Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

FOUNDATION INVESTIGATION AND DESIGN REPORT
DRIFTWOOD RIVER BRIDGE REPLACEMENT
HIGHWAY 577
ASSIGNMENT No. 5016-E-0038 - WO #13
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 5104-18-00, SITE 39E-096
GEOCRES NO. 42A-124

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Terraprobe Inc.

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DRIFTWOOD RIVER BRIDGE REPLACEMENT, SITE 39E-096
HIGHWAY 577
TOWNSHIP OF TAYLOR, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5016-E-0038 – WO #13, G.W.P. 5104-18-00

1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by WSP Canada Group Limited (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of detailed designs for the replacement of Driftwood River Bridge.

This project is based on the Ministry of Transportation, Ontario (MTO) Work Item Order titled *"Multi-Services Retainer – Agreement #5016E-0038, Work Item / Assignment #13, Foundation Investigations at Driftwood River Bridge on Highway 577"*. The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Work Item Order.

This report presents the factual data on the subsurface conditions at the Driftwood River Bridge, Site 39E-096 on Highway 577, Township of Taylor, District of Cochrane, Ontario.

2.0 SITE DESCRIPTION

The site (Latitude 48.551º; Longitude -80.681º) is located on Highway 577, approximately 1.6 km north of the highway's intersection with Highway 101 in the Township of Taylor, Ontario. The key plan on the Borehole Locations and Soil Strata Drawing, (Drawing 1) provides an overview of the site location and site photos are provided in Drawing 4.

The existing structure is a fourteen-span timber bridge that is 84± m long and 10± m wide, supported on timber piles. This bridge carries Highway 577 north bound and south bound traffic over Driftwood River. The Driftwood River meanders through this site flowing from west to east.

The terrain at the bridge site is flat and the vegetation consists primarily of deciduous trees and wild bush. There are minor areas of groomed grass on the north bank of the Driftwood River west of the bridge structure.

3.0 INVESTIGATION PROCEDURES

Terraprobe's staff staked out the borehole locations in the field relative to on-site features and, WSP surveyors established Control Point HCP 101 with a geodetic elevation of 263.959 m. The data from this control point was used by Terraprobe's staff to determine the ground surface elevations and coordinates of Boreholes 1 to 6. Under this assignment an additional borehole (BH 7) was drilled and two PiezoCone Penetration Tests (CPTus) were carried out at two locations shown as CPTu 18-1 and CPTu 18-2 on the borehole location plan. Borehole 7 and the CPTu locations were surveyed for coordinates and geodetic elevation with a Trimble R10 Receiver connected to the Global Navigation Satellite System. The borehole and CPTu data are summarized in the following table and the approximate borehole locations and CPTu soundings are shown on Drawing 1.

Borehole No.	MTM NAD 83 Coordinates		Ground Surface Elevation (m)	Depth (m)
	Northing (m)	Easting (m)		
BH1	5 379 214.06	328 355.20	263.8	44.2
BH2	5 379 247.46	328 351.57	259.1*	28.1
BH3	5 379 277.66	328 355.17	256.2*	23.3
BH4	5 379 310.66	328 351.57	263.9	29.0
BH5	5 379 229.60	328 350.60	262.6**	38.7

Borehole No.	MTM NAD 83 Coordinates		Ground Surface Elevation (m)	Depth (m)
	Northing (m)	Easting (m)		
BH6	5 379 302.30	328 356.20	262.8**	28.5
BH 7	5 379 230.8	328 354.5	262.3**	10.8
CPTu 18-1	5 379 213.1	328 350.6	263.9	24.0
CPTu 18-2	5 379 313.4	328 355.4	263.9	24.0

* River bed elevation. **Ground surface elevation below bridge deck.

The field work for this project was carried out in three stages. Four boreholes numbered Borehole 1, 2, 3 and 4 were drilled and sampled to depths ranging from 23.3 m to 44.2 m below ground surface between July 21 and 30, 2014 as part of the preliminary investigations. Two boreholes numbered Boreholes 5 and 6 were drilled and sampled to depths of 28.5 m and 38.7 m below ground surface between December 05 and 20, 2017, to supplement the preliminary investigation.

Under this assignment one borehole numbered Borehole 7 was drilled to a depth of 10.8 m below ground surface and two CPTu soundings numbered CPTu 18-1 and CPTu 18-2 were carried out to depths of 24.0 m below ground surface between November 19 and 23, 2018. It was intended to drill Borehole 7 to explore the depth of bedrock in the area of the south bridge abutment. However, this borehole was terminated at a depth of 10.8 m below ground surface because of misaligned casings and the extreme cold weather which froze drilling fluids in the exposed casing between the bridge deck and ground surface below.

The boreholes were drilled with a truck-mounted CME 75 drill rig supplied and operated by a specialist drilling contractor. Terraprobe's staff observed and recorded the drilling, sampling and in situ testing operations and logged the boreholes and rock cores. The CPTu soundings were carried out by DownUnder Geotechnical Limited (DownUnder) and further details on the CPTu field testing procedures are provided in DownUnder's appended Piezocone Penetration Testing Report.

Samples of the overburden soils were generally obtained at intervals of 0.75 m and 1.5 m depth using a 50 mm outer diameter (O.D.) split-spoon sampler in conjunction with the Standard Penetration Testing (SPT) procedures as specified in ASTM Method D 1586[1]. Relatively undisturbed samples of the clay soils were also collected with thin-walled Shelby Tube samplers. In the clay deposits an MTO 'N' vane was used to perform in-situ field vane tests, in order to determine the undrained shear strength of the soil. In Boreholes 1 and 5, cobbles and boulders were encountered within the till matrix and NQ-size diamond coring techniques were used to extend the borehole through the cobbles and boulders. Dynamic Cone Penetration tests were also performed in Borehole 1. The bedrock was cored by NQ-size diamond coring techniques.

Ground water conditions in the open boreholes were observed throughout the drilling operations. Since artesian conditions were encountered in Boreholes 1, 2, 3, 4, 5 and 6. The boreholes were backfilled immediately and sealed in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The recovered soil and rock samples were subjected to Visual Identification (VI) and select soil samples were also subjected to a laboratory testing programme consisting of natural moisture content, grain size distribution analyses, Atterberg limits determinations and one-dimensional consolidation testing in

1 ASTM D1586 – Standard Test Method for Standard Penetration Tests and Split Barrel Sampling of Soils.

accordance with MTO and/or ASTM Standards as appropriate. Soil samples were also submitted to SGS Canada Inc. (SGS) for chemical testing.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The surficial geology of the study area generally consists of glaciolacustrine sediments of the Barlow-Ojibway Formation and Matheson Till[2]. Fine-grained glaciolacustrine sediment, consisting of clay and silt, blankets most of the area and is on average 10 m to 15 m thick. Coarse-grained glaciolacustrine sediments, consisting of sand and minor gravel, are found mainly along the flanks of the major esker complexes and on bedrock uplands. The Matheson Till (deposited during the Late Wisconsinan period) is a silty sand till that varies in thickness from thin bands a few centimetres thick up to a maximum of 30 m. Most commonly, the Matheson Till is found beneath a thick cover of glaciolacustrine deposits.

The study area lies within the Abitibi Greenstone Belt of the Superior structural province of the Canadian Shield. The Abitibi Greenstone Belt consists of both volcanic and sedimentary rocks though typically dominated by mafic metavolcanic rocks. Several felsic and alkaline intrusions occur throughout the area.

4.2 Subsurface Conditions

Reference is made to the Record of Borehole Sheets in Appendix A. Details of the encountered soil and bedrock stratigraphy are presented in this appendix and on the “Borehole Locations and Soil Strata” drawings. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic sections are inferred from non-continuous soil sampling 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 summary, a flexible pavement and fill soils consisting of very loose to very dense gravelly sand to sand and very soft to very stiff silty clay were encountered at the site. The native overburden deposits consist of very loose organic silt, soft to stiff varved silty clay to clay, loose to very dense sand, and compact to very dense silty sand till. These soils are further underlain by argillite bedrock.

4.2.1 Flexible Pavement

Boreholes 1 and 4 were drilled through the Highway 577 bridge approach embankment. Both boreholes encountered a flexible pavement consisting of 150 mm and 175 mm thick asphalt surface treatment underlain by granular fill consisting of gravelly sand. The locations, thicknesses and base elevations of the granular pavement fill are summarized in the following table.

2 McClenaghan, M.B. 1990. Summary of results from the Black River – Matheson (BRiM) reconnaissance surface till sampling program; Ontario Geological Survey, Open File Report 5749, p. 197.

Borehole No.	Fill Thickness (mm)	Fill Base Elevation (m)
BH1	350	263.3
BH4	430	263.3

A Standard Penetration test carried out in the gravelly sand fill measured an SPT N-value of more than 50 blows for 0.3 m of penetration indicating a very dense relative density. The natural water contents of two samples of the granular fill are 2% and 6% by weight.

The grain size distribution curve of a sample of the gravelly sand fill is presented on Figure B1 in Appendix B. The results show a grain size distribution consisting of 21% gravel, 71% sand and 8% silt and clay size particles.

4.2.2 Fill – Gravelly Sand to Sand

Gravelly sand to sand fill was encountered at this site. The locations, thicknesses, depths and base elevations of the gravelly sand to sand fill are summarized in the following table.

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH1	1.6	2.1	261.7
BH4	2.3	2.9	261.0
BH5	1.6	3.2	260.7
BH6	2.1	3.2	260.7
BH7	2.0	3.8	260.3

Standard Penetration tests performed in the gravelly sand to sand fill measured SPT N-values that generally range from 1 to 50 blows for 0.3 m of penetration indicating a very loose to dense relative density. In Borehole 6 cobbles and boulders were encountered within the sand fill and the recorded SPT N-value in this cobble/boulder zone is 100 blows for less than 0.3 m of penetration indicating a very dense relative density. The natural water content of samples of the gravelly sand to sand fill range from 1% to 30% by weight.

Two samples of the sand fill were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figure B2 in Appendix B. The results show a grain size distribution consisting of 11% and 13% gravel, 80% and 84% sand, 5% silt, and 7% silt and clay size particles.

4.2.3 Fill – Silty Clay

Silty clay fill was encountered at this site, and the locations, thicknesses, depths and base elevations of the silty clay fill are summarized in the following table.

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH1	1.6	3.7	260.1
BH4	0.8	3.7	260.2
BH5	0.3	1.6	262.3
	1.0	4.2	259.7

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH6	0.5	3.7	260.2
BH7	1.0	4.8	259.3

Standard Penetration tests in the silty clay fill measured SPT N-values ranging from 1 to 18 blows for 0.3 m of penetration indicating a very soft to very stiff consistency. The natural water content (by weight) of samples of the silty clay fill range from 27% to 57% by weight.

The grain size distribution curves of two samples of the silty clay fill are depicted on Figure B3 in Appendix B. These results show a grain size distribution consisting of 0% and 2% gravel, 5% and 7% sand, 39% and 69% silt and, 26% and 52% clay size particles.

Atterberg limits tests were not possible because of insufficient sample quantities.

4.2.4 Organic Silt

Below the river bed there exists a layer of organic silt. Summarized below are the locations, thicknesses, depths and base elevations of the organic silt deposit.

Borehole No.	Organic Silt Thickness (m)	Organic Silt Depth (m)	Organic Silt Base Elevation (m)
BH2	1.4	3.7	257.7
BH3	0.7	5.9	255.5

Standard Penetration tests performed in the organic silt layer measured SPT N-values of 0 blows (weight of hammer) per 0.3 m of penetration indicating a very loose relative density. The natural water contents (by weight) of two samples of the organic silt are 73% and 90%.

The grain size distribution curve of a sample of the organic silt is depicted in Figure B4 in Appendix B. These results show a grain size distribution consisting of 0% gravel, 5% sand, 74% silt and 21% clay sized particles.

An Atterberg Limits test was also carried out on a sample of the organic silt and the results plotted on the plasticity chart on Figure B5 in Appendix B verify this classification i.e. organic silt. The results from the Atterberg Limits test are summarized below.

Liquid Limit:	49 %
Plastic Limit:	32 %
Plasticity Index:	17 %
Natural Moisture Content:	73 %

4.2.5 Silty Clay to Clay

The site is underlain by a varved silty clay to clay deposit. The deposit’s structure consists of fine grained clay soils interlayered with silt ranging from 1 mm to 50 mm in thickness. Photographs illustrating the varved clay matrix are provided in Figure B6 in Appendix B. The locations, thicknesses, depths and base elevations of the silty clay to clay deposit are summarized in the following table.

Borehole No.	Silty Clay to Clay Thickness (m)	Silty Clay to Clay Depth (m)	Silty Clay to Clay Base Elevation (m)
BH1	22.6	26.3	237.5
BH2	19.4	23.1	238.3
BH3	13.3	19.2	242.2
BH4	20.3	24.0	239.9
BH5	21.9	26.1	237.8
BH6	20.2	23.9	240.0
BH 7	6.0	10.8*	253.3

* Borehole termination depth.

The N-values of Standard Penetration tests carried out in the silty clay to clay deposit range from 0 blows (weight of hammer) to 16 blows per 0.3 m of penetration. Field vane tests measured in-situ undrained shear strengths that range from 12 kPa to 88 kPa as illustrated on Figure B7 in Appendix B. Based on these results the consistency of the silty clay to clay is described as generally soft to stiff. The sensitivity of the silty clay ranges from about 1.2 to 8.4, indicating a medium to extra sensitive soil class (April 01, 2018 errata to Canadian Foundation Engineering Manual [CFEM], 2006).

The variation of undrained shear strength with elevation plot depicted in Figure B7 illustrates higher undrained shear strength values ranging from 36 kPa to 118 kPa between elevation 260.0 m and elevation 258.0 m. Lower undrained shear strength values generally ranging from 12 kPa to 42 kPa were recorded between elevation 258.0 m and 248.0 m. Below elevation 248.0 m, there is a trend of increasing undrained shear strength values with depth, with shear strength values generally ranging from about 20 kPa to 88 kPa with higher CPTu values of up to 245 kPa being recorded between elevation 242.5 m and elevation 240.0 m.

Samples of the silty clay to clay soils were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figures B8 to B12 in Appendix B. The test results show a grain size distribution consisting of 0% gravel, 0% to 1% sand, 7% to 74% silt and 26% to 93% clay sized particles.

Samples of the silty clay to clay soils were also subjected to Atterberg limits tests and the results are plotted on the plasticity charts, Figures B13 to B17 in Appendix B. The results indicate a cohesive deposit of generally intermediate to high plasticity (CI to CH) with also layers of low plasticity clay (CL). The Atterberg limits test results are summarized below.

Liquid Limit:	27% to 71%
Plastic Limit:	17% to 27%
Plasticity Index:	10% to 44%
Natural Moisture Content:	27% to 80%

The Atterberg Limits tests results of the silty clay to clay deposit are also plotted against elevation in Figure B18. These results illustrate that the natural moisture contents of the tested samples are typically higher than the liquid limits. The moisture content of samples of the silty clay to clay varies between 21% and 80% and the unit weight of three tested samples range from 17.0 kN/m^3 to 18.2 kN/m^3.

Two one-dimensional consolidation tests were performed on samples of the silty clay to clay and the results are presented in Figures B19 to B26 in Appendix B. The results of the one-dimensional consolidation tests are summarized in the following table.

Borehole/Sample No.	Sample Depth/Elevation (m)	σ'_{vo} (kPa)	σ'_P (kPa)	C_c	C_r	e_o
BH4, Sample 10	9.4/254.5	90.9	88.0	0.547	0.074	1.35
BH5, Sample 6	6.2/257.7	47.7	46.0	0.276	0.023	1.11

Where: σ'_{vo} = effective overburden pressure
σ'_P = Preconsolidation pressure;
C_c = Compression index;
C_r = Recompression index; and
e_o = Initial void ratio.

The preconsolidation pressures derived from the consolidation test data are approximately equal to the effective overburden pressures suggesting that the silty clay to clay deposit is normally consolidated and this finding is also supported by the CPTu data. However, the recent CPTu data also indicates that there is an overconsolidated upper crust of silty clay to clay that extends to elevation 258.0 m±.

4.2.6 Sand

The varved silty clay to clay deposit is underlain by a sand layer. The locations, thicknesses, depths and base elevations of the sand deposit are summarized in the following table.

Borehole No.	Sand Thickness (m)	Sand Depth (m)	Sand Base Elevation (m)
BH1	2.1	28.4	235.4
BH2	1.6	24.7	236.7
BH3	1.1	20.3	241.1
BH4	1.7	25.7	238.2
BH6	1.0	24.9	239.0

The N-values of Standard Penetration tests carried out in the sand deposit range from 8 to more than 100 blows per 0.3 m of penetration, suggesting a loose to very dense relative density and, the moisture content of samples of this deposit range from 15% to 18% by weight.

Three samples of the sand deposit were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figure B27 in Appendix B. The test results show a grain size distribution consisting of 0% to 7% gravel, 80% to 93% sand, 17% silt, 4% and 5% silt and clay sized particles, and 3% clay.

4.2.7 Silty Sand Till

A silty sand till deposit was encountered in Boreholes 1 and 5. The locations, thicknesses, depths and base elevations of the silty sand till deposit are summarized in the following table.

Borehole No.	Silty Sand Till Thickness (m)	Silty Sand Till Depth (m)	Silty Sand Till Base Elevation (m)
BH1	12.5	40.9	222.9
BH5	12.6	38.7*	225.2

* Borehole termination depth.

Standard Penetration tests carried out in the silty sand till deposit gave N-values that range from 19 to more than 100 blows per 0.3 m of penetration indicating a compact to very dense relative density. The moisture content of samples from this stratum range from 6% to 27% by weight.

Grain size distribution tests were carried out on two samples from this deposit and the results illustrated in Figure B28, Appendix B; show a grain size distribution consisting of 0% gravel, 52% and 64% sand, 21% and 31% silt and 15% and 17% clay sized particles.

The matrix of the silty sand till contains cobble and boulder inclusions and, NQ-size diamond coring techniques were adopted in order to extend the boreholes into and below the cobbles and boulders. Photographs of the cobbles and boulders are provided in Figure B29 in Appendix B.

4.2.8 Bedrock

The overburden soils are underlain by argillite bedrock. Summarized below are the depths to bedrock and the bedrock surface elevations.

Borehole No.	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
BH1	40.9	222.9
BH2	24.7	236.7
BH3	20.3	241.1
BH4	25.7	238.2
BH6	24.9	239.0

The argillite bedrock is described as unweathered to slightly weathered, thinly to thickly bedded and its colour is greenish grey to dark grey. Photographs of the bedrock core samples are provided in Figures B30 to B33 in Appendix B. Summarized below are the Rock Quality Designation, Rock Mass Quality, Total Core Recovery and Solid Core Recovery.

Borehole No.	Rock Quality Designation (RQD)	Rock Mass Quality[3]	Total Core Recovery (TCR)	Solid Core Recovery (SCR)
BH1	0% to 20%	Very Poor	48% to 100%	40% to 86%
BH2	0%	Very Poor	33% to 61%	10% to 20%
BH3	44% and 100%	Poor to Excellent	73% and 100%	73% and 100%
BH4	79% to 100%	Good to Excellent	98% to 100%	92% to 100%
BH6	6% to 22%	Very Poor	65% to 100%	9% to 47%

Point Load Index Tests were carried out on the bedrock core samples and the interpreted unconfined compressive strength (UCS) results range from 114 MPa to 343 MPa. These UCS results classify the tested portions of the bedrock as very strong (R5 grade, 100 MPa to 250 MPa) to extremely strong (R6 grade, > 250 MPa) according to the rock strength classification in Table 3.5 of the *Canadian Foundation Engineering Manual 2006*.

3 Deere et al., 1967.

4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling. Artesian conditions were encountered in Boreholes 1, 2, 3, 4, 5 and 6 in the lower coarse-grained deposits (sand and silty sand till) overlying bedrock. The head of water (estimated from Boreholes 1, 4, 5 and 6) varies from 0.9 m to 2.4 m above ground corresponding to ground water elevations that range from 264.6 m to 266.3 m.

The ground water level at this site is estimated to be at an elevation of 261± m based on the soil moisture conditions and river water levels. The ground water level is expected to fluctuate seasonally and is expected to rise during wet periods of the year.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Landcore Drilling of Chelmsford, Ontario. The field operations were organized by Mr. Satyajit Manani, C.E.T. (Boreholes 1, 2, 3 and 4) and Ms Fatemeh Yazdandoust, MSc. (Boreholes 5, 6 and 7). The routine laboratory and one-dimensional consolidation testing were carried out at Terraprobe's Brampton laboratory. The CPTu soundings were carried out by DownUnder Geotechnical Limited of Maple, Ontario.

This report was prepared by Ms. Sepideh D-Monfared, P.Eng., and reviewed by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Principal with Terraprobe. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact, carried out an independent quality control review.

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PART B – FOUNDATION DESIGN REPORT

DRIFTWOOD RIVER BRIDGE REPLACEMENT, SITE 39E-096
HIGHWAY 577
TOWNSHIP OF TAYLOR, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5016-E-0038 – WO #13, G.W.P. 5104-18-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to carry out designs for the Driftwood River Bridge replacement. The discussion and recommendations presented in this report are based on our understanding of the project and our interpretation of the factual data obtained from the subsurface investigations. These geotechnical recommendations are intended for use by the Ministry of Transportation and their design consultants and shall not be used or relied upon for any other purposes or by any parties including contractors.

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 or operational constraints may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

The existing bridge is a fourteen-span timber structure supported on timber pile foundations, with a length of 84± m and a width of 10± m. The bridge carries Highway 577 north bound and south bound traffic over Driftwood River. A three-span bridge replacement on the existing alignment is proposed and the highway profile will also be raised by up to 0.7 m± to achieve an elevation of 264.6 m± at the bridge abutments.

6.2 Consequence and Site Understanding Classification

The proposed structure carries Highway 577 traffic with the potential to impact this transportation corridor as well as alternative transportation corridors or structures. Therefore, a “typical consequence level” is considered appropriate as outlined in Section 6.5 of the *Canadian Highway Bridge Design Code (CHBDC) S6-14*.

A “typical degree of site and prediction model understanding” has been utilized given the scope of the foundation investigation and laboratory testing programme.

The consequence factor (ψ) and geotechnical resistance factors (Φ_{gu} & Φ_{gs}) used for designs and stipulated in Clause 6.5.2 and Clause 6.9 of the CHBDC S6-14, are based on a “typical consequence level” and a “typical degree of site and prediction model understanding”.

6.3 Seismic Design

6.3.1 Importance Category & Seismic Site Classification

Based on Section 4.4.2 of the CHBDC, the proposed bridge has an importance category of *“Other Bridge”.* Ground conditions for seismic site characterization were established based on the field investigation and laboratory testing data. The soil average undrained shear strength in the upper 30 m of soil below founding level was used to define the seismic site classification in accordance with Table 4.1 of the CHBDC. Based on this methodology and the data, the structure shall be designed based on Site Class E.

6.3.2 Spectral Response Values

The CHBDC requires that the seismic hazard values associated with the design earthquake be established based on the National Building Code of Canada (NBCC). These values, Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Spectral Acceleration (Sa) can be obtained from the Geological Survey of Canada (GSC) *"2015 National Building Code of Canada Seismic Hazard Calculator"* and are for a reference ground condition of Site Class C.

In accordance with Section 4.4.3.3 of the CHBDC, the NBCC values were adjusted to reflect local site conditions i.e. Site Class E. As per Section 4.4.3.3 of the CHBDC, the value of PGA_{ref} for use with Tables 4.2 to 4.9 was taken as 80% of the PGA since the Sa(0.2)/PGA ratio is less than 2.0. A PGA_{ref} value of 0.085 for the 2,475 year return was used. The NBCC spectral response values and the site-specific design values are tabulated below.

NBCC Seismic Hazard Values 2% Exceedance in 50 years (2,475 Year Return Period)							
PGA (g)	**PGV (m/s)**	**Sa (0.2) (g)**	**Sa (0.5) (g)**	**Sa (1.0) (g)**	**Sa (2.0) (g)**	**Sa (5.0) (g)**	**Sa (10.0) (g)**
0.106	0.073	0.166	0.092	0.050	0.024	0.006	0.0025
Site Specific Design Seismic Hazard Values Site Class E 2% Exceedance in 50 years (2,475 Year Return Period)							
0.192	0.180	0.272	0.227	0.141	0.070	0.018	0.006

6.3.3 Liquefaction Assessment

As per Table 4.10 of the CHBDC, the bridge has been assigned to Seismic Performance Category 2. The overburden silty clay to clay soils are non-susceptible to liquefaction but these soils can exhibit cyclic failure due to strain softening. The potential for cyclic failure was assessed based on the research done by Seed et al (2003)[4]. The assessment indicates that there is a potential for cyclic failure of the silty clay to clay overburden under earthquake loads.

From a foundation engineering perspective, no concerns are anticipated since the bridge will be supported on pile foundations. Further analysis on the effects of earthquake events on embankments are provided in Section 6.9.4.

6.4 Foundation Alternatives

The advantages, disadvantages, risks and consequences of foundation options for supporting a bridge are presented in Table 1. These foundation alternatives are summarized below.

- Spread footings;
- Augered Caissons (drilled shafts); and
- Driven piles.

[4] Recent Advances in Soil Liquefaction Engineering: A Unified Consistent Framework" Keynote Presentation, 26th Annual ASCE.

6.4.1 Spread Footings

At the abutment and pier locations the soft to stiff silty clay to clay deposit is unsuitable for supporting the bridge on spread footings. The geotechnical resistance of the silty clay to clay deposit is low and spread footings will experience large time dependent consolidation settlements. There are also no advantages in founding spread footings on an engineered fill pad because the geotechnical resistance of the silty clay to clay deposit remains low with increasing depth. Consequently, spread footings are not considered to be a feasible foundation alternative.

6.4.2 Augered Caissons (Drilled Shafts)

Augered caisson foundations were considered as a foundation scheme. The caissons will have to be founded into the underlying very dense silty sand till and/or on the bedrock at depths in the order of 20± m to 27.5± m below ground surface. Artesian conditions exist in the sand and the silty sand till through which the caissons will have to be extended and, the silty sand till matrix also contains cobbles and boulders.

Given these sub-surface conditions, it would be difficult to seal the bottom of the liner to exclude ground water because of the artesian pressure in the permeable sand and silty sand deposits as well as the presence of cobbles and boulders in the silty sand till. Attempts at dewatering the caisson excavation and maintaining a sufficiently dry excavation to permit cleaning, inspection and high-quality construction, would be challenging and most likely impractical. Therefore, caisson foundations are not recommended for supporting the structure.

6.4.3 Driven Piles

The subsurface conditions at the site are considered suitable for the design of foundations supported on close ended steel tube piles and/or steel H-piles. When selecting the pile type i.e. steel tube piles and/or H-piles the designer should consider the following issues:

- Steel H-piles and concrete filled steel tube piles are feasible and practical foundation alternatives for supporting the north abutment and piers;
- Close ended, concrete filled steel tube piles have a higher probability of being installed successfully on the bedrock at the north bridge abutment and pier locations but not at the south abutment; and
- Steel tube piles are not recommended for supporting the south abutment. There is a very dense silty sand till unit with cobble and boulder inclusions in this area making it very difficult (maybe impractical), to drive “high displacement” steel tube piles into this deposit to the depth required to achieve the desired load carrying capacity.

6.4.3.1 Axial Resistance

The concentric axial factored ultimate geotechnical design resistance at the Ultimate Limit State (ULS), the factored serviceability geotechnical resistance at the Serviceability Limit State (SLS), and estimated pile tip elevations are provided in the following table. Based on an average bedrock UCS value of 200 MPa, the factored ULS geotechnical resistance of the bedrock is 3,500 kN which is higher than the structural capacity of the pile. Therefore, the structural capacity of the pile will govern and should be checked by the structural engineer.

Location	Reference Borehole	Pile Type	Factored Geotechnical Resistance at U.L.S (kN)	Factored Geotechnical Resistance at SLS (25 mm Settlement) (kN)*	Estimated Pile Tip Elevation (m)	Founding Stratum
South Abutment	BH5	HP 310x110	1800	1500	233.0±	Silty Sand Till
		HP 360x132	2100	1600	232.5±	
South Pier	BH2	610mm Steel Tube	3500	N/A*	236.7±	Bedrock
		HP 310x110	2000			
		HP 360x132	2400			
North Pier	BH3	610mm Steel Tube	3500	N/A*	241.1±	Bedrock
		HP 310x110	2000			
		HP 360x132	2400			
North Abutment	BH6	610mm Steel Tube	3500	N/A*	239.0±	Bedrock
		HP 310x110	2000			
		HP 360x132	2400			

* The bedrock is "unyielding" and the SLS condition will not govern. ULS values provided for H-piles founded on bedrock are the pile's structural axial resistance.

Pile installation shall be carried out in accordance with OPSS.PROV 903 as amended by Special Provision No. 109F57. For piles driven to bedrock the Contractor shall adequately seat the pile on bedrock without damaging the pile as specified in OPSS.PROV 903 "Driving to Bedrock". The appropriate pile driving note is "Pile to be driven to bedrock".

Steel H-piles will be driven to practical refusal in the silty sand till at the south abutment. Since the till matrix contains cobbles and boulders, piles may encounter effective refusal in this stratum without reaching the predicted pile tip elevations. Pile driving at the south abutment location should be controlled by the Hiley Formula and an Ultimate Pile Resistance (R) to be specified by the structural engineer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is "Piles to be driven in accordance with Standard SS 103-11 using an ultimate geotechnical resistance of "R" kN per pile". For design purposes an "R" value of 3600 kN is recommended for HP 310x110 piles and an "R" value of 4200 kN is recommended for HP 360x132 piles.

Artesian conditions exist in the lower silty sand till and the sand deposits overlying bedrock. However, pile driving (including the installation of steel tube piles) through the upper varved silty clay to clay deposit will cause significant remoulding and adhesion of clay to the pile shafts. A watertight barrier will therefore be formed at the soil/pile interface that will prevent upward movement of ground water under artesian pressure. Therefore, an inverted granular filter below pile caps is not required.

6.4.3.2 Downdrag

The remaining post construction consolidation settlement (up to 25 mm) that occurs after construction is complete will impart downdrag loads on the driven piles. Excavations will extend to underside abutment elevations of 260.1± m at both bridge abutments. An HP 310x110 pile section shall be designed for an unfactored downdrag load of 250 kN per pile.

6.4.3.3 Pile Tips

Pile tip reinforcement is not required at the piers and north abutment. However, the tips of piles installed at the south abutment should be fitted with Rock Injector points from an approved manufacturer such as Titus Steel Company (“R” Series “H” or “Pipe”) or Associated Pile & Fitting Corp (APF Hard Bite). The use of Rock Injector points or rock points is recommended for the following reasons:

- The piles will penetrate into soil containing cobbles and boulders and these aggressive driving conditions require a higher level of tip protection; and
- Rock points will provide increased cutting ability to the pile sections, reduce the probability of misalignment and increase the probability of achieving the desired penetration in the silty sand till.

6.4.3.4 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. If deemed necessary by the structural engineer, the upper 3 m of the piles should be surrounded by a 600 mm diameter CSP as specified by MTO’s integral abutment design procedures.

The space between the pile and the CSP should be filled with sand. A Non Standard Special Provision (NSSP) will be required specifying the gradation of the sand according to the data tabulated below.

MTO Sieve Designation		Percentage Passing
2 mm	#10	100%
600 µm	#30	80%-100%
425 µm	#40	40%-80%
250 µm	#60	5%-25%
150 µm	#100	0%-6%

6.4.3.5 Lateral Resistance

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and the ultimate lateral resistance (p_{ult}) as outlined in the following equations:

	k_s	=	n_h z / D [cohesionless soils]	(kN/m^3)
	k_s	=	67 S_u/D [cohesive soils]	(kN/m^3)
	p_{ult}	=	3 γ z K_p [cohesionless soils]	(kPa)
	p_{ult}	=	9 S_u [cohesive soils]	(kPa)
where	z	=	depth of pile embedment	(m)
	D	=	pile width	(m)
	S_u	=	undrained shear strength	(kPa)
	n_h	=	coefficient of horizontal subgrade reaction	(kN/m^3)
	γ	=	unit weight	(kN/m^3)
	K_p	=	passive earth pressure coefficient	(dimensionless)

The spring constant K, for analysis of a pile segment or element of length L metres, can be obtained from the expression, K = k_s x L x D (kN/m). The ultimate lateral resistance P_{ult}, of a pile segment or element of length L metres, can be obtained from the expression, P_{ult} = p_{ult} x L x D.

The equations provided above and the soil parameters provided in the following table, may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the pile. For design purposes a maximum horizontal passive resistance of 120 kN (ULS) is recommended.

Area Reference Borehole No	Applicable Elevation (m)	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m^3)*
South Abutment BH 5	260.1 – 259.7	Fill – Silty Clay	18	0	10	–
	259.7 – 258.0	Silty Clay to Clay	17	0	40	–
	258.0 – 242.0	Silty Clay to Clay	17	0	20	–
	242.0 – 237.8	Silty Clay to Clay	17	0	70	–
	237.8 – 225.2	Silty Sand Till	21	35	–	11000
South Abutment BH 7	260.1 – 259.3	Fill – Silty Clay	18	0	10	–
	259.3 – 258.0	Silty Clay to Clay	17	0	40	–
	258.0 – 253.3	Silty Clay to Clay	17	0	20	–
North Abutment BH 6	260.1 – 258.0	Silty Clay to Clay	17	0	40	–
	258.0 – 242.0	Silty Clay to Clay	17	0	20	–
	242.0 – 240.0	Silty Clay to Clay	17	0	70	–
	240.0 – 239.0	Sand	20	32	–	4400

* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd edition, 1992.

Since the piles are end bearing, their vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to CHBDC Clause 6.11.4.7.

For lateral soil/pile group interaction analysis, the equation for k_s quoted in this section may be used in conjunction with appropriate reduction factors. Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre

Where a pile group is oriented parallel to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D*	1.00
6 D*	0.70
4 D*	0.40
3 D*	0.25

* D is the width of the pile, and spacing is measured centre to centre

Intermediate values of the horizontal subgrade reaction reduction factor R may be obtained by interpolation. For conventional bridge abutments, battered piles are recommended to provide lateral resistance.

6.4.4 Recommended Foundation Scheme

From a geotechnical point of view, it is recommended that the bridge be supported on deep foundations consisting of steel H-piles. The advantages, disadvantages, risks and consequences of deep foundation alternatives are provided in Table 1.

As noted in Table 1, a working mat will be required to support pile driving equipment in areas where weak soils exist. Therefore, geotechnical assessments shall be carried out by a geotechnical engineer retained by the Contractor, to ensure that proper construction equipment is selected such that embankment and foundation failures do not occur during construction. A Special Provision for this aspect of the work is provided in Appendix H.

6.4.5 Design Frost Depth

Pile caps and footings should be founded at a minimum depth of 2.5 m of earth cover below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSD 3090.100. In addition, the footings should extend below any existing fill and surficial organic materials, where present.

6.5 Erosion Protection

The free water level in Driftwood River is at elevation 260.9 m±, with the potential to rise higher during storm events as well as the time of the year. This water has the potential to submerge and erode the forward and side slopes at the bridge abutments if the slopes are not protected. Design of an erosion protection scheme will depend on hydrologic, hydraulic and/or other concerns. We recommend using rip-rap to armour the embankment slopes with which creek water is likely to be in contact. The rip-rap should be installed in accordance with OPSS.PROV 511.

Surface water can also cause erosion beneath the rip-rap and loss of fines through the rip-rap. Therefore, a properly designed filter should be installed between the rip-rap and the embankment material.

We recommend that a qualified Hydraulics Engineer be consulted to estimate the scour depth and to provide inputs on the design thickness and lateral extent of rip-rap protection.

6.6 Lateral Earth Pressure

6.6.1 Static Conditions

Earth pressures are generally calculated using the following expression:

$$P_h = K(\gamma h + q)$$

P_h = horizontal pressure on the wall (kPa)
K = lateral earth pressure coefficient
γ = unit weight of retained soil (kN/m^3)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

For conventional backfill to abutments earth pressures acting on the structure should be computed in accordance with Clause 6.12 of the CHBDC S6-14 and according to Clause 6.12.3 of the CHBDC S6-14;

a compaction surcharge should also be added. For soils with an angle of internal friction ranging from 30º to 35º the magnitude should be 12 kPa at the top of the fill decreasing linearly to 0 kPa at a depth of 1.7 m; or decreasing linearly to 0 kPa at a depth of 2.0 m for soils with an angle of internal friction that exceeds 35º. Compaction equipment including hand operated vibratory equipment should comply with OPSS.PROV 501.

The backfill to the bridge abutments should be in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150, and rock backfill should be placed to the extents shown in OPSD 3101.200.

The lateral earth pressure coefficients are dependent on the material used as backfill and typical values are provided in the following table.

Wall Condition	**Lateral Earth Pressure Coefficient (K)**					
	OPSS Granular A or OPSS Granular B Type II ϕ = 35°; γ = 22.8 kN/m^3		OPSS Granular B Type I ϕ = 32°; γ = 21.2 kN/m^3		Rock Fill ϕ = 42°; γ = 19.0 kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.30	0.46*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* For wing walls.

The lateral earth pressure coefficients provided in the table above are "ultimate" values that require certain structural movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the CHBDC S6.1-14.

EPS when used as backfill applies insignificant vertical load to structure foundations. Furthermore, because of the light weight of EPS and since the EPS blocks are stable when stacked vertically, virtually no lateral load is applied to bridge abutments provided that the EPS extends beyond the limits of the active earth pressure wedge.

6.6.2 Seismic Conditions

In accordance with Section 4.6 of the CHBDC, seismic loads shall be considered in the design. The designs shall take into consideration:

- The wall should be designed to withstand the combined static lateral loads plus the earthquake induced loads;
- The horizontal seismic coefficient (k_h) used to calculate the seismic active pressure coefficient is taken as 1.0 times the PGA for structures that do not permit lateral yielding and 0.5 times PGA for structures that permit lateral yielding; and

- Where sloping backfill exists above the top of the wall, the weight of the backfill above the top of the wall should be treated as a surcharge when calculating the lateral earth pressure under seismic conditions.

The Mononobe-Okabe (M-O) method was used to calculate the active earth pressure coefficients for yielding and non-yielding walls assuming that the angle of friction between the wall and backfill material is 0.5 ϕ. The seismic active earth pressure coefficients provided in the following table may be used for designs.

Wall Condition	Seismic Active Earth Pressure Coefficients (K)		
	OPSS Granular A or OPSS Granular B Type II ϕ = 35°; δ = 17.5° γ = 22.8 kN/m^3	OPSS Granular B Type I ϕ = 32°; δ = 16.0° γ = 21.2 kN/m^3	Rock Fill ϕ = 42°; δ = 21.0° γ = 19.0 kN/m^3
	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall
K_{AE} (Yielding Wall)	0.30	0.34	0.23
K_{AE} (Non-Yielding Wall)	0.37	0.41	0.29

6.7 Excavations

All excavations must be carried out in accordance with the guidelines outlined in the *Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects*. Where workers must enter excavations extending deeper than 1.2 m, the trench walls must be suitably sloped and/or braced in accordance with the OHSA. Within the envisaged depths of temporary excavations (i.e. up to elevation 259.5± m), the OHSA soil classifications are:

- Embankment fill – Type 3 soils above ground water and Type 4 soils below ground water; and
- Silty Clay to Clay – Type 4 soils.

The side slopes of temporary excavations may be formed no steeper than 1H:1V for Type 3 soils and 3H:1V or flatter for Type 4 soils. Excavations should be carried out in accordance with OPSS 902.

6.8 Ground Water Control

6.8.1 General

Artesian conditions were encountered in all of the boreholes in the lower granular deposits, but excavations are not expected to extend to these depths. Nevertheless, surface water and ground water control will be required to enable construction below the ground water table. Around the perimeter of the excavations, interceptor perimeter trenches and/or cofferdams may also be required to prevent surface water from entering excavations. The design, installation, operation and maintenance of the dewatering system is the Contractor's responsibility.

The Ontario Ministry of Environment and Climate Change (MOECC) requires a Permit to Take Water (PTTW) for any ground water and storm water takings in excess of 400 m^3/day. If the ground water and storm water taking is between 50 m^3/day and 400 m^3/day, then the activity must be registered on the Environmental Activity and Sector Registry (EASR).

At the abutment locations, excavations will extend through the existing embankment fill terminating either in the silty clay fill material or in the cohesive silty clay to clay deposit. A suitable dewatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

Cofferdams (if required) in Driftwood River, will have to be unwatered to permit construction. The excavation base will consist of relatively impermeable silty clay to clay soils that are not anticipated to yield significant volumes of water. The Contractor must ensure that suitably sized dewatering equipment is used and, adequately sized emergency “stand-by” equipment should also be supplied in case of sudden failure of the primary dewatering equipment.

6.8.2 Ground Water Taking Volumes - Open Cut Excavations

Daily ground water taking volumes for open cut excavations at the bridge abutments and approach embankments were estimated based on the following assumptions:

- Estimated hydraulic conductivity values of:
  - 1.0 e-07 m/s for silty clay fill and overconsolidated silty clay to clay; and
  - 1.0 e-09 m/s for silty clay to clay.
- A surface water elevation of 260.1 m± in the Driftwood River and a ground water elevation of 261.0 m± in the footprint area of the approach embankments;
- Open cut excavations extending to elevation 260.0 m± at both approach embankments with a 3H:1V side slope geometry and measuring 10 m in base width and 20 m in base length and 3.8 m in vertical height; and
- A 46 mm daily precipitation value (equivalent to the average rainfall reported on *Canadian Climate Normals 1981-2010 Station Data* for *TIMMINS VICTOR POWER A Station*).

The analysis indicates the estimated daily ground water taking volume (assuming that excavations are carried out at both abutments simultaneously) is 60 m^3±. Therefore, a PTTW is not required but, the project will require a registration on the EASR.

6.9 Approach Embankments

6.9.1 General

Varved clays described as silty clay to clay in the borehole logs exist at this site. The microstratigraphy of varved clays has a direct impact on their geotechnical properties and behaviour. The properties of varved deposits can vary significantly and are difficult to assess from field and laboratory testing.

The Highway 577 grade will also be raised. At the north approach, a grade raise of 550 ±mm is proposed at the new bridge abutment gradually reducing to 0 mm at Station 1+650. At the south approach, a grade raise of 670 ±mm is proposed at the new bridge abutment gradually reducing to 0 mm at Station 1+404.

Three embankment options were initially presented to the design team to permit further discussions with the intent of arriving at a cost effective and constructible solution that meets the one-year construction schedule. In addition to Option 1 (conventional embankment) which is presented as a baseline, the two embankment alternatives that were considered to be feasible and practical are also described below. Other alternatives such as staged construction with preloading and also wick drains were considered but do not appear to be advantageous when compared to the options described below. The advantages, disadvantages, risk/consequences and relative costs of the three options are summarized in Table 2.

- ***Option 1 (Conventional Embankment)*** – An embankment constructed with OPSS 1010 Select Subgrade Material (SSM unit weight of 20 kN/m^3) that is overbuilt in the footprint area of the bridge abutment and surcharged with an additional 1 m high layer of SSM to elevation 265.5 m in order to accelerate settlement. The pile driving operations and abutment construction will be carried out after consolidation settlement is essentially complete recognizing that in the 0 m – 20 m Transition Point Distance the post construction settlement cannot exceed 25 mm;
- ***Option 2 (EPS/Gran. B Embankment, North Approach Only)*** – A 20 m long embankment constructed with Granular B Type I from elevation 260.0 m to 262.0 m with Expanded Polystyrene (EPS) blocks installed on the Granular B Type I material. No preloading of the embankment is required but, the EPS installation shall be carried out at the same time when EPS is installed at the south abutment. The pile driving operations and abutment construction will be carried out prior to installing the EPS blocks, recognizing that in the 0 m – 20 m Transition Point Distance the post construction settlement cannot exceed 25 mm; and
- ***Option 3 (RSS/EPS Embankment, South Approach Only)*** – A Reinforced Soil System (RSS) constructed with Granular A from elevation 260.0 m to 263.5 m within the confines of the bridge abutment and extending a horizontal distance of 5 m measured from the inner face of the abutment. Beyond the RSS and for a distance of 15 m, Granular A will be used to construct the approach embankment from elevation 260.0 m to 263.5 m. The RSS and Granular A constructed to 263.5 m will remain in place for a defined preload period and will then be removed to elevation 262.0 m and replaced with EPS blocks installed on the Granular A and RSS. The RSS is required to ensure that lateral loads from the Granular A are not transferred to the abutment and consequently, the pile driving operations and abutment construction can be carried out before the RSS is installed. In the 0 m – 20 m Transition Point Distance the post construction settlement cannot exceed 25 mm.

6.9.2 Settlement

The engineering data used for the analyses were established using CPTu data, data obtained from two consolidation tests as well as empirical correlations of undrained shear strengths, laboratory index tests and soil moisture contents. This data and the equations used are provided in Figure D1 in Appendix D.

The preconsolidation pressures (σ'_p) derived from the consolidation test data are approximately equal to the effective overburden pressure suggesting that the silty clay to clay deposit is normally consolidated. However, the recent CPTu data indicates that there is an overconsolidated upper crust of silty clay to clay that extends to elevation 258.0 m±. From elevation 258.0 m± to 245.0 m±, the CPTu results indicate a normally consolidated clay and the CPTu pore pressure dissipation tests suggest that the silt layers within the varved silty clay to clay deposit are relatively thin and cannot be relied upon as drainage paths that will quickly dissipate excess pore water pressures that develop due to embankment loads. Below elevation 245.0 m±, there is a visible increase in CPTu tip resistance data which is an indicator of thicker silt seams that may act as drainage paths for quicker excess pore water pressure dissipation.

The engineering properties that were used to calculate the magnitude and time rate of settlement are summarized in the following table.

Parameter	Overconsolidated Silty Clay to Clay (above elevation 258± m)	Varved Silty Clay to Clay (elevation 258± m to 245± m)	Varved Silty Clay to Clay (below elevation 245± m)
Overconsolidation Ratio	5.0	1.0	1.0
Compression Index - C_c	0.25	0.3	0.18
Recompression Index - C_r	0.04	0.05	0.035
Initial Void Ratio - e_o	0.95	1.35	1.0
Coefficient of Consolidation - C_v (cm^2/s)	0.01	0.006-0.009H* and 0.0015	0.0015+0.0031H*

* Refer to Figure D1 in Appendix D for clay layer thickness (H) and corresponding elevations.

To predict the magnitude and time rate of settlement of the underlying silty clay to clay soils the commercially available program Settle 3D developed by Rocscience Inc. was used. Spreadsheet calculations were also carried out using the constrained modulus values derived from the CPTu data.

