otherwise agreed in writing.

Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
This Certificate shall not be reproduced except in full, without the written approval of the laboratory.
Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

Maxxam Analytics International Corporation o/a Maxxam Analytics Burnaby: 4606 Canada Way V5G 1K5 Telephone(604) 734-7276 Fax(604) 731-2386

Your Project #: MB7J9789
Site Location: 1670846
Your C.O.C. #: B7J9789-M058-01-01

Attention:EMA GITEJ

MAXXAM ANALYTICS
CAMPOBELLO
6740 CAMPOBELLO ROAD
MISSISSAUGA, ON
CANADA L5N 2L8

Report Date: 2017/09/18
Report #: R2445858
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B780085
Received: 2017/09/16, 12:10

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Letitia Prefontaine, B.Sc., Senior Project Manager
Email: LPrefontaine@maxxam.ca
Phone# (604)639-2616
==
This report has been generated and distributed using a secure automated process.
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Maxxam Job #: B780085
Report Date: 2017/09/18

MAXXAM ANALYTICS
Client Project #: MB7J9789
Site Location: 1670846
Sampler Initials: DH

RESULTS OF CHEMICAL ANALYSES OF SOIL

Maxxam ID		RZ2662	RZ2662	RZ2663		RZ2664		
Sampling Date		2017/08/23	2017/08/23	2017/09/07		2017/09/06		
COC Number		B7J9789-M058-01-01	B7J9789-M058-01-01	B7J9789-M058-01-01		B7J9789-M058-01-01		
	UNITS	**ACB-03 SA4**	**ACB-03 SA4 Lab-Dup**	**ACC1-03 SA2**	**RDL**	**ACCS-03 SA2**	**RDL**	**QC Batch**
MISCELLANEOUS								
Sulphide	ug/g	0.69 (1)	<0.50	0.52	0.50	1.06 (2)	0.55	8761700

RDL = Reportable Detection Limit
Lab-Dup = Laboratory Initiated Duplicate
(1) Matrix spike exceeds acceptance limits due to matrix interference. Re-analysis yields similar results.
(2) RDL raised due to high sample moisture content.

Maxxam ID		RZ2665	RZ2666		RZ2667		
Sampling Date		2017/07/16	2017/07/11		2017/08/23		
COC Number		B7J9789-M058-01-01	B7J9789-M058-01-01		B7J9789-M058-01-01		
	UNITS	**MRB-04 SA3**	**MRB-03 SA5**	**RDL**	**DCC-01 SA2**	**RDL**	**QC Batch**
MISCELLANEOUS							
Sulphide	ug/g	<0.50	0.52	0.50	0.68 (1)	0.55	8761700

RDL = Reportable Detection Limit
(1) RDL raised due to high sample moisture content.

Maxxam ID		RZ2668	RZ2669		
Sampling Date		2017/07/29	2017/08/02		
COC Number		B7J9789-M058-01-01	B7J9789-M058-01-01		
	UNITS	**MCC-03 SA1**	**WRC-01 SA3**	**RDL**	**QC Batch**
MISCELLANEOUS					
Sulphide	ug/g	0.78	0.57	0.50	8761700

RDL = Reportable Detection Limit

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Maxxam Job #: B780085
Report Date: 2017/09/18

MAXXAM ANALYTICS
Client Project #: MB7J9789
Site Location: 1670846
Sampler Initials: DH

PHYSICAL TESTING (SOIL)

Maxxam ID		RZ2662	RZ2663	RZ2664	RZ2665		
Sampling Date		2017/08/23	2017/09/07	2017/09/06	2017/07/16		
COC Number		B7J9789-M058-01-01	B7J9789-M058-01-01	B7J9789-M058-01-01	B7J9789-M058-01-01		
	UNITS	**ACB-03 SA4**	**ACC1-03 SA2**	**ACCS-03 SA2**	**MRB-04 SA3**	**RDL**	**QC Batch**
Physical Properties							
Moisture	%	24	22	28	8.2	0.30	8761682
RDL = Reportable Detection Limit							