Since the overburden soils are more sensitive to settlement at the south approach embankment and higher grade raises are being proposed in this area, the magnitude and time rate of settlement including the construction schedule, will be governed by the performance of the south approach embankment. It is reasonable to assume that consolidation/recompression will occur quickly in the overconsolidated upper silty clay to clay layer and the settlement due to recompression will likely be complete in a very short time period. Therefore, the rate of consolidation will be primarily controlled by the coefficient of consolidation and thickness of the varved silty clay to clay that exists between elevation 258.0 m± and 237.5 m±.

The results of the settlement analysis carried out for the three embankment alternatives as well as geometrical details of the models are tabulated below.

Parameter	Option 1 Conventional Embankment	Option 2 North Approach	Option 3 South Approach
Base Elevation	260.1 m	260.1 m	260.1 m
Top Elevation (during preload period)	265.5 m	262.0 m	263.5 m
Side Slope Geometry	2H:1V	2H:1V*	2H:1V*
Recommended Preload Period	24 months	Not Required	7 months
Settlement of normally consolidated layer (mm) during preload	300	Negligible	115
Predicted post construction settlement including creep	< 25	< 25	< 25
Total Settlement (mm)	325	< 25	<140

* Embankment side slopes to be constructed at 2H:1V during RSS/Granular A or Granular B build up to preload height. Embankment side slopes to be reconstructed to 2.5H:1V during EPS installation.

For Option 1, using the more refined coefficient of consolidation values established from CPTu pore pressure dissipation tests; a surcharge period of 24 months is required in order to achieve a post construction settlement of 25 mm. Since the time required to achieve the 25 mm post construction settlement requirement will exceed the one-year construction schedule, this option is not recommended. Approaches that mitigate settlement such that the bridge can be built in a one-year construction schedule are provided as Options 2 and 3.

Option 2 and Option 3 approach embankment design concepts will comply with MTO's *"Embankment Settlement Criteria For Design, Table 1.2, Post Construction Settlement Criteria for Transitions"* dated

July 02, 2010 since the estimated post construction settlements are < 25 mm within the 0 m – 20 m Transition Point Distance.

The north approach embankment design concept i.e. EPS supported on Granular B Type I will not subject the underlying silty clay to clay to any significant stress increase. Therefore, the total settlement (including creep) is estimated to be less than 25 mm over a 20-year design period.

South approach embankment construction will subject the underlying silty clay to clay to increased stress under the new loads and a Granular A/RSS preload applied over a 7-month period is required to accelerate primary consolidation. The silty clay to clay soils will rebound when the preload is removed and the lighter EPS is installed. Sometime after the rebound, the silty clay to clay will experience secondary compression (creep) under the constant effective stress. The effect of the preload combined with the load reduction by using EPS will reduce the secondary compression index which will in turn reduce the magnitude of secondary settlement that occurs over a 20-year design period. The longer the surcharge can be left in place, the greater the reduction in the magnitude of secondary compression and, the time delay will be longer before the onset of secondary compression. Therefore, we recommend that a 7-month preload period be instituted at the south approach.

To comply with MTO's *"Embankment Settlement Criteria For Design, Table 1.2, Post Construction Settlement Criteria for Transitions"* dated July 02, 2010; settlement analyses were carried out at sections in the 20 m – 50 m Transition Point Distance where grade raises are proposed. The analysis indicates that at the south approach surcharging with Granular B Type I material placed 1 m high above the design pavement profile is required over a 7-month period prior to paving.

A closure of the Stock Concession Road 2/Highway 577 intersection is not possible and access ramps will be required if a 1 m high surcharge is placed at the north approach. However, rebuilding the north approach embankment in this area and raising the grade to the design profile, will result in 15± mm of settlement over a 7-month period and the estimated post construction settlement will be 50± mm. It is probable that the post construction settlement could exceed 50± mm but, this approach at settlement mitigation is most feasible compared to closing the intersection or providing access ramps. The settlement mitigation treatment of the two approaches for a 7-month period are:

- South Approach – Sta. 1+404 to Sta. 1+470 surcharge with 1 m of Granular B Type I; and
- North Approach – Sta. 1+585 to Sta. 1+650 rebuild embankment and raise grade to the design pavement profile.

To mitigate the potential for non-uniform embankment performance, we recommend carrying out grade raises, surcharging and embankment side slope construction in the 20 m – 50 m Transition Point Distance at the same time that the approach embankments are being constructed. A recommended Operational Constraint for the preload and surcharging of embankments is provided in Appendix H. Typical EPS embankment construction details for the north and south approaches are provided in Appendix F and a Special Provision for Rigid Expanded Polystyrene Embankment Fill is included in Appendix H.

Embankments constructed with non-cohesive earth fill will also settle during construction (fill compression) and, the magnitude of this settlement is expected to be about 1% of the fill height. This settlement should be immediate in nature and essentially be complete shortly after construction is complete.

Settlement monitoring shall be carried out to determine the construction timing for pre and post paving operations and a settlement monitoring and instrumentation plan is provided in Appendix G. Special Provisions for the supply and installation of monitoring instruments by the Contractor and for monitoring by the Contractor Administrator are provided in Appendix H.

6.9.3 Stability (Static Conditions)

The global, internal and surficial stability of the embankment will depend on the embankment height, the slope geometry, the subsurface soils, and material used to construct the embankment. For the purpose of embankment stability analyses, the commercially available slope stability program Slide 7.0 developed by Rocscience Inc. was used.

The Morgenstern-Price and Spencer methods for stability analysis were employed and the target factors of safety for temporary and permanent conditions were derived based on the site consequence factor (ψ) and the geotechnical resistance factors (Φ_{gu}) provided in Table 6.2 of the CHBDC. Accordingly, minimum target factors of safety of 1.3 and 1.5 were established for temporary (short term) and permanent (long term) conditions respectively.

In addition to the short term and long term analyses, we also believe that there is an intermediate condition when the shear strength of the overconsolidated silty clay to clay layer quickly reduces to the effective stress state and the underlying normally consolidated silty clay to clay remains in a total stress state. Stability analyses were also carried out for this intermediate condition which accounts for strain compatibility between the stronger overconsolidated silty clay to clay layer and the underlying weaker silty clay to clay.

The soil parameters used for the slope stability analyses and the factors of safety that were obtained are provided in the following tables. The slope stability models depicting the corresponding factors of safety for selected embankment sections in the 0 m – 20 m and 20 m – 50 m Transition Point Distances as well as the forward slopes extending from the abutment wing walls to the river banks adjacent to the bridge abutments are provided in Figures E1 to E5 in Appendix E. The analyses indicate that the factors of safety will be equal to or greater than the minimum target factors of safety, provided that the embankments are constructed at a minimum side slope geometry of 2.5 Horizontal to 1 Vertical (2.5H:1V).

Material Type	Total Stress Analysis		Effective Stress Analysis		Unit Weight
	ϕ (degrees)	c (kPa)	ϕ' (degrees)	c' (kPa)	γ (kN/m^3)
Embankment Fill (Sand)	30	0	30	0	20
Embankment Fill (Silty Clay)	28	0	28	0	18
Granular A	35	0	35	0	22.8
Granular B Type I	32	0	32	0	21.2
EPS	0	15	0	15	1.0
Overconsolidated Silty Clay/Clay (above Elev.258 m)	0	40	28	0	17
Silty Clay to Clay (Elev. 258 m to 248 m)	0	20	28	0	17
Silty Clay to Clay (below Elev. 248 m)	0	3.64H+20*	28	0	17
Sand	29 to 32	0	29	0	19 to 20
Silty Sand Till	35	0	35	0	21
Design Factors of Safety	1.5 to 2.0		1.5 to 1.7		-

* Refer to Figure D1 in Appendix D. H = clay layer thickness below elevation 248 m for the corresponding design Cu

Material Type	Intermediate Condition				Unit Weight
	ϕ (degrees)	c (kPa)	ϕ' (degrees)	c' (kPa)	γ (kN/m³)
Embankment Fill (Sand)	-	-	30	0	20
Embankment Fill (Silty Clay)	-	-	28	0	18
Granular A	35	0	35	0	22.8
Granular B Type I	32	0	32	0	21.2
EPS	0	15	0	15	1.0
Overconsolidated Silty Clay/Clay (above Elev.258 m)	-	-	28	0	17
Silty Clay to Clay (Elev. 258 m to 248 m)	0	20	-	-	17
Silty Clay to Clay (below Elev. 248 m)	0	3.64H+20*	-	-	17
Sand	-	-	29	0	19 to 20
Silty Sand Till	-	-	35	0	21
Design Factors of Safety	1.5 to 1.7				-

* Refer to Figure D1 in Appendix D. H = clay layer thickness below elevation 248 m for the corresponding design Cu

The permanent forward slopes at the new bridge abutments were also assessed for stability taking into consideration the river's static water level of 260.1 m and a ground water level of 261.0 m. The analyses at the north east, north west, south east and south west quadrants of the approach embankments yielded factors of safety that were either equal to or greater than the target factors of safety of 1.3 (short term) and 1.5 (long term) provided that the forward slope geometry is constructed at 2H:1V or flatter.

The forward and side slopes at the south approach embankments including the RSS Wall were also assessed for the preload condition. The analysis indicates that the target factor of safety will be equal to or greater than 1.3 provided that the side slope geometry is constructed at 2H:1V or flatter.

6.9.4 Stability (Seismic Conditions)

Under earthquake conditions, embankment stability can be assessed using conventional pseudo-static methods of slope stability analysis under the earthquake-induced peak ground acceleration. A calculated factor of safety of 1.1 to 1.3 indicates that the slope is considered to be generally stable and meets the seismic design requirements. A calculated factor of unity or less does not necessarily indicate full-scale slope failure because the soil mass is subjected to the peak load in a given direction for only a fraction of a second.

Because soil slopes are not rigid and the peak acceleration generated during an earthquake lasts for only a very short period of time, seismic coefficients used in practice generally correspond to acceleration values well below the predicted peak accelerations.

For a 2 % probability of exceedance in 50 years, the derived site-specific peak ground acceleration (PGA) is 0.192g consistent with Site Class E. The horizontal and vertical seismic coefficients shall not be less than one-half of the corresponding peak ground acceleration resulting in a design seismic coefficient value of 0.096 i.e. 50% of the site-specific PGA.

The pore water pressure in the subsurface soils will increase under earthquake conditions. In granular, cohesionless deposits the pore water pressures are expected to dissipate very quickly due to the soils relatively high permeability, and the effective stress parameters of these soils were used for the pseudo-static analyses. For silty clay to clay soils however, total stress parameters were used for the pseudo-static analysis to account for excess pore water pressures generated during earthquake conditions.

Pseudo-static seismic slope stability analyses carried out on embankments with a 2.5H:1V side slope geometry indicate that the embankments will have factors of safety equal to or greater than 1.0. The forward slopes were also analysed for seismic events and factors of safety of 0.8 and 0.9 were obtained for a 2H:1V side slope geometry. The results of the seismic stability analyses are presented in Appendix E, Figures E6 and E7.

Since the pseudo-static limit equilibrium analyses indicates a factor of safety less than unity, the CHBDC requires an assessment of the permanent slope deformation. The permanent slope deformation was assessed using the Newmark sliding block analysis, and an estimated deformation value of 75 mm was obtained. Shallow sloughing and toe failure could occur during seismic events. This sloughing and toe failure is expected to be limited, would not impair the use of the highway, and would mainly be a maintenance issue. The potential for sloughing following seismic events could be reduced by providing well-vegetated side slopes.

6.9.5 South Abutment RSS Global Stability

The global stability of the RSS constructed adjacent to the south bridge abutment is dependent on the characteristics of the RSS, preload, EPS, and the underlying foundation soils. The RSS will be in the form of a rectangular block that extends to a maximum height of 2.0 m± with EPS blocks installed on the RSS. This RSS will have a length of 5 m± measured from the inner face of the bridge abutment. Stability analyses were carried out on the RSS configuration, taking into consideration the following variables:

- RSS base founded at a design elevation of 260.1 ±m;
- The top elevation of the final RSS is horizontal and at an elevation of 262.0 m;
- Reinforcement installed across the entire block width and block length;
- Preload consisting of RSS constructed with Granular A to elevation of 263.5 m with Granular A simultaneously placed adjacent to the sides of the RSS/Granular A wall build-up at a 2H:1V side slope geometry;
- Ground water elevation at 261.0 ±m; and
- Removal of RSS preload to elevation 262.0 m and installation of EPS and concrete slab to pavement design subgrade.

The Morgenstern-Price and Spencer methods for stability analysis were employed and the target factors of safety for temporary and permanent conditions were derived based on the site consequence factor (ψ) and the geotechnical resistance factors (Φ_{gu}) provided in Table 6.2 of the CHBDC. Accordingly, a minimum target factor of safety of 1.3 and 1.5 were established for temporary (short term) and permanent (long term) conditions. The soil parameters used for the slope stability analyses and the factors of safety that were obtained are provided in Section 6.9.2 of this report.

The analysis carried out on the RSS indicates that an RSS block reinforced across its entire block width and block length, will achieve a factor of safety of 1.5 for permanent conditions and will also achieve a factor of safety of 1.3 for short term conditions i.e. during the preload period.

The actual design configuration must be checked for global stability prior to finalization. The internal stability of the RSS wall should be analyzed by the supplier/designer of the proprietary product that is selected.

Since the silty clay to clay soils will be susceptible to loosening/softening and degradation on exposure to water and construction traffic, a geotextile fabric should be installed at the silty clay/RSS subgrade interface to prevent soil migration. The RSS should also be placed on undisturbed subsurface soil.

6.9.6 Embankment Construction

Materials used for embankment construction should be placed in lifts not exceeding 300 mm (before compaction), and each lift should be uniformly compacted to at least 95 % of the material's SPMDD. Embankment construction should be carried out in accordance with OPSS.PROV 209, OPSS.PROV 501 and OPSS.PROV 206. Borrow material shall meet the requirements of OPSS.PROV 212 and bonding between existing fill and new fill shall be carried out by benching in accordance with OPSD 208.010.

Proper erosion control measures shall be implemented both during construction and permanently. Temporary erosion and sediment control must be provided in accordance with OPSS 805 and embankment slopes shall be reinstated with permanent erosion protection in accordance with OPSS 803 and OPSS.PROV 804.

In the 0 m – 20 m Transition Point Distance where excavations are required to elevation 260.0 m±, the silty clay to clay soils will be susceptible to loosening/softening and degradation on exposure to water and construction traffic. Therefore, embankment fill should be placed expeditiously and a non-woven geotextile fabric with a Filtration Opening Size (FOS) of 100 microns shall be installed at the silty clay/embankment fill interface to prevent soil migration. The footprint area of the embankment should also be reinforced with a biaxial geogrid to provide stability and support for construction equipment during fill placement and compaction. A Special Provision for the geogrid is provided in Appendix H.

6.10 Temporary Cuts

It is envisaged that temporary cuts extending to elevation 260.1 m will be required at the abutment locations in order to construct the pile caps and bridge abutments as well as to construct the approach embankments. The global stability of a 4.0 m± high cut perpendicular to the highway was assessed using the commercially available slope stability program Slide 7.0 developed by Rocscience Inc., using the soil properties provided in Section 6.9.2 of this report.

The Morgenstern-Price and Spencer methods for stability analysis were employed and the target factor of safety for temporary conditions were derived based on the site consequence factor (ψ) and the geotechnical resistance factor (Φ_{gu}) provided in Table 6.2 of the CHBDC. Accordingly, a minimum target factor of safety of 1.3 was established for a temporary condition. The analyses indicate that for a 4.0 m± high cut, a side slope geometry of 3H:1V is required to achieve the minimum factor of safety of 1.3. Further analysis by the Contractor's geotechnical consultant will be required to assess the stability of excavations due to construction loads.

6.11 Temporary Protection Systems

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary protection systems should be designed in accordance with OPSS.PROV 539 and the designs should be carried out by a licensed Professional Engineer experienced in shoring design. Based on the subsurface conditions, it is envisaged that an interlocking sheet pile option would be practical and can be used to construct cofferdams in the river.

The shape of the soil pressure distribution diagram behind a temporary protection system depends upon the type of soil to be supported and the amount of movement that can be permitted. The protection system can be restrained, fixed or flexible and the sequence of work will alter the shape of the pressure diagram during the various construction phases.

Earth pressure computations must also take into account the ground water level. Above the ground water level, earth pressure is computed using the bulk unit weight of the retained soil. Below the ground water level, the earth pressures are computed using the submerged unit weight of the soil. A hydrostatic pressure is also applied if the retained soil is not fully drained.

Flexible shoring should be designed on the basis of the active earth pressure coefficient (K_a). In this case, the performance level should be Level 2 – Angular Distortion 1:200 but shall not be more than 25 mm. Where limited shoring movement (Performance Level 1A or 1B) is required the design should be based on the at rest earth pressure coefficient (K_o). For "kick out" design the lateral resistance should be computed on the basis of the passive earth pressure coefficient (K_p). It should be noted that the lateral earth pressure coefficients chosen for design require certain movements for the active and passive conditions to be mobilized.

The appropriate lateral earth pressure parameters for use in the design of structures subject to unbalanced earth pressures are provided in the following table. The active earth pressure coefficients are based on the assumption that the ground surface behind the temporary protection system is horizontal. Where the retained ground is sloping, the lateral earth pressure coefficients must be adjusted to account for the slope and, these earth pressure coefficients can be estimated from the equations provided on Figures C6.17 and C6.18 of the CHBDC 2006. A ground water elevation of 261.0 m± should be used for design.

Stratigraphic Unit	**Friction Angle ϕ (degrees)**	**Unit Weight γ (kN/m^3)**	**Active Earth Pressure Coefficient**	**At - Rest Earth Pressure Coefficient**	**Passive Earth Pressure Coefficient**
			K_a	**K_o**	**K_p**
Fill – Sand	30	20	0.33	0.50	3.00
Fill – Silty Clay	28	18	0.36	0.53	2.77
Silty Clay to Clay	28	17	0.36	0.53	2.77
Sand	30	20	0.33	0.50	3.00
Silty Sand Till	35	21	0.27	0.43	3.69

6.12 Removal of Existing Bridge

The existing bridge shall be removed in order to construct the new structure. This work shall be carried out in accordance with OPSS.PROV 510. Bridge footings shall be removed as outlined in Section 510.07.02.01 of OPSS.PROV 510.

6.13 Soil Corrosivity and Sulphate Test Results

Three soil samples were submitted for soil chemical tests consisting of pH, water soluble sulphate, sulphide, chloride, resistivity and electrical conductivity analyses and the laboratory report is provided in Appendix B. This suite of tested parameters is used for assessing the corrosivity potential of the native soil to ductile iron pipe in accordance with the 10-point soil evaluation procedure described in ANSI/AWWA C105/A21.5 Standard[5]. Based on this soil corrosivity scale, a total of 10 points or more indicates that the soil is corrosive to as-manufactured ductile iron pipe (DIP), and additional corrosion protection measures are recommended. The Corrosivity Indices of the tested samples are reported as 1 and 4.

The water soluble sulphate concentrations of the tested samples were compared to Table 3 of the Canadian Standards Association A23-1-09. The results indicate that the degree of exposure to water soluble sulphate in soil is low i.e. less than the moderate range.

7.0 CLOSURE

This report was prepared by Ms. Sepideh D-Monfared, P.Eng., and Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Principal with Terraprobe. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact, carried out an independent quality control review.

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[5] American Water Works Association (AWWA) C-105 (2005) Standard, "Polyethylene Encasement for Ductile-Iron Pipe Systems" Catalog No. 43105. AWWA Denver, CO.

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Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification For Grading.
OPSS.PROV 209	Construction Specification For Embankments Over Swamps And Compressible Soils.
OPSS.PROV 212	Construction Specification For Earth Borrow.
OPSS.PROV 501	Construction Specification For Compacting.
OPSS.PROV 510	Construction Specification For Removal.
OPSS 511	Construction Specification For Rip-Rap, Rock Protection, And Granular Sheeting.
OPSS.PROV 539	Construction Specification For Temporary Protection Systems.
OPSS 803	Construction Specification For Sodding.
OPSS.PROV 804	Construction Specification For Seed and Cover.
OPSS 805	Construction Specification For Temporary Erosion And Sediment Control Measures.

OPSS 902	Construction Specification For Excavating and Backfilling – Structures.
OPSS 903	Construction Specification For Deep Foundations.
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material.

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching Of Earth Slopes.
OPSD 3090.100	Foundation, Frost Penetration Depths For Northern Ontario
OPSD 3101.150	Walls Abutment Backfill, Minimum Granular Requirement
OPSD 3101.200	Walls, Abutment, Backfill, Rock

TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES

Foundation Element	Pile Foundations (H-Pile Sections)	Pile Foundations (Steel Tube Sections)	Augered Caissons
North and South Abutments	***Advantages:*** ▪ Reliable performance expected. ▪ High geotechnical resistances available by driving piles to refusal or driving piles to bedrock. ▪ Allows for the design of an integral abutment structure. ▪ Shallow excavation depth, reduced excavation volume and reduced dewatering requirements. ***Disadvantages:*** ▪ Construction concerns related to the possibility of piles being obstructed by boulders during driving at the south abutment location. ▪ A working mat is required to support pile driving equipment in areas where weak soils exist.	***Advantages:*** ▪ Reliable performance expected. ▪ High geotechnical resistances available by founding piles on the bedrock but only at the north abutment. ▪ Shallow excavation depth, reduced excavation volume and reduced dewatering requirements. ***Disadvantages:*** ▪ Not recommended for supporting the south bridge abutment because of construction concerns related to installing steel tube piles in the relatively thick silty sand till deposit that contains cobble and boulder inclusions. ▪ Does not allow for the design of an integral abutment bridge. ▪ A working mat is required to support pile driving equipment in areas where weak soils exist.	***Advantages:*** ▪ High geotechnical resistances available by founding caissons on competent soils or bedrock. ▪ Allows for the design of a semi integral abutment. ***Disadvantages:*** ▪ A working mat is required to support caisson equipment in areas where weak soils exist. ▪ Requires a permanent liner to maintain side wall support. ▪ Artesian conditions exist at depth. Attempts at dewatering the caisson excavation and maintaining a sufficiently dry excavation to permit cleaning, inspection and high-quality construction, would be challenging and most likely impractical.
Piers	***Advantages:*** ▪ Reliable performance expected. ▪ High geotechnical resistances available by driving piles to bedrock. ▪ No problems associated with scour. ***Disadvantages:*** ▪ Cofferdam required to facilitate construction if H-piles are used. ▪ Steel piles will require protection from corrosion and a relatively large pile cap will be required. ▪ Pier construction requires significant in-river work.	***Advantages:*** ▪ Reliable performance expected. ▪ High geotechnical resistances available by founding piles on the bedrock. ▪ Cofferdam not required to facilitate steel tube pile installations. ▪ Requires minimal in-river work compared to H-pile installations. ▪ No problems associated with scour. ***Disadvantages:*** ▪ Corrosion must be taken into consideration when selecting the steel tube wall thickness. ▪ Pier construction requires in-river work.	***Advantages:*** ▪ Does not require corrosion protection compared to steel piles. ▪ No problems associated with scour. ***Disadvantages:*** ▪ Pier construction requires significant in-river work. ▪ Requires a permanent liner to maintain side wall support. ▪ Artesian conditions exist at depth. Attempts at dewatering the caisson excavation and maintaining a sufficiently dry excavation to permit cleaning, inspection and high quality construction, would be challenging and most likely impractical.
North and South Abutments and Piers	***Risks/Consequences*** ▪ Very low risk of bearing capacity failure. ▪ Very low risk that total settlement will exceed 25 mm.	***Risks/Consequences*** ▪ Very low risk of bearing capacity failure. ▪ Very low risk that total settlement will exceed 25 mm.	***Risks/Consequences*** ▪ Very low risk of bearing capacity failure. ▪ Very low risk that total settlement will exceed 25 mm. ▪ Artesian conditions will increase the level of construction effort.

TABLE 2
EVALUATION OF EMBANKMENT ALTERNATIVES

Option 1 **Conventional Embankment (SSM)**	**Option 2** **North Approach**	**Option 3** **South Approach**
Advantages: ▪ Reliable performance expected. ***Disadvantages:*** ▪ Time to complete consolidation settlement such that the remaining post construction settlement is equal to or less than 25 mm exceeds the one year construction schedule and requires the highway to remain closed for more than a year. ▪ For a one-year construction schedule this design concept will not satisfy MTOs embankment settlement criteria. ▪ Requires installing an interlocking sheet pile arrangement parallel to the banks of Driftwood River to mitigate global stability failure of the forward slopes. Alternatively, staged construction is required. ▪ Requires implementing a settlement monitoring and instrumentation programme to assess the time rate and magnitude of settlement and to determine the timing for construction operations.	***Advantages:*** ▪ Reliable performance expected. ▪ Settlement can be mitigated to meet the one-year construction schedule. ▪ Negligible stress increase on the compressible soils due to a Granular B/EPS embankment. Therefore, settlement due to secondary compression will be minimal and preloading/surcharging is not required. ▪ This design concept will satisfy MTOs embankment settlement criteria within the 0 m – 20 m Transition Point Distance. ▪ Low construction effort required to install the EPS blocks. ▪ EPS blocks will be placed above the ground water table. Hence, buoyancy is not a concern. ▪ Allows the substructure construction to proceed immediately since settlement will be negligible. ***Disadvantages:*** ▪ EPS cannot be placed until the 7-month surcharge period is complete since the grade raise in the 20 m – 50 m Transition Point Distance needs to be surcharged for 7 months.	***Advantages:*** ▪ Reliable performance expected. ▪ Settlement can be mitigated to meet the one-year construction schedule. ▪ Load imparted on the compressible soils after the preload period will be less than the load applied during the preload period. Therefore, settlement due to secondary compression will be minimal. ▪ This design concept will satisfy MTOs embankment settlement criteria within the 0 m – 20 m Transition Point Distance. ▪ Low construction effort required to install the EPS blocks. ▪ EPS blocks will be placed above the ground water table. Hence, buoyancy is not a concern ▪ Allows the substructure construction to proceed without waiting on settlement to be complete. ***Disadvantages:*** ▪ Additional construction effort and geotextile material required to construct the RSS Granular A core to preload elevation and this material will have to be removed partially after preloading is complete. ▪ Requires implementing a settlement monitoring and instrumentation programme to assess the time rate and magnitude of settlement and to determine the timing for construction operations.
Risks/Consequences ▪ Very high risk that settlement will not be completed within the one-year construction schedule. ▪ High risk of global stability failure of the forward slopes adjacent to the banks of Driftwood River that requires mitigation with a temporary interlocking sheet pile arrangement. ▪ Settlement due to creep (secondary compression) may result in a post construction settlement that exceeds 25 mm over a 20 year design period.	***Risks/Consequences*** ▪ Risk that settlement in the 20 m – 50 m Transition Point Distance will not be completed within the 7-month surcharge period. ▪ Low risk of global stability failure of the forward slopes adjacent to the banks of Driftwood River	***Risks/Consequences*** ▪ Risk that settlement will not be completed within the one-year construction schedule is low compared to Option 1. ▪ Low risk of global stability failure of the forward slopes adjacent to the banks of Driftwood River.
Relative Costs ▪ High cost. A large amount of SSM is required including the cost associated with supplying and installing a temporary interlocking sheet pile arrangement.	***Relative Costs*** ▪ Cost anticipated to be higher than Option 1 since there is an additional cost for the supply and installation of EPS blocks. ▪ Overall construction cost may be reduced compared to Option 1 since substructure construction can proceed immediately and settlement is negligible.	***Relative Costs*** ▪ Cost anticipated to be higher than Option 1 since there is an additional cost for the supply and installation of EPS blocks. ▪ Cost anticipated to be higher than Option 2 since a RSS wall is required. ▪ Overall construction cost may be reduced compared to Option 1 since substructure construction can proceed without waiting on settlement to be complete.

DRAWINGS
&
SITE PHOTOGRAPHS

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN

CONT: WP 5104-18-01

DRIFTWOOD RIVER BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET 16

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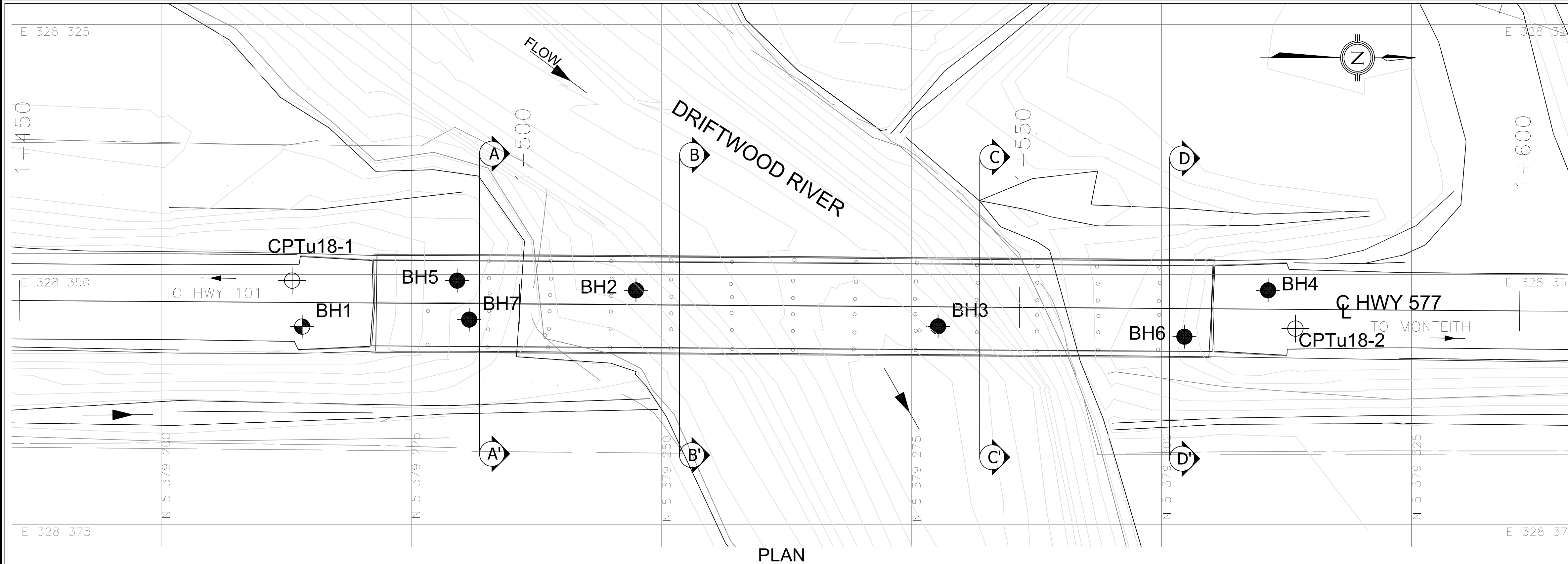
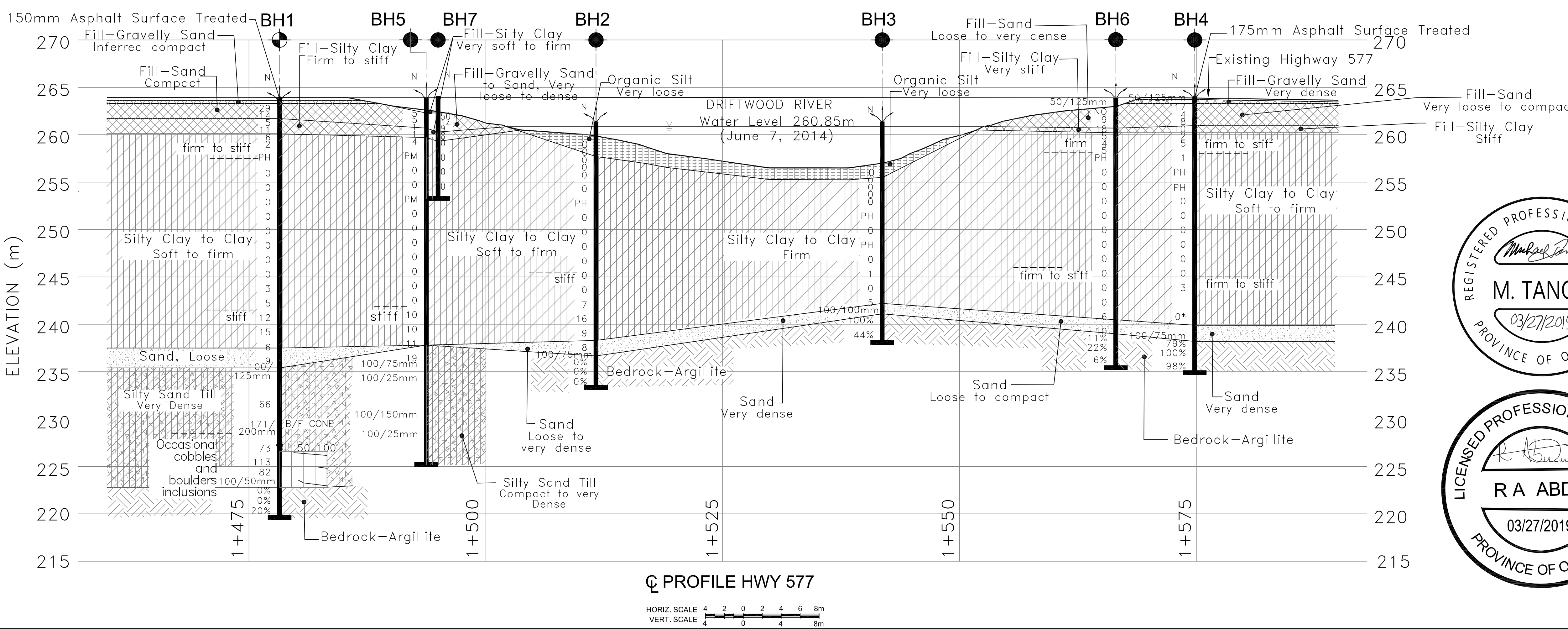


PLAN



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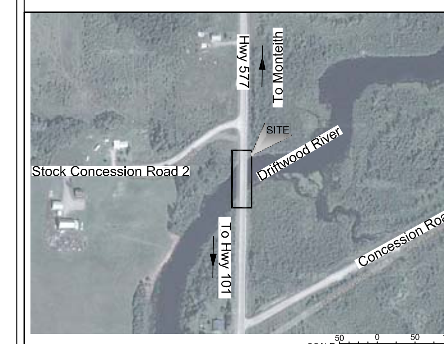

KEY PLAN

LEGEND

- Bore Hole
- Cone Penetration Test
- Bore Hole And 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
- WL in Piezometer
- Piezometer
- 90% Rock Quality Designation
- A/R Auger Refusal
- Artesian Condition

No	ELEV. (m)	COORDINATES (MTM, Zone 12)	
		NORTHING (m)	EASTING (m)
1	263.8	5 379 214.1	328 355.2
2	261.4	5 379 247.5	328 351.6
3	261.4	5 379 277.7	328 355.2
4	263.9	5 379 310.7	328 351.6
5	263.9	5 379 229.6	328 350.6
6	263.9	5 379 302.3	328 356.2
7	264.1	5 379 230.8	328 354.5
CPTu 18-1	263.9	5 379 213.1	328 350.6
CPTu 18-2	263.9	5 379 313.4	328 355.4

NOTE

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REGISTERED PROFESSIONAL ENGINEER
M. TANOS
03/27/2019
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R A ABDUL
03/27/2019
PROVINCE OF ONTARIO

REFERENCE

Drawings provided in digital format by MMM Group Ltd. (Assignment 5013-E-0018 Preliminary Design for Rehab/Replacement of 12 Structures on Highways in New Liskeard Area) drawing files B8450577001, DTM8450577001, received September 11, 2014
And drawings provided in digital format by WSP via email on Nov. 26, 2017.

REVISIONS

DATE	BY	DESCRIPTION

HWY. 577	PROJECT No. 1-18-0689		GEOCRES No.: 42A-124
SUBM'D. SD	CHKD. RA	DATE: March 2019	SITE: 39E-096
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 1

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN

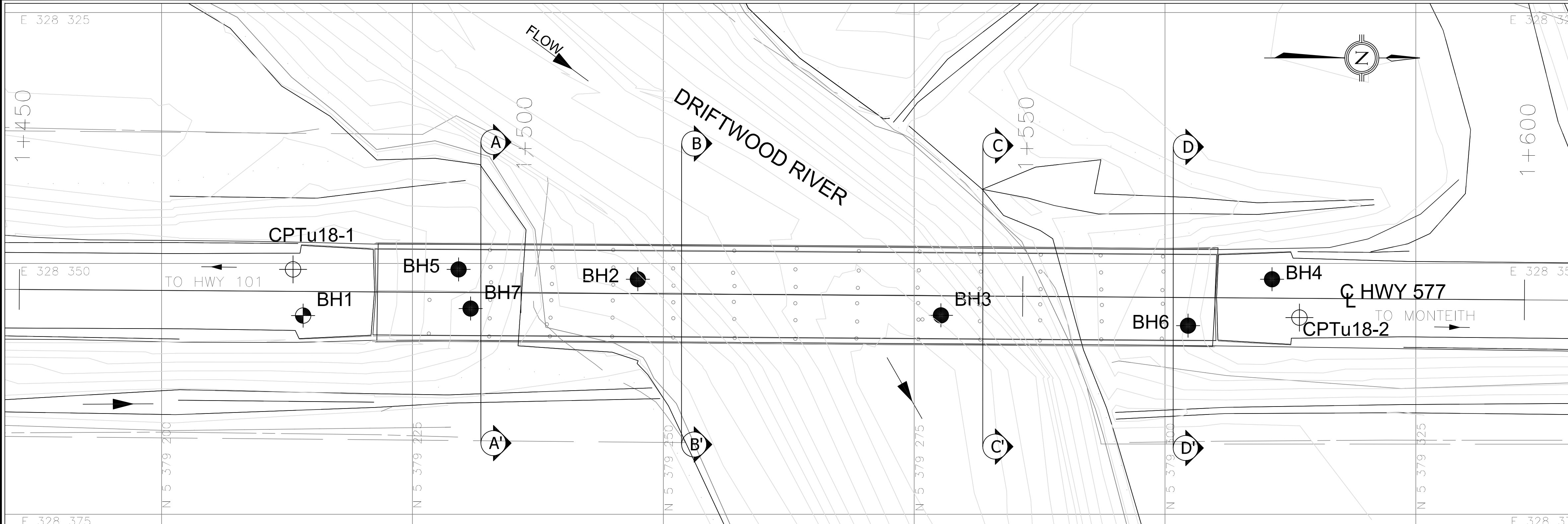
CONT:
WP 5104–18–01

DRIFTWOOD RIVER BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

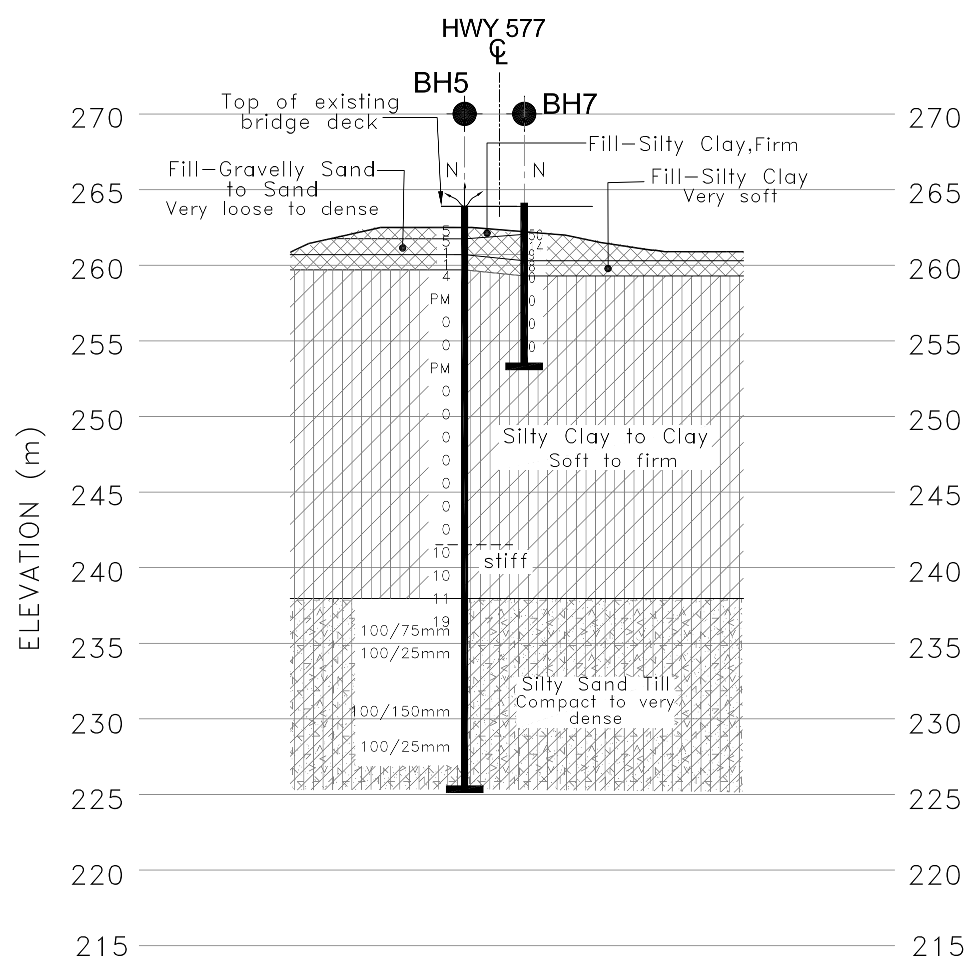
SHEET
17

wsp

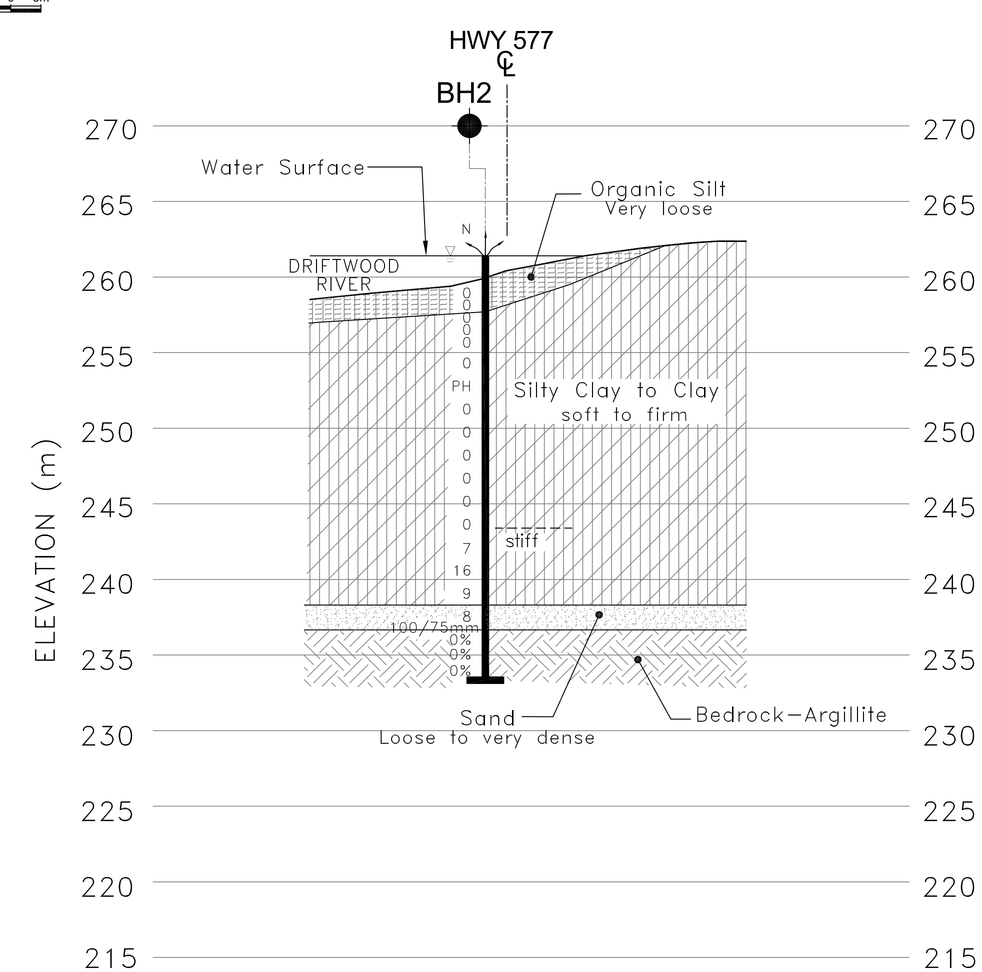
Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650



PLAN



SECTION A-A'

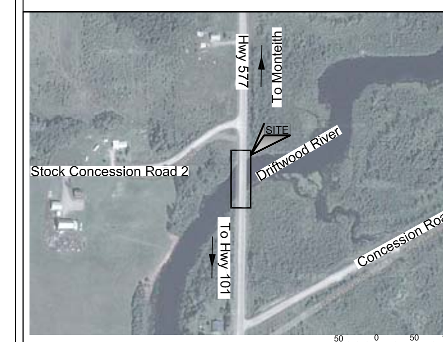


SECTION B-B'

HORIZ. SCALE 4 2 0 2 4 6 8m
VERT. SCALE 4 0 4 8m

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
03/27/2019
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R A ABDUL
03/27/2019
PROVINCE OF ONTARIO



KEY PLAN

LEGEND

	Bore Hole
	Cone Penetration Test
	Bore Hole And 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
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal
	Artesian Condition

No	ELEV. (m)	COORDINATES (MTM, Zone 12)	
		NORTHING (m)	EASTING (m)
1	263.8	5 379 214.1	328 355.2
2	261.4	5 379 247.5	328 351.6
3	261.4	5 379 277.7	328 355.2
4	263.9	5 379 310.7	328 351.6
5	263.9	5 379 229.6	328 350.6
6	263.9	5 379 302.3	328 356.2
7	264.1	5 379 230.8	328 354.5
CPTu 18-1	263.9	5 379 213.1	328 350.6
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REVISIONS	DATE	BY	DESCRIPTION