Maxxam ID		RZ2666	RZ2667	RZ2668	RZ2669		
Sampling Date		2017/07/11	2017/08/23	2017/07/29	2017/08/02		
COC Number		B7J9789-M058-01-01	B7J9789-M058-01-01	B7J9789-M058-01-01	B7J9789-M058-01-01		
	UNITS	**MRB-03 SA5**	**DCC-01 SA2**	**MCC-03 SA1**	**WRC-01 SA3**	**RDL**	**QC Batch**
Physical Properties							
Moisture	%	13	32	14	17	0.30	8761682
RDL = Reportable Detection Limit							

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TEST SUMMARY

Maxxam ID: RZ2662
Sample ID: ACB-03 SA4
Matrix: Soil

Collected: 2017/08/23
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

Maxxam ID: RZ2662 Dup
Sample ID: ACB-03 SA4
Matrix: Soil

Collected: 2017/08/23
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

Maxxam ID: RZ2663
Sample ID: ACC1-03 SA2
Matrix: Soil

Collected: 2017/09/07
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

Maxxam ID: RZ2664
Sample ID: ACCS-03 SA2
Matrix: Soil

Collected: 2017/09/06
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

Maxxam ID: RZ2665
Sample ID: MRB-04 SA3
Matrix: Soil

Collected: 2017/07/16
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

Maxxam ID: RZ2666
Sample ID: MRB-03 SA5
Matrix: Soil

Collected: 2017/07/11
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

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Maxxam Job #: B780085
Report Date: 2017/09/18

MAXXAM ANALYTICS
Client Project #: MB7J9789
Site Location: 1670846
Sampler Initials: DH

TEST SUMMARY

Maxxam ID: RZ2667
Sample ID: DCC-01 SA2
Matrix: Soil

Collected: 2017/08/23
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

Maxxam ID: RZ2668
Sample ID: MCC-03 SA1
Matrix: Soil

Collected: 2017/07/29
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

Maxxam ID: RZ2669
Sample ID: WRC-01 SA3
Matrix: Soil

Collected: 2017/08/02
Shipped:
Received: 2017/09/16

Test Description	Instrumentation	Batch	Extracted	Date Analyzed	Analyst
Moisture	BAL/BAL	8761682	2017/09/18	2017/09/18	Lolita Obusan
Sulphide in Soil	SPEC/COL	8761700	2017/09/18	2017/09/18	Prabhleen Sodhi

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GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	9.0°C
Package 2	6.0°C

Sample RZ2662 [ACB-03 SA4] : Sample was extracted past method specified hold time for Moisture. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Sample received past method specified hold time for Sulphide in Soil.

Sample RZ2663 [ACC1-03 SA2] : Sample analyzed past method specified hold time for Sulphide in Soil. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Sulphide in Soil.

Sample RZ2664 [ACCS-03 SA2] : Sample analyzed past method specified hold time for Sulphide in Soil. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Sulphide in Soil.

Sample RZ2665 [MRB-04 SA3] : Sample was extracted past method specified hold time for Moisture. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Sample received past method specified hold time for Sulphide in Soil.

Sample RZ2666 [MRB-03 SA5] : Sample was extracted past method specified hold time for Moisture. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Sample received past method specified hold time for Sulphide in Soil.

Sample RZ2667 [DCC-01 SA2] : Sample was extracted past method specified hold time for Moisture. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Sample received past method specified hold time for Sulphide in Soil.

Sample RZ2668 [MCC-03 SA1] : Sample was extracted past method specified hold time for Moisture. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Sample received past method specified hold time for Sulphide in Soil

Results relate only to the items tested.