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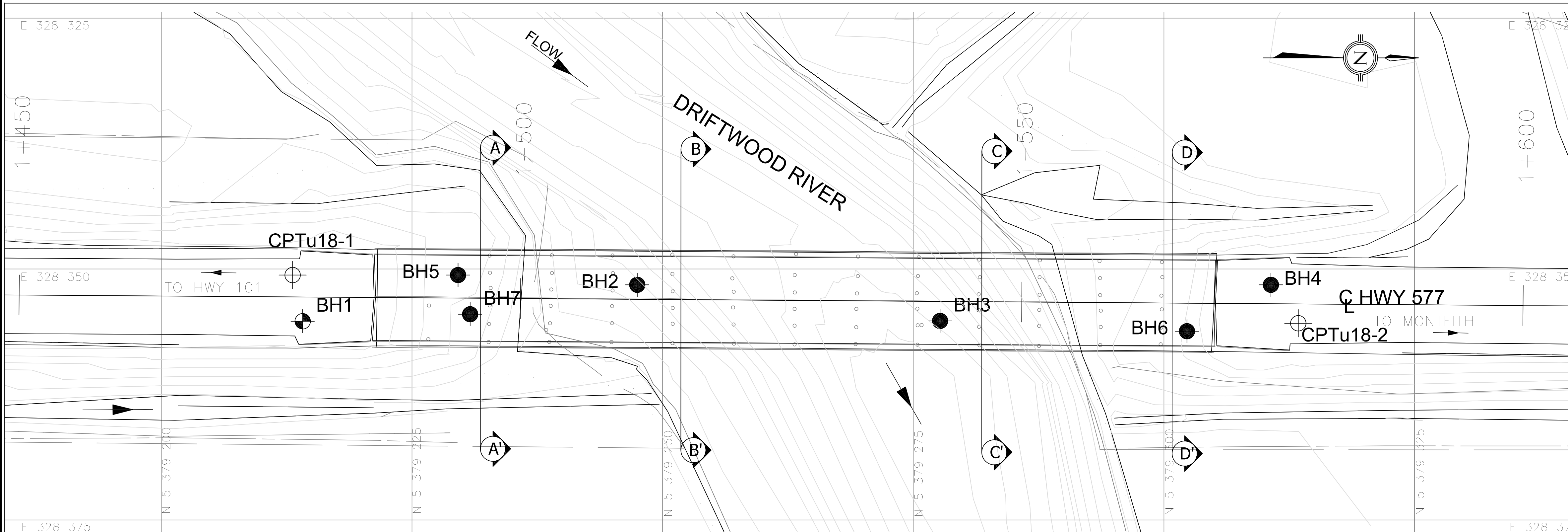
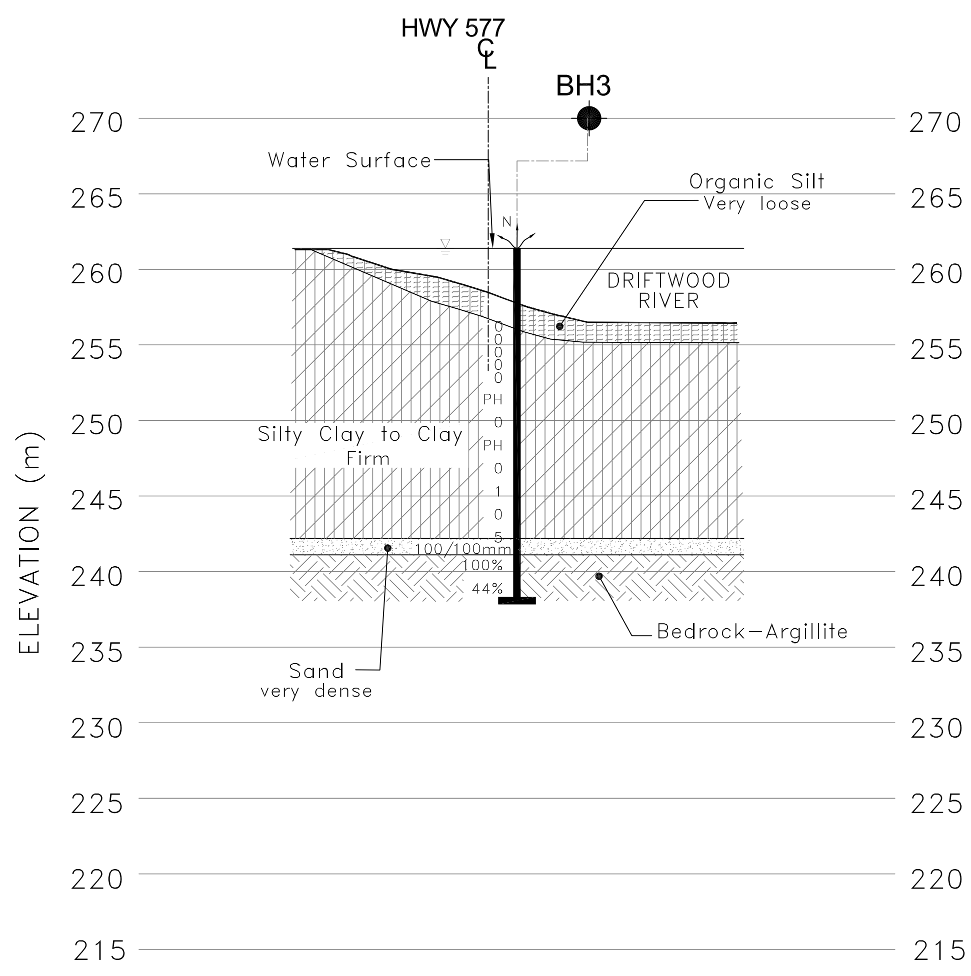
SUBM'D. SD	CHKD. RA	DATE: March 2019	SITE: 39E–096
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 2

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN

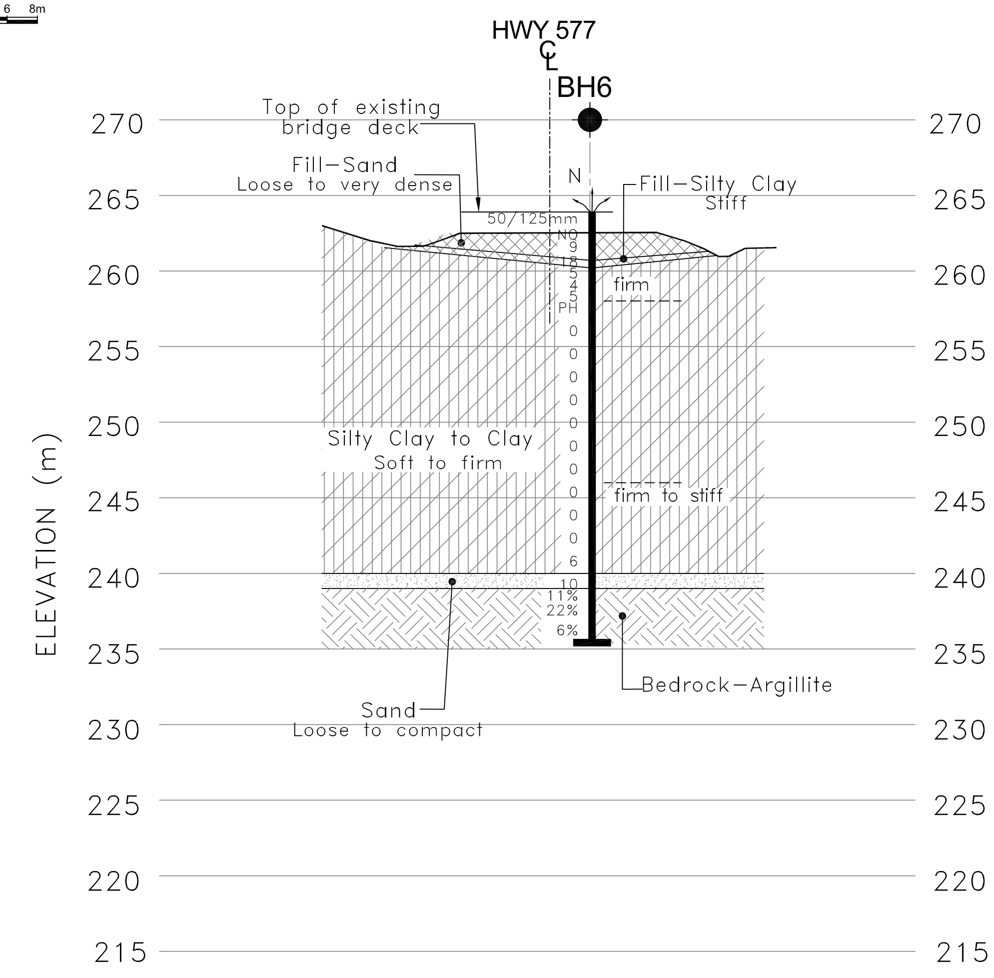


PLAN



SECTION C-C'

HORIZ. SCALE
VERT. SCALE



SECTION D-D'

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
03/27/2019
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R A ABDUL
03/27/2019
PROVINCE OF ONTARIO

CONT:
WP 5104–18–01

DRIFTWOOD RIVER BRIDGE REPLACEMENT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET
18

wsp

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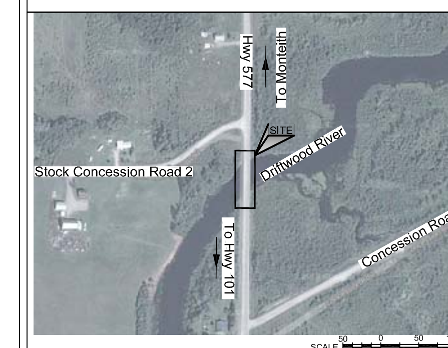

KEY PLAN

	LEGEND
	Bore Hole
	Cone Penetration Test
	Bore Hole And 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
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal
	Artesian Condition

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		NORTHING (m)	EASTING (m)
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REVISIONS	DATE	BY	DESCRIPTION

HWY. 577	PROJECT No. 1–18–0689		GEOCRES No.: 42A–124
SUBM'D. SD	CHKD. RA	DATE: March 2019	SITE: 39E–096
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 3

Drifwood Bridge - Existing Hwy 577 - Looking South - May 27, 2014

Drifwood Bridge - East Side of Highway 577 - Looking North - May 27, 2014

Driftwood Bridge - West Side of Highway 577 - Looking North - May 27, 2014

Driftwood Bridge - East Side of Highway 577 - Looking North - May 27, 2014

Terraprobe Inc. Consulting Geotechnical & Environmental Engineering Construction Materials, Inspection & Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	HWY 577 DRIFTWOOD RIVER BRIDGE, SITE 39E-096		
	G.W.P.: 417-91-00	DATE: February, 2018	
	SUBM'D: SD	CHKD: RA	APPD :MT
	Project No: 1-17-0864	Drawing: 4	

APPENDIX A
Record of Borehole Sheets

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 ú.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60/ CONE ANGLE) DRIVEN BY 475J 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.

<u>CONSISTENCY</u>: 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*

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

N (BLOWS/0.3m)	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.

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

<u>MODIFIED RECOVERY</u>: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY IS:

RQD (%)	0 – 25	25 – 50	50 – 75	75 – 90	90 – 100
	VERY POOR	*POOR*	*FAIR*	*GOOD*	*EXCELLENT*

<u>JOINTING AND BEDDING</u>:

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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	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
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	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 = c_u / τ_r

PHYSICAL PROPERTIES OF SOIL

r_s	kg/m^3	DENSITY OF SOLID PARTICLES	e	1,%	VOID RATIO	e_{min}	1,%	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1,%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
r_w	kg/m^3	DENSITY OF WATER						
γ_w	kN/m^3	UNIT WEIGHT OF WATER	w	1,%	WATER CONTENT	D	mm	GRAIN DIAMETER
r	kg/m^3	DENSITY OF SOIL	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
r_d	kg/m^3	DENSITY OF DRY SOIL	w_P	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	w_S	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
r_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_P	%	PLASTICITY INDEX = $(w_L - w_P)$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_P)/I_P$	i	1	HYDRAULIC GRADIENT
r'	kg/m^3	DENSITY OF SUBMERGED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w)/I_P$	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL	e_{max}	1,%	VOID RATIO IN LOOSEST STATE	j	kN/m^3	SEEPAGE FORCE

Ministry of Transportation Ontario

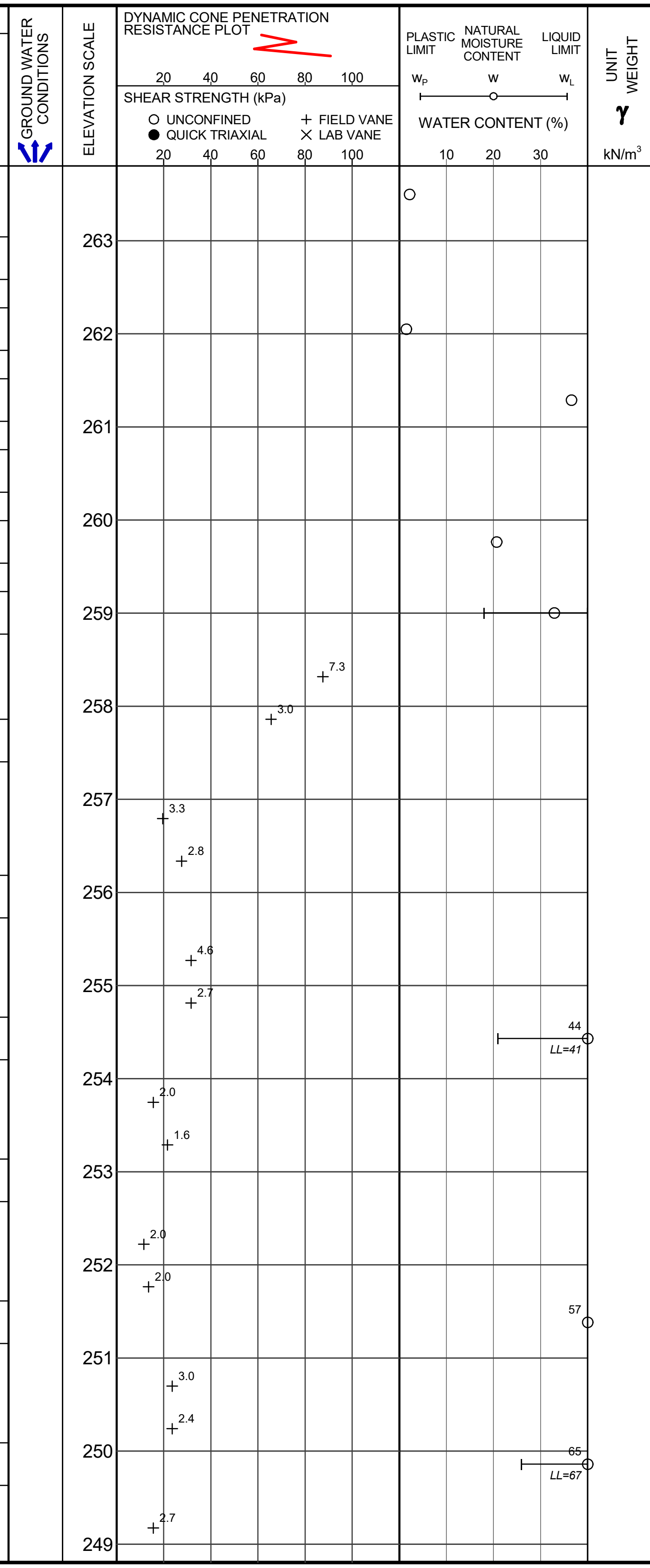
Terraprobe

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328355.2 N:5379214.06		ORIGINATED BY S.M
DIST	HWY 577	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-21 - 2014-7-24		CHECKED BY R.A

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
263.8	**GROUND SURFACE**				
263.6 / 0.2	150mm **ASPH. SURFACE TREATED**				
263.3 / 0.5	350mm **FILL**, gravelly sand, trace silt, inferred compact, brown, dry	1	AS		
	FILL, sand, some gravel, trace silt, compact, brown, dry	2	SS	29	
		3	SS	14	commence casing and wash boring 13 80 (7)
261.7 / 2.1	**FILL**, silty clay, trace sand, trace gravel, containing organics, firm to stiff, dark brown, wet	4	SS	5	2 7 39 52
		5	SS	11	
260.1 / 3.7	**SILTY CLAY TO CLAY** (varved)	6	SS	6	
	trace sand, trace organics, firm to stiff, moist to wet	7	SS	2	0 0 58 42
		8	TW	PH	
	— —				
	SILTY CLAY TO CLAY (varved) containing 1mm to 10mm thick silt layers, soft to firm, grey, wet	9	SS	0*	
		10	SS	0*	0 0 21 79
		11	SS	0*	July 21, 2014 ------------------ July 22, 2014
		12	SS	0*	
		13	SS	0*	0 1 11 88



Continued Next Page

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

file: 11-14-4066 (39e-096) driftwood river bridge - copy.gpj

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328355.2 N:5379214.06	ORIGINATED BY S.M
DIST HWY 577	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-21 - 2014-7-24	CHECKED BY R.A

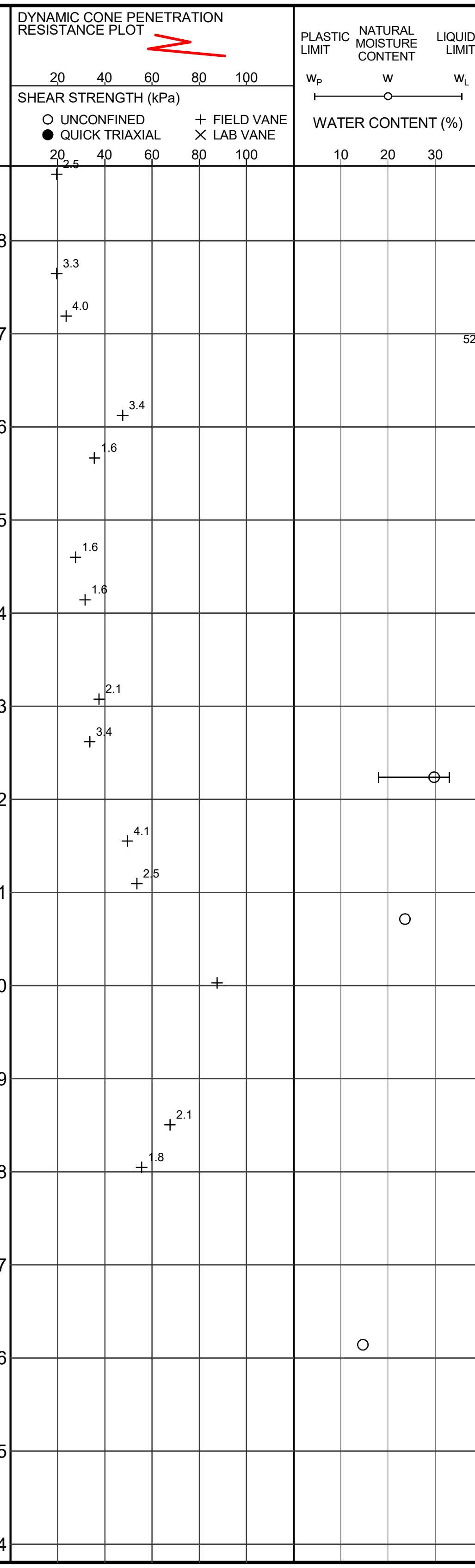
SOIL PROFILE

ELEV DEPTH (m)	DESCRIPTION (continued)
	SILTY CLAY TO CLAY (varved) containing 1mm to 10mm thick silt layers, soft to firm, grey, wet
	— — frequent silt layers, stiff
237.5 / 26.3	**SAND**, trace silt, trace gravel, loose, grey, wet
235.4 / 28.4	**SILTY SAND**, some clay, containing cobbles and boulders, very dense, grey, moist to wet (GLACIAL TILL)

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	Water content / Remarks (GR SA SI CL)
14	SS	0*	
15	SS	0*	w = 52
16	SS	0*	
17	SS	3	
18	SS	5	0 1 51 48
19	SS	12	
20	SS	15	
21	SS	6	
22	SS	9	2 93 (5)
23	SS	100 / 125mm	

Elevation scale: 248, 247, 246, 245, 244, 243, 242, 241, 240, 239, 238, 237, 236, 235, 234



UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

Continued Next Page

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

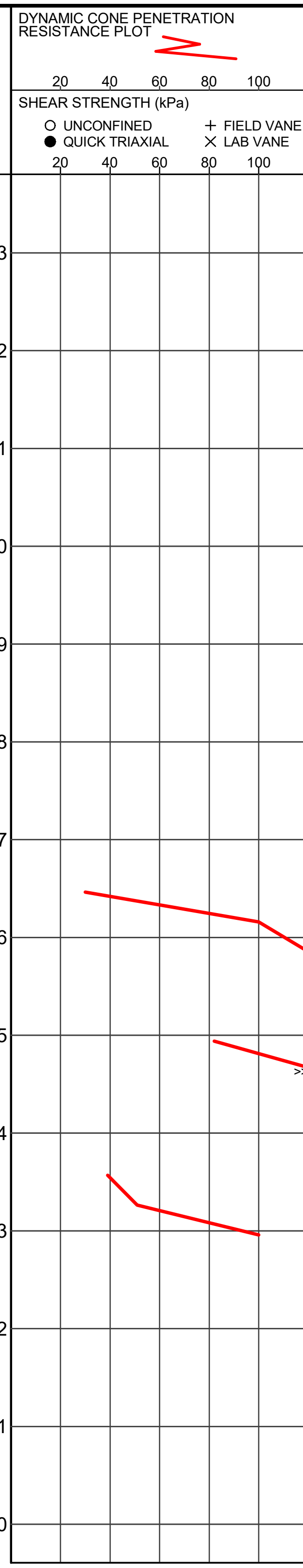
RECORD OF BOREHOLE No 1

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328355.2 N:5379214.06		ORIGINATED BY S.M
DIST	HWY 577	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-21 - 2014-7-24		CHECKED BY R.A

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued) **SILTY SAND**, some clay, containing cobbles and boulders, very dense, grey, moist to wet (GLACIAL TILL)				233		
					232		July 22, 2014 ------------------ July 23, 2014
		24	SS	66		o	0 52 31 17
					231		
					230		
					229		NQ Coring
		25	SS	171 / 200mm			
		26	RC		228	o	
		27	SS	73	227		
					226		
		28	SS	113		o	
					225		
		29	SS	82			
					224		
222.9 40.9	**BEDROCK-ARGILLITE**, slightly weathered, thickly bedded, greenish grey to dark grey, inferred very strong to extremely strong. Fragmented at 40.9m to 41.8m, 42.2m to 42.4m and 43.0m to 43.2m.	30	SS	100 / 50mm	223		July 23, 2014 ------------------ July 24, 2014 NQ Coring
		1	RUN	NQ	222		RUN# 1 TCR=100% SCR=86% RQD=0%
		2	RUN	NQ	221		RUN# 2 TCR=90% SCR=75% RQD=0%
219.6 44.2		3	RUN	NQ	220		RUN# 3 TCR=48% SCR=40% RQD=20%

END OF BOREHOLE



Other column headings: STRAT PLOT; GROUND WATER CONDITIONS; PLASTIC LIMIT W_P; NATURAL MOISTURE CONTENT W; LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

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

O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328355.2 N:5379214.06		ORIGINATED BY S.M
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DATUM GEODETIC	DATE 2014-7-21 - 2014-7-24		CHECKED BY R.A

SOIL PROFILE			SAMPLES									
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O 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 (%) 10 20 30	UNIT WEIGHT γ kN/m^3	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	

*Sampler sinking under weight of hammer and/ or rods.

Dynamic cone penetration test (DCPT) performed from 37.2m to 38.1m, 38.6m to 39.2m and 40.2m to 40.7m.

Hydrostatic uplift encountered at 22.9m and ground water flow observed about 1.5m above ground surface upon completion of drilling. Borehole was sealed/grouted with bentonite slurry mixture.

Insufficient sample available for Atterberg limits test at SS4.

file: 11-14-4066 (39e-096) driftwood river bridge - copy.gpj

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328351.57 N:5379247.46		ORIGINATED BY S.M
DIST	HWY 577	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-28 - 2014-7-30		CHECKED BY R.A

SOIL PROFILE

ELEV DEPTH (m)	DESCRIPTION
261.4	**WATER SURFACE**
	RIVER BED
259.1 2.3	**ORGANIC SILT**, some clay, trace sand, very loose, dark brown, wet
257.7 3.7	**SILTY CLAY TO CLAY** , (varved), containing 1mm to 20mm thick silt layers, soft to firm, grey, wet

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	0*	73 (LL=49)	0 5 74 21
2	SS	0*		
3	SS	0*	69	
4	SS	0*	51	0 0 74 26
5	SS	0*	62	
6	SS	0*		
7	TW	PH		
8	SS	0*	80 (LL=71)	0 0 17 83
9	SS	0*		
10	SS	0*	61	
11	SS	0*		

ELEVATION SCALE: 261, 260, 259, 258, 257, 256, 255, 254, 253, 252, 251, 250, 249, 248, 247

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE — 20 40 60 80 100

Field vane sensitivities (top to bottom): 2.7, 2.9, 2.2, 2.2, 2.3, 4.0, 2.0, 2.3, 3.3, 3.1, 1.6, 2.1

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

REMARKS: July 28, 2014 ------------------ July 29, 2014

Continued Next Page

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

file: 11-14-4066 (39e-096) driftwood river bridge - copy.gpj

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328351.57 N:5379247.46		ORIGINATED BY S.M
DIST	HWY 577	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-28 - 2014-7-30		CHECKED BY R.A

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION (continued)	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	SHEAR STRENGTH (kPa) / FIELD VANE (sensitivity)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY TO CLAY (varved), containing 1mm to 20mm thick silt layers, stiff, grey, wet				246	+ 2.6; + 2.9		
		12	SS	0*	245		60	
					244	+ 2.5; + 2.7		
		13	SS	0*	243	+ 3.6; + 1.9		
		14	SS	7	242			0 0 64 36
					241	+ 3.5; + 2.6		July 29, 2014 ----------------- July 30, 2014
		15	SS	16	240			
		16	SS	9	239			
238.3 23.1	**SAND**, trace silt, trace gravel, loose to very dense, grey, wet				238			
		17	SS	8	237			
		18	SS	100 / 75mm				
236.7 24.7	**BEDROCK-ARGILLITE**, slightly weathered, thickly bedded, greenish grey, inferred very strong to extremely strong. Fragmented from 24.7m to 28.1m.							NQ Coring
		1	RUN	NQ	236			RUN# 1 TCR=33% SCR=20% RQD=0%
		2	RUN	NQ	235			RUN# 2 TCR=39% SCR=10% RQD=0%
		3	RUN	NQ	234			RUN# 3 TCR=61% SCR=13% RQD=0%
233.3 28.1	**END OF BOREHOLE**							

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): ○ UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

UNIT WEIGHT γ kN/m³

*Sampler sinking under weight of hammer and/or rods.

Artesian conditions encountered and water flow observed about 1.4m above bridge deck upon completion of drilling. Borehole was sealed/grouted with bentonite slurry mixture.

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

RECORD OF BOREHOLE No 3

METRIC

G.W.P. 417-91-00 | LOCATION Coords: E:328355.17 N:5379277.66 | ORIGINATED BY S.M

DIST | HWY 577 | BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING | COMPILED BY H.A

DATUM GEODETIC | DATE 2014-7-21 - 2014-7-23 | CHECKED BY R.A

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
261.4	**WATER SURFACE**						
256.2 / 5.2	RIVER BED						
	ORGANIC SILT, some clay, trace sand, very loose, dark brown, wet	1	SS	0*		90	
255.5 / 5.9	**SILTY CLAY to CLAY**, (varved) occasional silt layers up to 10mm thick, firm, grey, wet	2	SS	0*			
		3	SS	0*		70, LL=58	0 0 7 93
		4	SS	0*			
		5	SS	0*		68	
					+ 2.3; + 2.9		
		6	TW	PH			
					+ 1.2; + 1.6		
		7	SS	0*		51, LL=43	0 0 29 71
					+ 4.0; + 1.7		
		8	TW	PH			
					+ 2.1; + 3.1		
		9	SS	0*			0 0 29 71

Continued Next Page

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

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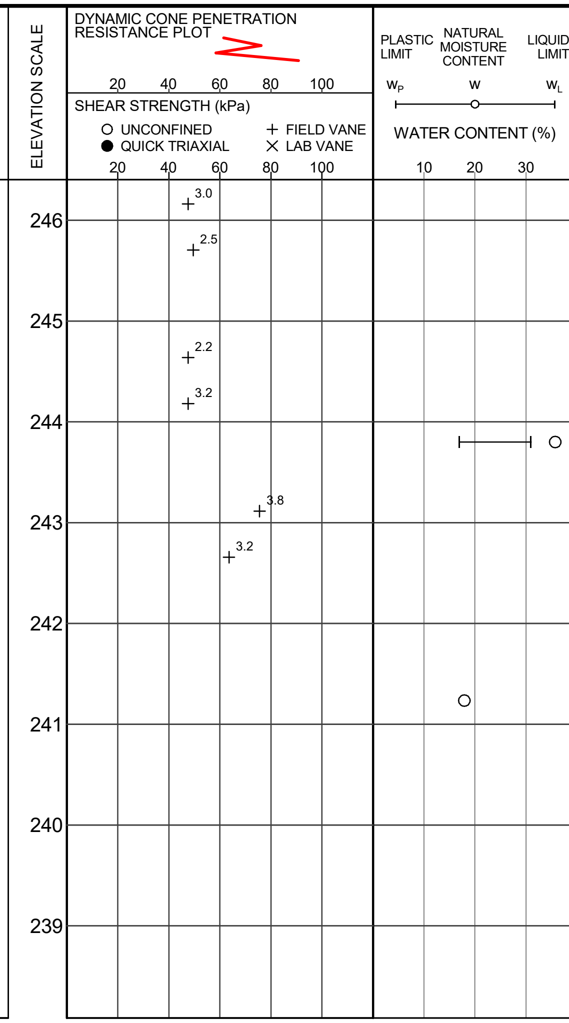
RECORD OF BOREHOLE No 3

METRIC

G.W.P.	417-91-00	LOCATION	Coords: E:328355.17 N:5379277.66	ORIGINATED BY	S.M
DIST		HWY 577	BOREHOLE TYPE: HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY	H.A
DATUM	GEODETIC	DATE	2014-7-21 - 2014-7-23	CHECKED BY	R.A

ELEV DEPTH (m)	SOIL PROFILE DESCRIPTION (continued)	SAMPLES NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY, (varved) occasional silt layers up to 10mm thick, firm, grey, wet	10	SS	1	July 21, 2014 ------------------ July 22, 2014
	containing frequent 20mm to 30mm thick silt layers	11	SS	0*	0 0 54 46
242.2 / 19.2		12	SS	5	
	SAND, trace silt, trace gravel, very dense, grey, wet	13	SS	100 / 100mm	7 89 (4)
241.1 / 20.3	**BEDROCK-ARGILLITE**, unweathered, thickly bedded, dark grey, very strong to extremely strong. Mechanically fragmented below 22.5m.	1	RUN	NQ	NQ Coring RUN# 1 TCR=100% SCR=100% RQD=100% UCS**= 165 - 333 (MPa) July 22, 2014 ------------------ July 23, 2014
		2	RUN	NQ	RUN# 2 TCR=73% SCR=73% RQD=44% UCS**= 186 - 342 (MPa)
238.1 / 23.3					



END OF BOREHOLE

*Sampler sinking under weight of hammer and/or rods.

Artesian conditions encountered at 20.1m and water flow observed about 1.2m above bridge deck upon completion of drilling. Borehole was sealed/grouted with bentonite slurry mixture.

**Uniaxial Compressive Strength determined from Point Load Strength Index values.

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

file: 11-14-4066 (39e-096) driftwood river bridge - copy.gpj

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RECORD OF BOREHOLE No 4

METRIC

G.W.P. 417-91-00 | LOCATION Coords: E:328351.57 N:5379310.66 | ORIGINATED BY S.M

DIST ____ HWY 577 | BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING | COMPILED BY H.A

DATUM GEODETIC | DATE 2014-7-22 - 2014-7-23 | CHECKED BY R.A

SOIL PROFILE

ELEV DEPTH (m)	DESCRIPTION
263.9	GROUND SURFACE
263.7 0.2	175mm **ASPH. SURFACE TREATED**
263.3 0.6	430mm **FILL**, gravelly sand, trace silt, very dense, brown, dry
	FILL, sand, some gravel, trace silt, very loose to compact, brown, wet
261.0 2.9	**FILL**, silty clay, trace sand, trace gravel, containing organics, stiff, dark brown, moist
260.2 3.7	firm to stiff, brown **SILTY CLAY TO CLAY** (varved) soft to firm, grey, wet

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE
1	SS	50 / 125mm
2	SS	17
3	SS	4
4	SS	8
5	SS	10
6	SS	2
7	SS	5
8	SS	1
9	TW	PH
10	TW	PH
11	SS	0*
12	SS	0*
13	SS	0*

GROUND WATER CONDITIONS

ELEVATION SCALE: 263, 262, 261, 260, 259, 258, 257, 256, 255, 254, 253, 252, 251, 250, 249

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH (kPa): O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100

Field vane sensitivities plotted: 4.9, 2.7, 5.6, 4.0, 2.6, 4.0, 2.3, 3.3, 2.7, 2.2, 4.0, 2.5, 3.3

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

WATER CONTENT (%): 10 20 30

Plotted: LL=50; 48, LL=46; 58, LL=56

UNIT WEIGHT γ (kN/m³): 17.0

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL	Remarks
1	21	71	(8)		
3					commence casing and wash boring
6	0	1	53	46	
8	0	0	26	74	
10	0	0	29	71	July 22, 2014 / July 23, 2014
12	0	1	14	85	

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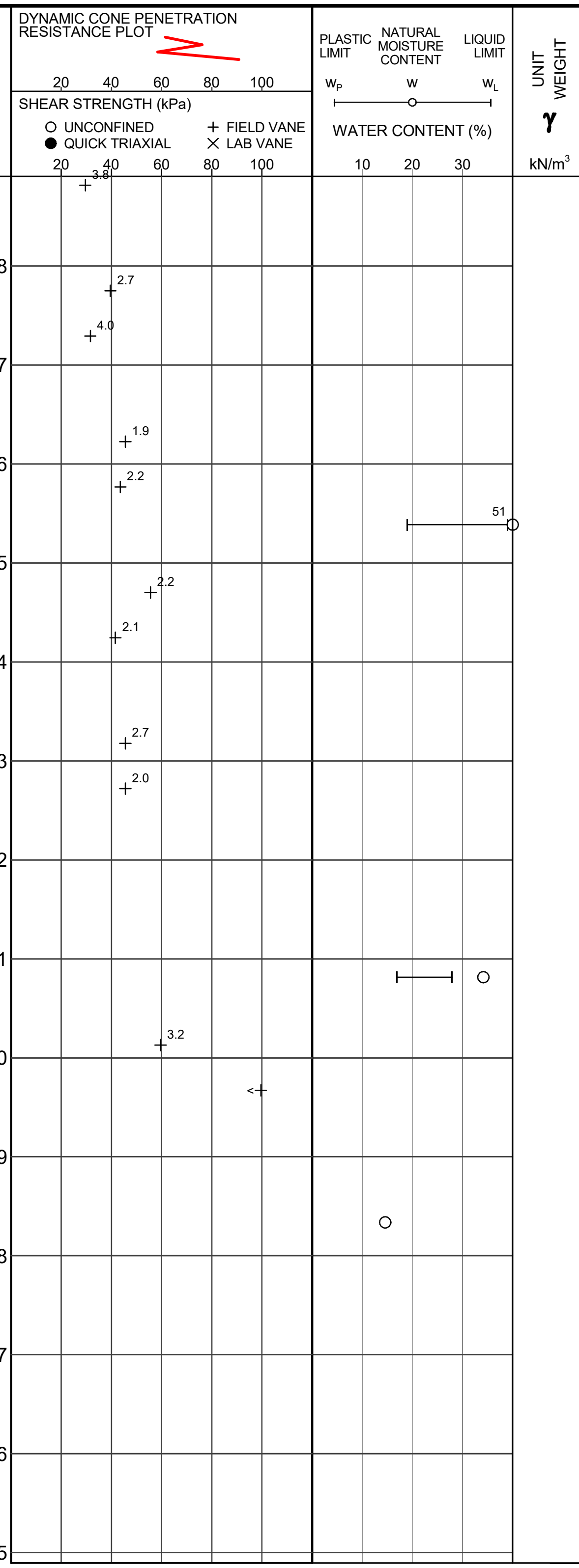
+3, ×3: Numbers refer to Sensitivity   O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328351.57 N:5379310.66		ORIGINATED BY S.M
DIST	HWY 577	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY H.A
DATUM GEODETIC	DATE 2014-7-22 - 2014-7-23		CHECKED BY R.A

ELEV DEPTH (m)	DESCRIPTION (continued)	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY TO CLAY (varved) soft to firm, grey, wet	14	SS	0*	248	
		15	SS	0*	247	
	— — containing silt layers up to 20mm thick, firm to stiff	16	SS	0*	246	0 0 31 69
		17	SS	3	245, 244	
		18	SS	0*	243, 242, 241	0 0 58 42
239.9 24.0	**SAND**, trace silt, trace gravel, very dense, grey, wet				240, 239	
238.2 25.7	**BEDROCK-ARGILLITE**, unweathered, thickly bedded, dark grey, very strong to extremely strong.	19	SS	100/ 75mm		NQ Coring
		1	RUN	NQ	238	RUN #1 TCR=100% SCR=92% RQD=79% UCS**= 343 (MPa)
		2	RUN	NQ	237	RUN #2 TCR=100% SCR=100% RQD=100% UCS**= 114 - 321 (MPa)
		3	RUN	NQ	236, 235	RUN #3 TCR=98% SCR=98% RQD=98% UCS**= 192 - 279 (MPa)
234.9 29.0	**END OF BOREHOLE**					



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

RECORD OF BOREHOLE No 4

METRIC

G.W.P.	417-91-00	LOCATION	Coords: E:328351.57 N:5379310.66	ORIGINATED BY	S.M
DIST	HWY 577	BOREHOLE TYPE	HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING	COMPILED BY	H.A
DATUM	GEODETIC	DATE	2014-7-22 - 2014-7-23	CHECKED BY	R.A

SOIL PROFILE			SAMPLES								
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O 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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

*Sampler sinking under weight of hammer and/or rods.

Consolidation test performed on TW10.

Artesian conditions encountered at 25.6m and water flow observed about 0.9m above ground surface upon completion of drilling. Borehole was sealed/grouted with bentonite slurry mixture.

**Uniaxial Compressive Strength determined from Point Load Strength Index values.

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

RECORD OF BOREHOLE No 5

METRIC

G.W.P. 417-91-00 LOCATION Coords: E:328350.6 N:5379229.6 ORIGINATED BY FY

DIST HWY 577 BOREHOLE TYPE CASING AND WASH BORING / NQ CORING COMPILED BY SD

DATUM GEODETIC DATE 2017-12-5 - 2017-12-10 CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
263.9	**BRIDGE DECK (TOP OF PAVEMENT)**							
					263			
262.6	**GROUND SURFACE**							
1.3	**FILL**, silty clay, some sand to sandy, trace gravel, firm, brown, wet	1	SS	5				
262.3 / 1.6	**FILL**, sand, some gravel, trace silt, very loose to loose, brown, wet	2	SS	5	262			
		3	SS	1	261			11 84 5 0
260.7 / 3.2	**FILL**, silty clay, trace sand, very soft, grey, wet	4	SS	1	260	57		0 5 69 26
259.7 / 4.2	**SILTY CLAY TO CLAY** (varved) containing 1mm to 10mm thick silt layers, soft to firm, grey, wet	5	SS	4	259			
		6	TW	PM	258	41	17.7	0 0 67 33
		7	SS	0*	257 / 256			0 0 66 34
		8	SS	0*	255			
		9	TW	PM	254 / 253	59	18.2	0 0 36 64
		10	SS	0*	252			
		11	SS	0*	251 / 250 / 249	74		

Field vane sensitivities (plotted): 6.0, 2.8, 3.2, 2.7, 5.0, 2.9, 3.2, 2.0, 2.3, 2.8, 2.0, 2.8, 5.0, 3.6

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 (%) 10 20 30

Continued Next Page

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

Ministry of Transportation Ontario

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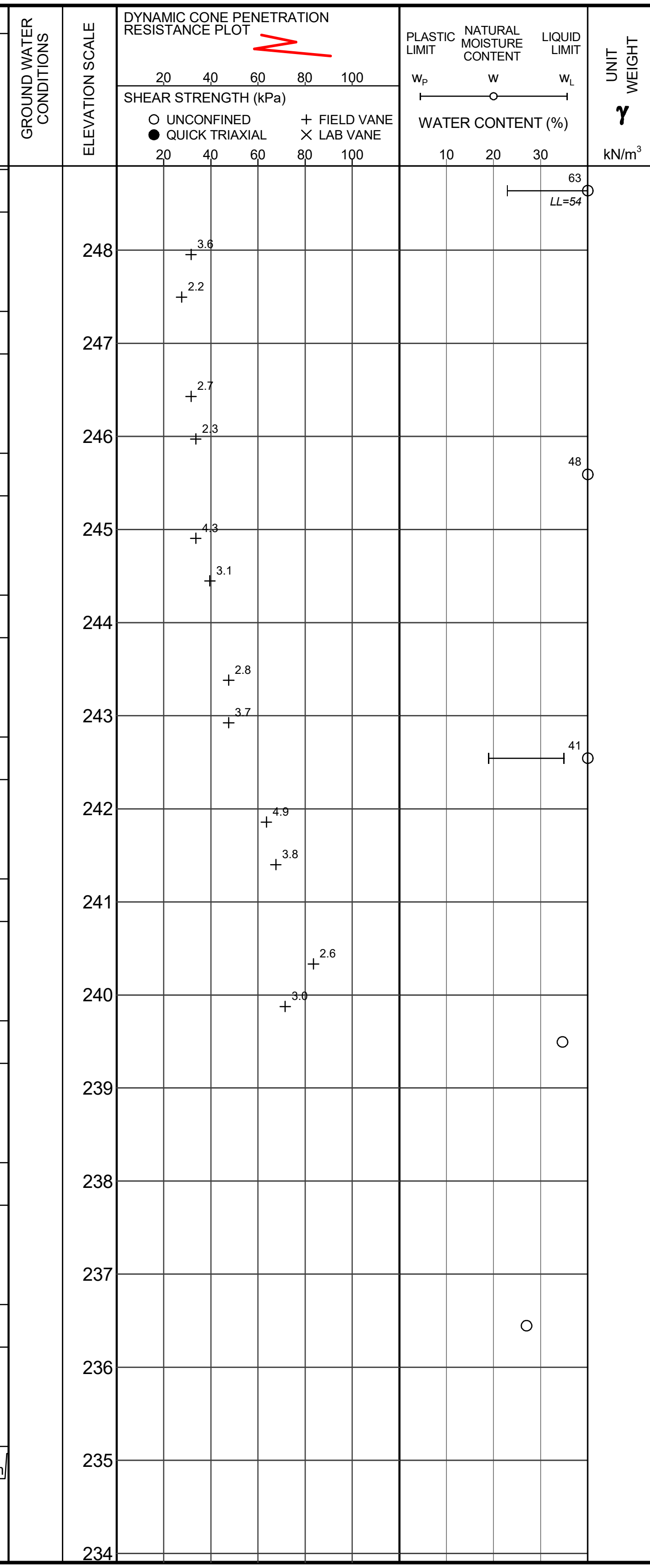
RECORD OF BOREHOLE No 5

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328350.6 N:5379229.6		ORIGINATED BY FY
DIST	HWY 577	BOREHOLE TYPE CASING AND WASH BORING / NQ CORING	COMPILED BY SD
DATUM GEODETIC	DATE 2017-12-5 - 2017-12-10		CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued)					
	SILTY CLAY TO CLAY (varved) containing 1mm to 10mm thick silt layers, soft to firm, grey, wet		12	SS	0*	0 1 15 84
			13	SS	0*	Dec 05, 2017 Dec 07, 2017
			14	SS	0*	
			15	SS	0*	
			16	SS	0*	0 0 45 55
	containing frequent 30mm to 45mm thick silt layers, stiff		17	SS	10	
			18	SS	10	
237.8 26.1			19	SS	11	
	SILTY SAND, some clay, trace gravel, containing cobbles and boulders, compact to very dense, grey, wet (GLACIAL TILL)		20	SS	19	Dec 07, 2017 Dec 08, 2017 0 64 21 15
			21	SS	100 / 75mm	Dec 08, 2017 Dec 09, 2017
			22	RC	NQ	



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+3, ×3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 5

METRIC

G.W.P. 417-91-00 LOCATION Coords: E:328350.6 N:5379229.6 ORIGINATED BY FY

DIST HWY 577 BOREHOLE TYPE CASING AND WASH BORING / NQ CORING COMPILED BY SD

DATUM GEODETIC DATE 2017-12-5 - 2017-12-10 CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION (continued)	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH (kPa)	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY SAND, some clay, trace gravel, containing cobbles and boulders, compact to very dense, grey, wet (GLACIAL TILL)		22	RC	NQ						
			23	SS	100 / 25mm						
			24	RC	NQ		233				
							232				
			25	RC	NQ		231				
			26	SS	100/ 150mm		230				
			27	RC	NQ		229				
							228				
			28	SS	100 / 25mm		227				Dec 09, 2017 Dec 10, 2017
			29	RC	NQ		226				
225.2 38.7											

Legend: DCPT scale 20 40 60 80 100; O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE; shear strength 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; water content 10 20 30.

END OF BOREHOLE

*Sampler sinking under weight of hammer and / or rods.

Hydrostatic uplift encountered at 22.9m and ground water flow observed about 2m above bridge deck upon completion of drilling. Borehole was sealed /grouted with bentonite, slurry mixture.

Consolidation test performed on TW6.

Unable to retrieve core samples from 37.0m to 38.7m below ground surface. Inferred cobbles.

Insufficient sample available for Atterberg Limits test at SS4.

file: 1-17-0864-01_borehole logs.gpj

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

Ministry of Transportation Ontario

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RECORD OF BOREHOLE No 6

METRIC

G.W.P. 417-91-00 | LOCATION Coords: E:328356.2 N:5379302.3 | ORIGINATED BY FY

DIST | HWY 577 | BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING / NQ CORING | COMPILED BY SD

DATUM GEODETIC | DATE 2017-12-18 - 2017-12-20 | CHECKED BY RA

SOIL PROFILE

ELEV DEPTH (m)	DESCRIPTION
263.9	BRIDGE DECK (TOP OF PAVEMENT)
262.8	**GROUND SURFACE**
1.1	**FILL**, sand, some gravel to gravelly, trace silt, containing cobbles and boulders, loose to very dense, brown, wet
260.7 3.2	**FILL**, silty clay, trace sand, very stiff, brown, wet
260.2 3.7	**SILTY CLAY TO CLAY** (varved) firm, brown containing 1mm to 5mm thick silt layers, soft to firm, grey, wet

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE
1	SS	100 / 125mm
2	RC	NQ
3	SS	9
4	SS	18
5	SS	5
6	SS	4
7	SS	5
8	TW	PH
9	SS	0*
10	SS	0*
11	SS	0*
12	SS	0*
13	SS	0*

GROUND WATER CONDITIONS

ELEVATION SCALE: 263, 262, 261, 260, 259, 258, 257, 256, 255, 254, 253, 252, 251, 250, 249

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

Field vane sensitivities (top to bottom): 8.4, 5.0, 2.4, 2.3, 2.3, 4.3, 2.9, 4.8, 2.2, 2.9, 1.9, 5.3

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

WATER CONTENT (%): 10 20 30

Plotted values: 48, LL=45; 75, LL=51; 61, LL=56

UNIT WEIGHT γ kN/m³

REMARKS & GRAIN SIZE DISTRIBUTION (%)

GR SA SI CL

- commence casing and wash boring (at sample 2)
- 0 0 67 33 (sample 6)
- Dec 18, 2017 / Dec 19, 2017 — 0 0 26 74 (sample 9)
- 0 0 10 90 (sample 11)
- 0 0 10 90 (sample 13)

Continued Next Page

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

file: 1-17-0864-01_borehole logs.gpj

RECORD OF BOREHOLE No 6

METRIC

G.W.P. 417-91-00	LOCATION Coords: E:328356.2 N:5379302.3		ORIGINATED BY FY
DIST	HWY 577	BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING / NQ CORING	COMPILED BY SD
DATUM GEODETIC	DATE 2017-12-18 - 2017-12-20		CHECKED BY RA

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100) | SHEAR STRENGTH (kPa): O UNCONFINED, ● QUICK TRIAXIAL, + FIELD VANE, X LAB VANE (20, 40, 60, 80, 100) | PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L – WATER CONTENT (%) (10, 20, 30) | UNIT WEIGHT γ kN/m³ | REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued) **SILTY CLAY TO CLAY** (varved) containing 1mm to 5mm thick silt layers, soft to firm, grey, wet	14	SS	0*	248	
		15	SS	0*	247	
	— —	16	SS	0*	246, 245	
	containing frequent 20mm to 50mm thick silt layers, firm to stiff	17	SS	0*	244	0 0 56 44
		18	SS	0*	243, 242	
		19	SS	6	241	
240.0 / 23.9	**SAND**, some silt, trace clay, loose to compact, grey, wet	20	SS	10	240	0 80 17 3
239.0 / 24.9	**BEDROCK - ARGILLITE** unweathered, thinly to medium bedded, dark grey, very strong to extremely strong, fragmented from 26.5m to 26.7m	1	RUN	NQ	239, 238	Dec 19, 2017 / Dec 20, 2017 Run #1 TCR: 100% SCR: 47% RQD: 11% UCS**=114-192 (MPa)
		2	RUN	NQ	237	Run #2 TCR: 90% SCR: 44% RQD: 22% UCS**=236-284 (MPa)
235.4 / 28.5		3	RUN	NQ	236	Run #3 TCR: 65% SCR: 9% RQD: 6%

END OF BOREHOLE

Field vane sensitivities (top to bottom): 3.8, 4.9, 3.8, 3.0, 3.1, 3.1, 2.9, 2.9, 3.3, 3.1, 3.7, 3.8

Water content labels: 57, 40

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

RECORD OF BOREHOLE No 6

METRIC

G.W.P. 417-91-00 LOCATION Coords: E:328356.2 N:5379302.3 ORIGINATED BY FY

DIST HWY 577 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING / NQ CORING COMPILED BY SD

DATUM GEODETIC DATE 2017-12-18 - 2017-12-20 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) O 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 (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	**DESCRIPTION** **BRIDGE DECK (TOP OF PAVEMENT)**	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE						

END OF BOREHOLE

*Sampler sinking under weight of hammer and / or rods.

**Uniaxial Compressive Strength determined from Point Load Strength Index values.

Artesian conditions encountered at 24.4m and ground water flow observed about 2.4m above bridge deck upon completion of drilling. Borehole was sealed /grouted with bentonite, slurry mixture.

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 7

METRIC

G.W.P. 5104-18-00 LOCATION Coords: E:328354.5 N:5379230.8 ORIGINATED BY FY

DIST HWY 577 BOREHOLE TYPE CASING AND WASH BORING COMPILED BY SD

DATUM GEODETIC DATE 2018-11-21 - 2018-11-23 CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	WATER CONTENT / PLASTIC–LIQUID LIMIT	SHEAR STRENGTH (FIELD VANE, sensitivity)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
264.1	**BRIDGE DECK (TOP OF PAVEMENT)**						
262.3 / 1.8	**FILL**, gravelly sand to sand, trace to some silt, trace to some clay, loose to dense, brown, moist to wet	1	SS	50			
		2	SS	14			
		3	SS	9			
260.3 / 3.8	**FILL**, silty clay, trace sand, firm to stiff, grey, wet	4	SS	8			
259.3 / 4.8	Stiff **SILTY CLAY TO CLAY** (varved) containing 5mm to 20mm thick silt layers, soft to firm, grey, wet	5	SS	0		6.5; 3.0	
		6	SS	0		2.7; 2.7	0 0 58 42
		7	SS	0		3.8; 3.3	0 0 60 40
		8	SS	0		4.0; 2.0	0 1 66 33 Nov. 21, 2018 Nov. 23, 2018
253.3 / 10.8	**END OF BOREHOLE**						

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 (%) 10 20 30

UNIT WEIGHT γ kN/m^3

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

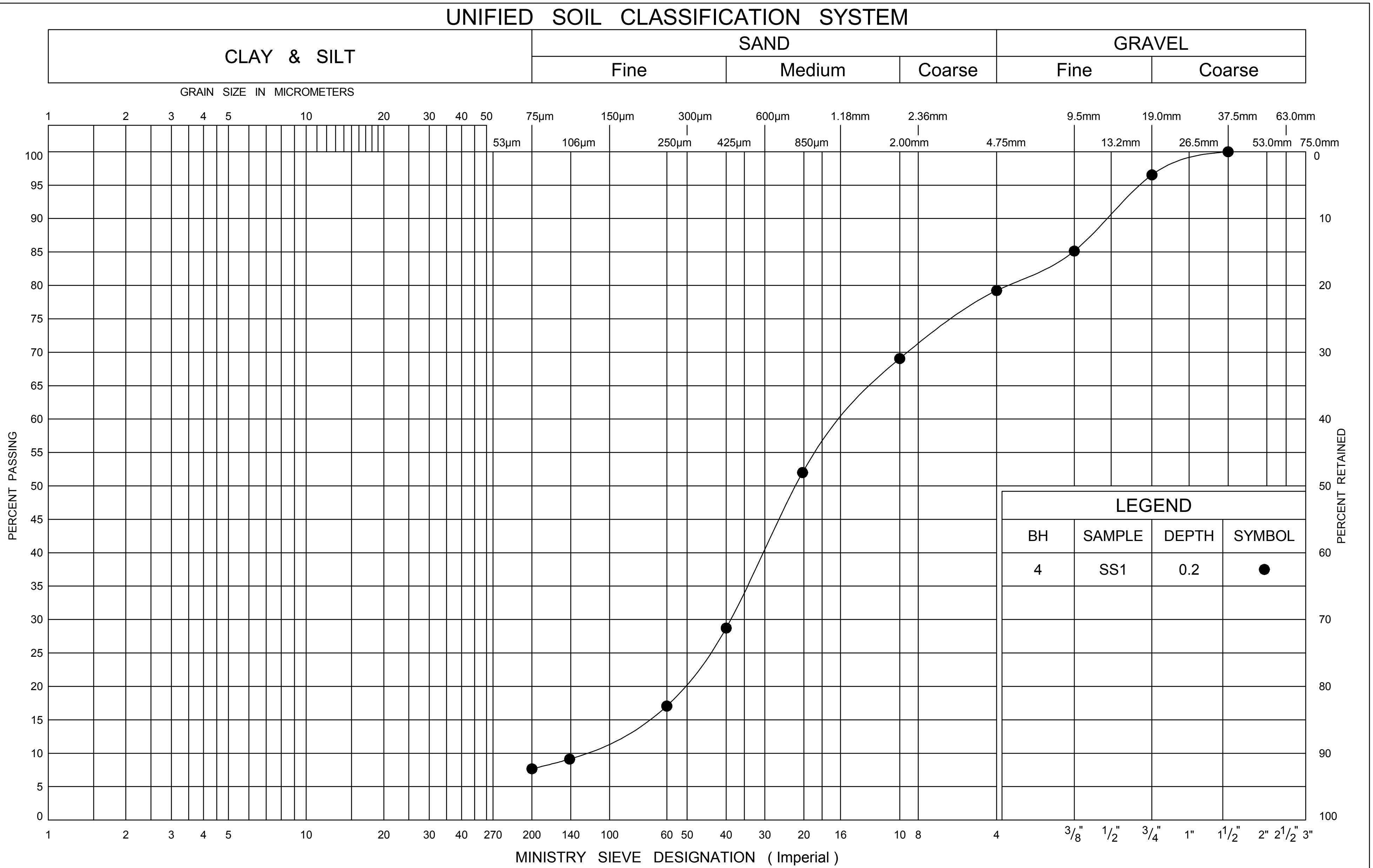
APPENDIX B

Field & Laboratory Test Results

&

Photographs

Terraprobe



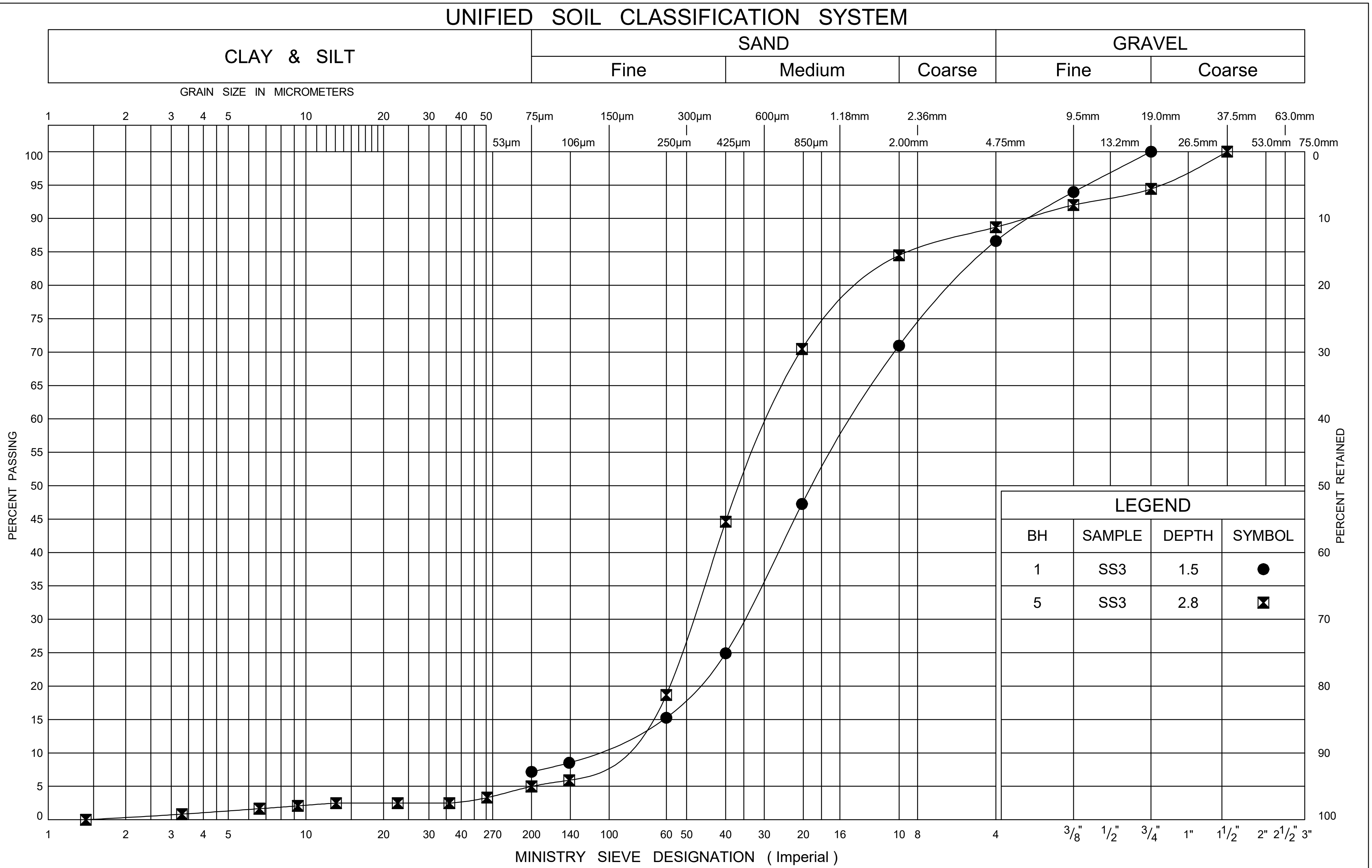
Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
FILL-GRAVELLY SAND

FIG No B1

G W P 5104-18-00

Driftwood River Bridge Site 39E-096



Ministry of Transportation

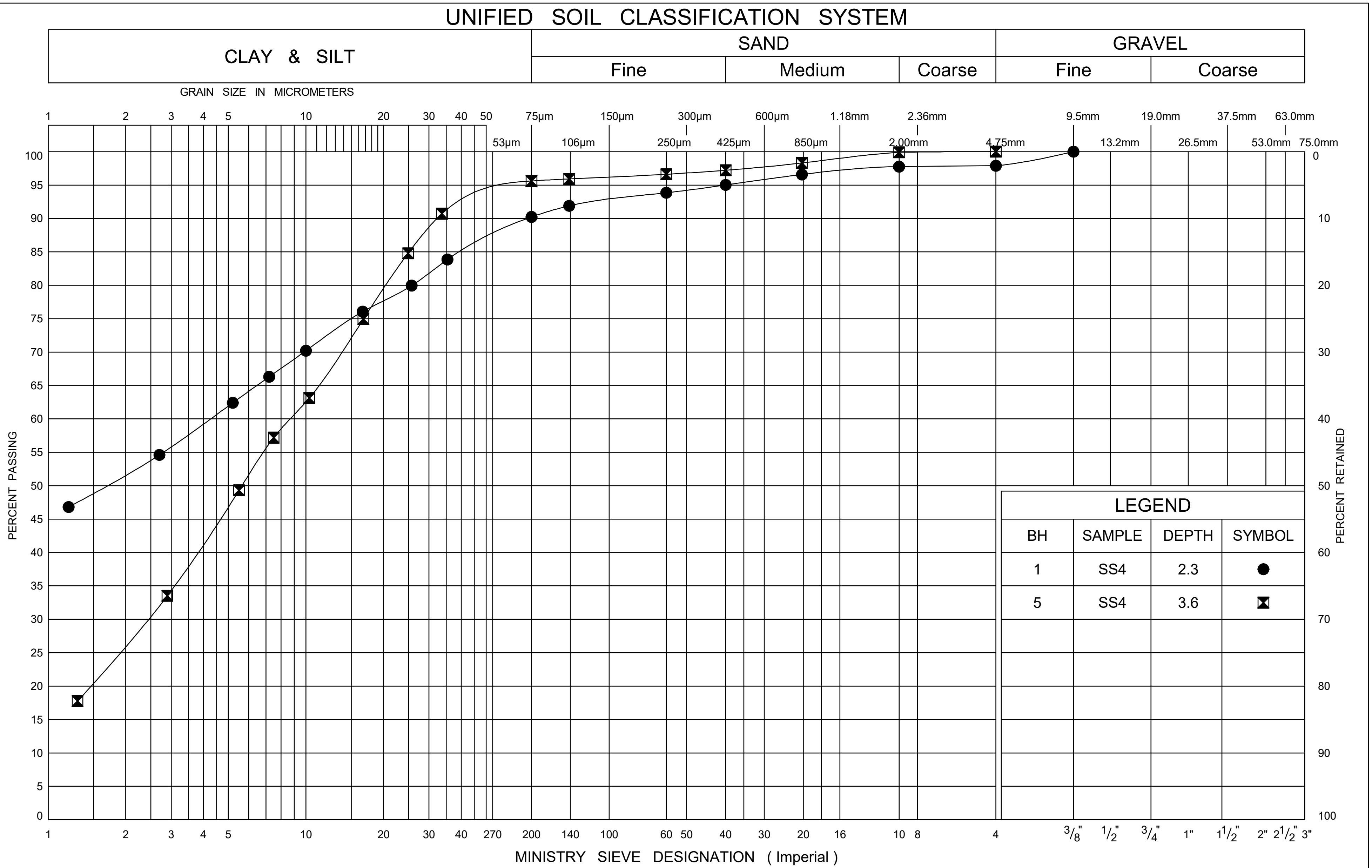
Ontario

GRAIN SIZE DISTRIBUTION

FILL - SAND

FIG No	B2
G W P	5104-18-00
Driftwood River Bridge Site 39E-096	

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

FILL - SILTY CLAY

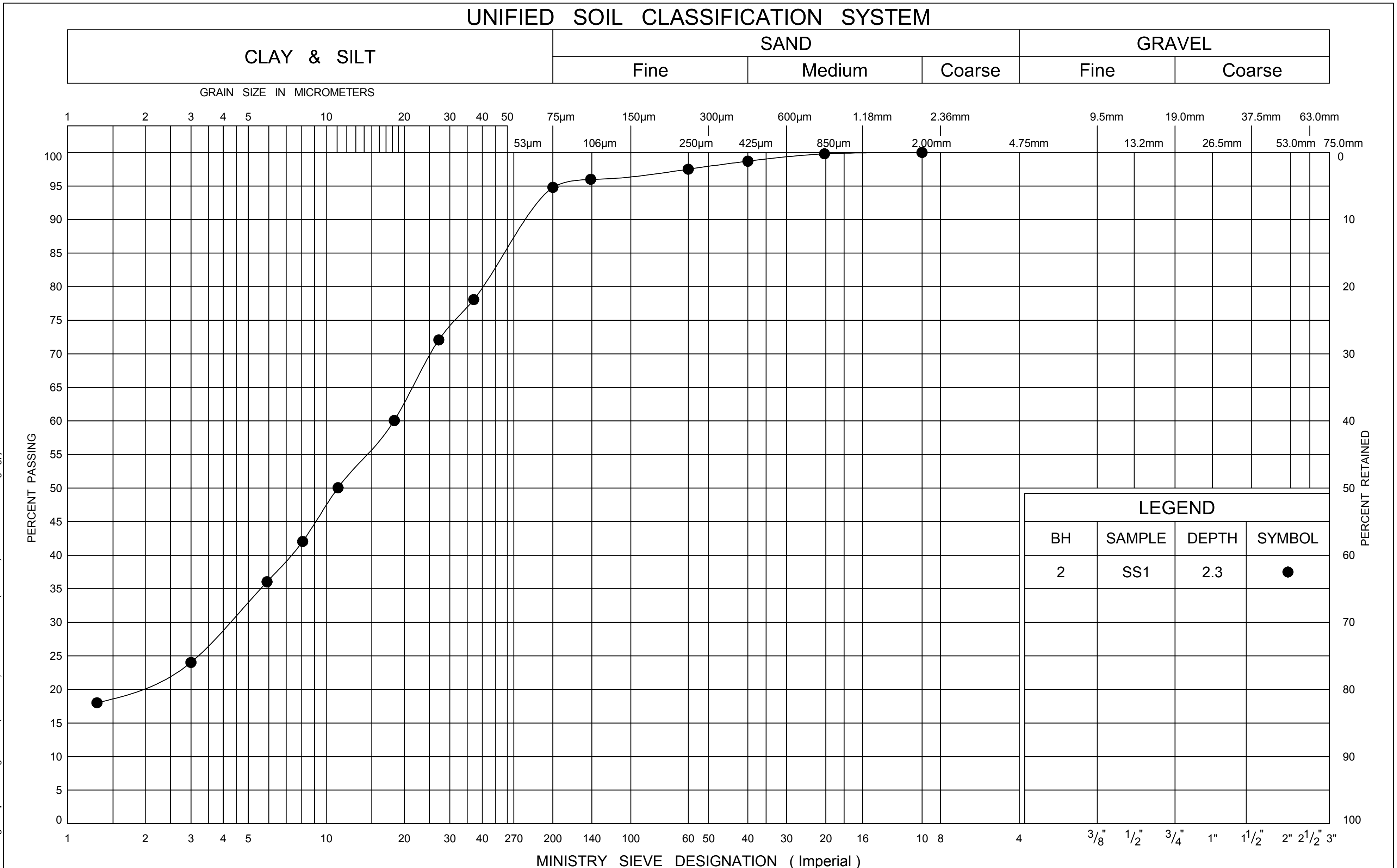
FIG No B3

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj

Terraprobe



Ministry of Transportation
Ontario

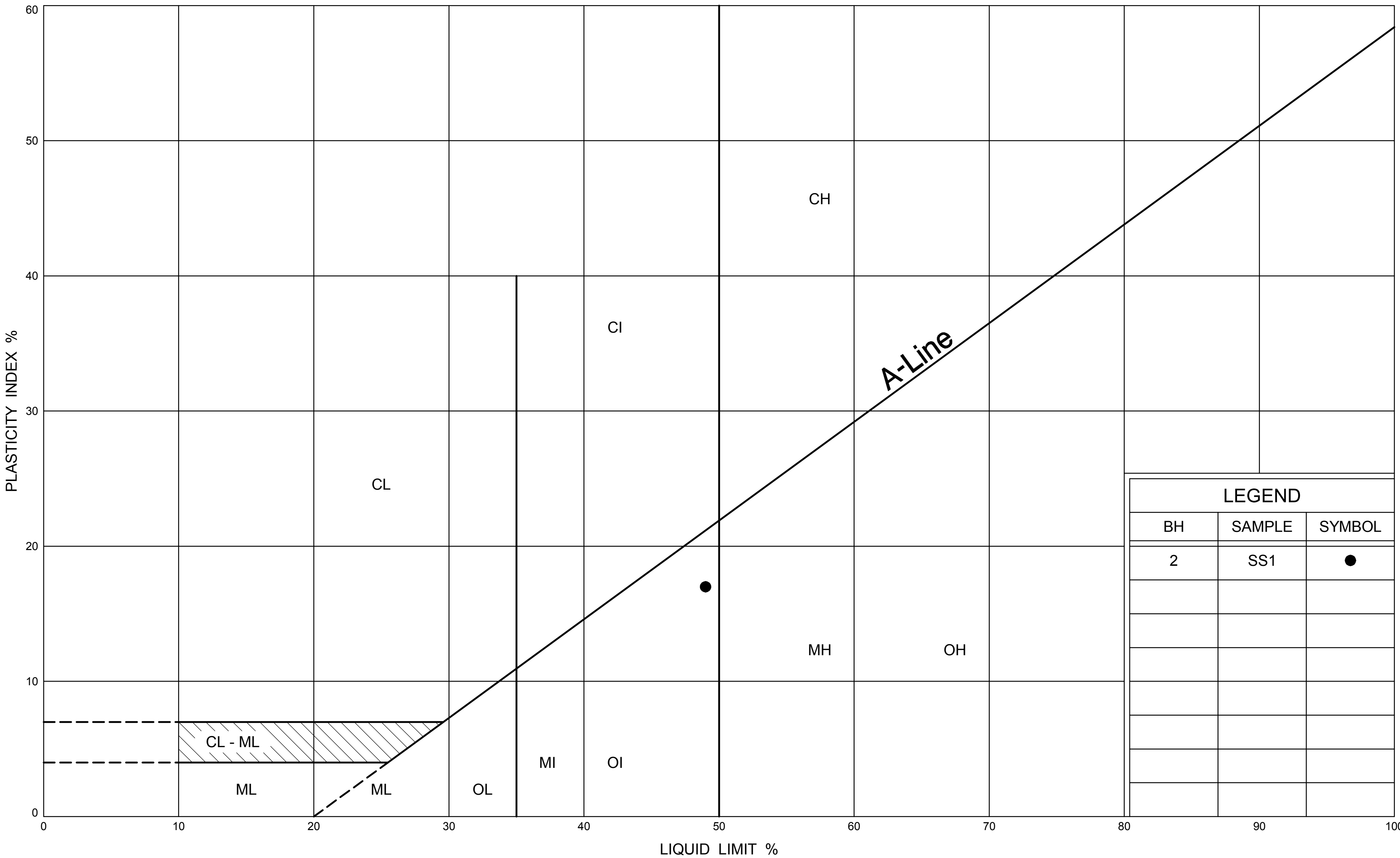
GRAIN SIZE DISTRIBUTION

ORGANIC SILT

FIG No B4

G W P 5104-18-00

Driftwood River Bridge Site 39E-096



LEGEND

BH	SAMPLE	SYMBOL
2	SS1	●

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PLASTICITY CHART

ORGANIC SILT

FIG No B5

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

VARVED SILTY CLAY TO CLAY SAMPLES	FIGURE B6

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

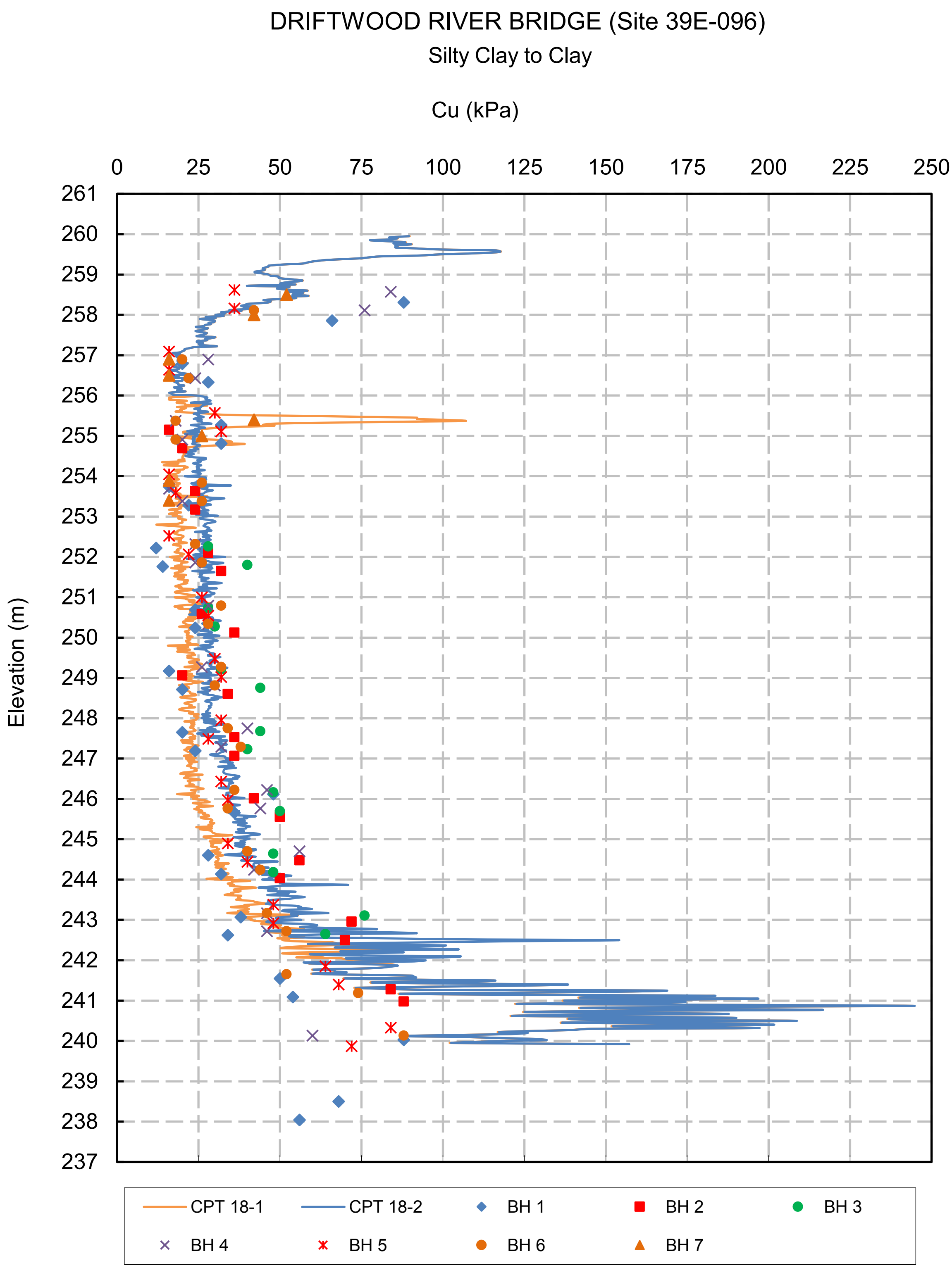
Silty Clay to Clay

BH1 SS18

BH2 SS13

Project No. :	1-18-0689	Terraprobe Inc.	Prepared by :	SD
Date :	January, 2019		Checked by :	RA

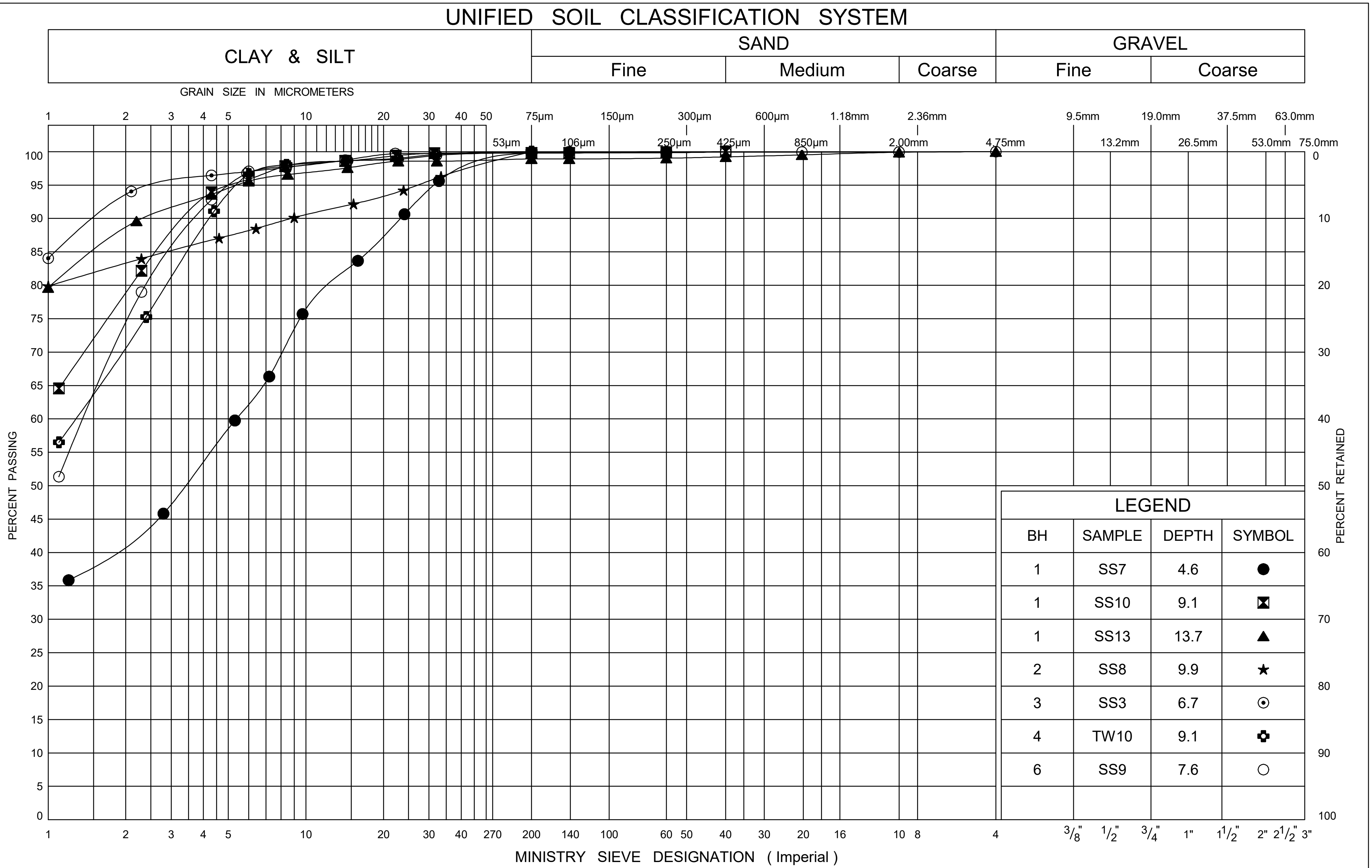
UNDRAINED SHEAR STRENGTH	FIGURE B7



Project No. :	1-18-0689	Prepared by :	SD
Date :	January, 2019	Checked by :	RA

Terraprobe Inc.

Terraprobe



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GRAIN SIZE DISTRIBUTION

SILTY CLAY TO CLAY

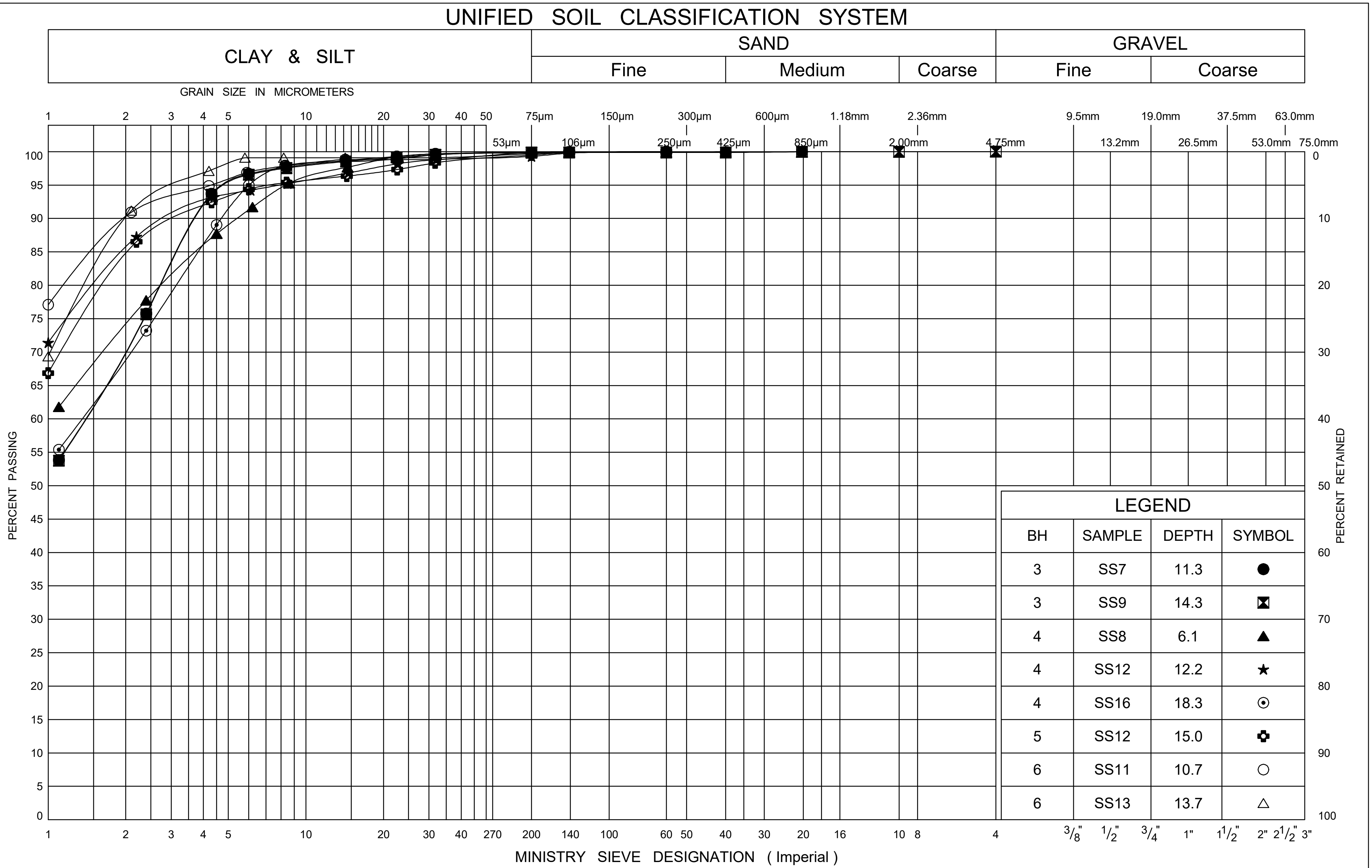
FIG No B8

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj

Terraprobe



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

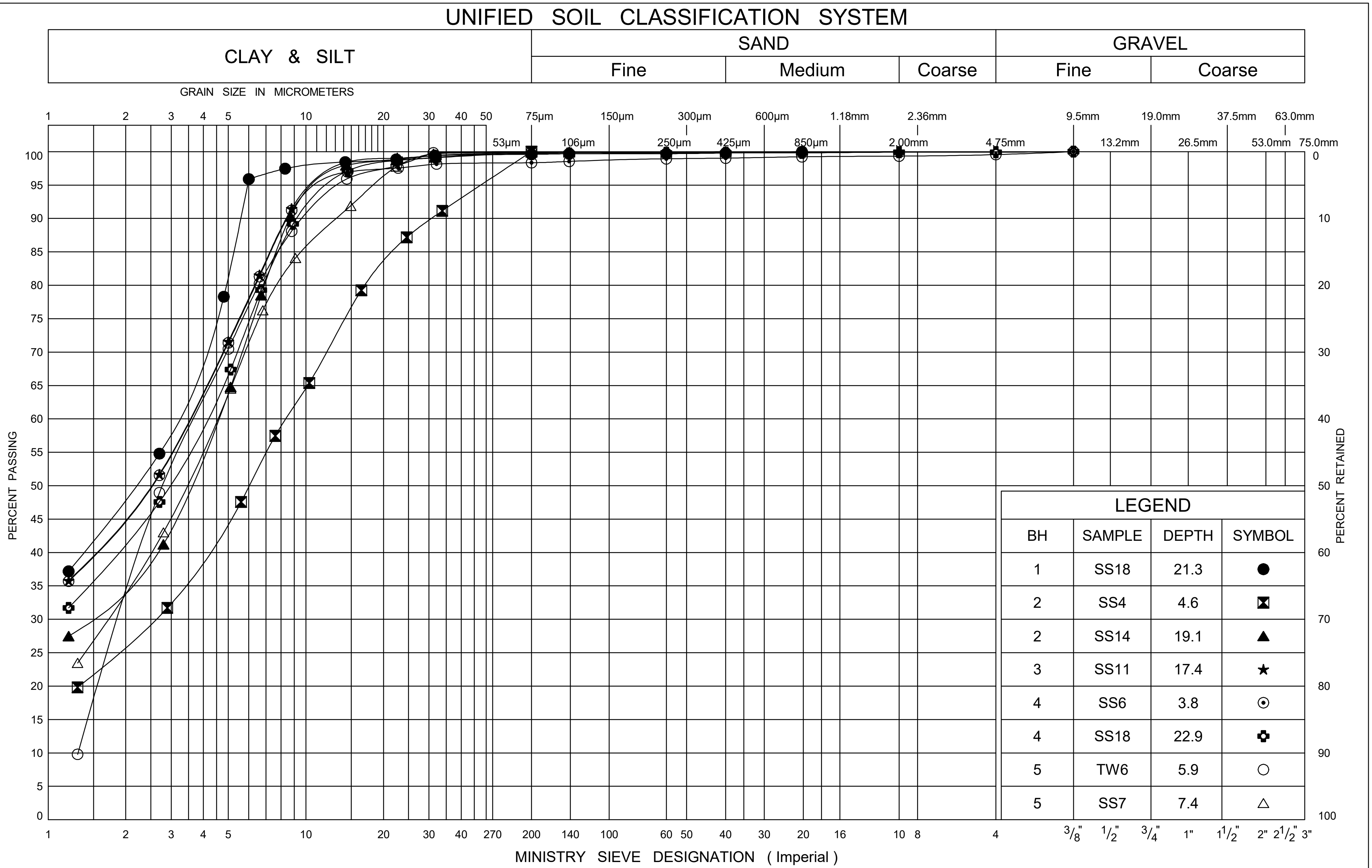
SILTY CLAY TO CLAY

FIG No B9

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
1	SS18	21.3	●
2	SS4	4.6	⊠
2	SS14	19.1	▲
3	SS11	17.4	★
4	SS6	3.8	⊙
4	SS18	22.9	✿
5	TW6	5.9	○
5	SS7	7.4	△

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Ontario

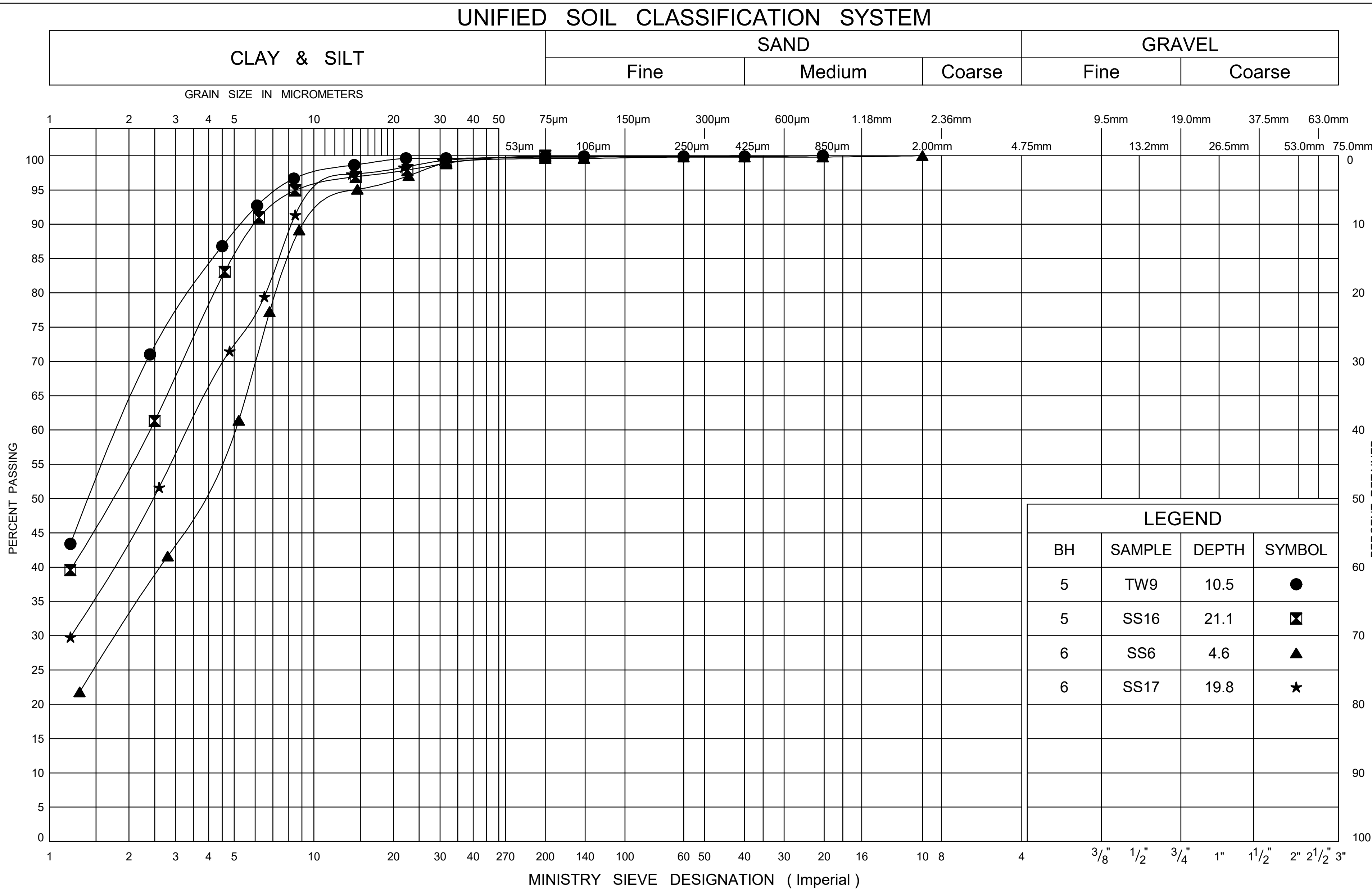
GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