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Maxxam Job #: B780085
Report Date: 2017/09/18

QUALITY ASSURANCE REPORT

MAXXAM ANALYTICS
Client Project #: MB7J9789
Site Location: 1670846
Sampler Initials: DH

			Matrix Spike		Spiked Blank		Method Blank		RPD	
QC Batch	Parameter	Date	% Recovery	QC Limits	% Recovery	QC Limits	Value	UNITS	Value (%)	QC Limits
8761682	Moisture	2017/09/18					<0.30	%	0 (1)	20
8761700	Sulphide	2017/09/18	39 (2,3)	75 - 125	84	75 - 125	<0.50	ug/g	NC (4)	30

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

NC (Duplicate RPD): The duplicate RPD was not calculated. The concentration in the sample and/or duplicate was too low to permit a reliable RPD calculation (absolute difference <= 2x RDL).

(1) Duplicate Parent ID

(2) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.

(3) Matrix Spike Parent ID [RZ2662-01]

(4) Duplicate Parent ID [RZ2662-01]

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Maxxam Job #: B780085
Report Date: 2017/09/18

MAXXAM ANALYTICS
Client Project #: MB7J9789
Site Location: 1670846
Sampler Initials: DH

VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Andy Lu, Ph.D., P.Chem., Scientific Specialist

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Your Project #: MB7L2287
Site Location: 1670846
Your C.O.C. #: B7L2287-M058-01-01

Attention:SUBCONTRACTOR

MAXXAM ANALYTICS
OTTAWA
32 COLONNADE RD N
UNIT 1000
NEPEAN, ON
CANADA K2E7J6

Report Date: 2017/10/04
Report #: R2454826
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B785668
Received: 2017/10/02, 08:55

Sample Matrix: Soil
Samples Received: 2

Analyses	Quantity	Date Extracted	Date Analyzed	Laboratory Method	Analytical Method
Moisture	2	2017/10/03	2017/10/03	BBY8SOP-00017	BCMOE BCLM Dec2000 m
Sulphide in Soil	2	2017/10/02	2017/10/04	BBY6SOP-00006	SM 22 4500 S2- D m

Remarks:

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All work recorded herein has been done in accordance with procedures and practices ordinarily exercised by professionals in Maxxam's profession using accepted testing methodologies, quality assurance and quality control procedures (except where otherwise agreed by the client and Maxxam in writing). All data is in statistical control and has met quality control and method performance criteria unless otherwise noted. All method blanks are reported: unless indicated otherwise, associated sample data are not blank corrected.

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Solid sample results, except biota, are based on dry weight unless otherwise indicated. Organic analyses are not recovery corrected except for isotope dilution methods.
Results relate to samples tested.
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Reference Method suffix "m" indicates test methods incorporate validated modifications from specific reference methods to improve performance.
* RPDs calculated using raw data. The rounding of final results may result in the apparent difference.

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Your Project #: MB7L2287
Site Location: 1670846
Your C.O.C. #: B7L2287-M058-01-01

Attention:SUBCONTRACTOR
MAXXAM ANALYTICS
OTTAWA
32 COLONNADE RD N
UNIT 1000
NEPEAN, ON
CANADA K2E7J6

Report Date: 2017/10/04
Report #: R2454826
Version: 1 - Final

CERTIFICATE OF ANALYSIS

MAXXAM JOB #: B785668
Received: 2017/10/02, 08:55

Encryption Key

Please direct all questions regarding this Certificate of Analysis to your Project Manager.
Letitia Prefontaine, B.Sc., Senior Project Manager
Email: LPrefontaine@maxxam.ca
Phone# (604)639-2616
==
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Maxxam Job #: B785668
Report Date: 2017/10/04

MAXXAM ANALYTICS
Client Project #: MB7L2287
Site Location: 1670846

RESULTS OF CHEMICAL ANALYSES OF SOIL

Maxxam ID		SC4339		SC4340	SC4340		
Sampling Date		2017/08/26		2017/09/09	2017/09/09		
COC Number		B7L2287-M058-01-01		B7L2287-M058-01-01	B7L2287-M058-01-01		
	UNITS	**DCC-04 SA-2**	**RDL**	**ACB-06 SA-3**	**ACB-06 SA-3 Lab-Dup**	**RDL**	**QC Batch**
MISCELLANEOUS							
Sulphide	ug/g	0.92	0.55	0.60	0.50	0.50	8779137

RDL = Reportable Detection Limit
Lab-Dup = Laboratory Initiated Duplicate

Maxxam Analytics International Corporation o/a Maxxam Analytics Burnaby: 4606 Canada Way V5G 1K5 Telephone(604) 734-7276 Fax(604) 731-2386

Maxxam Job #: B785668
Report Date: 2017/10/04

MAXXAM ANALYTICS
Client Project #: MB7L2287
Site Location: 1670846

PHYSICAL TESTING (SOIL)

Maxxam ID		SC4339	SC4340	SC4340		
Sampling Date		2017/08/26	2017/09/09	2017/09/09		
COC Number		B7L2287-M058-01-01	B7L2287-M058-01-01	B7L2287-M058-01-01		
	UNITS	**DCC-04 SA-2**	**ACB-06 SA-3**	**ACB-06 SA-3 Lab-Dup**	**RDL**	**QC Batch**
Physical Properties						
Moisture	%	29	18	17	0.30	8779668

RDL = Reportable Detection Limit
Lab-Dup = Laboratory Initiated Duplicate

GENERAL COMMENTS

Each temperature is the average of up to three cooler temperatures taken at receipt

Package 1	7.3°C

Samples received past hold time for sulphide in soil analysis.

Sample SC4339 [DCC-04 SA-2] : Sample was extracted past method specified hold time for Moisture. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Sample received past method specified hold time for Sulphide in Soil. Sample analyzed past method specified hold time for Moisture.

Sample SC4340 [ACB-06 SA-3] : Sample was extracted past method specified hold time for Moisture. {Exceedance of hold time increases the uncertainty of test results but does not necessarily imply that results are compromised.} Sample received past method specified hold time for Moisture. Sample analyzed past method specified hold time for Sulphide in Soil. Sample received past method specified hold time for Sulphide in Soil. Sample analyzed past method specified hold time for Moisture.

Results relate only to the items tested.

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Maxxam Job #: B785668
Report Date: 2017/10/04

MAXXAM ANALYTICS
Client Project #: MB7L2287
Site Location: 1670846

QUALITY ASSURANCE REPORT

QA/QC Batch	Init	QC Type	Parameter	Date Analyzed	Value	Recovery	UNITS	QC Limits
8779137	KAB	Matrix Spike [SC4340-01]	Sulphide	2017/10/04		33 (1)	%	75 - 125
8779137	KAB	Spiked Blank	Sulphide	2017/10/04		114	%	75 - 125
8779137	KAB	Method Blank	Sulphide	2017/10/04	<0.50		ug/g	
8779137	KAB	RPD [SC4340-01]	Sulphide	2017/10/04	17		%	30
8779668	LO1	Method Blank	Moisture	2017/10/03	<0.30		%	
8779668	LO1	RPD [SC4340-01]	Moisture	2017/10/03	5.0		%	20

Duplicate: Paired analysis of a separate portion of the same sample. Used to evaluate the variance in the measurement.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate sample matrix interference.

Spiked Blank: A blank matrix sample to which a known amount of the analyte, usually from a second source, has been added. Used to evaluate method accuracy.

Method Blank: A blank matrix containing all reagents used in the analytical procedure. Used to identify laboratory contamination.

(1) Recovery or RPD for this parameter is outside control limits. The overall quality control for this analysis meets acceptability criteria.

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VALIDATION SIGNATURE PAGE

The analytical data and all QC contained in this report were reviewed and validated by the following individual(s).

Rob Reinert, B.Sc., Scientific Specialist

Maxxam has procedures in place to guard against improper use of the electronic signature and have the required "signatories", as per section 5.10.2 of ISO/IEC 17025:2005(E), signing the reports. For Service Group specific validation please refer to the Validation Signature Page.

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APPENDIX E

Non-Standard Special Provisions

WORKING SLAB – Item No.