FIG No B10

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj



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Ontario

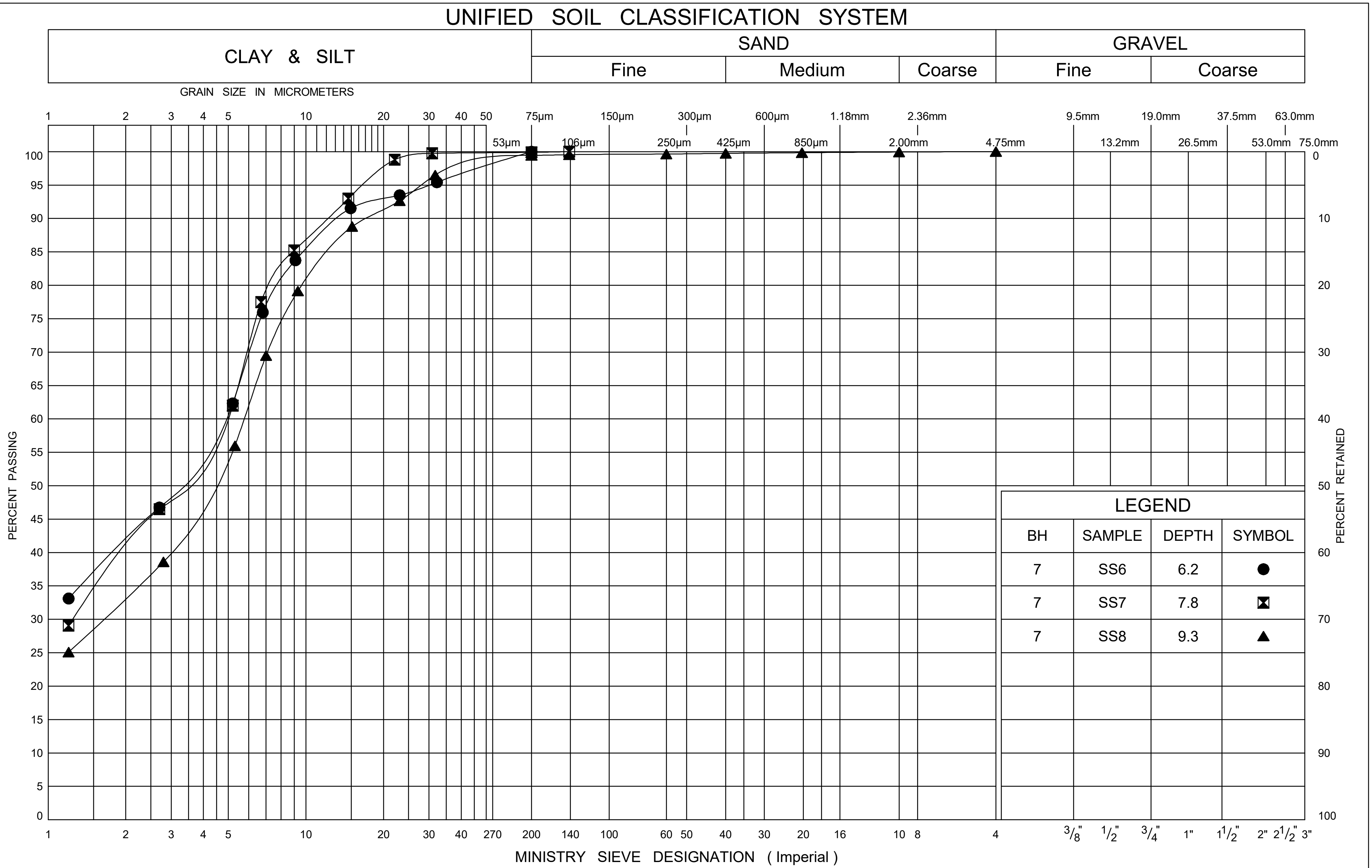
GRAIN SIZE DISTRIBUTION

SILTY CLAY TO CLAY

FIG No B11

G W P 5104-18-00

Driftwood River Bridge Site 39E-096



LEGEND			
BH	SAMPLE	DEPTH	SYMBOL
7	SS6	6.2	●
7	SS7	7.8	⊠
7	SS8	9.3	▲

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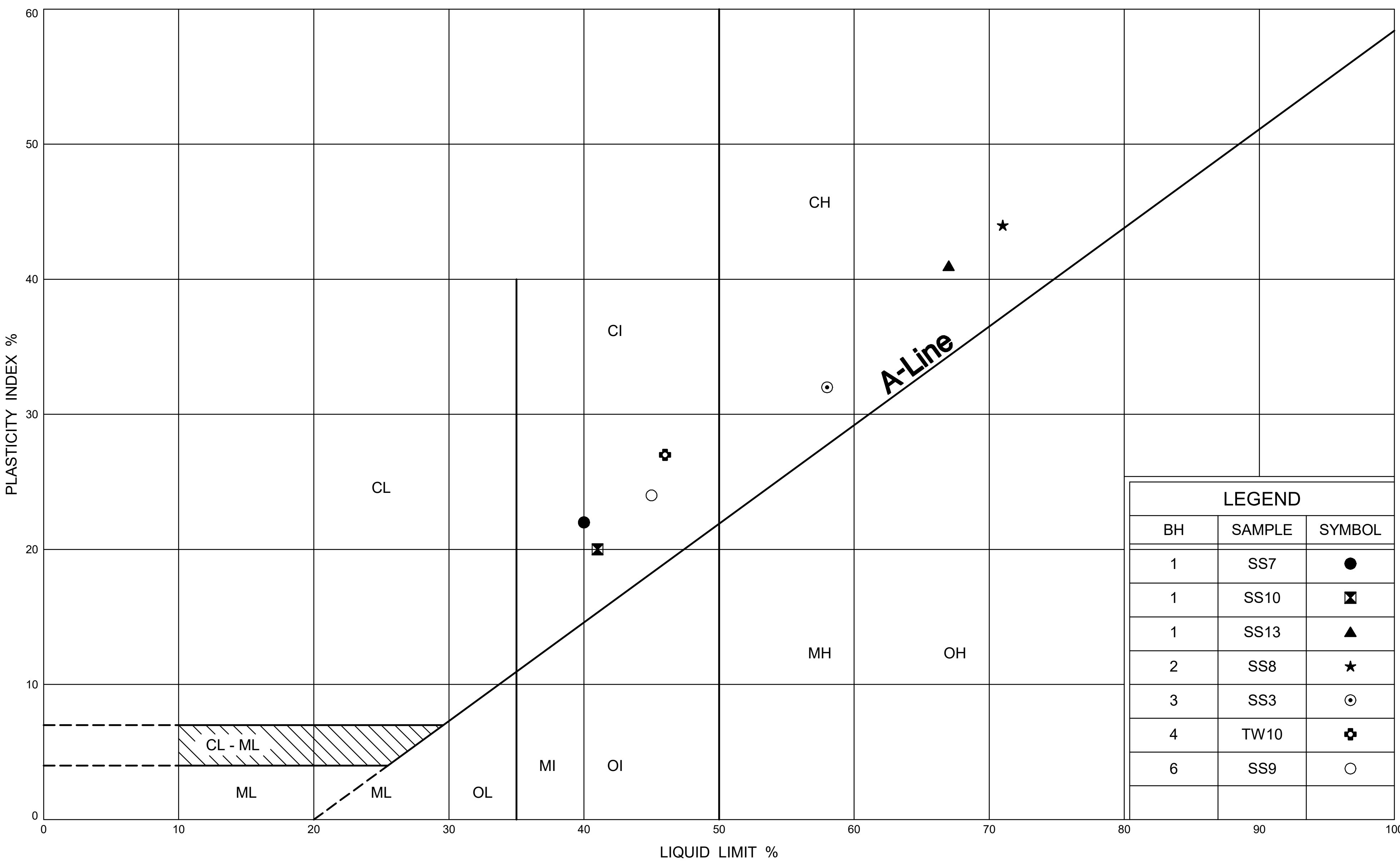
Ontario

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

FIG No B12

G W P 5104-18-00

Driftwood River Bridge Site 39E-096



LEGEND

BH	SAMPLE	SYMBOL
1	SS7	●
1	SS10	⧗
1	SS13	▲
2	SS8	★
3	SS3	⊙
4	TW10	✿
6	SS9	○

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Ontario

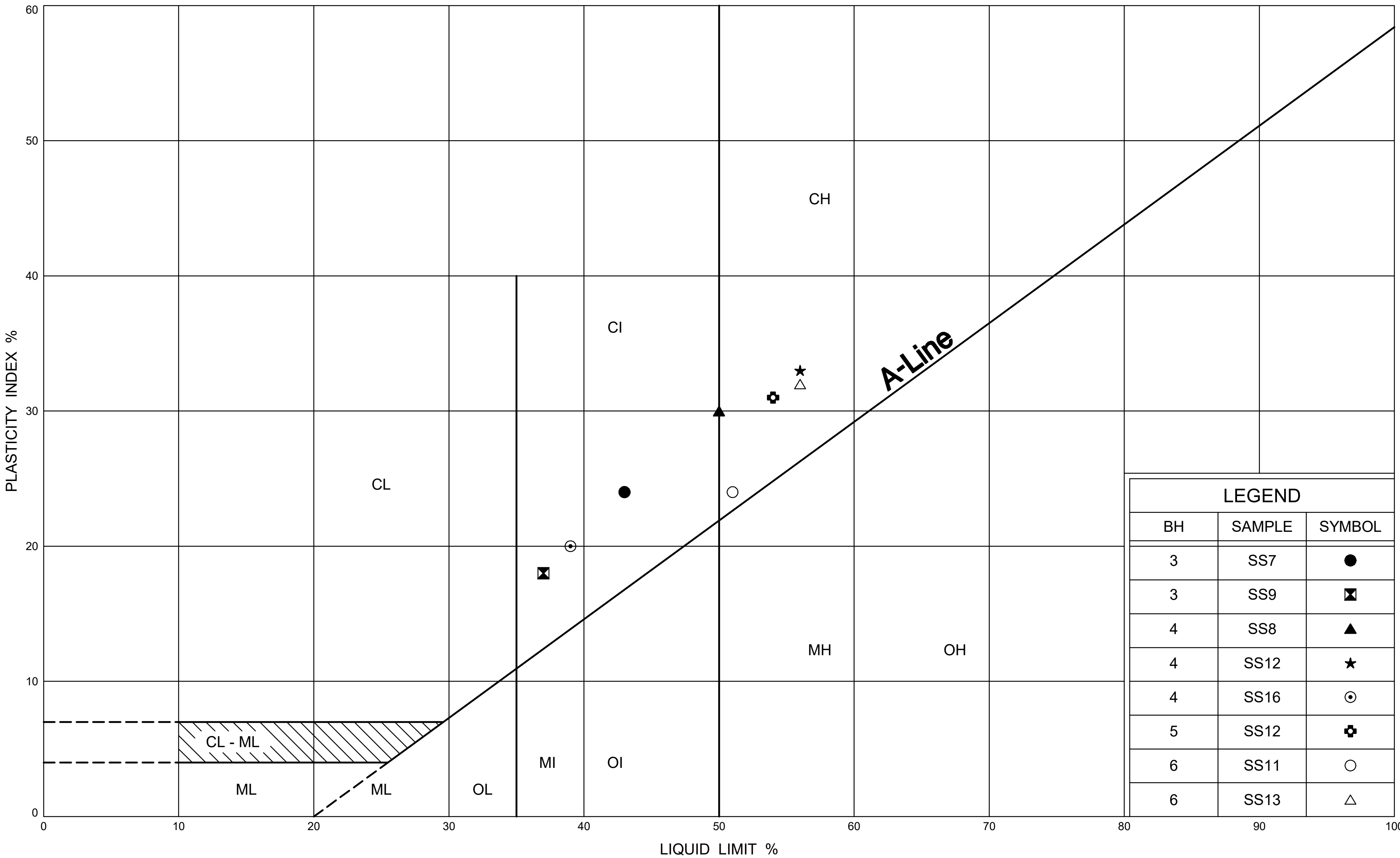
PLASTICITY CHART
SILTY CLAY TO CLAY

FIG No B13

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj



LEGEND

BH	SAMPLE	SYMBOL
3	SS7	●
3	SS9	⧗
4	SS8	▲
4	SS12	★
4	SS16	⊙
5	SS12	✿
6	SS11	○
6	SS13	△

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PLASTICITY CHART

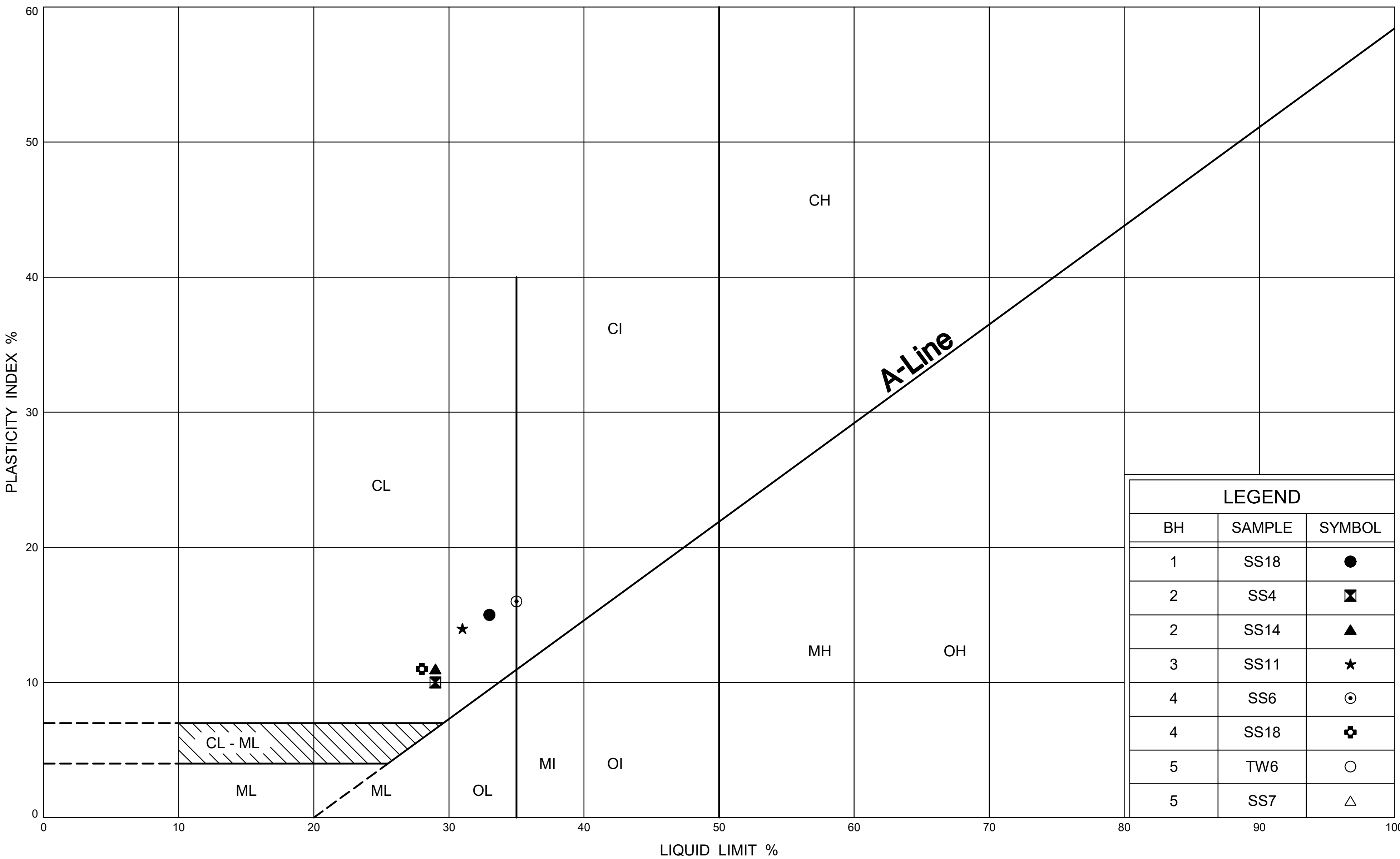
SILTY CLAY TO CLAY

FIG No B14

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

Terraprobe



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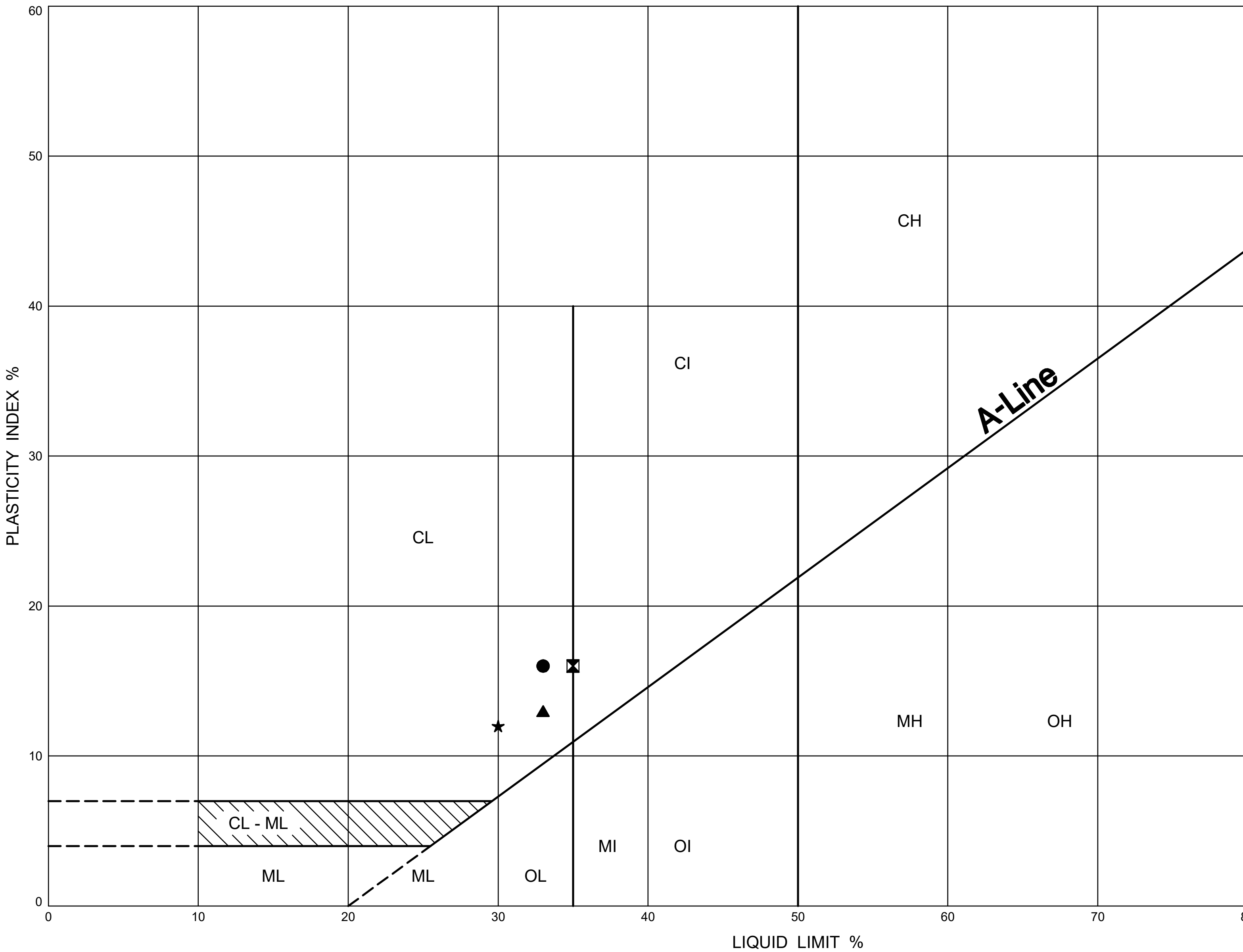
PLASTICITY CHART
SILTY CLAY TO CLAY

FIG No B15
G W P 5104-18-00
Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj



LEGEND

BH	SAMPLE	SYMBOL
5	TW9	●
5	SS16	⊠
6	SS6	▲
6	SS17	★

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PLASTICITY CHART

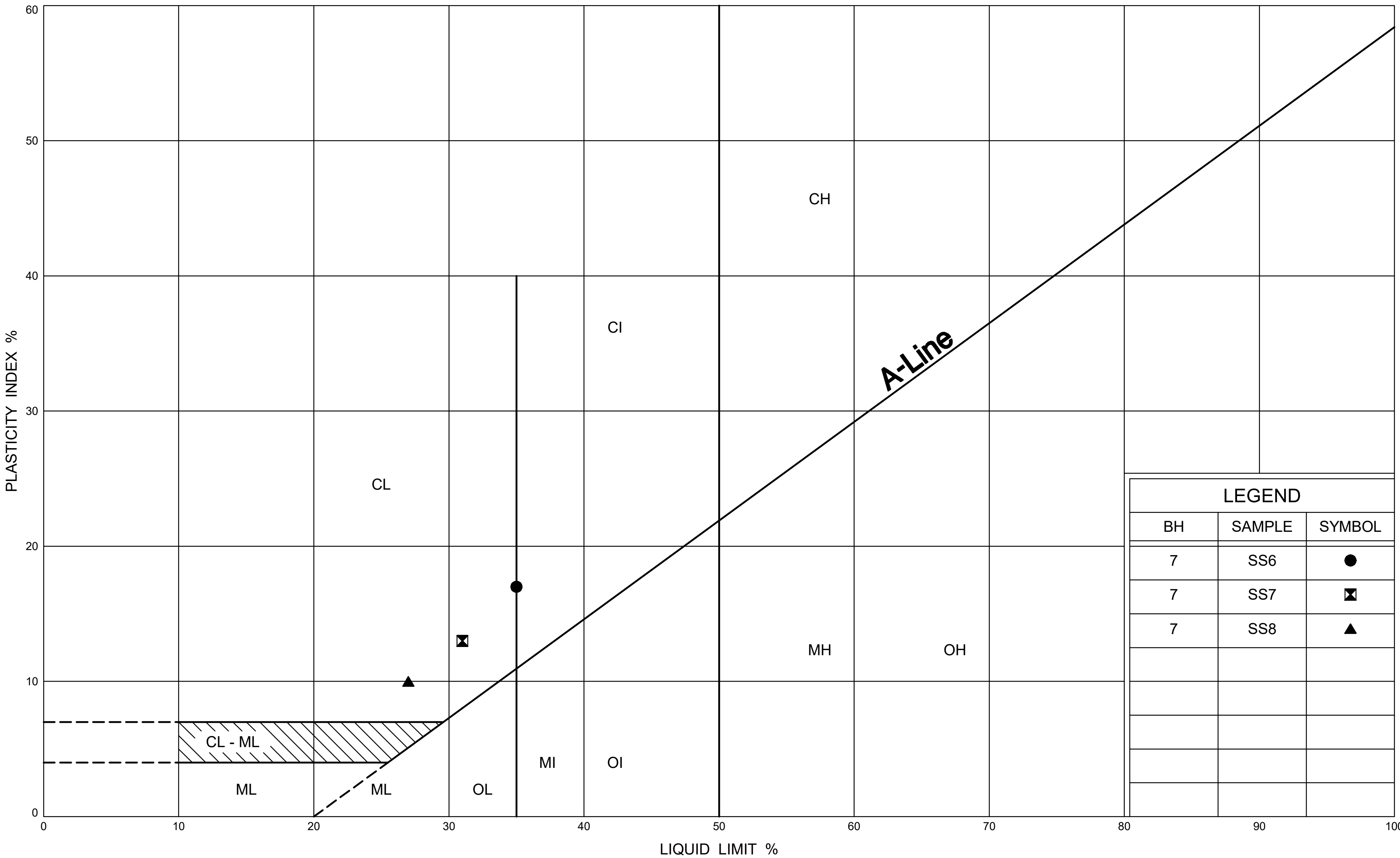
SILTY CLAY TO CLAY

FIG No B16

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj



LEGEND

BH	SAMPLE	SYMBOL
7	SS6	●
7	SS7	⧗
7	SS8	▲

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PLASTICITY CHART

SILTY CLAY TO CLAY

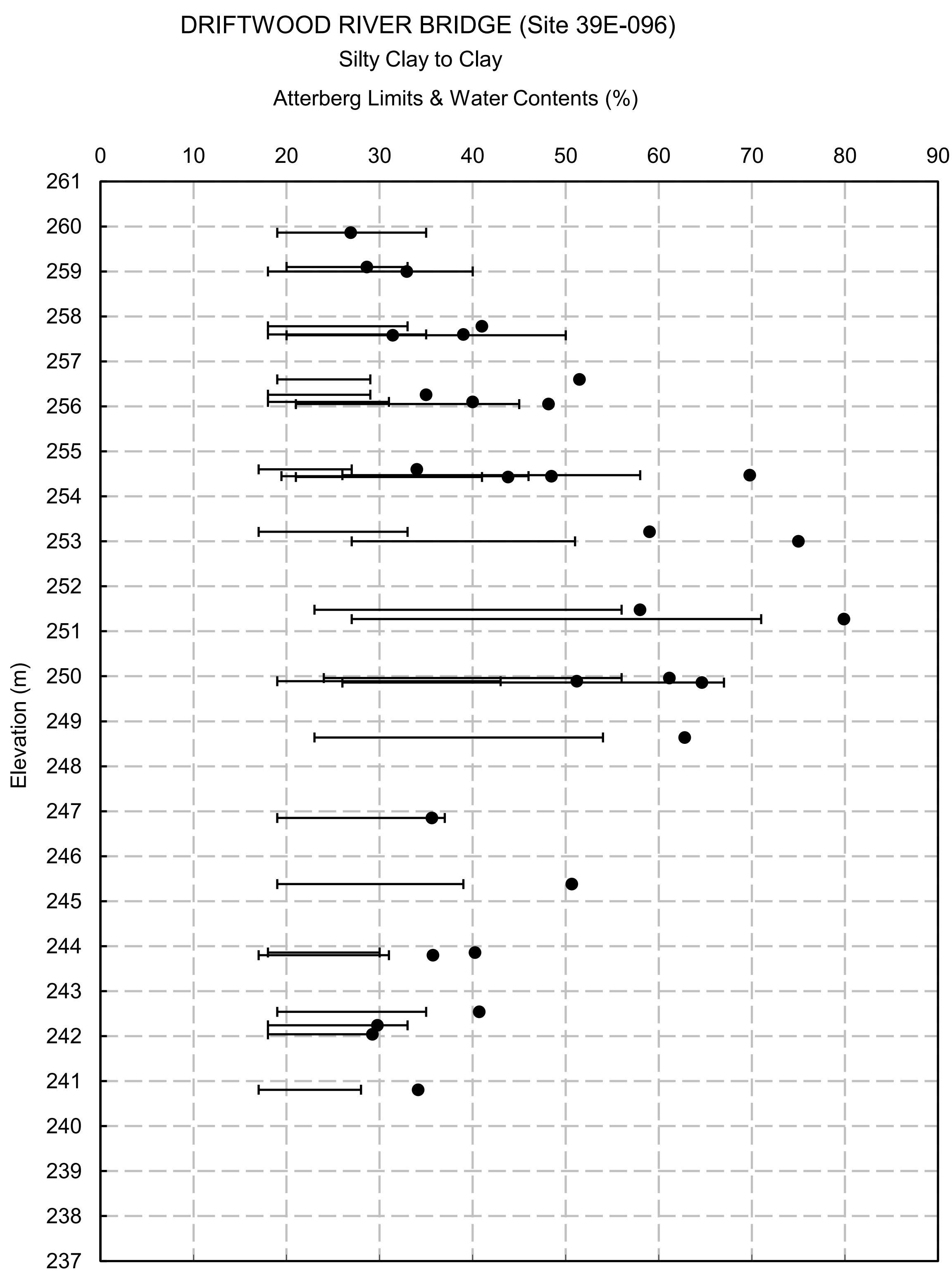
FIG No B17

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-18-0689_bh logs (new and old all boreholes).gpj

ATTERBERG LIMITS AND WATER CONTENTS	FIGURE B18



Project No. : 1-18-0689
Date : January, 2019

Terraprobe Inc.

Prepared by : SD
Checked by : RA

CONSOLIDATION TEST SUMMARY	FIGURE B19

SAMPLE IDENTIFICATION

Borehole No. :	4	Sample No. :	TW10
		Sample Depth (m) :	9.1- 9.6

TEST CONDITIONS

Test Type :	Laboratory Standard	Date Started :	27-Oct-14
Load Duration (hr) :	24	Date Completed :	7-Nov-14

SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL

Sample Height (mm) :	19.04	Unit Weight (kN/m^3) :	16.87
Sample Diameter (mm) :	63.44	Dry Unit Weight (kN/m^3) :	11.36
Area (cm^2) :	31.61	Specific Gravity :	2.73
Volume (cm^3) :	60.18	Solid Height (mm) :	8.09
Water Content (%) :	48.50	Volume of Solids (cm^3) :	25.58
Wet Mass (g) :	103.52	Volume of Voids (cm^3) :	34.61
Dry Mass (g) :	69.70	Degree of Saturation (%) :	97.73

TEST COMPUTATIONS

Stress (kPa)	Initial Height (mm)	Final Height (mm)	Void Ratio	t_{90} (min)	C_v (cm^2/s)	m_v (m^2/kN)	k (cm/s)
1.6	19.04	19.04	1.35				
18.7	19.04	18.72	1.31	9.00	1.38E-03	9.87E-04	1.30E-07
35.8	18.72	18.47	1.28	18.06	6.70E-04	7.67E-04	5.00E-08
70.1	18.47	18.01	1.23	16.00	7.20E-04	7.30E-04	5.20E-08
138.6	18.01	16.84	1.08	49.00	2.10E-04	9.46E-04	1.90E-08
275.7	16.84	15.56	0.92	33.06	2.60E-04	5.54E-04	1.40E-08
549.8	15.56	14.52	0.79	21.16	3.60E-04	2.44E-04	8.50E-09
1098.0	14.52	13.63	0.68	12.25	5.40E-04	1.12E-04	6.00E-09
2194.4	13.63	12.83	0.59	7.56	7.80E-04	5.40E-05	4.10E-09
549.8	12.83	13.06	0.61				
138.6	13.06	13.46	0.66				
35.8	13.46	13.91	0.72				

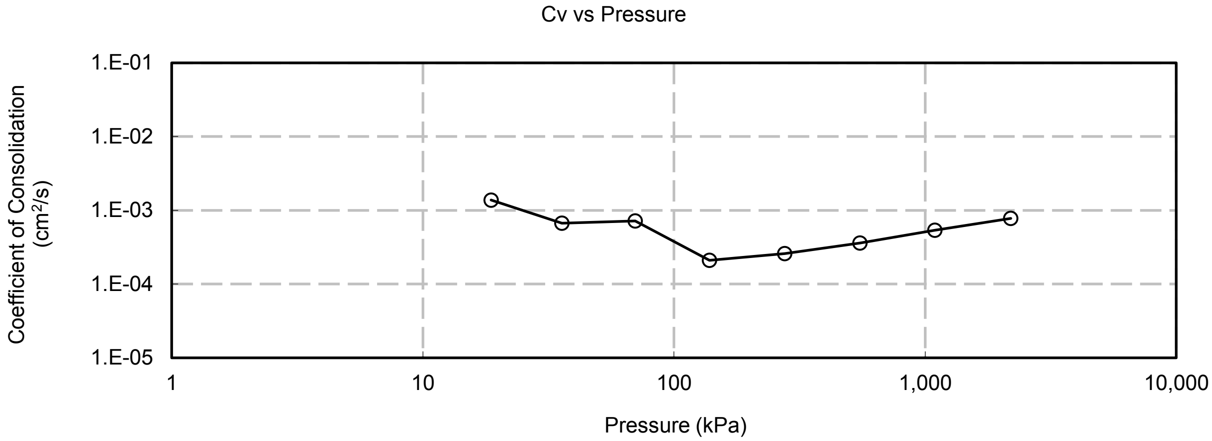
SAMPLE DIMENSIONS AND PROPERTIES _ FINAL

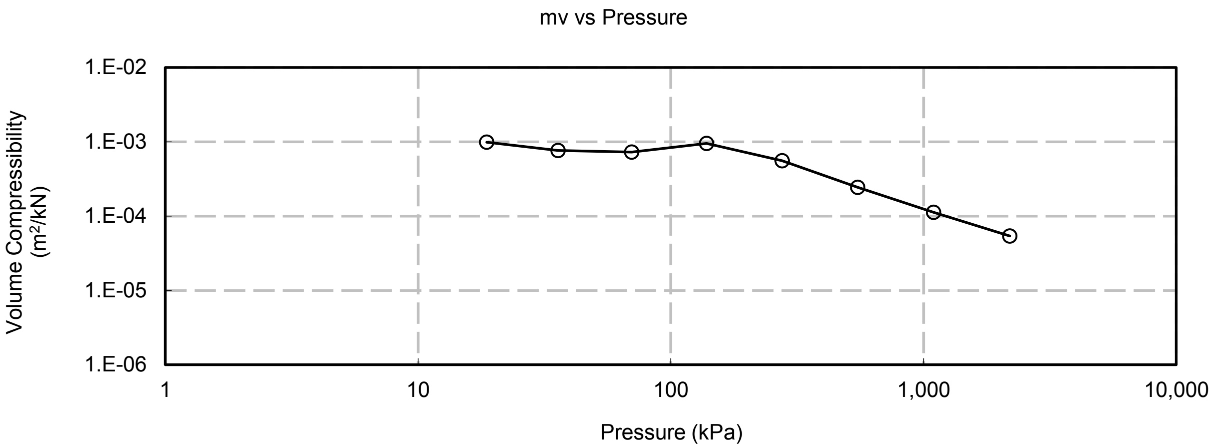
Sample Height (mm) :	13.91	Unit Weight (kN/m^3) :	19.74
Sample Diameter (mm) :	63.44	Dry Unit Weight (kN/m^3) :	15.29
Area (cm^2) :	31.61	Specific Gravity :	2.73
Volume (cm^3) :	43.97	Solid Height (mm) :	8.09
Water Content (%) :	29.14	Volume of Solids (cm^3) :	25.15
Wet Mass (g) :	88.51	Volume of Voids (cm^3) :	18.82
Dry Mass (g) :	68.54		

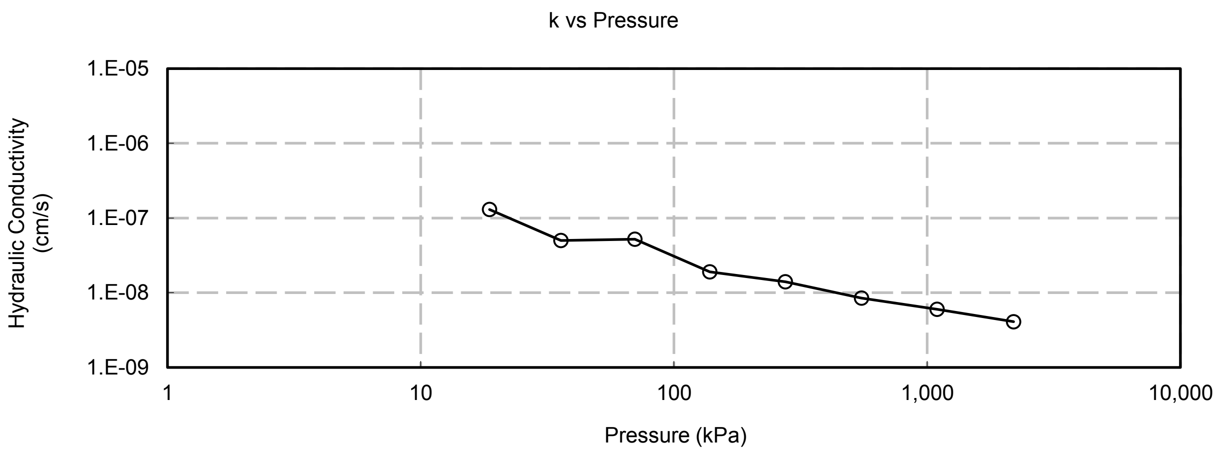
Project No. :	1-18-0689	Terraprobe Inc.	Prepared By :	SD
Date :	January 2019		Checked By :	RA

CONSOLIDATION TEST	FIGURE B20

DRIFTWOOD RIVER BRIDGE (Site 39E-096)
BH 4, TW 10







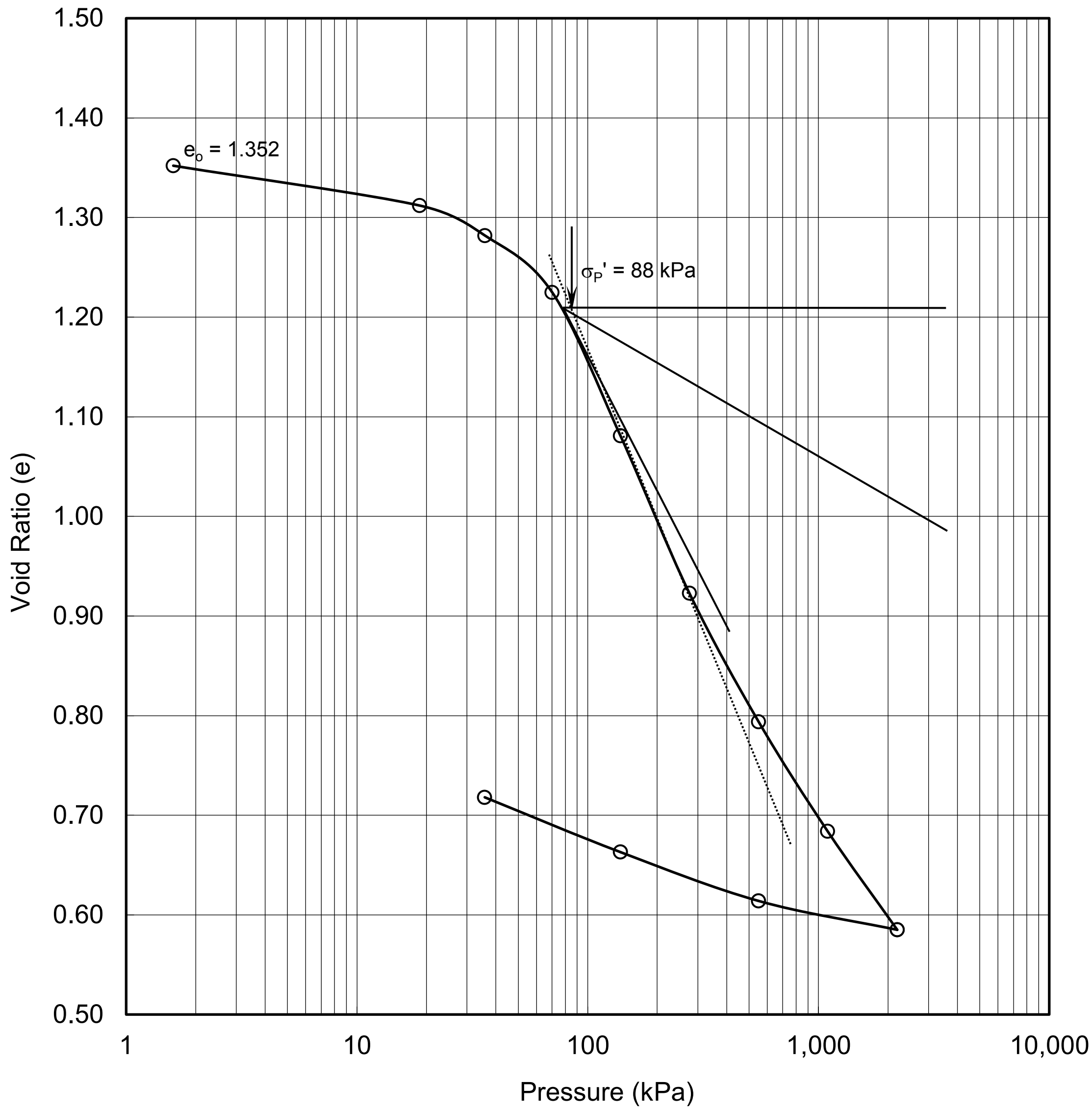
Project No. :	1-18-0689	Terraprobe Inc.	Prepared By :	SD
Date :	January 2019		Checked By :	RA

CONSOLIDATION TEST	FIGURE B21

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

BH 4, TW 10

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

e_o =	1.35	ω_L =	46%	σ_{v0}' =	90.9 kPa
ω =	48%	ω_P =	19%	σ_P' =	88.0 kPa
γ =	17.0 kN/m^3	PI =	27%		
Gs =	2.73				

Project No. : 1-18-0689
Date : January 2019

Terraprobe Inc.

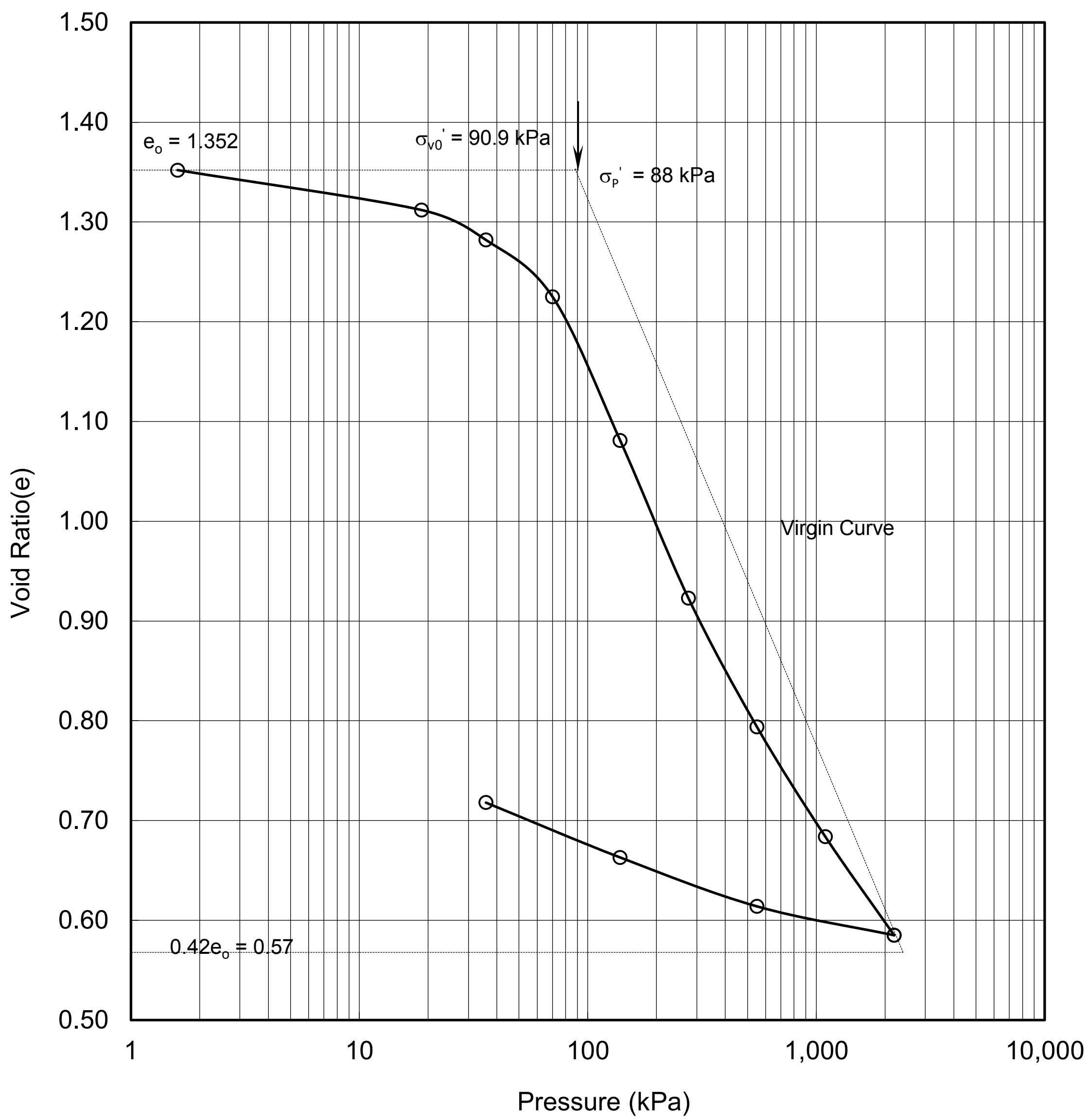
Prepared By : SD
Checked By : RA

CONSOLIDATION TEST	FIGURE B22

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

BH 4, TW 10

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

e_o =	1.35	ω_L =	46%	σ_{v0}' =	90.9 kPa
ω =	48%	ω_P =	19%	σ_P' =	88.0 kPa
γ =	17.0 kN/m^3	PI =	27%	C_c =	0.547
Gs =	2.73			C_r =	0.074

Project No. :	1-18-0689	Terraprobe Inc.	Prepared By :	SD
Date :	January 2019		Checked By :	RA

CONSOLIDATION TEST SUMMARY

FIGURE B23

SAMPLE IDENTIFICATION

Borehole No. :	5	Sample No. :	TW6
		Sample Depth (m) :	5.9 - 6.4

TEST CONDITIONS

Test Type :	Laboratory Standard	Date Started :	12-Jan-18
Load Duration (hr) :	24	Date Completed :	23-Jan-18

SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL

Sample Height (mm) :	25.27	Unit Weight (kN/m^3) :	17.87
Sample Diameter (mm) :	63.35	Dry Unit Weight (kN/m^3) :	12.71
Area (cm^2) :	31.52	Specific Gravity :	2.74
Volume (cm^3) :	79.65	Solid Height (mm) :	11.96
Water Content (%) :	40.70	Volume of Solids (cm^3) :	37.68
Wet Mass (g) :	145.18	Volume of Voids (cm^3) :	41.97
Dry Mass (g) :	103.20	Degree of Saturation (%) :	100.02

TEST COMPUTATIONS

Stress (kPa)	Initial Height (mm)	Final Height (mm)	Void Ratio	t_{90} (min)	C_v (cm^2/s)	m_v (m^2/kN)	k (cm/s)
1.2	25.27	25.27	1.11				
18.4	25.27	24.96	1.09	22.56	9.80E-04	7.23E-04	6.90E-08
35.6	24.96	24.70	1.07	18.06	1.20E-03	5.92E-04	7.00E-08
69.9	24.70	24.11	1.02	30.25	6.80E-04	7.00E-04	4.70E-08
138.7	24.11	23.13	0.94	31.36	6.10E-04	5.88E-04	3.50E-08
276.1	23.13	22.26	0.86	16.00	1.10E-03	2.74E-04	3.00E-08
551.0	22.26	21.47	0.80	9.00	1.82E-03	1.29E-04	2.30E-08
1100.7	21.47	20.74	0.74	7.56	2.02E-03	6.20E-05	1.20E-08
276.1	20.74	20.85	0.74				
69.9	20.85	21.03	0.76				
18.4	21.03	21.24	0.78				

SAMPLE DIMENSIONS AND PROPERTIES _ FINAL

Sample Height (mm) :	21.24	Unit Weight (kN/m^3) :	19.11
Sample Diameter (mm) :	63.35	Dry Unit Weight (kN/m^3) :	15.08
Area (cm^2) :	31.52	Specific Gravity :	2.74
Volume (cm^3) :	66.94	Solid Height (mm) :	11.96
Water Content (%) :	26.70	Volume of Solids (cm^3) :	37.59
Wet Mass (g) :	130.43	Volume of Voids (cm^3) :	29.35
Dry Mass (g) :	102.96		

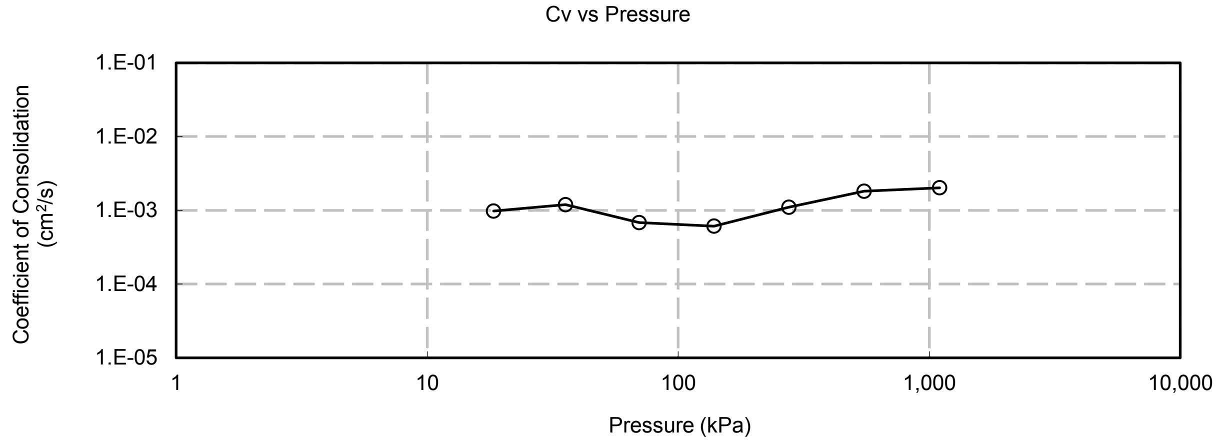
Project No. : 1-18-0689
Date : January 2019

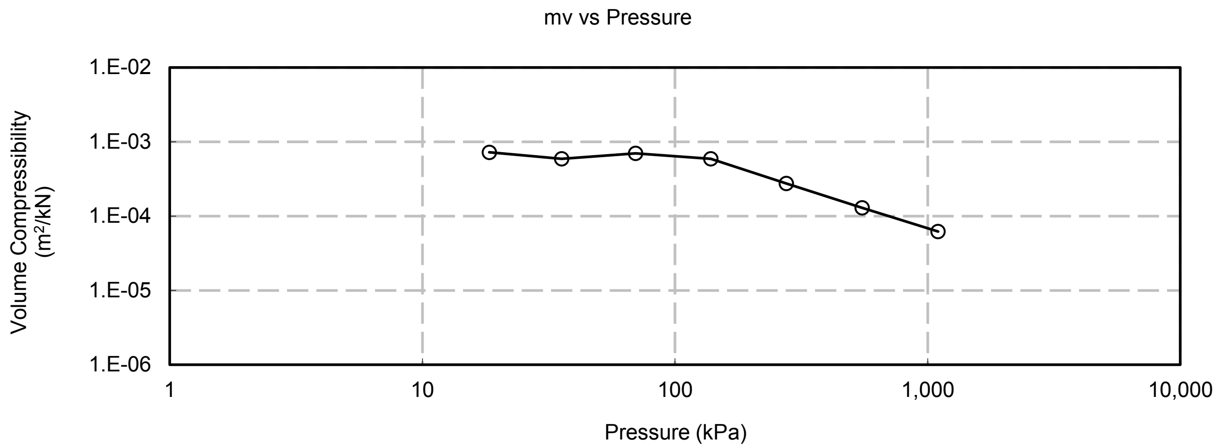
Terraprobe Inc.