Non-Standard Special Provision

Amendment to OPSS.PROV 902, November 2010

Construction Specification for Excavating and Backfilling - Structures

902.07.05.02 Excavation for Foundations

Section 902.07.05.02 of OPSS.PROV 902 shall be amended by the addition of the following after the second paragraph:

The subgrade soils within the footprint of the proposed shallow foundations at the south and north bridge abutments of the Achigan Creek replacement bridge and the temporary modular bridge may be susceptible to disturbance and loosening/softening from construction traffic and ponded water.

If the footings are not placed on the prepared subgrade within four hours of its inspection and approval, a concrete working slab of 20 MPa compressive strength at 28-days with minimum thickness of 100 mm, shall be placed on the foundation subgrade. A minimum 75 mm thick uncompacted levelling pad consisting of Granular 'A' material (OPSS.PROV 1010) or concrete fine aggregate (meeting the grading requirements specified in OPSS.PROV 1002) shall be provided on top of the concrete working slab if a pre-cast concrete footing is constructed at the bridge abutments.

<u>DEEP FOUNDATIONS – Item No.</u>

Non-Standard Special Provision

Amendment to OPSS.PROV 903, April 2016

Deep Foundations

903.07 CONSTRUCTION

Section 903.07.03.02 of OPSS.PROV 903 shall be amended by the addition of the following:

The Contactor shall be alerted to the presence of cobbles and boulders within the lower granular deposit encountered below the extensive cohesive deposit. Consideration of the presence of these obstructions must be made in the selection of appropriate equipment and procedures for driving steel H-piles or tube piles, such that the piles are not damaged and design pile tip levels are achieved.

VIBRATION MONITORING - Item No.

Special Provision

TABLE OF CONTENTS

1.0 SCOPE

This special provision describes requirements for vibration monitoring for the following components of the Contract:

- Deep foundation installation for the Achigan Creek replacement bridge.
- Deep foundation installation, if required, for the temporary modular bridge over Achigan Creek.

2.0 REFERENCES

The subsurface conditions at the site are described in the following Foundation Investigation Report:

1. Foundation Investigation Report; Structural Bundle – 11 Structures om Highways 129, 532 and 556; Highway 532 – Achigan Creek Bridge Replacement, 5.1 km North of Highway 556 (Site No. 38S-041); Lat. 46.789744° ; Long. -84.054775°; Hodgins and Gaudette Townships, Algoma District, Ontario; Ministry of Transportation, Ontario; GWP 5378-11-00 ; WP 151-97-01.

3.0 DEFINITIONS

For the purposes of this specification, the following definitions apply:

Contractor’s Engineer means an Engineer with a minimum of five (5) years of experience in the field of installation of piling and vibration monitoring or, alternatively, with expertise demonstrated by providing satisfactory quality verification services for a minimum of two (2) projects of similar scope to the Contract. The Contractor’s Engineer shall be retained by the Contractor to ensure general conformance with the Contract Documents and issue certificates of conformance.

Peak Particle Velocity (PPV) means the maximum component velocity in millimetres per second (m/sec) that ground particles move as a result of energy released from vibratory construction operations.

Pre-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, prior to the commencement of vibratory or vibration-inducing construction operations.

Post-Construction Condition Survey means a detailed record, accompanied by film or video, as necessary, of the condition of private or public property, after completion of vibratory or vibration-inducing construction operations.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.1 Submission Requirements

The Contractor/Contractor’s Engineer shall submit details of the vibration monitoring plan to the Contract Administrator for information purposes. The submittals shall satisfy the specifications and at a minimum contain the following specific information:

a) Equipment and methods used by the Contractor to perform the work that may cause undue vibration.
b) Qualifications of vibration monitoring specialist.
c) Details regarding proposed instrumentation.
d) Proposed location of instruments adjacent to the on the residences, utilities, wells, or other potentially vibration-sensitive structures within a 250 m radius from the Achigan Creek replacement bridge and the temporary modular bridge.
e) Proposed frequency of readings.
f) Action plan to be taken to adjust deep foundation installation methods or if readings show vibrations exceeding tolerable levels.