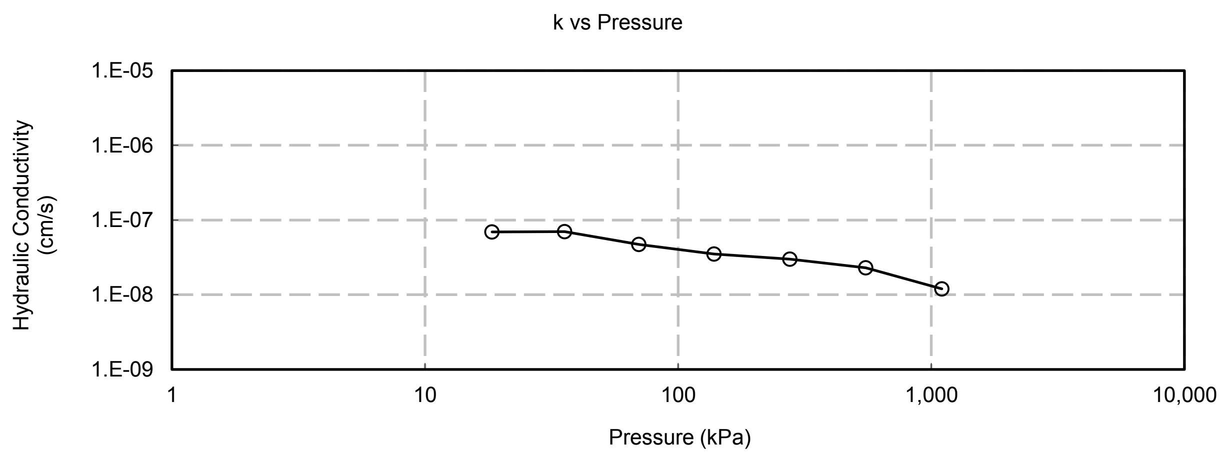
Prepared By : SD
Checked By : RA

CONSOLIDATION TEST	FIGURE B24

DRIFTWOOD RIVER BRIDGE (Site 39E-096)
BH 5, TW 6





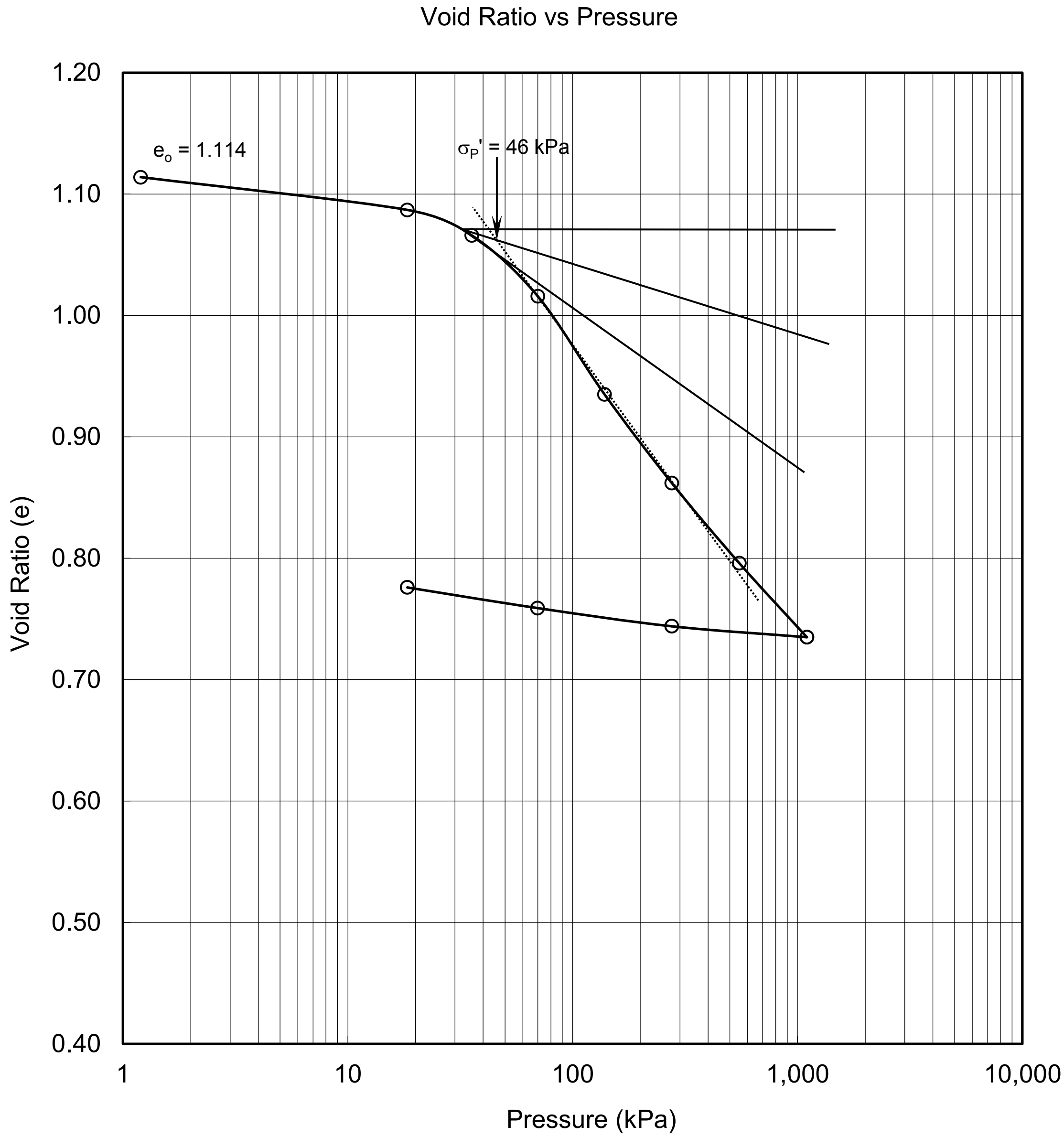


Project No. :	1-18-0689	Terraprobe Inc.	Prepared By :	SD
Date :	January 2019		Checked By :	RA

CONSOLIDATION TEST	FIGURE B25

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

BH 5, TW 6



Soil Type : SILTY CLAY to CLAY

e_o =	1.11	ω_L =	33%	σ_{v0}' =	47.7 kPa
ω =	41%	ω_P =	18%	σ_P' =	46.0 kPa
γ =	17.9 kN/m^3	PI =	15%		
Gs =	2.74				

Project No. : 1-18-0689

Date : January 2019

Terraprobe Inc.

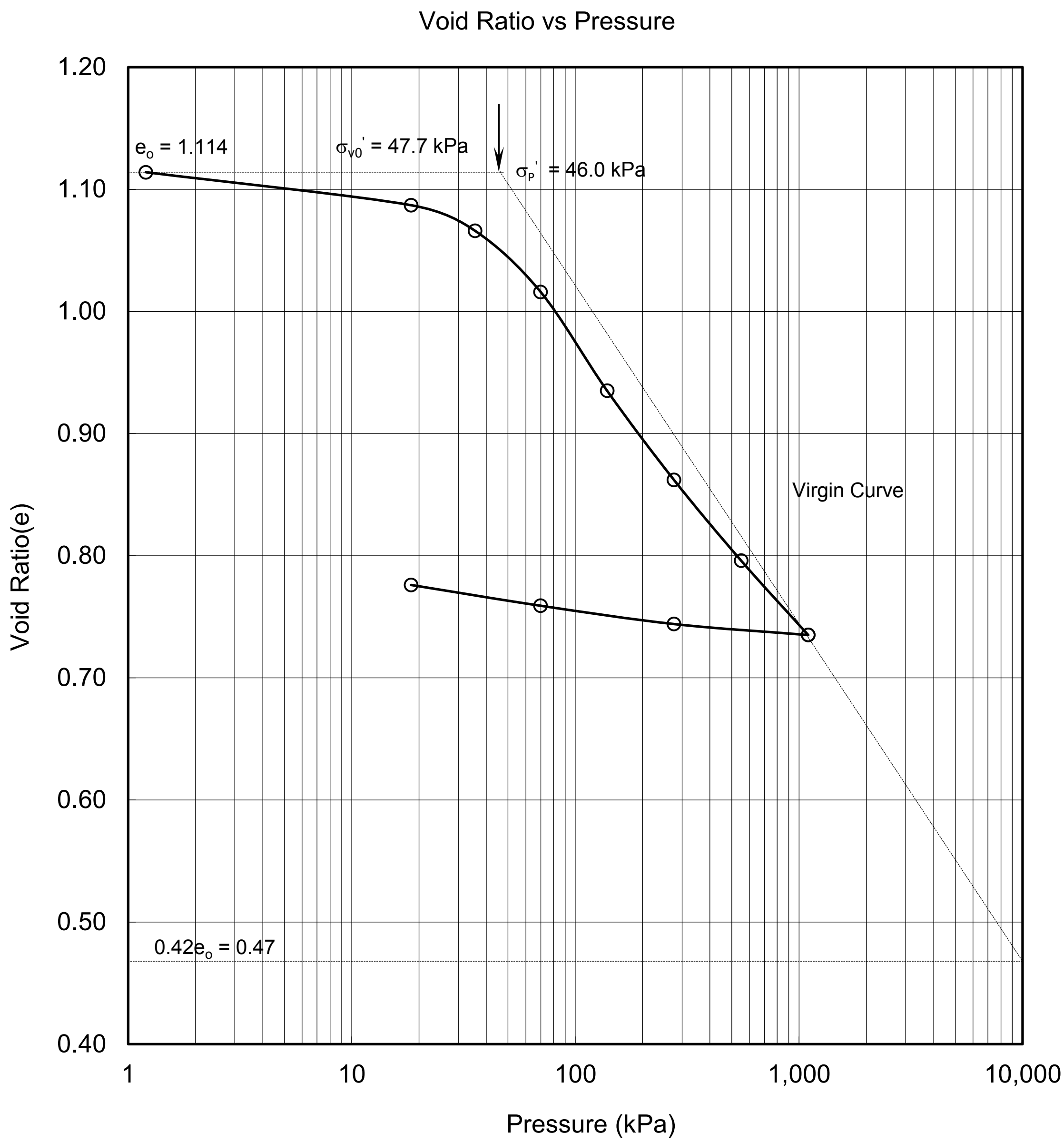
Prepared By : SD

Checked By : RA

CONSOLIDATION TEST	FIGURE B26

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

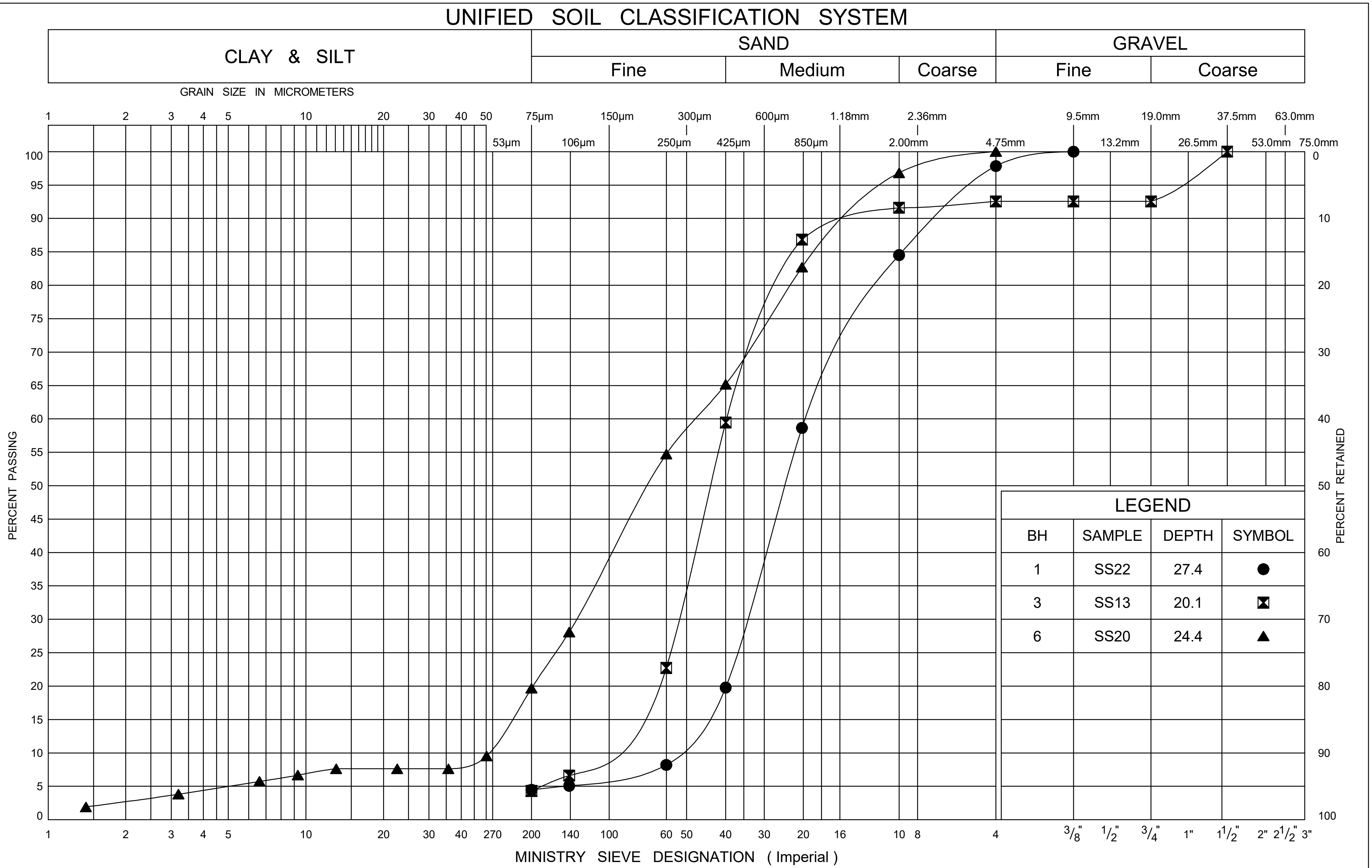
BH 5, TW 6



Soil Type : SILTY CLAY to CLAY

e_o =	1.11	ω_L =	33%	σ_{v0}' =	47.7 kPa
ω =	41%	ω_P =	18%	σ_P' =	46.0 kPa
γ =	17.9 kN/m^3	PI =	15%	C_c =	0.277
Gs =	2.74			C_r =	0.023

Project No. :	1-18-0689	Terraprobe Inc.	Prepared By :	SD
Date :	January 2019		Checked By :	RA



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SAND

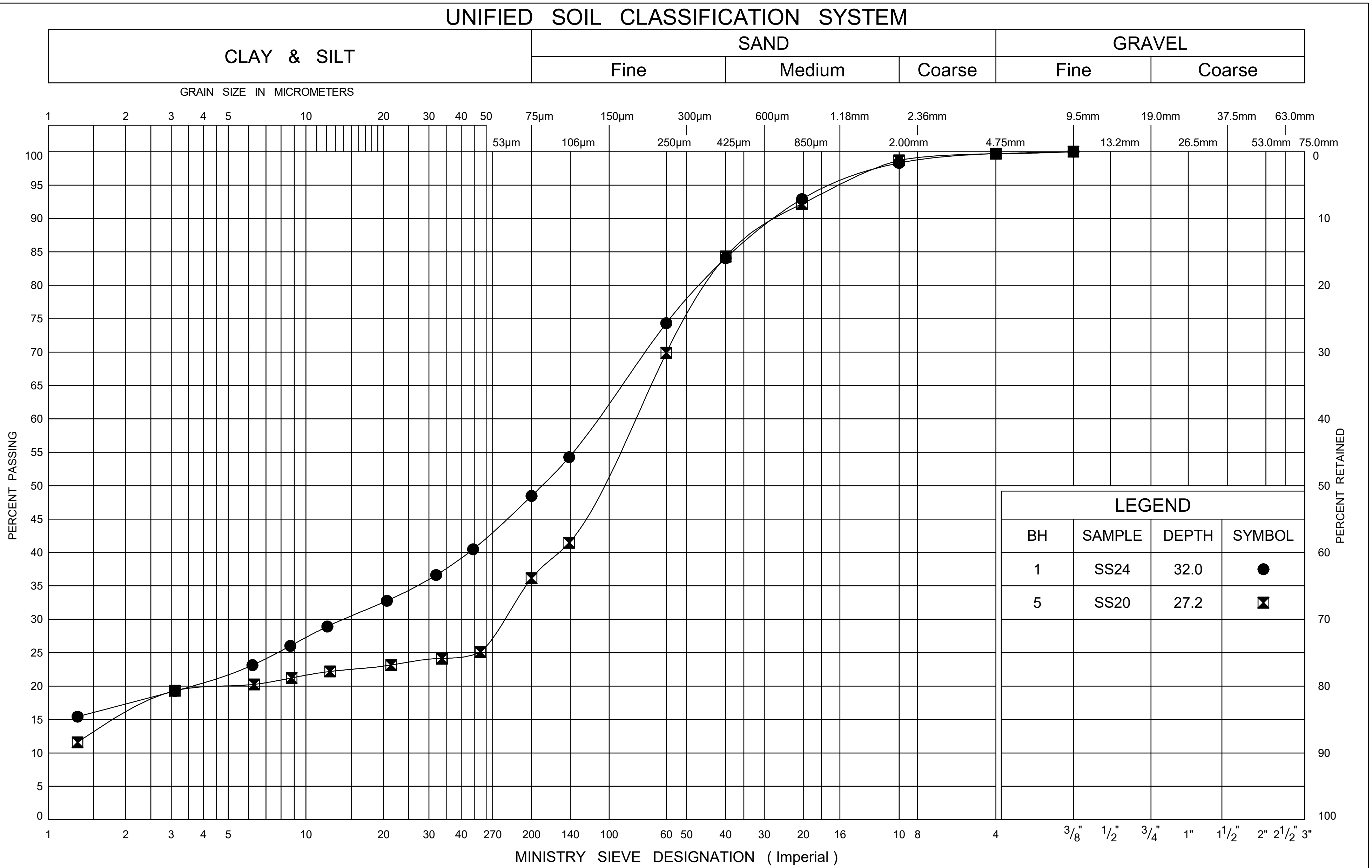
FIG No B27

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj

Terraprobe



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

SILTY SAND TILL

FIG No B28

G W P 5104-18-00

Driftwood River Bridge Site 39E-096

file: 1-17-0864-01_bh logs (new and old all boreholes).gpj

PHOTOGRAPH OF COBBLES AND BOULDERS	FIGURE B29

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Borehole No. 1

Borehole No. 5

Project No. : 1-18-0689
Date : January, 2019

Terraprobe Inc.

Prepared by : SD
Checked by : RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B30

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Project No. : 1-18-0689
Date : January, 2019

Terraprobe Inc.

Prepared by : SD
Checked by : RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B31

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Project No. :	1-18-0689	Terraprobe Inc.	Prepared by :	SD
Date :	January, 2019		Checked by :	RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B32

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Project No. : 1-18-0689
Date : January, 2019

Terraprobe Inc.

Prepared by : SD
Checked by : RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B33

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Borehole No. 6

Project No. : 1-18-0689

Date : January, 2019

Terraprobe Inc.

Prepared by : SD

Checked by : RA

FINAL REPORT

CA14223-FEB18 R

1-17-0864

Prepared for

Terraprobe Inc

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Terraprobe Inc	Project Specialist	Deanna Edwards, B.Sc, C.Chem
		Laboratory	SGS Canada Inc.
Address	11 Indell Lane Brampton, ON L6T 3Y3, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Sepideh D_Monfared	Telephone	705-652-2000
Telephone	(905) 796-2650	Facsimile	705-652-6365
Facsimile	(905) 796-2250	Email	deanna.edwards@sgs.com
Email	smonfared@terraprobe.ca	SGS Reference	CA14223-FEB18
Project	1-17-0864	Received	02/14/2018
Order Number		Approved	02/15/2018
Samples	Soil (2)	Report Number	CA14223-FEB18 R
		Date Reported	02/15/2018

COMMENTS

Temperature of Sample upon Receipt: 9 degrees C
Cooling Agent Present: Yes
Custody Seal Present: Yes

Chain of Custody Number:00288

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Deanna Edwards, B.Sc, C.Chem

Deanna Edwards

TABLE OF CONTENTS

PACKAGE: **REG153 - 1.1.6 PHCs** (SOIL)

L1 = REG153 / SOIL / COARSE - TABLE 1 - Agricultural/Other - UNDEFINED

			Sample Number	5	6
			Sample Name	BH5-SS5 (14'4"-15'10")	BH6-SS5-(12.5-14)
			Sample Matrix	Soil	Soil
			Sample Date	05/12/2017	18/12/2017
Parameter	**Units**	**RL**	**L1**	**Result**	**Result**
1.1.6 PHCs					
Moisture Content	%	0.1		26.7	20.6

PACKAGE: **REG153 - 1.3 Other (ORP)** (SOIL)

L1 = REG153 / SOIL / COARSE - TABLE 1 - Agricultural/Other - UNDEFINED

			Sample Number	5	6
			Sample Name	BH5-SS5 (14'4"-15'10")	BH6-SS5-(12.5-14)
			Sample Matrix	Soil	Soil
			Sample Date	05/12/2017	18/12/2017
Parameter	**Units**	**RL**	**L1**	**Result**	**Result**
1.3 Other (ORP)					
Chloride	µg/g	0.4		34	61

PACKAGE: **REG153 - Corrosivity Index** (SOIL)

L1 = REG153 / SOIL / COARSE - TABLE 1 - Agricultural/Other - UNDEFINED

			Sample Number	5	6
			Sample Name	BH5-SS5 (14'4"-15'10")	BH6-SS5-(12.5-14)
			Sample Matrix	Soil	Soil
			Sample Date	05/12/2017	18/12/2017
Parameter	**Units**	**RL**	**L1**	**Result**	**Result**
Corrosivity Index					
Corrosivity Index	none	1		1	4
Soil Redox Potential	mV	-		212	253
Sulphide	%	0.02		< 0.02	< 0.02
pH	no unit	0.05		7.77	8.66
Resistivity (calculated)	ohms.cm	-9999		12800	7890

FINAL REPORT

CA14223-FEB18 R

Client: Terraprobe Inc

Project: 1-17-0864

Project Manager: Sepideh D_Monfared

Samplers: Fatemeh

PACKAGE: **REG153 - General Chemistry** (SOIL)

L1 = REG153 / SOIL / COARSE - TABLE 1 - Agricultural/Other - UNDEFINED

			Sample Number	5	6
			Sample Name	BH5-SS5 (14'4"-15'10")	BH6-SS5- (12.5-14)
			Sample Matrix	Soil	Soil
			Sample Date	05/12/2017	18/12/2017
Parameter	**Units**	**RL**	**L1**	**Result**	**Result**
General Chemistry					
Conductivity	uS/cm	2		78	127

PACKAGE: **REG153 - Metals and Inorganics** (SOIL)

L1 = REG153 / SOIL / COARSE - TABLE 1 - Agricultural/Other - UNDEFINED

			Sample Number	5	6
			Sample Name	BH5-SS5 (14'4"-15'10")	BH6-SS5- (12.5-14)
			Sample Matrix	Soil	Soil
			Sample Date	05/12/2017	18/12/2017
Parameter	**Units**	**RL**	**L1**	**Result**	**Result**
Metals and Inorganics					
Sulphate	µg/g	0.4		11	7.1

EXCEEDANCE SUMMARY

No exceedances are present above the regulatory limit(s) indicated

FINAL REPORT

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0173-FEB18	µg/g	0.4	<0.4	14	20	97	80	120	118	75	125
Sulphate	DIO0173-FEB18	µg/g	0.4	<0.4	7	20	99	80	120	95	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide	ECS0021-FEB18	%	0.02	<0.02	NV	20	102	80	120			

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0187-FEB18	uS/cm	2	< 2	0.00	10	0.99	90	110	NA		

QC SUMMARY

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

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

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.
RL Reporting Limit.
↑ Reporting limit raised.
↓ Reporting limit lowered.
NA The sample was not analysed for this analyte
ND Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act" published by the Ministry and dated March 9, 2004 as amended.

-- End of Analytical Report --

FINAL REPORT

CA14068-DEC18 R2

1-18-0689 Hwy 577, Driftwood River Bridge, Hwy 577, Taylor, Ontario

Prepared for

Terraprobe Inc

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Terraprobe Inc	Project Specialist	Rob Irwin B.Sc., C.Chem
		Laboratory	SGS Canada Inc.
Address	11 Indell Lane Brampton, ON L6T 3Y3. Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Sepideh D_Monfared	Telephone	2361
Telephone	(905) 796-2650	Facsimile	705-652-6365
Facsimile	(905) 796-2250	Email	
Email	smonfared@terraprobe.ca	SGS Reference	CA14068-DEC18
Project	1-18-0689 Hwy 577, Driftwood River Bridge, Hwy 577, Taylor, (	Received	12/04/2018
Order Number		Approved	12/11/2018
Samples	Soil (1)	Report Number	CA14068-DEC18 R2
		Date Reported	12/13/2018

COMMENTS

Temperature of Sample upon Receipt: 5 degrees C
Cooling Agent Present: yes
Custody Seal Present: no

Chain of Custody Number: 002759

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

SIGNATORIES

Rob Irwin B.Sc., C.Chem

ASSOCIATION OF THE CHEMICAL PROFESSION OF ONTARIO CHARTERED Robert Irwin CHEMIST

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FINAL REPORT

CA14068-DEC18 R2

Client: Terraprobe Inc

Project: 1-18-0689 Hwy 577, Driftwood River Bridge, Hwy 577,

Project Manager: Sepideh D_Monfared

Samplers: Fatemah Yazdandoust

PACKAGE: **- Corrosivity Index** (SOIL)

Sample Number	5
Sample Name	BH7_SS5B-(15'9"-17')
Sample Matrix	Soil
Sample Date	21/11/2018

Parameter	Units	RL	Result
Corrosivity Index			
Soil Redox Potential	mV	-	179
Sulphide	%	0.02	< 0.02
pH	pH Units	0.05	7.98
Resistivity (calculated)	ohms.cm	-9999	6530

PACKAGE: **- General Chemistry** (SOIL)

Sample Number	5
Sample Name	BH7_SS5B-(15'9"-17')
Sample Matrix	Soil
Sample Date	21/11/2018

Parameter	Units	RL	Result
General Chemistry			
Conductivity	uS/cm	2	153

PACKAGE: **- Metals and Inorganics** (SOIL)

Sample Number	5
Sample Name	BH7_SS5B-(15'9"-17')
Sample Matrix	Soil
Sample Date	21/11/2018

Parameter	Units	RL	Result
Metals and Inorganics			
Moisture Content	%	0.1	25.7
Sulphate	µg/g	0.4	12

FINAL REPORT

CA14068-DEC18 R2

Client: Terraprobe Inc

Project: 1-18-0689 Hwy 577, Driftwood River Bridge, Hwy 577,

Project Manager: Sepideh D_Monfared

Samplers: Fatemah Yazdandoust

PACKAGE: **- Other (ORP)** (SOIL)

Sample Number	5
Sample Name	BH7_SS5B-(15'9"-17')
Sample Matrix	Soil
Sample Date	21/11/2018

Parameter	Units	RL	Result
Other (ORP)			
Chloride	μg/g	0.4	8.8

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0050-DEC18	µg/g	0.4	<0.4	3	20	100	80	120	119	75	125
Sulphate	DIO0050-DEC18	µg/g	0.4	<0.4	1	20	96	80	120	92	75	125

Carbon/Sulphur

Method: ASTM E1915-07A | Internal ref.: ME-CA-[ENV]ARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide	ECS0014-DEC18	%	0.02	<0.02	14	20	116	80	120			

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0055-DEC18	uS/cm	2	< 0.002	1	10	99	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 I Internal ref.: ME-CA-[ENV]EWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0055-DEC18	pH Units	0.05	NA	1		101			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

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

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.
RL Reporting Limit.
↑ Reporting limit raised.
↓ Reporting limit lowered.
NA The sample was not analysed for this analyte
ND Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. “Temperature Upon Receipt” is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” published by the Ministry and dated March 9, 2004 as amended.

-- End of Analytical Report --

APPENDIX C

Piezocone Penetration Testing Report

PIEZOCONE PENETRATION TESTING
PROPOSED BRIDGE OVER DRIFTWOOD RIVER
HIGHWAY 577, MATHESON, ONTARIO

For:
Terraprobe Inc.
11 Indell Lane
Brampton, Ontario
L6T 3Y3

January 2019
Ref. No. D18138

DownUnder Geotechnical Limited
P.O. Box 96737, Jane/Major Mackenzie P.O., 2943 Major Mackenzie Drive, Maple, Ontario L6A 0A2
Tel 905-553-2483 Toll Free Fax 1-866-478-4593 Email office@downundergeotechnical.com

TABLE OF CONTENTS

1.0 INTRODUCTION

Downunder Geotechnical Limited (Downunder Geotechnical) was retained by Terraprobe to carry out two PiezoCone Penetration Tests (CPTus), with dissipation tests, at the proposed Highway 577 bridge over Driftwood River near Matheson, Ontario. This report contains the findings of piezocone soundings advanced by Downunder Geotechnical.

2.0 FIELD TESTING PROCEDURES

The CPTu soundings were carried out on November 19 and 20, 2018. Two CPTs (CPT-1 and CPT-2) soundings were carried out in general accordance with ASTM standards (D 5778). The CPT soundings were carried out using a CME drill rig owned and operated by Landcore Drilling of Chelmsford, Ontario, under the full-time supervision of ConeTec Investigations Ltd.

A 35mm diameter instrumented cone and friction sleeve assembly was hydraulically thrust into the soil at a rate of about 2 cm/s. The soundings were conducted using a 15 tonne capacity ConeTec cone with a tip area of 15 cm^2, a friction sleeve area of 225 cm^2 and a u_2 filter location. The pore pressure filter was pre-saturated with glycerine. The cone measures tip resistance, friction and pore pressures. Measurements were taken at about 2.5 cm depth intervals during penetration.

Figure No.1 presents the approximate CPTu locations. The CPTu soundings are included graphically in Appendix A.

3.0 CPT RESULTS

The results of the sounding are presented in Appendix A. The sounding logs comprise the measured results and soil behaviour classification. Interpreted geotechnical parameters are discussed in Section 4.0. The following provides a brief discussion on each of the measured results.

Tip Resistance

The CPTu provides a continuous measurement of the cone resistance, q_c. The measured cone resistance is corrected to total cone resistance, q_t, using the following equation,

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

where u_2 = pore pressure acting behind the cone
a = cone area ratio = A_n/A_c
= 0.80 for ConeTec cone
A_n = cross-sectional area of the load cell or shaft
A_c = projected area of the cone

Sleeve Friction and Friction Ratio

The friction along the cone sleeve, f_s, is continuously measured during cone penetration. Friction Ratio is a commonly used parameter for determination of soil profiling and classification. Friction ratio is determined by the following equation.

$$FR\ (\%) = \frac{f_s}{q_t}$$

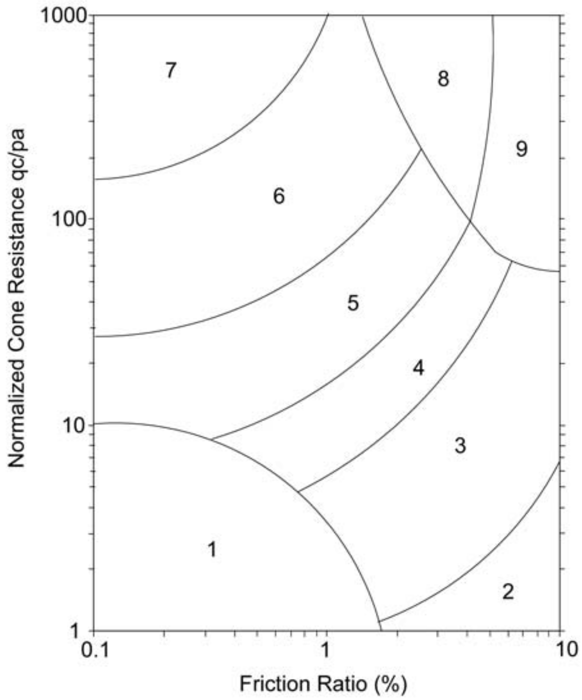
Pore Pressure

Continuous measurements of porewater pressure are taken during penetration. Due to the dynamic nature of the cone penetration, the porewater pressure measurements within fine grained soils are not representative due to undrained conditions and may even be negative in overconsolidated soils or dilatant silts.

Dissipation tests within fine grained soils are carried out by stopping penetration and measuring the change in excess porewater pressure over time. These results can provide an indication of hydraulic conductivity and consolidation characteristics, as well as soil behaviour – drained or undrained. In normally consolidated soils the excess porewater pressures dissipate during the test. In heavily overconsolidated or dilatant soils there is a delay in porewater pressure dissipation due to redistribution of the excess pore pressure behind the shoulder of the cone tip and the excess porewater pressures increase to a maximum before dissipating. The time for dissipation is also an indicator of drained or undrained behaviour. Sixteen (16) dissipation tests were carried out during stoppage in penetration. The results are summarized in Appendix B.

Soil Behaviour Type

One of the main applications of CPTu soundings is for rapid soil profiling and classification. Normalized soil behaviour type (SBTn) on the sounding logs is based on the classification chart by Roberston (1990). A reproduction of one of the charts and the soil behaviour types are presented in the chart below. The chart is typically a 2-chart system, one assessing normalized cone resistance vs. friction ratio and the second chart assessing normalized cone resistance vs. pore pressure ratio (which is not presented).



NORMALIZED
SOIL BEHAVIOUR TYPE
(after Robertson 1990)

ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty Clay to Clay
5		Silty Sand to Sandy Silt
6		Sand to Silty Sand
7		Sand
8		Very dense/stiff soil*
9		Very dense/stiff soil*

* heavily overconsolidated and/or cemented

To simplify the SBTn charts, Jefferies and Davies (1993) proposed a CPTu Soil Index I_C, which is also used as an indicator for soil stratigraphy, and was further normalized by Robertson (2009).

$$I_C = [(3.47-\log(Q_t))^2 + (1.22 + (\log F))^2]^{0.5}$$

where Q_t = normalized tip resistance = $(q_t - \sigma_{V0}) / \sigma_{V0}'$
F = normalized sleeve friction = $f_s / (q_t - \sigma_{V0})$

It should be noted that the above chart is an indication of soil behaviour and not an indication of grain size distribution.

4.0 INTERPRETATION

Undrained Shear Strength

The relationship between cone resistance and undrained shear strength can be empirically represented by the following equation.

$$S_u = \frac{(q_t - \sigma_V)}{N_{kt}}$$

where S_u = undrained shear strength (kPa)
σ_V= vertical stress (kPa)
N_{kt}= dimensionless constant

Typically N_{kt} varies from 10 to 20, with higher results in fissured clay, silts or varved clay deposits. Published empirical correlations also exist relating undrained shear strength, in situ effective vertical stress and OCR. A N_{kt} of 16.5 provides excellent correlation with the in situ shear vane test results.

Equivalent N_{60} SPT Value

Based on Jefferies and Davies (1993) the following empirical equation is used to correlate to equivalent Standard Penetration Test results.

$$N_{60} = \frac{q_C}{0.85 \times (1 - I_C/4.75)}$$

where q_C = tip resistance (MPa)
I_C = Soil Classification Index

Overconsolidation Ratio (OCR)

The estimate of the overconsolidation ratio, OCR, in clays is based on the following equation,

$$OCR = k\,(q_t - \sigma_v)/\sigma'_v$$

Where k is constant typically ranging from 0.3 to 0.5 for clays. A ‘k’ value of 0.5 was used for the clay deposits at the site, which is generally the upper range for ‘k’ values in clays. This value was used to ensure the lower bound of the interpreted OCR was at least 1.0.

Constrained Modulus

The constrained modulus, M, represents the deformation characteristics of soils for preconsolidation stresses, and is a function of the stress history, drainage condition and the stress path direction of the soil. The estimate of M for clayey soils can be estimated using the method proposed by Senneset et al (1982).

$$M = 1/m_v = \alpha_m\,(q_t - \sigma_V)$$

where m_v = coefficient of volume change
α_m = constant

Senneset et al Method:

For SBTn <6 (Silts, Clays and Clayey Silts):
$\alpha_m = 4.5$

The α_m value above was selected based on odeometer testing in similar soils.

Coefficient of Consolidation

The horizontal coefficient of consolidation (C_h) of the soil can be estimated from the pore pressure dissipation test results. Montonic and dilatory excess pore pressure dissipation was observed in the sixteen (16) tests carried out at the site. The method by Houlsby and Teh (1988) was used to determine C_h, as follows.

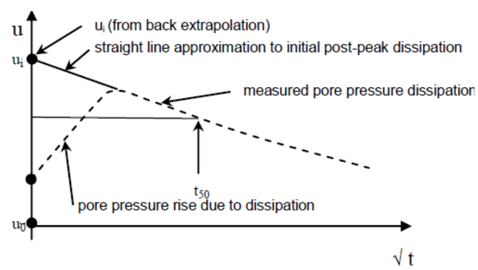
$$C_h = \frac{T^*_{50}}{t_{50}} \, r^2 \, I_R^{0.5}$$

where T^*_{xx}= time factor from theoretical solutions
t_{xx}= measured time for xx% dissipation
r = penetrometer radius = 22 mm
I_R = undrained rigidity index (based on plasticity)
G = shear modulus

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

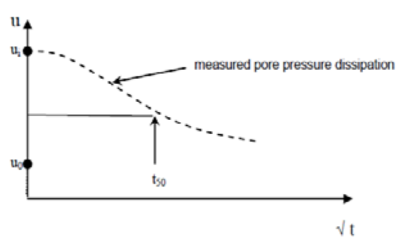
Degree of Dissipation	20%	30%	40%	50%	60%	70%	80%
$T^*(u_2)$	0.038	0.078	0.142	0.245	0.439	0.804	1.60

Due to the dilative nature of most of the soils, the excess pore pressure increased during the test before dissipating (dilatory behaviour). In order to determine the 20 to 50% dissipation the test measurements were plotted for excess pore pressure vs root time scale. The initial excess pore pressure was then estimated by extrapolating back to time zero as presented in the following sketch.



Overconsolidated or Dilative Soils

In other tests an increase in pore pressure was not observed and the following sketch represents the monotonic behaviour observed.



From the initial pore pressure estimation, the normalized excess pore pressure was determined and plotted vs time. Normalized excess pore pressure was determined based on the following equation.

$$U = \frac{u_t - u_0}{u_i - u_0}$$

where u_t = excess pore water pressure measurement at time t
u_0= in situ pore pressure based on the CPT results
u_i = initial excess pore water pressure at beginning of dissipation test

To correlate C_h to the vertical coefficient of consolidation (C_v) the following equation was used:

$$C_v = C_h\, k_v/k_h$$

where k_v/k_h ratio is suggested in the table below from Jamiolkowski (1985).

Nature of Clay	k_h/k_v
No macrofabric or slightly developed macrofabric (homogeneous deposit)	1 to 1.5
Fairly well to well developed macrofabric (eg. sedimentary clays with discontinuous lenses and layers of more permeable material)	2 to 4
Varved clays and other deposits containing embedded and more or less continuous permeable layers	3 to 15

The results are considered to be approximate and reasonable to within an order of magnitude.

5.0 RESULTS

The CPTu soundings penetrated a soft to very stiff silty clay deposit. The following average values are provided for the silty clay based on the CPTu results. Selection of soil parameters should be based on engineering judgement (average values, characteristic values, etc.).

Stiff Silty Clay "Crust"

CPT	q_t (MPa)	N_{60}	OCR	M (MPa)	S_u (kPa)	C_h (m^2/day)
18-1	1.1	4	9.3	4.8	65	-
18-2	1.1	4	6.8	4.8	65	-

Soft to Firm Stiff Silty Clay

CPT	q_t (MPa)	N_{60}	OCR	M (MPa)	S_u (kPa)	C_h (m^2/day)
18-1	0.6	3	1.8	1.9	25	0.060
18-2	0.7	3	1.5	2.1	28	0.067

6.0 REFERENCES

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7.0 LIMITATION OF REPORT

Subsurface and groundwater conditions beyond the CPTu locations may differ from those encountered at the CPTu location. The information herein in no way reflects on the environmental aspects of the project.

This report has been prepared for this specific project and the information herein is not applicable to any other project or site location. This report is for use by the client. Any use of this report by another third party, or any reliance on or decisions to be made based on it, are the responsibility of such third parties. Downunder Geotechnical does not take any responsibility for the use of the soil parameters summarized in this report unless consulted during geotechnical design.