6.0 EQUIPMENT

6.1 Vibration Monitoring Equipment

All vibration monitoring equipment shall be capable of measuring and recording ground vibration PPV up to 200 mm/s in the vertical, transverse, and radial directions. The equipment shall have been calibrated within the last 12 months either by the manufacturer or other qualified agent. Proof of calibration shall be submitted to the Contract Administrator prior to commencement of any monitoring operations.

7.0 CONSTRUCTION

7.1 Pre- and Post-Construction Condition Surveys

A Pre-Construction Condition Survey and Post-Construction Condition Survey shall be prepared for all buildings, utilities, structures, water wells, and facilities within a 250 m radius from the Achigan Creek replacement bridge and the temporary modular bridge.

7.1.1 Pre-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

The Pre-Construction Condition Survey, at each structure/well within a 250 m radius from the Achigan Creek bridge and the temporary modular bridge, shall be completed a minimum of two (2) weeks prior to commencement of installation of the deep foundations. Only one Pre-Construction Condition Survey per structure or facility is required to be carried out in advance of deep foundation installation, unless more than six (6) months will elapse between these operations, in which case an interim inspection will be required.

The Pre-Construction Condition Survey shall include, as a minimum, the following information:

a) Type of structure, including type of construction and if possible, the date when built.
b) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
c) Digital photographs or digital video or both, as necessary, to record areas of significant concern.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Pre-Construction Construction Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request.

7.1.2 Post-Construction Condition Surveys

The standard inspection procedure shall include the provision of an explanatory letter to the owner or occupant and owner with a formal request for permission to carry out an inspection.

A Post-Construction Condition Survey at each structure within a 250 m radius from the Achigan Creek bridge and the temporary modular bridge, is required within two (2) months of completion of the installation of deep foundations.

The Post-Construction Condition Survey shall include, as a minimum, the following information:

a) Identification and description of existing differential settlements, including visible cracks in walls, floors, and ceilings, including a diagram, if applicable, room-by-room. All other apparent structural and cosmetic damage or defects shall also be noted. Defects shall be described, including dimensions, wherever possible.
b) Digital photographs or digital video or both, as necessary, to record areas of significant concern.
c) Comparison between pre-condition survey documented concerns and post-condition concerns.

Photographs and videos shall be clear and shall accurately represent the condition of the property. Each photograph or video shall be clearly labelled with the location and date taken.

A copy of the Post-Construction Condition Survey limited to a single residence or property, including copies of any photographs or videos that may form part of the report, shall be provided to the owner of that residence or property, upon request. The report shall confirm that there have been no changes to the property between the Pre-Construction Condition Survey and the Post-Construction Condition Survey as a result of the installation of deep foundations.

7.2 Monitoring

The vibration monitoring equipment shall be placed on the ground surface at radial distances of 25 m, 50 m, and 100 m from the bridge structures toward the receptors (e.g., buildings, sensitive utilities). The Contractor shall take readings continuously during pile driving for the deep foundation elements, and shall immediately notify the Contract Administrator if the vibrations exceed the limits specified herein.

The vibrations measured on private structures, wells, etc. shall not exceed 25 mm/s. Those measured on utilities, if applicable, shall not exceed 10 mm/s.

If the readings are not within the limits stated above, the Contractor must alter the installation procedures until the vibrations at the various locations are within acceptable levels.

7.3 Records

The Contractor/Contractor's Engineer shall submit details of the vibration monitoring to the Contract Administrator as follows:

a) The time/duration of each reading.
b) Construction operations (i.e. installation of sheet piling) and timing of such relative to the readings.
c) Details of exceedances and modifications to operations.
d) Final report containing all relevant data including vibration monitoring and Pre- and Post-Construction Condition Surveys.

10.0 BASIS OF PAYMENT

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

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