Report prepared by:

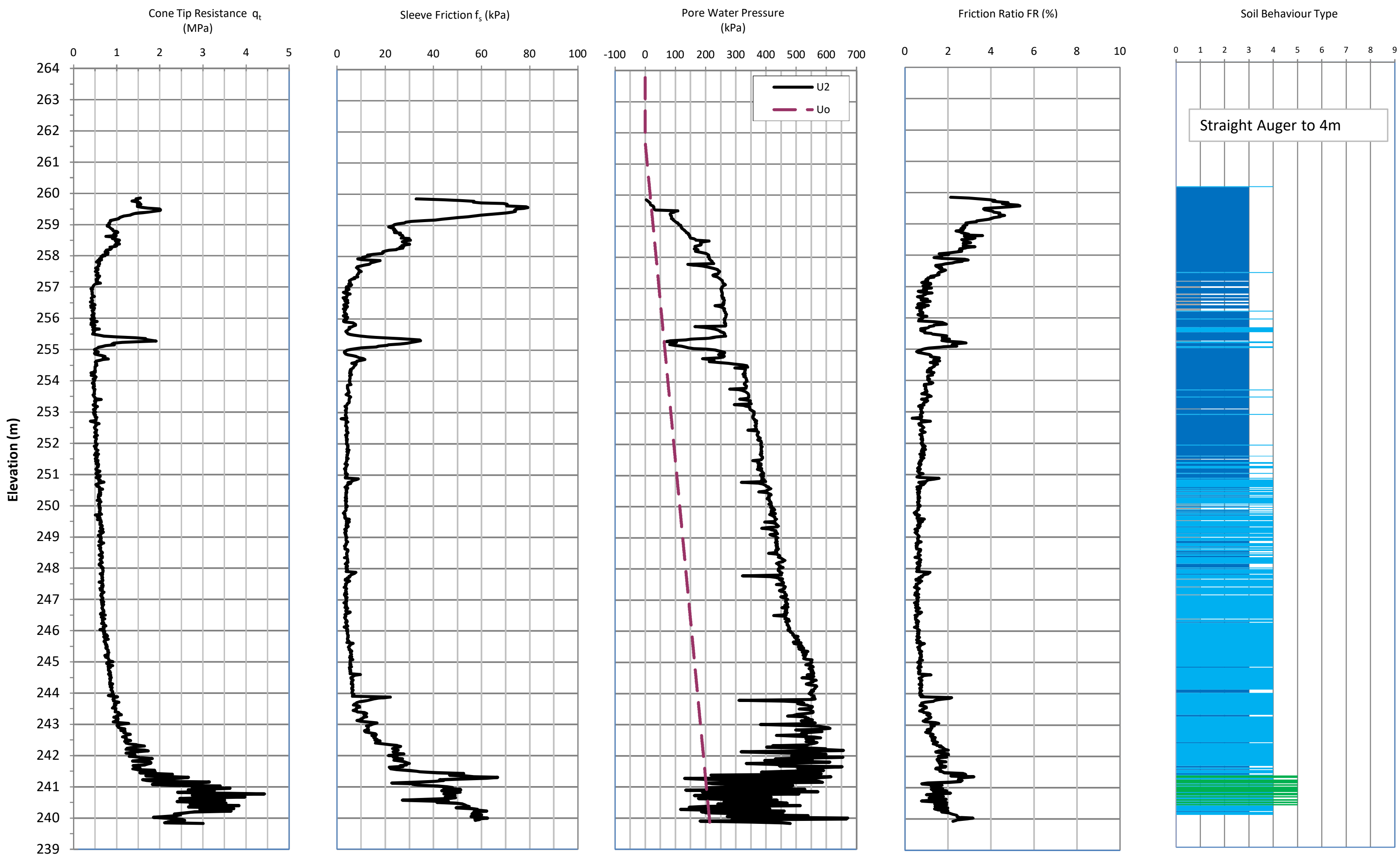
LICENSED PROFESSIONAL ENGINEER
17 January 2019
A.R. DREVININKAS
90502154
PROVINCE OF ONTARIO

Andrew Drevininkas, P. Eng.
President

FIGURE NO. 1
CPT LOCATION PLAN

APPENDIX A – Piezocone Sounding

PiezoCone Penetration Test



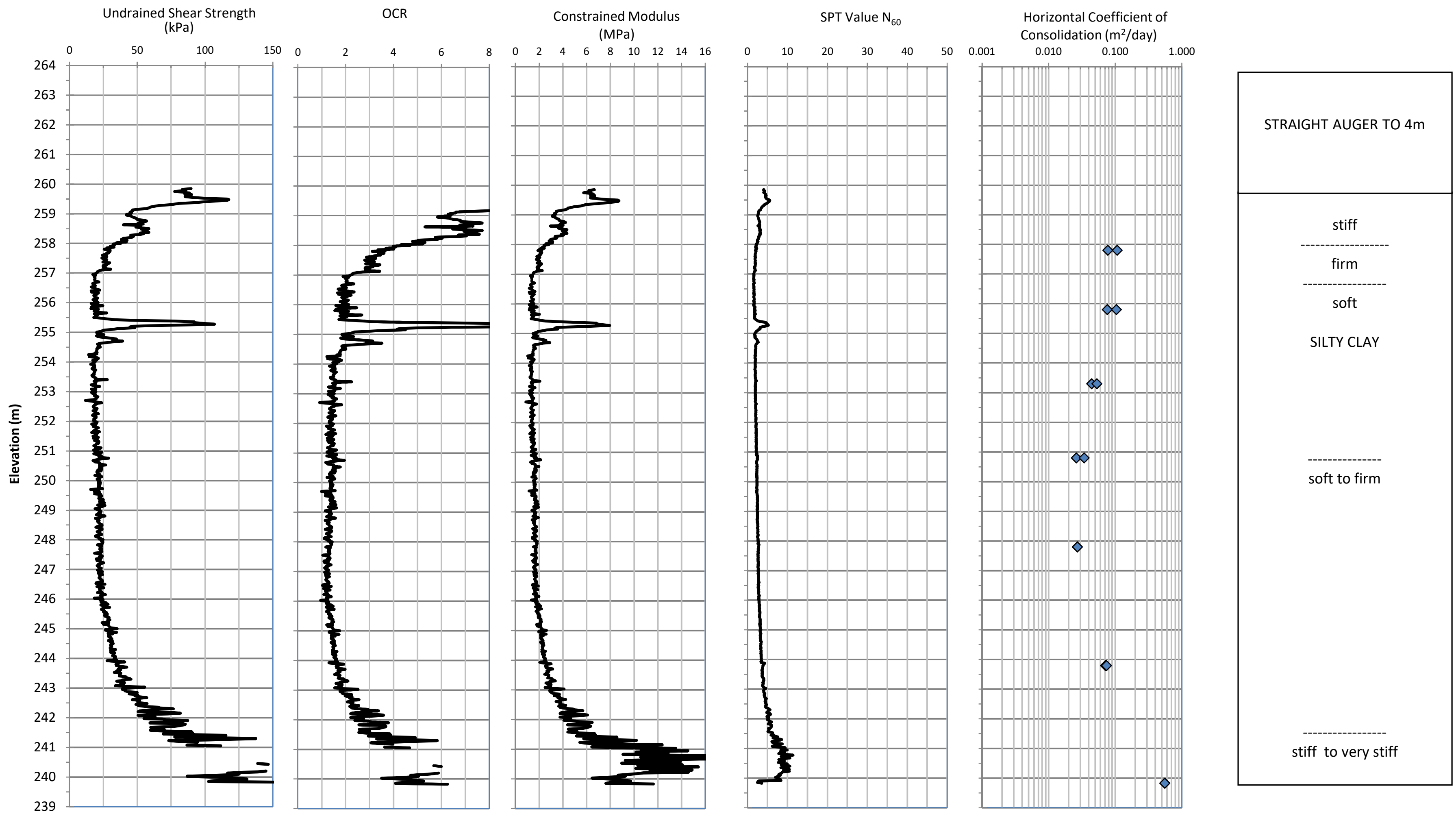
CPT-18-1

Date: November 19, 2018
Location: Highway 577 at Driftwood River near Matheson, Ontario
Engineer: Conetec Investigations
Cone: Conetec 15 tonne
Tip Area: 15 cm^2
Friction Sleeve Area: 225 cm^2
Filter Location: U_2
MTM NAD 83 (Zone 12) Northing 5379213.1 Easting 328350.6 Elevation 263.8m

CPT Probe 472:T1500F15U1K

DownUnder Geotechnical Limited

PiezoCone Penetration Test



Date: November 19, 2018
Location: Highway 577 at Driftwood River near Matheson, Ontario
Engineer: Conetec Investigations
Cone: Conetec 15 tonne
Tip Area: 15 cm^2
Friction Sleeve Area: 225 cm^2
Filter Location: U_2

CPT-18-1

CPT Probe 472:T1500F15U1K

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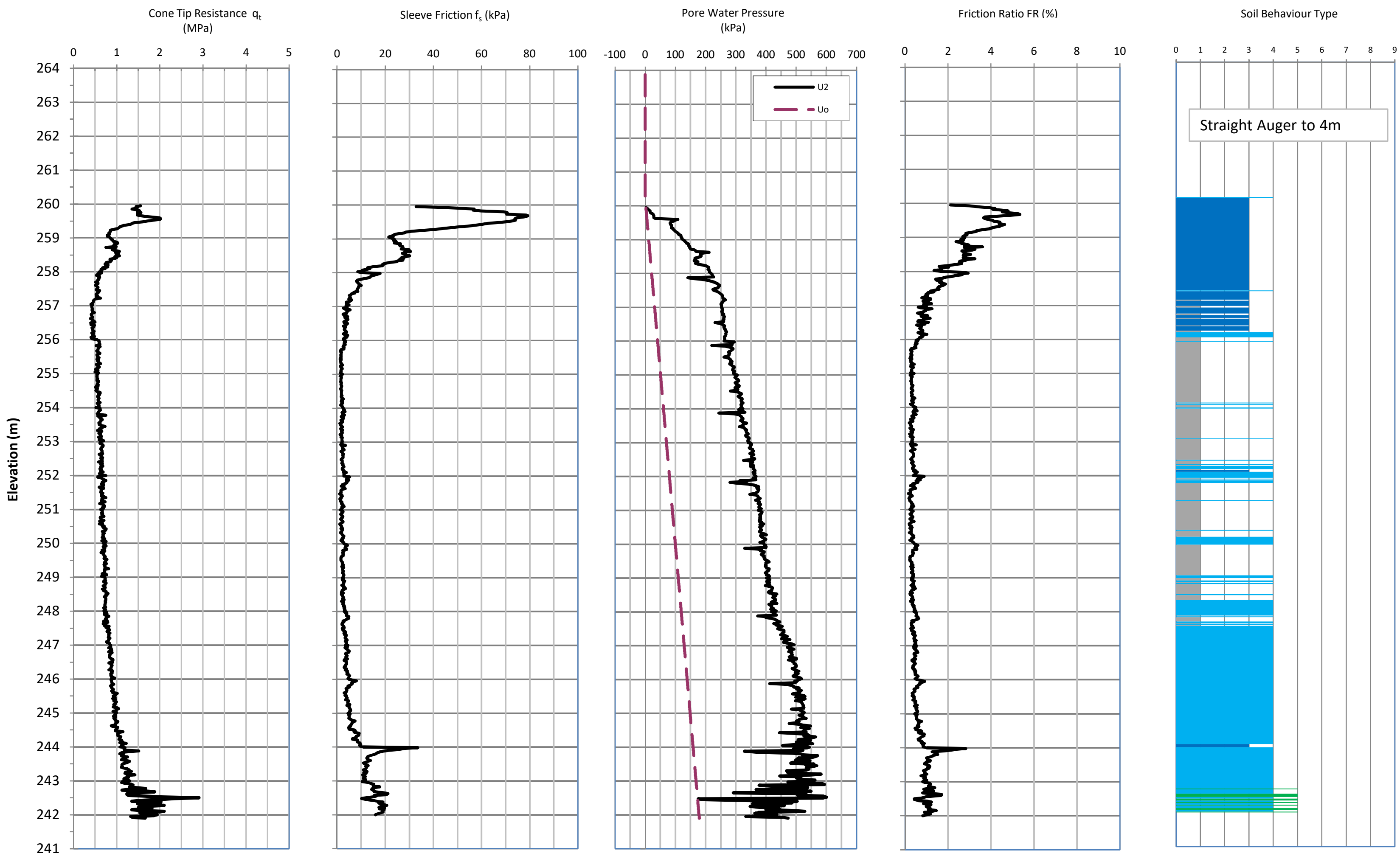
PiezoCone Penetration Test



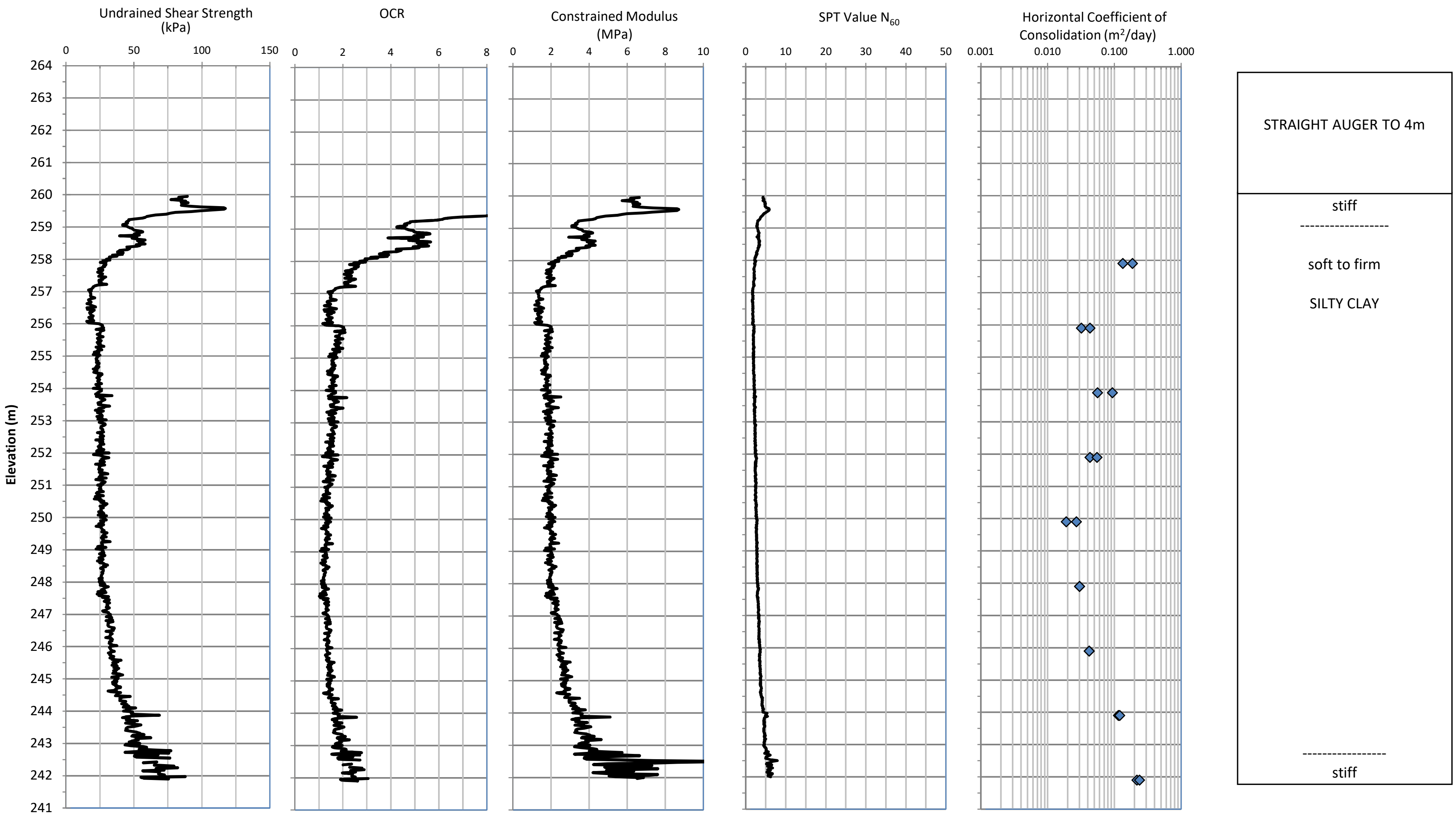
CPT-18-2

Date: November 20, 2018
Location: Highway 577 at Driftwood River near Matheson, Ontario
Engineer: Conetec Investigations
Cone: Conetec 15 tonne
Tip Area: 15 cm^2
Friction Sleeve Area: 225 cm^2
Filter Location: U_2
MTM NAD 83 (Zone 12) Northing 5379313.4 Easting 328355.4 Elevation 263.9m

CPT Probe 472:T1500F15U1K

DownUnder Geotechnical Limited

PiezoCone Penetration Test



Date: November 20, 2018
Location: Highway 577 at Driftwood River near Matheson, Ontario
Engineer: Conetec Investigations
Cone: Conetec 15 tonne
Tip Area: 15 cm^2
Friction Sleeve Area: 225 cm^2
Filter Location: U_2

CPT-18-2

CPT Probe 472:T1500F15U1K

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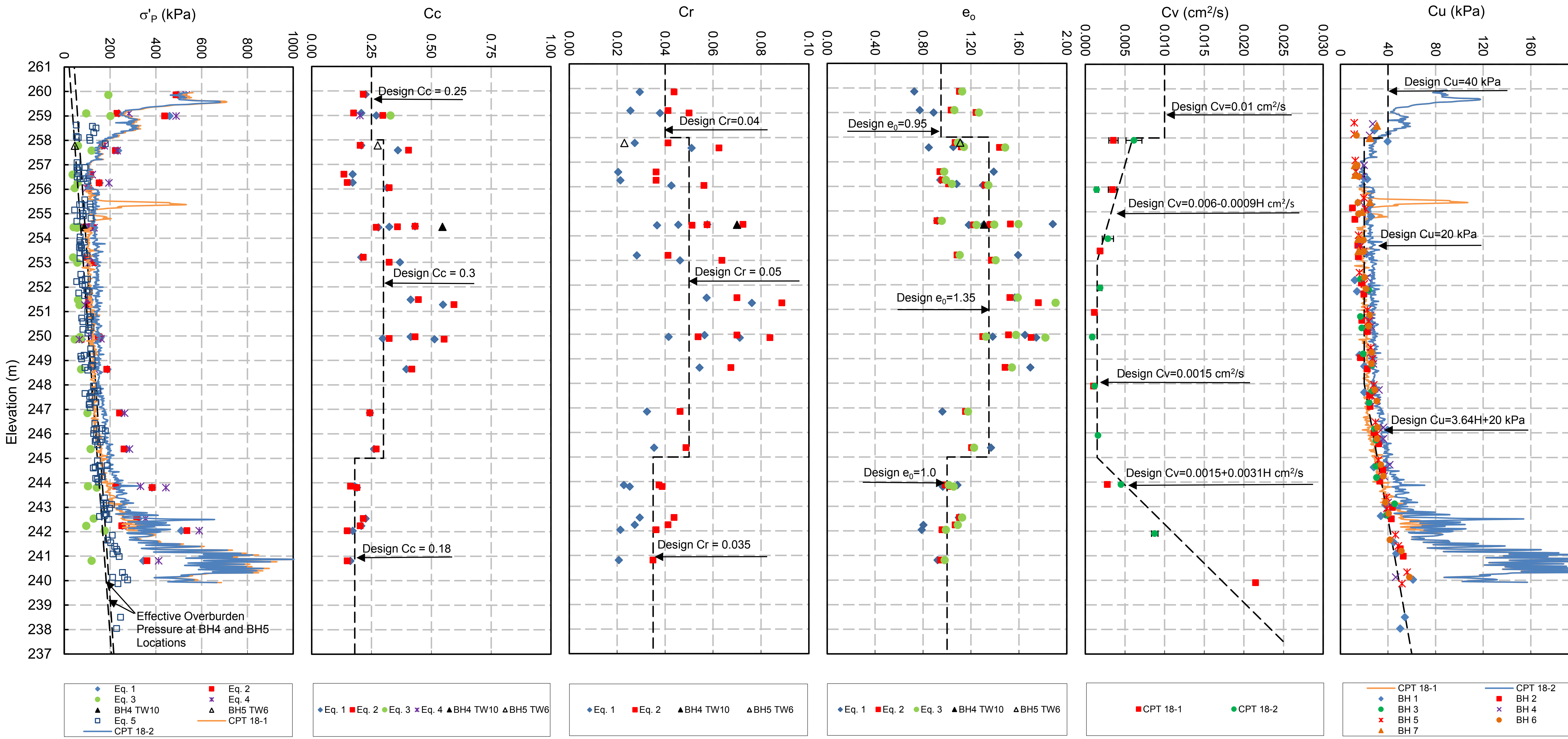
APPENDIX B – Dissipation Test Results

Summary of Dissipation Test Results

CPT	Depth (m)	u_i (kPa)	u_0 (kPa)	t_{xx} (min)	I_r (kPa)	C_h (cm^2/min)	C_h (m^2/day)	Response
18-1	6.0	290	37	T_{60} = 13.3	107 207	0.530 0.740	0.077 0.107	Dilatory
	8.0	248	57	t_{80} = 4.2	143 268	0.528 0.723	0.076 0.104	Monotonic
	10.5	322	81	t_{80} = 7.2	144 205	0.308 0.367	0.044 0.053	Monotonic
	13.0	405	106	t_{80} = 7.8	60 97	0.184 0.234	0.026 0.034	Monotonic
	16.0	455	135	t_{80} = 10.0	106	0.189	0.027	Monotonic
	20.0	557	175	t_{80} = 5.6	224 245	0.493 0.516	0.071 0.074	Monotonic
	24.0	720	214	t_{50} = 4.3	197	3.841	0.553	Dilatory
18-2	6.0	271	40	t_{60} = 7.7	107 207	0.927 1.290	0.134 0.186	Monotonic
	8.0	294	60	t_{80} = 10.0	143 267	0.220 0.301	0.032 0.043	Monotonic
	10.0	330	79	t_{80} = 4.8	101 279	0.389 0.647	0.056 0.093	Monotonic
	12.0	376	99	t_{80} = 4.8	60 97	0.300 0.381	0.043 0.055	Monotonic
	14.0	412	119	t_{80} = 11.7	68 144	0.130 0.189	0.019 0.027	Monotonic
	16.0	445	138	t_{80} = 9.0	106	0.210	0.030	Monotonic
	18.0	529	158	t_{80} = 8.3	172	0.289	0.042	Monotonic
	20.0	565	178	t_{70} = 7.1	224 245	0.798 0.834	0.115 0.120	Monotonic
	22.0	510	197	t_{60} =6.6	209 251	1.509 1.654	0.217 0.238	Monotonic

u_i = initial measured excess pore pressure for Monotonic response
= extrapolated maximum excess pore pressure for Dilatory response (as per Houlsby and Teh)
u_0 = pore water pressure at rest (assumed hydrostatic) with water levels at 1.9 to 2.2m below grade
t_{xx} = time for 20 to 50% excess pore water pressure dissipation
I_r = Undrained Rigidity Index - based on plasticity index
Assumed Bulk Unit Weight (γ) ~ 17.5 kN/m^3

APPENDIX D
Soil Design Parameters



Eq. 1 $\sigma'_P = Cu / (0.11 + 0.0037 * I_P)$
(Chandler (1988))
Eq. 2 $\sigma'_P = 22*(I_p^{-0.48})*Cu$
(Mayne and Mitchell (1988))
Eq. 3 derived from $OCR=3.22*Cu/\sigma'_{v0}$
(Mayne and Mitchell (1988))
Eq. 4 $\sigma'_P = 222 / LL * Cu$
(Hansbo (1957))
Eq. 5 $\sigma'_P = Cu(mob)/0.22$
(Mesri (1975))
CPTu data derived from $OCR=k(q_t-\sigma_v)/\sigma'_v$

Eq. 1 $Cc = 0.009 * (LL - 10)$
(Terzaghi & Peck (1967))
Eq. 2 $Cc = I_p / 74$
(Kulhaway & Mayne (1990))
Eq. 3 $Cc = 0.141 * Gs^{1.2} * ((1 + eo) / Gs)^{2.38}$
(Rendon - Herrero (1983))
Eq. 4 $Cc = 0.141 * Gs * (\gamma_w / \gamma_d)^{2.4}$
(Herrero (1983))

Eq. 1 $Cr = Cc / 5 \sim Cc / 10$
(Das (1993))
Eq. 2 $Cr = 0.000463*LL*Gs$
(Nagaraj & Murty (1985))

Eq. 1 $e_o = w * Gs$
(when saturated)
Eq. 2 $e_o = (Cc / 0.141)^{0.4202} * Gs^{0.4958} - 1$
(derived from Rendon - Herrero (1983))
Eq. 3 $e_o = Cc / 0.30 + 0.27$
(derived from Hough (1957))

$Cv = Ch/3$ New Liskeard Varved Clays
(Leroueil et. al. (1990))

Field vane shear strengths were corrected based on Aas, et al. (1986)
CPTu data derived from $Cu=(q_t-\sigma_v)/N_{kt}$

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HWY. 577		
DRIFTWOOD RIVER BRIDGE, SITE 39E-096		
G.W.P.: 5104-18-00	Date: January 2019	
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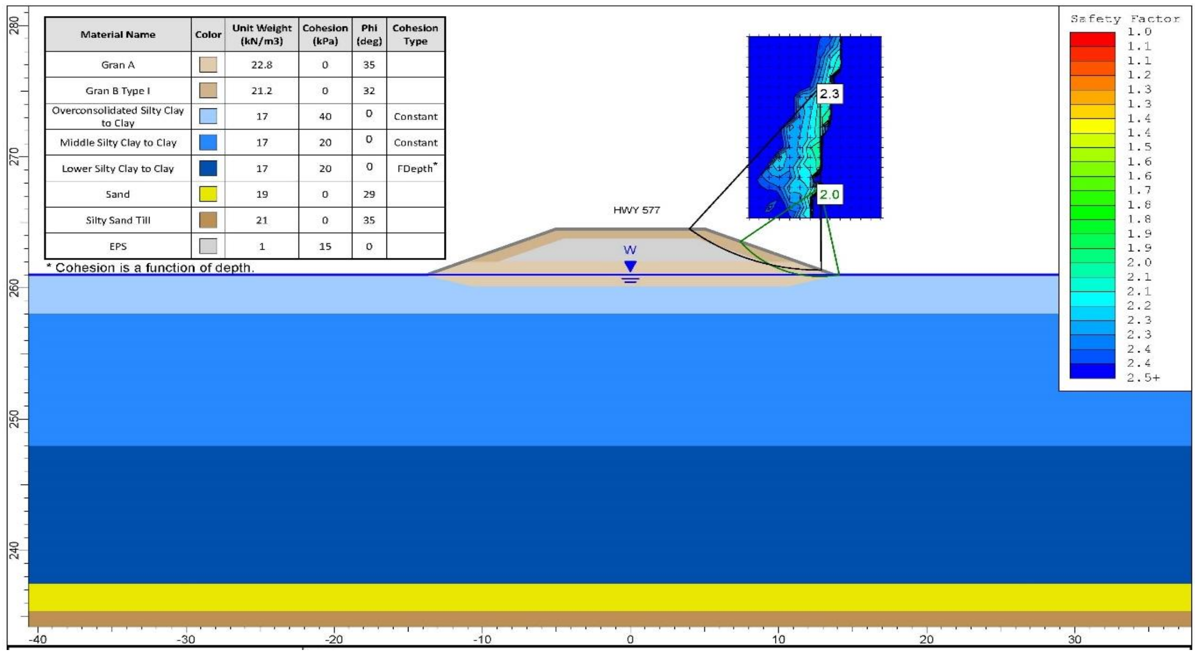
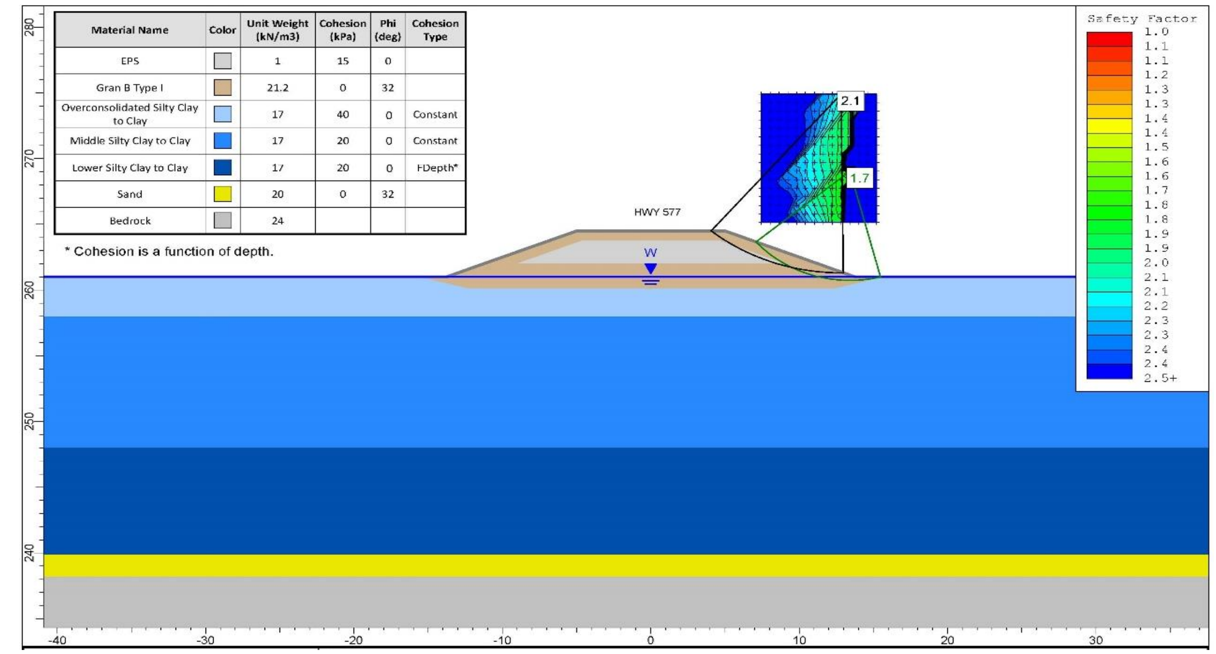
File No: 1-18-0689	Figure: D1	

APPENDIX E

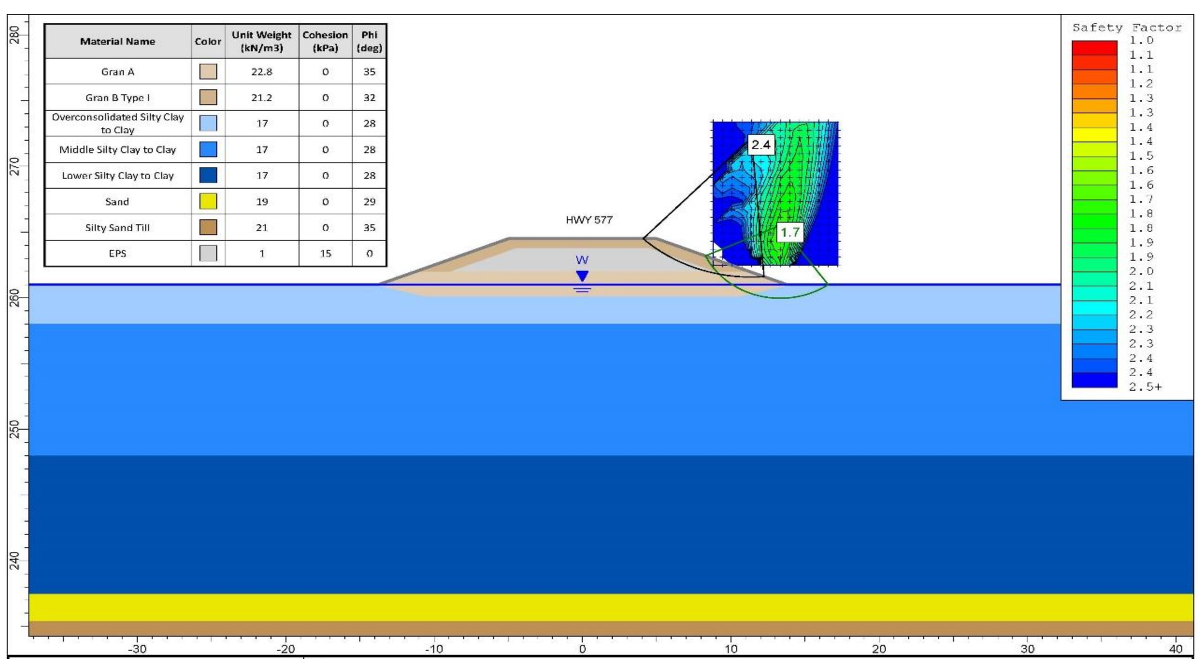
Slope Stability Models & Results



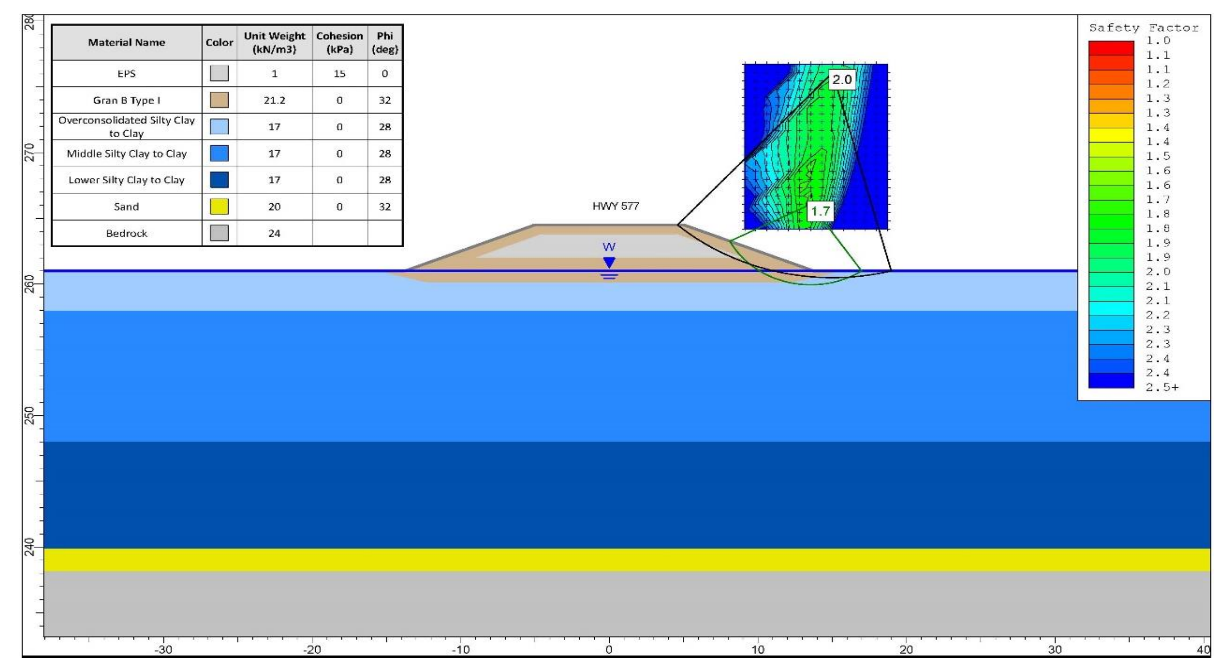
South Approach Embankment - Hwy 577 - Total Stress Analysis
(0m - 20m Transition Point Distance)



North Approach Embankment - Hwy 577 - Total Stress Analysis
(0m - 20m Transition Point Distance)



South Approach Embankment - Hwy 577 - Effective Stress Analysis
(0m - 20m Transition Point Distance)



North Approach Embankment - Hwy 577 - Effective Stress Analysis
(0m - 20m Transition Point Distance)

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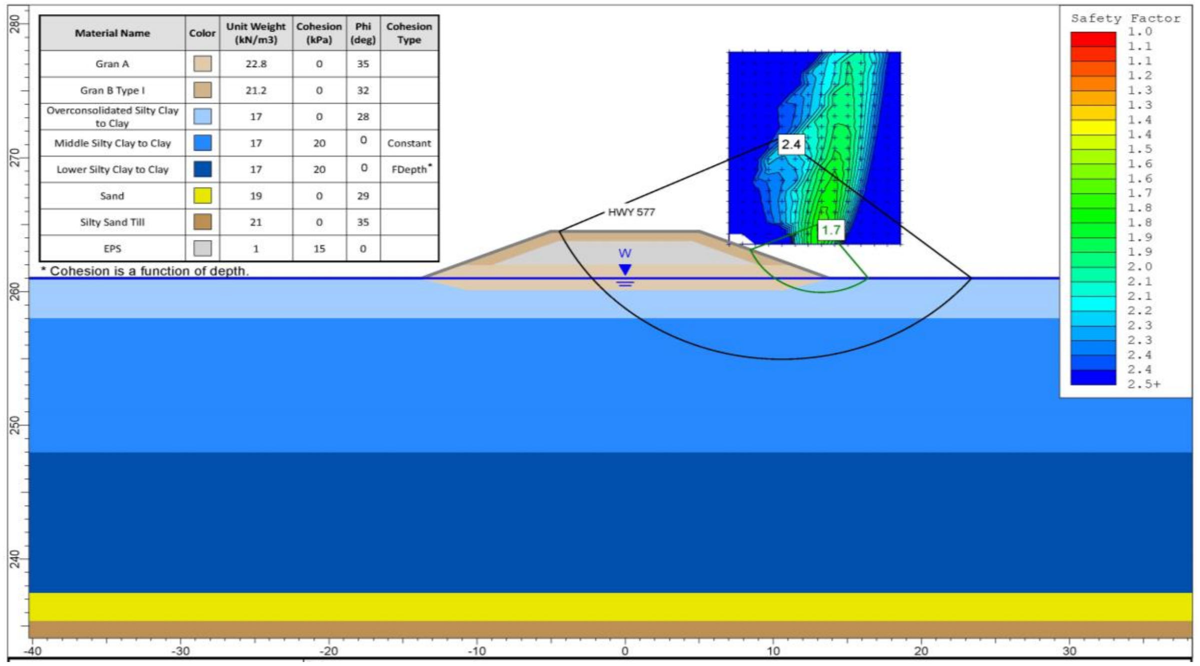
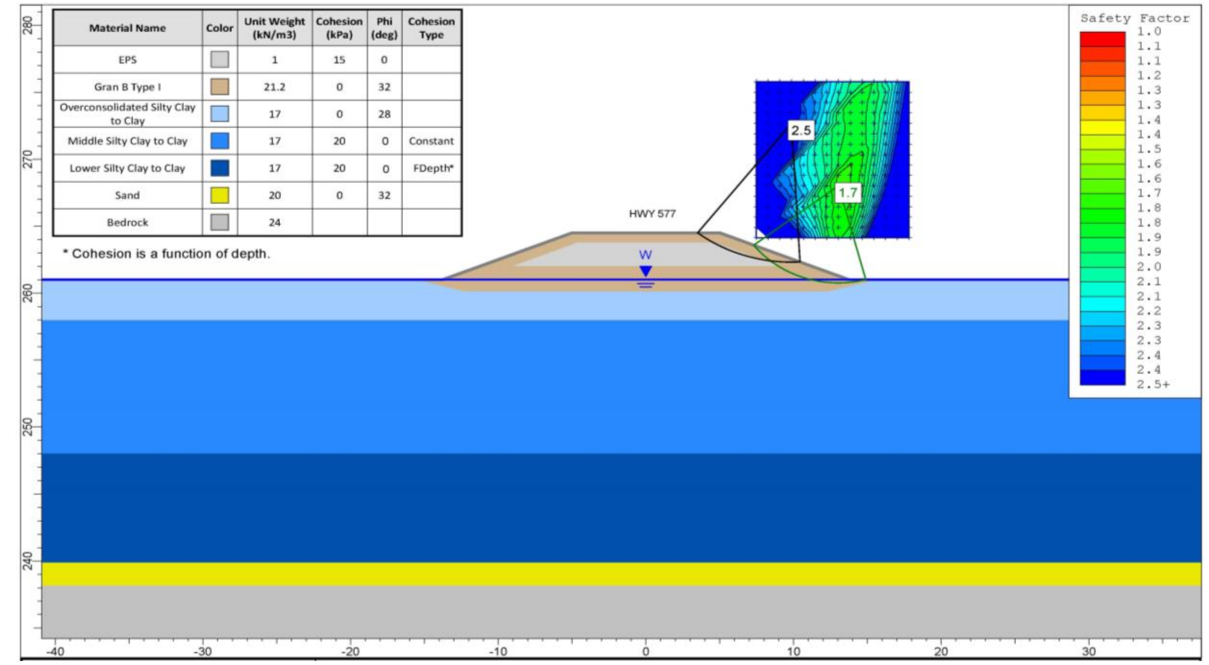
Project No: 1-18-0689		Figure: E1	



South Approach Embankment - Hwy 577 - Intermediate Condition
(0m - 20m Transition Point Distance)



North Approach Embankment - Hwy 577 - Intermediate Condition
(0m - 20m Transition Point Distance)

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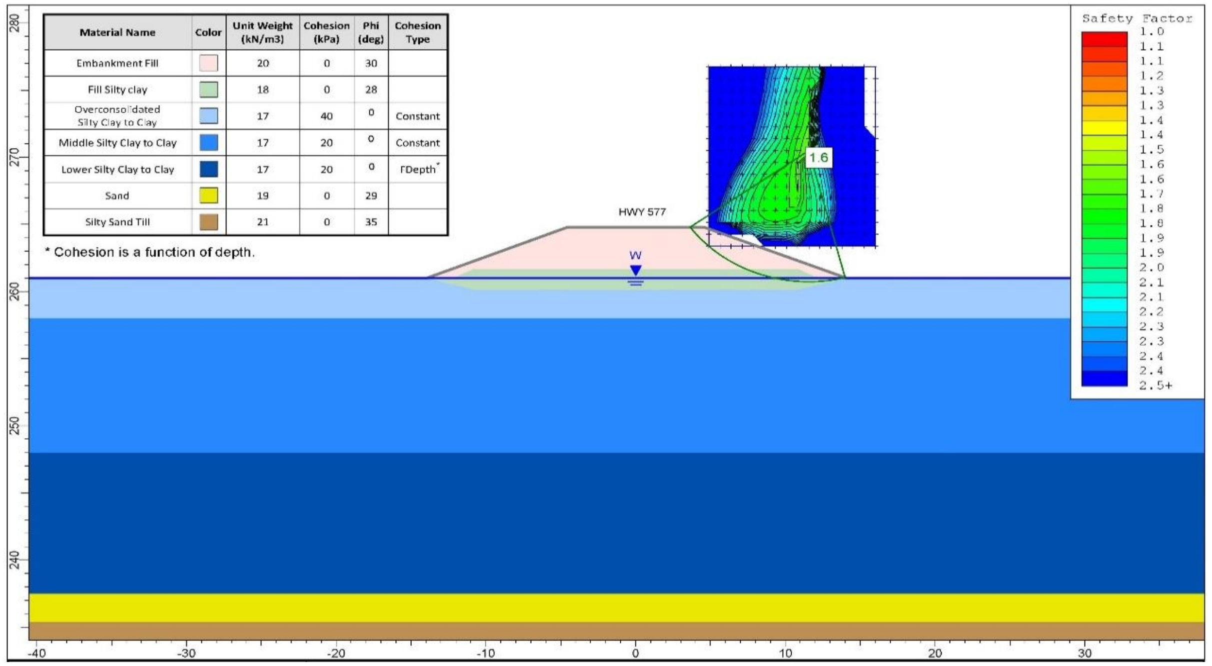
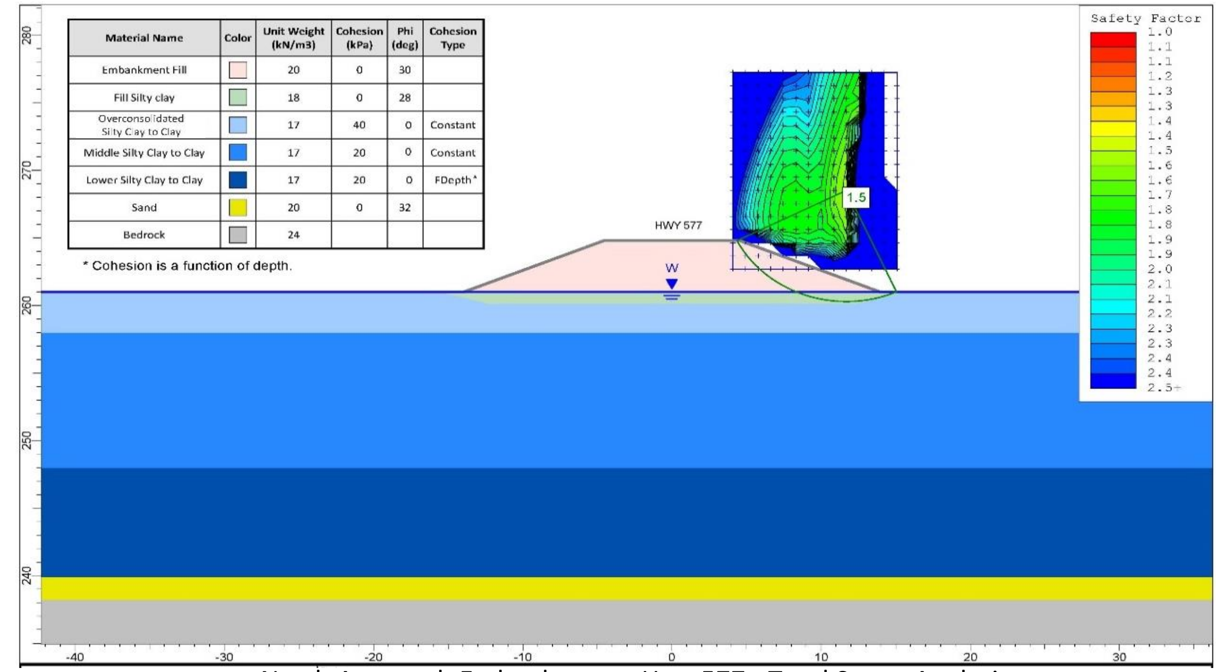
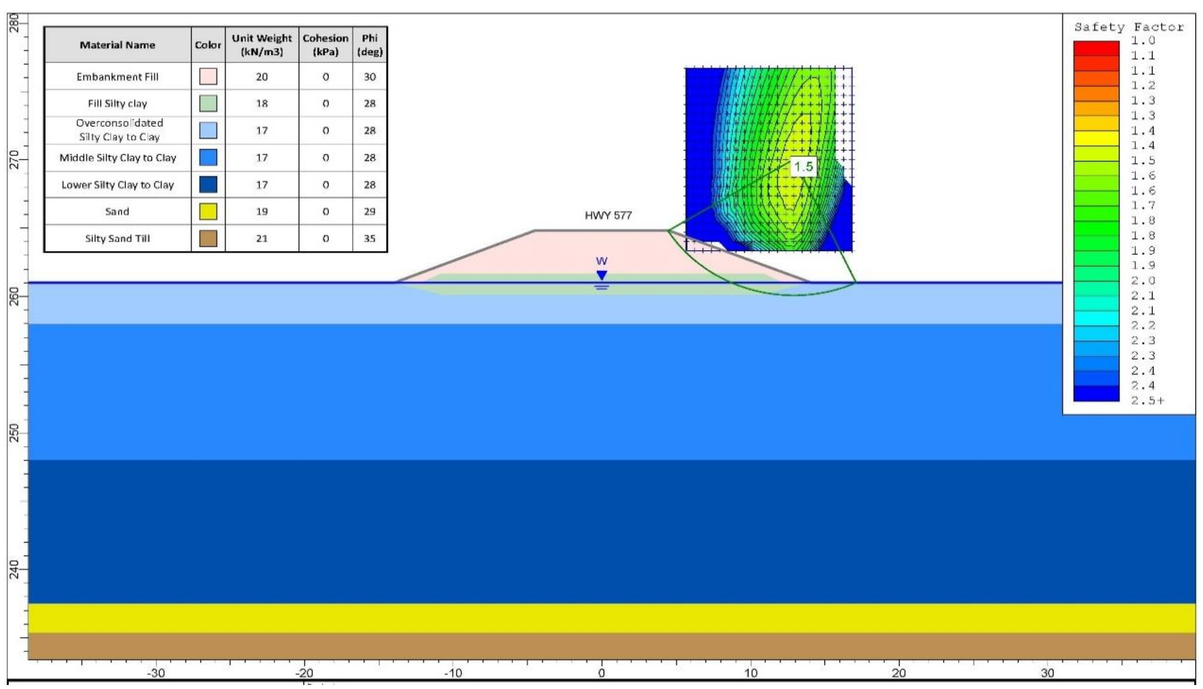


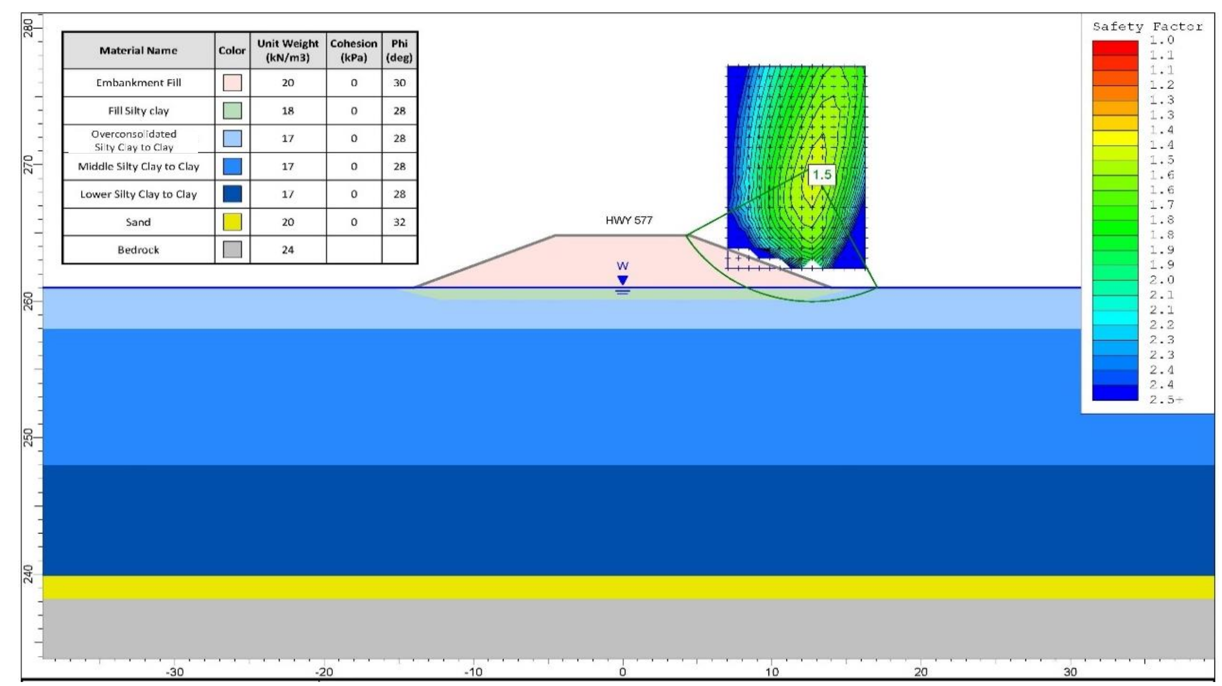
South Approach Embankment - Hwy 577 - Total Stress Analysis
(20m - 50m Transition Point Distance)



North Approach Embankment - Hwy 577 - Total Stress Analysis
(20m - 50m Transition Point Distance)



South Approach Embankment - Hwy 577 - Effective Stress Analysis
(20m - 50m Transition Point Distance)



North Approach Embankment - Hwy 577 - Effective Stress Analysis
(20m - 50m Transition Point Distance)

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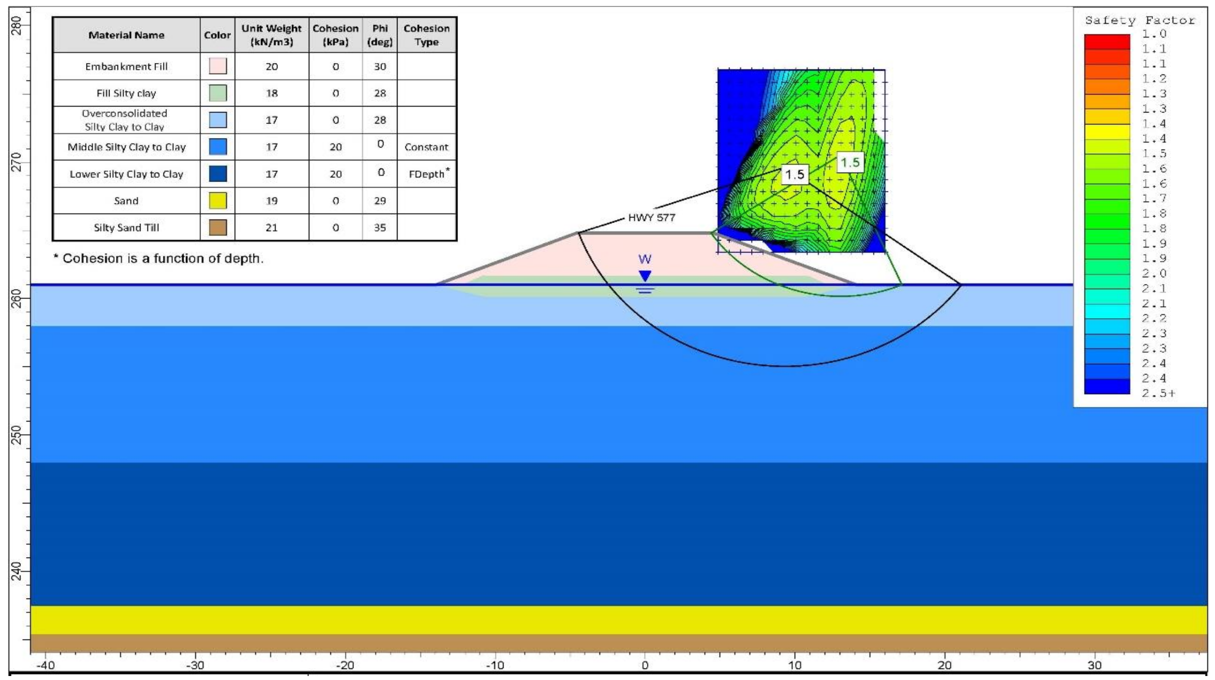
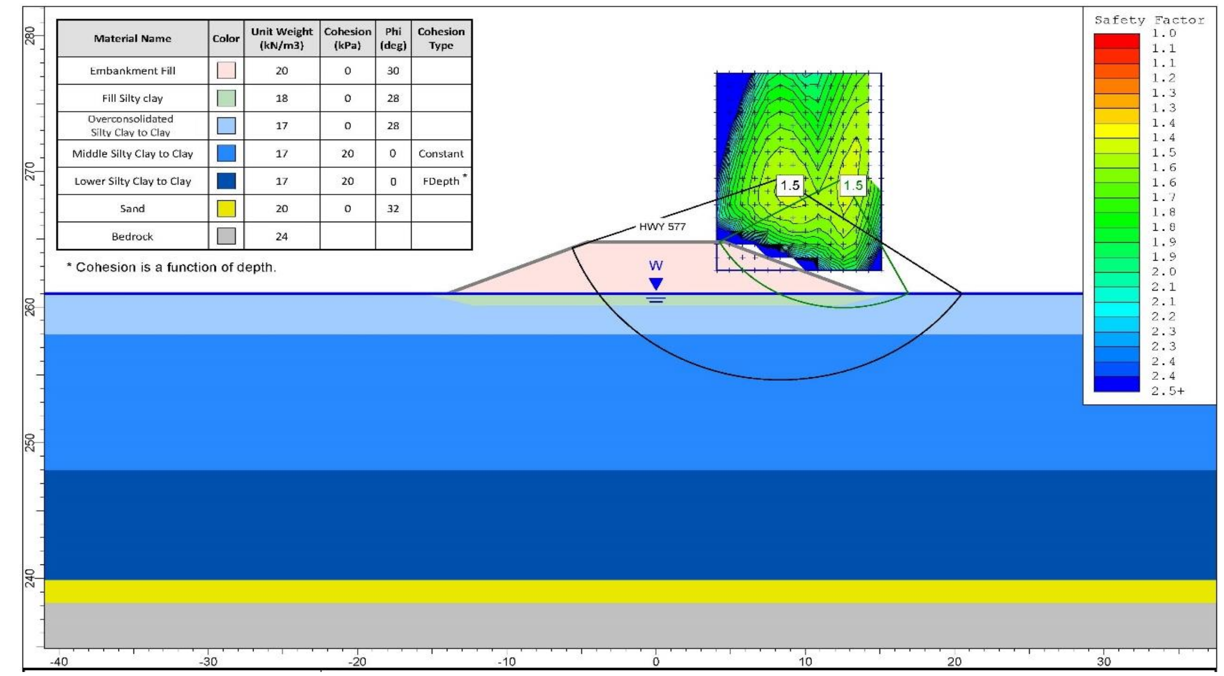
	SUBM'D: SD	CHKD: RA	APPD: MT
	Project No: 1-18-0689	Figure: E3	



South Approach Embankment - Hwy 577 - Intermediate Condition
(20m - 50m Transition Point Distance)



North Approach Embankment - Hwy 577 - Intermediate Condition
(20m - 50m Transition Point Distance)

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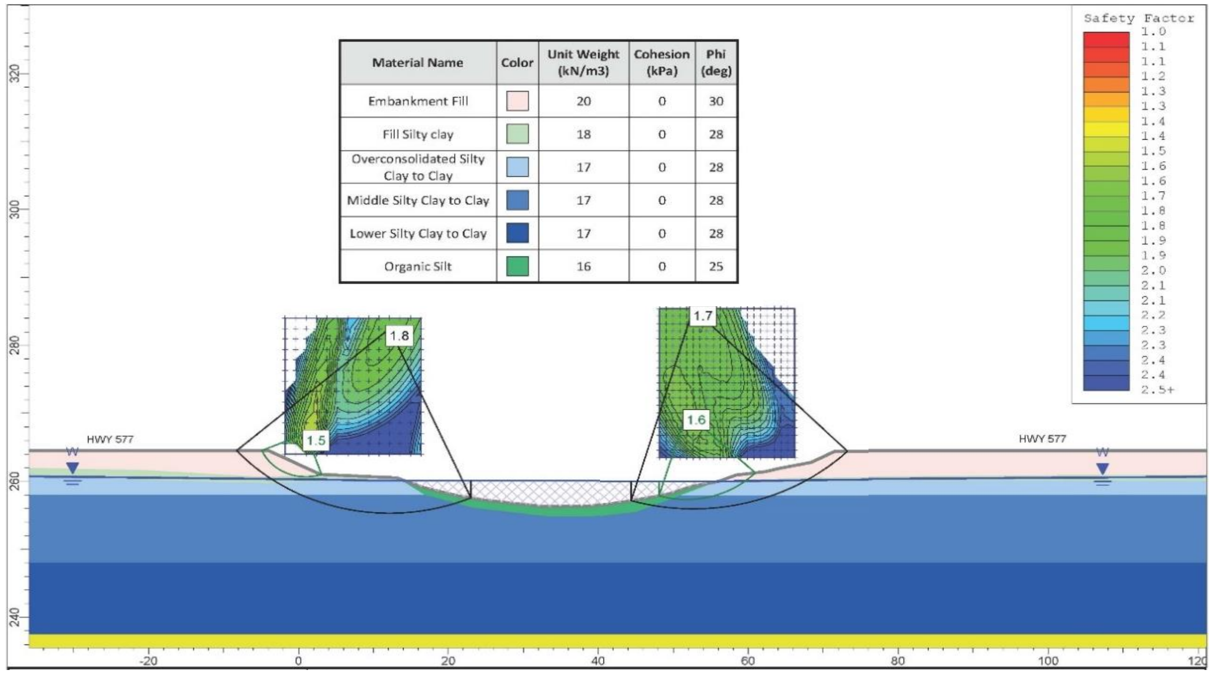
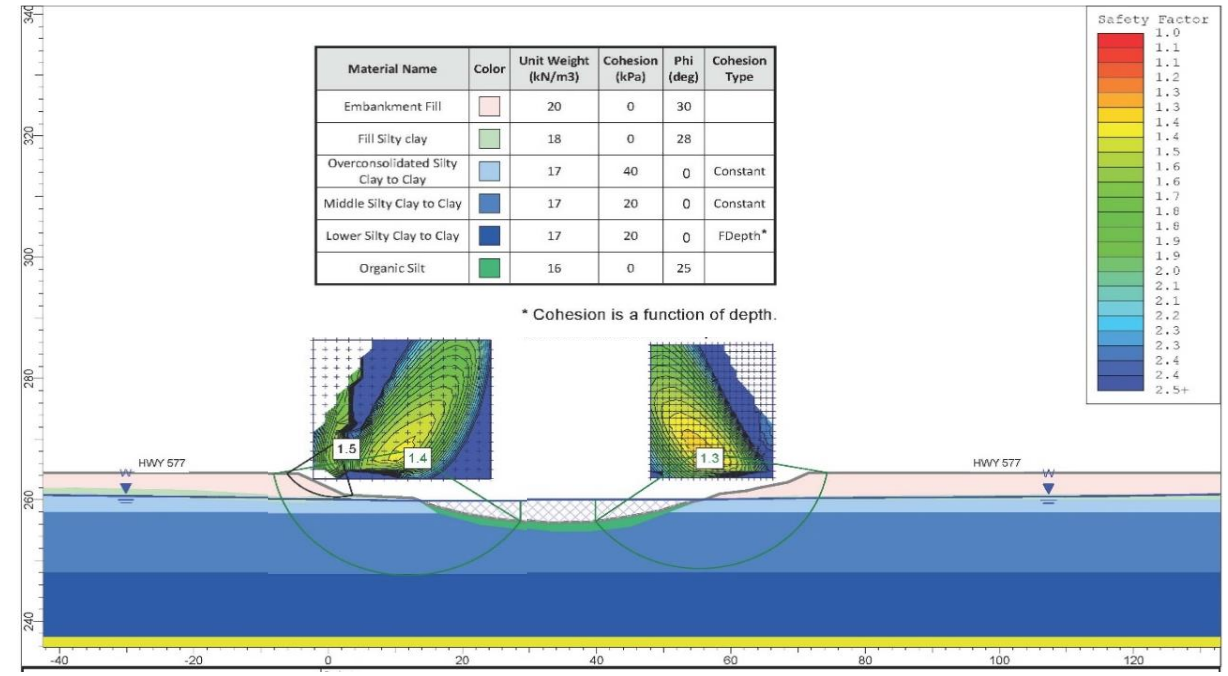
	Project No: 1-18-0689		Figure: E4



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Embankment Fill		20	0	30
Fill Silty clay		18	0	28
Overconsolidated Silty Clay to Clay		17	0	28
Middle Silty Clay to Clay		17	0	28
Lower Silty Clay to Clay		17	0	28
Organic Silt		16	0	25

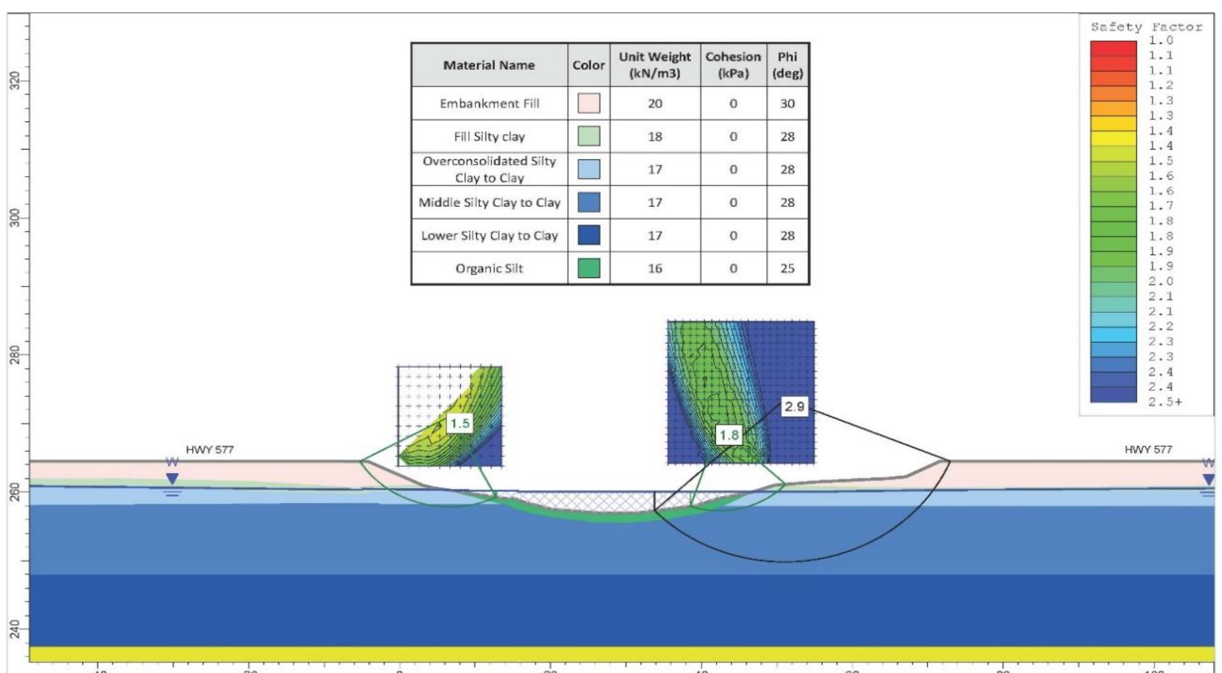
Forward Slopes - East Side of Hwy 577 - Effective Stress Analysis



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)	Cohesion Type
Embankment Fill		20	0	30	
Fill Silty clay		18	0	28	
Overconsolidated Silty Clay to Clay		17	40	0	Constant
Middle Silty Clay to Clay		17	20	0	Constant
Lower Silty Clay to Clay		17	20	0	FDepth*
Organic Silt		16	0	25	

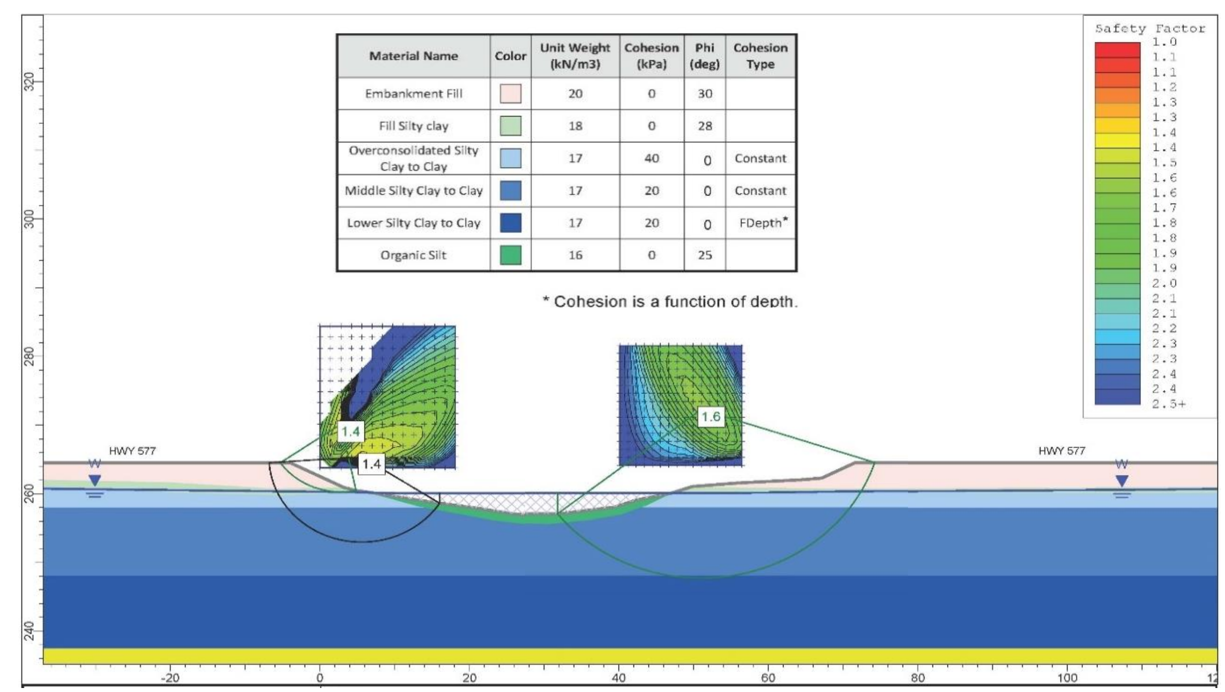
* Cohesion is a function of depth.

Forward Slopes - East Side of Hwy 577 - Total Stress Analysis



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Embankment Fill		20	0	30
Fill Silty clay		18	0	28
Overconsolidated Silty Clay to Clay		17	0	28
Middle Silty Clay to Clay		17	0	28
Lower Silty Clay to Clay		17	0	28
Organic Silt		16	0	25

Forward Slopes - West Side of Hwy 577 - Effective Stress Analysis



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)	Cohesion Type
Embankment Fill		20	0	30	
Fill Silty clay		18	0	28	
Overconsolidated Silty Clay to Clay		17	40	0	Constant
Middle Silty Clay to Clay		17	20	0	Constant
Lower Silty Clay to Clay		17	20	0	FDepth*
Organic Silt		16	0	25	

* Cohesion is a function of depth.

Forward Slopes - West Side of Hwy 577 - Total Stress Analysis

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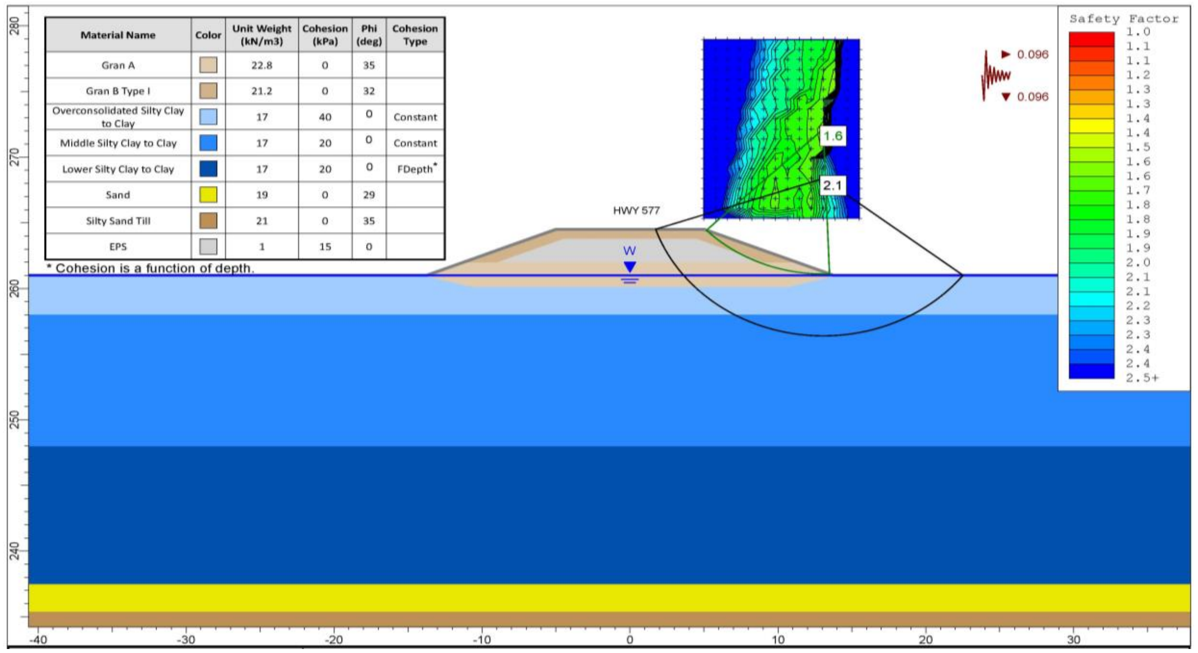
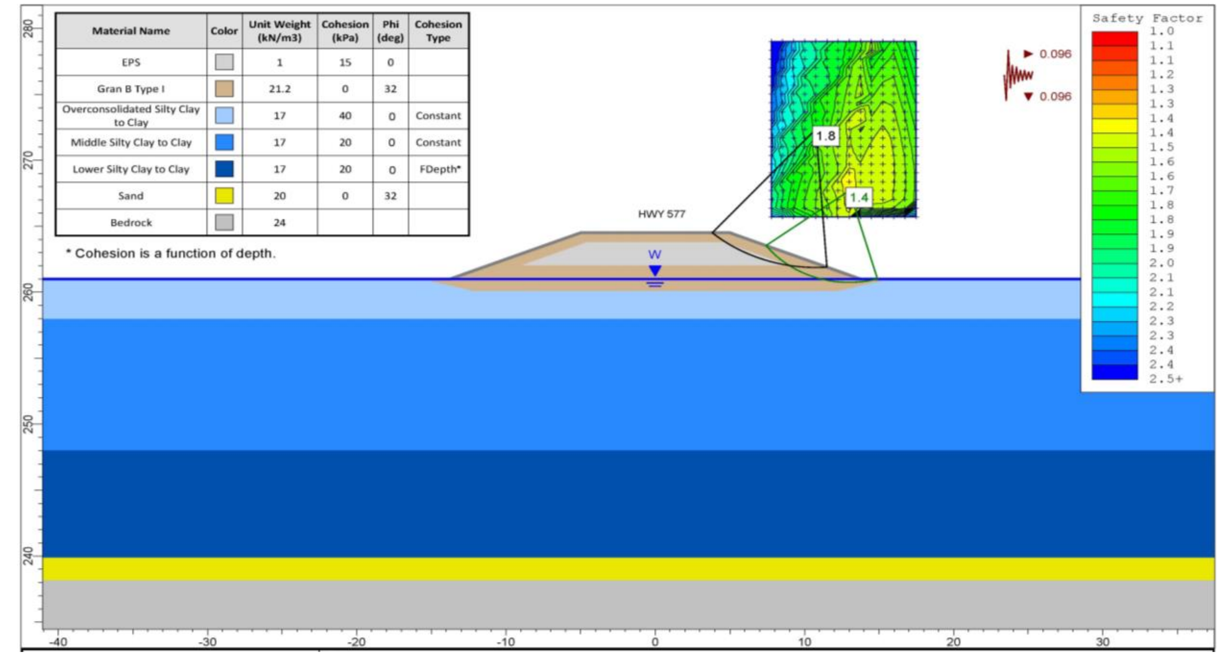
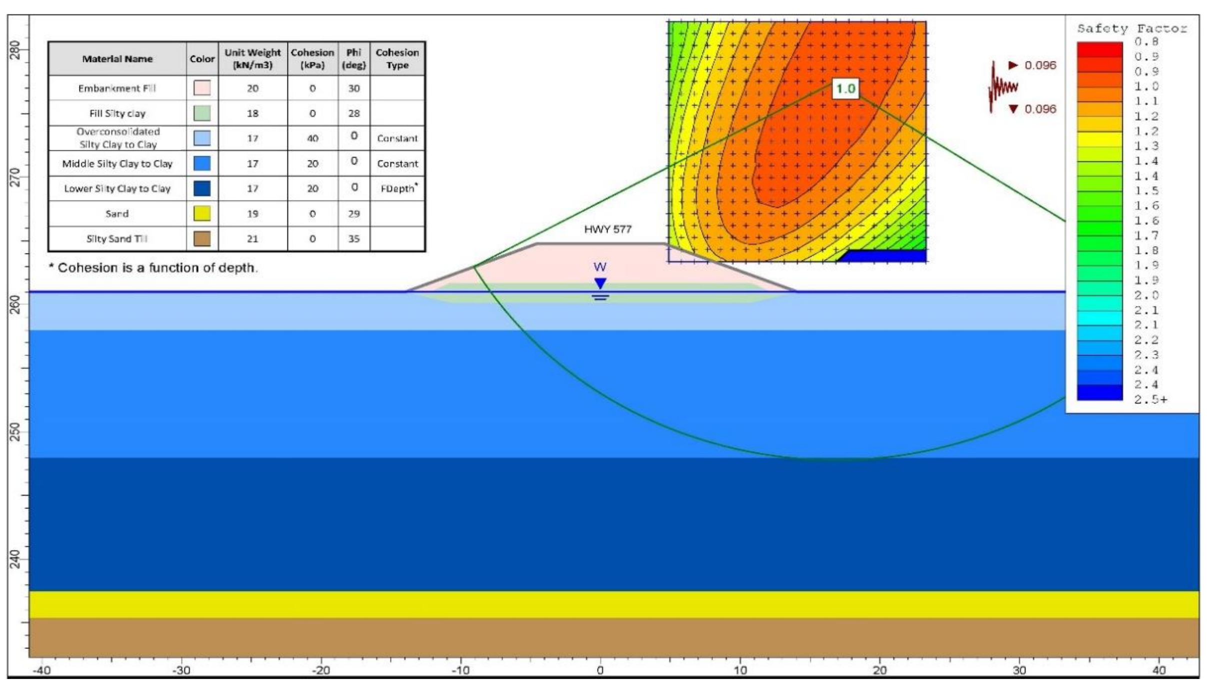
	G.W.P: 5104-18-00	DATE: February, 2019	
	SUBM'D: SD	CHKD: RA	APPD: MT
	Project No: 1-18-0689	Figure: E5	



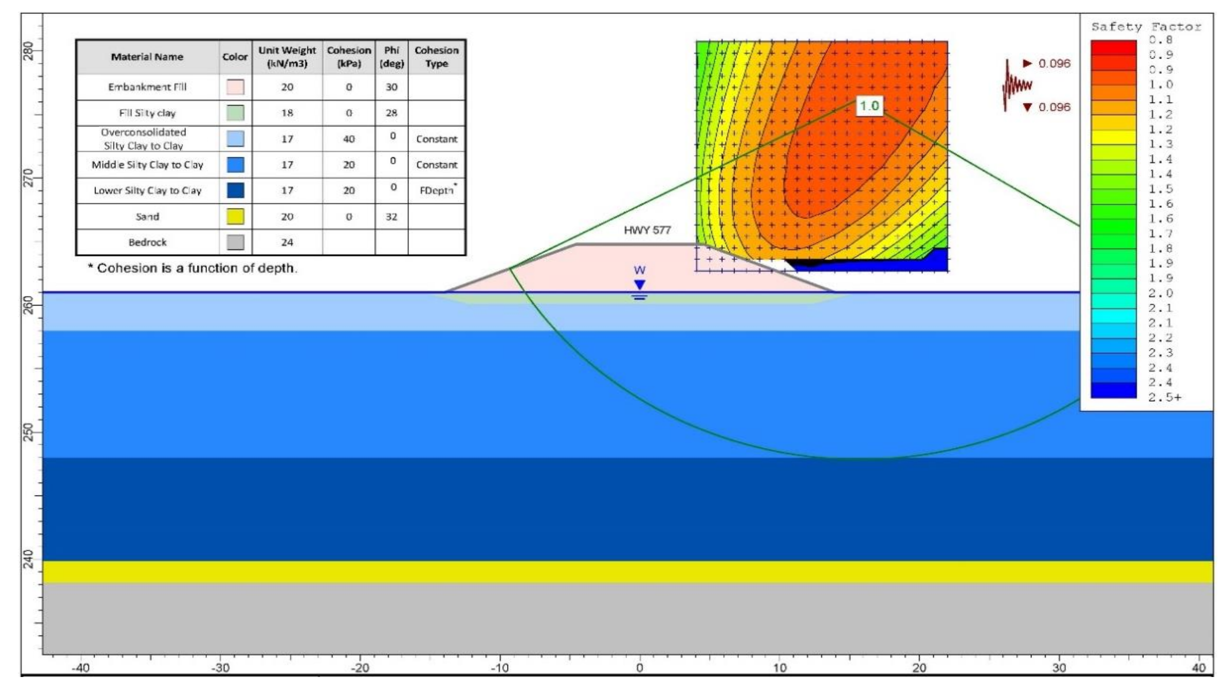
South Approach Embankment - Hwy 577 - Seismic Analysis
(0m - 20m Transition Point Distance)



North Approach Embankment - Hwy 577 - Seismic Analysis
(0m - 20m Transition Point Distance)



South Approach Embankment - Hwy 577 - Seismic Analysis
(20m - 50m Transition Point Distance)



North Approach Embankment - Hwy 577 - Seismic Analysis
(20m - 50m Transition Point Distance)

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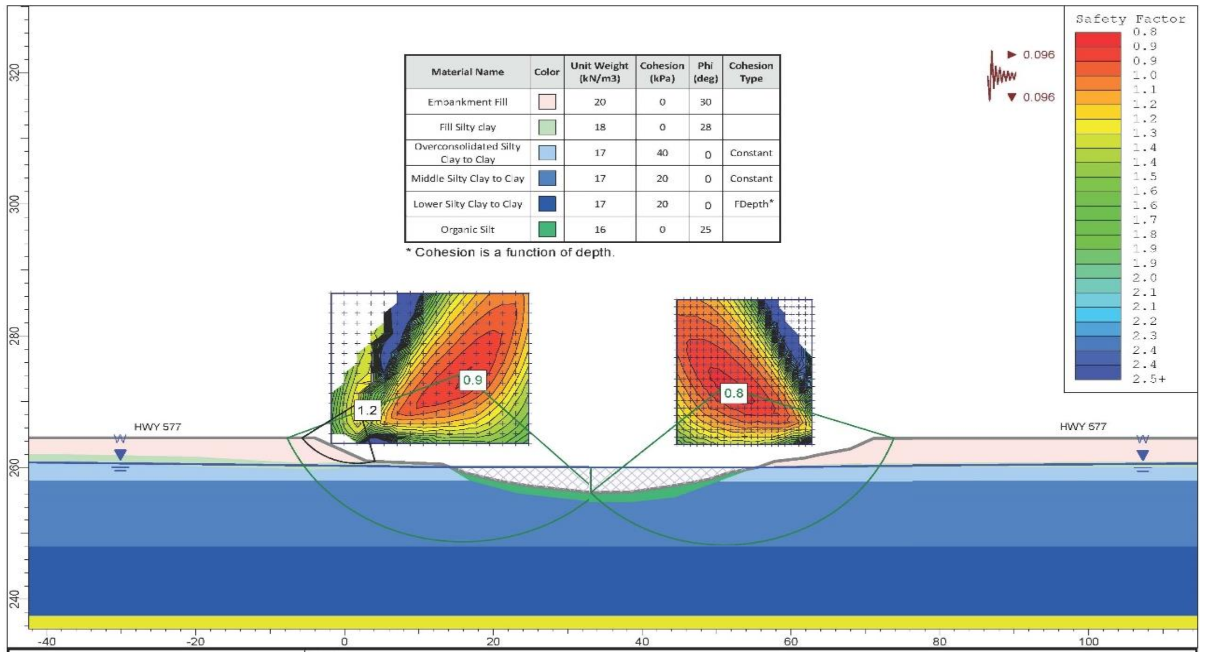
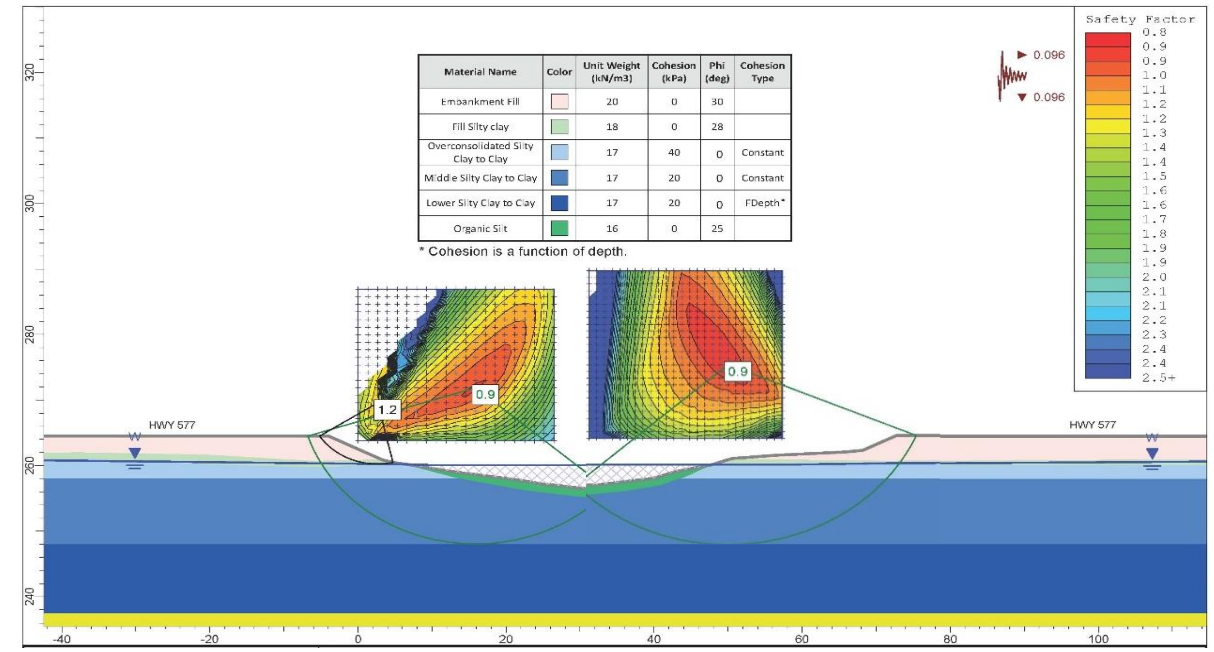
	G.W.P: 5104-18-00	DATE: February, 2019	
	SUBM'D: SD	CHKD: RA	APPD: MT
	Project No: 1-18-0689	Figure: E6	



Forward Slopes - East Side of Hwy 577 - Seismic Analysis



Forward Slopes - West Side of Hwy 577 - Seismic Analysis

<table>
<tr><td rowspan="4">Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650</td><td colspan="3">HWY 577
DRIFTWOOD RIVER BRIDGE, SITE 39E-096</td></tr>
<tr><td>G.W.P: 5104-18-00</td><td colspan="2">DATE: February, 2019</td></tr>
<tr><td>SUBM'D: SD</td><td>CHKD: RA</td><td>APPD: MT</td></tr>
<tr><td>Project No: 1-18-0689</td><td colspan="2">Figure: E7</td></tr>
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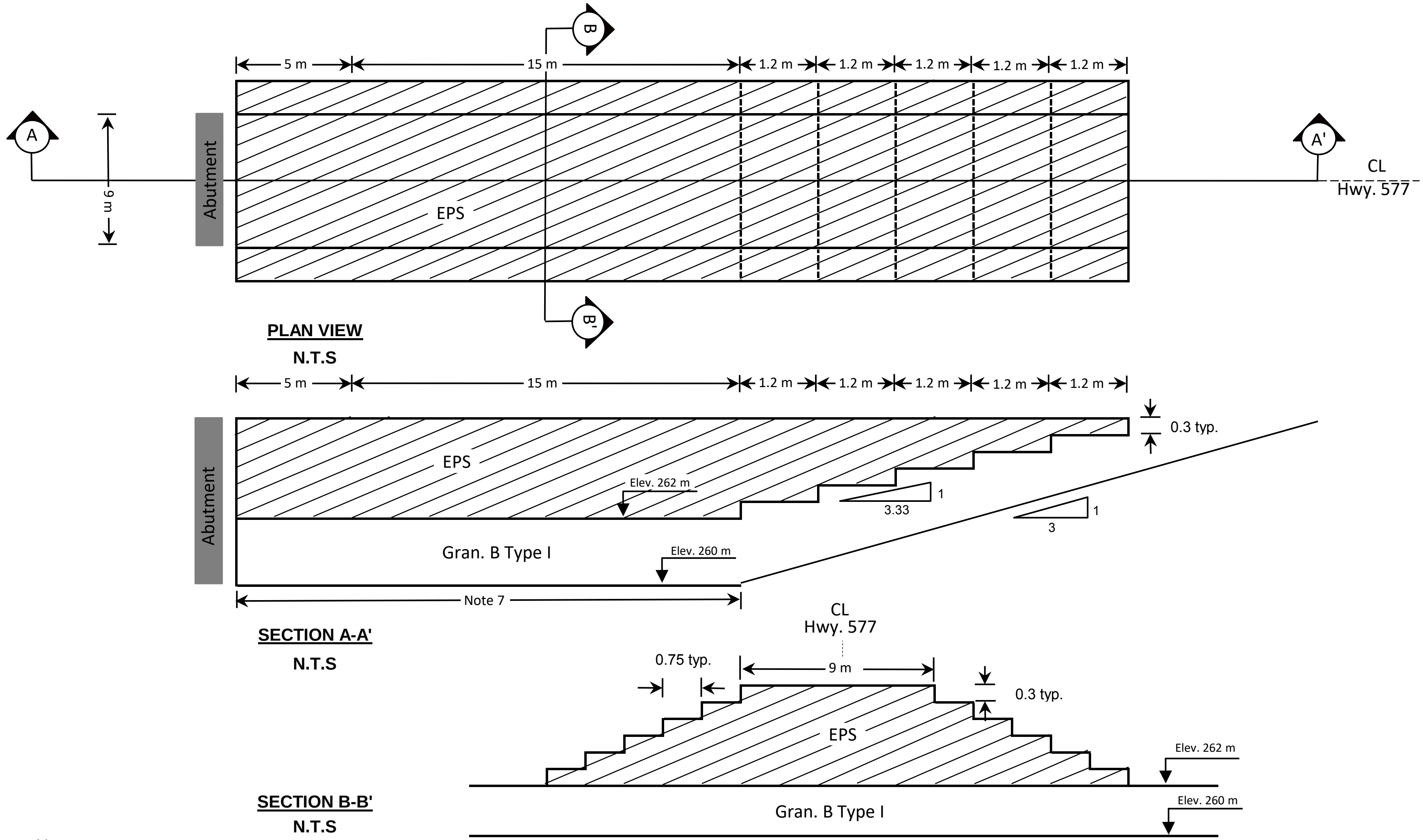
APPENDIX F

EPS Construction Details



PLAN VIEW
N.T.S

SECTION A-A'
N.T.S

SECTION B-B'
N.T.S

Notes:

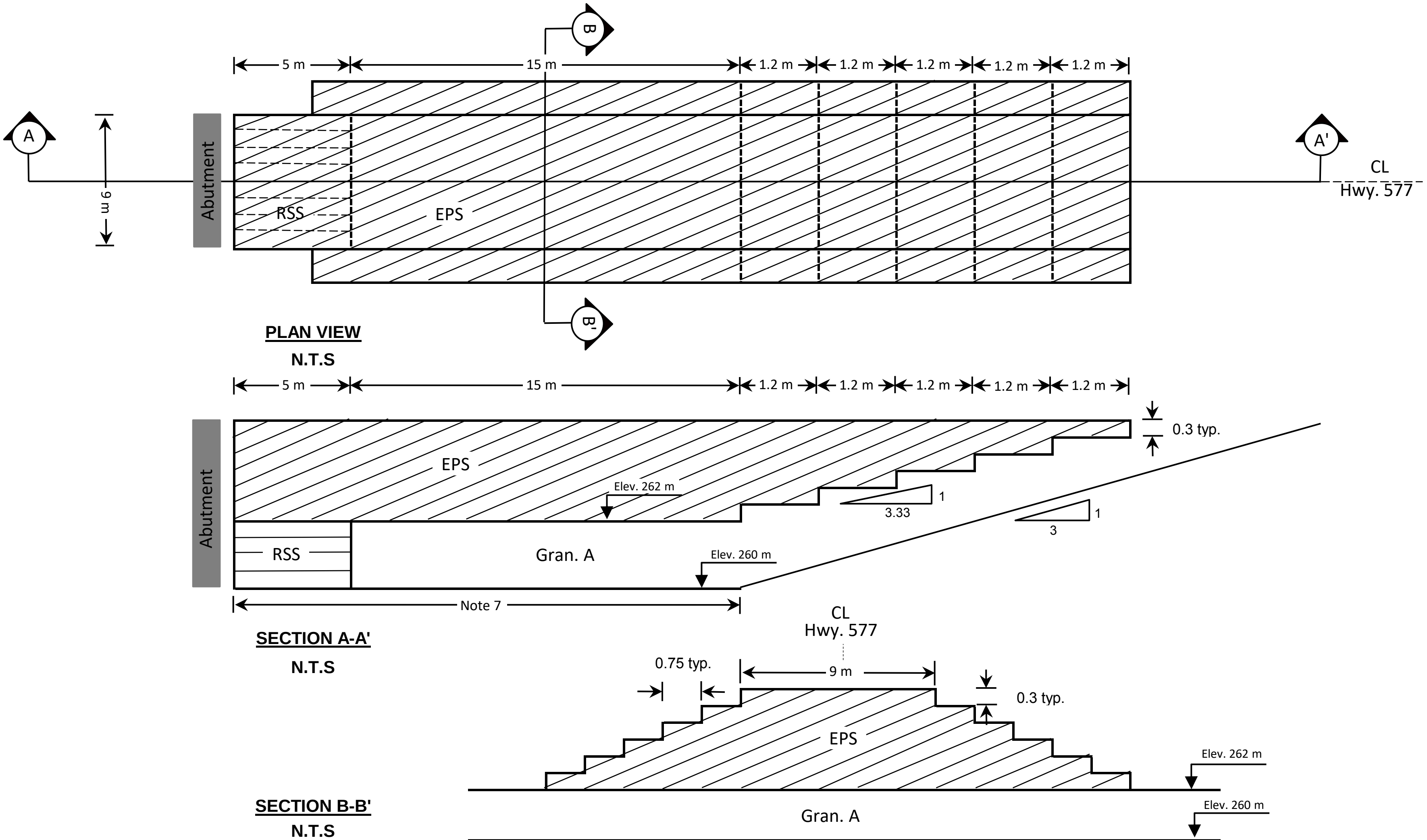
1. All units are in metres.
2. Refer to accompanying Foundation Design Report.
3. Grade and compact Gran. A and RSS surface to create a level surface for EPS installation.
4. A minimum 300 mm thick layer of Granular 'B' Type I should be placed above the side slopes of the EPS.
5. All EPS side slopes should be protected with a minimum 1.0 m soil cover and 6mm thick polyethylene sheeting.
6. A 125 mm thick concrete slab is required on the top of the highest level of EPS blocks.
7. Place non-woven geotextile fabric (FOS = 100 microns) over biaxial geogrid prior to placing Gran B Type I.

PROJECT: DRIFTWOOD RIVER BRIDGE REPLACEMENT
HWY. 577, Site 39E-096

TITLE: RECOMMENDED EPS CONFIGURATION
(North Approach Embankment)

Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650

Project No.: 1-18-0689			G.W.P. : 5104-18-00
DESIGN	RA	Feb-19	W.P. : 5104-18-01
CADD	-	-	Figure F1
CHECK	-	-	
REVIEW	-	-	



PLAN VIEW

N.T.S

SECTION A-A'

N.T.S

SECTION B-B'

N.T.S

Notes:

1. All units are in metres.
2. Refer to accompanying Foundation Design Report.
3. Grade and compact Gran. A and RSS surface to create a level surface for EPS installation.
4. A minimum 300 mm thick layer of Granular 'B' Type I should be placed above the side slopes of the EPS.
5. All EPS side slopes should be protected with a minimum 1.0 m soil cover and 6mm thick polyethylene sheeting.
6. A 125 mm thick concrete slab is required on the top of the highest level of EPS blocks.
7. Place non-woven geotextile fabric (FOS = 100 microns) over biaxial geogrid prior to placing Gran A.

PROJECT:	DRIFTWOOD RIVER BRIDGE REPLACEMENT HWY. 577, Site 39E-096			
TITLE:	RECOMMENDED EPS CONFIGURATION (South Approach Embankment)			
Terraprobe Inc. Consulting Geotechnical & Environmental Engineering Construction Materials, Inspection & Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	Project No.: 1-18-0689			G.W.P. : 5104-18-00
	DESIGN	RA	Feb-19	W.P. : 5104-18-01
	CADD	-	-	**Figure F2**
	CHECK	-	-	
	REVIEW	-	-	

APPENDIX G

Monitoring and Instrumentation Plan

METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN

CONT:
WP 5104-18-01

DRIFTWOOD RIVER BRIDGE REPLACEMENT
MONITORING AND INSTRUMENTATION PLAN

SHEET
–

wsp

Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650

PLAN

SCALE 4 2 0 2 4 6 8 10m

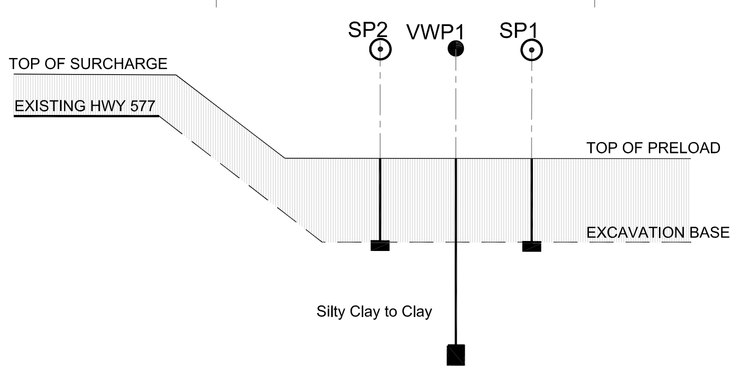
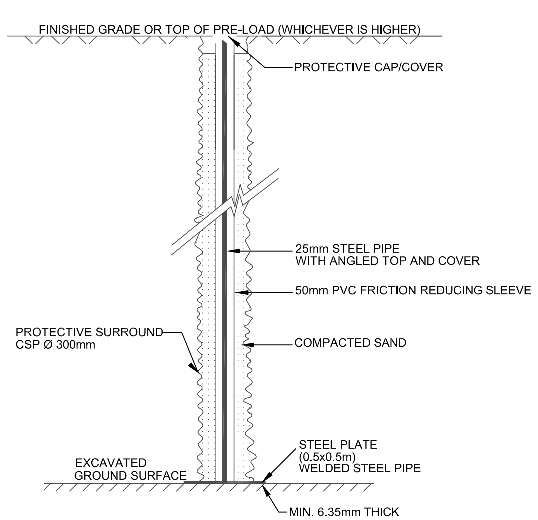


ILLUSTRATION OF MONITORING INSTRUMENTS
N.T.S



SETTLEMENT PLATE (SP)
N.T.S.

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
03/27/2019
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R A ABDUL
03/27/2019
PROVINCE OF ONTARIO

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH TERRAPROBE INC. REPORT TITLED: DRIFTWOOD RIVER BRIDGE REPLACEMENT, HWY. 577, ASSIGNMENT NO. 5016-E-0038, MINISTRY OF TRANSPORTATION, ONTARIO, DATED MARCH 2019, AND ALSO THE SPECIAL PROVISIONS, SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT.
2. THE MONITORING LOCATIONS SHOULD BE CONSIDERED APPROXIMATE AND MUST BE CONFIRMED BY THE CONTRACTOR. PRIOR TO INSTALLATION/CONSTRUCTION AND MAY HAVE TO BE ADJUSTED IN THE FIELD TO SUIT LOCAL CONDITIONS/CONSTRAINTS.
3. THE CONTRACTOR SHALL RETAIN A SURVEYOR REGISTERED IN ONTARIO FOR ESTABLISHING AND SURVEYING THE SETTLEMENT PLATES AS SPECIFIED IN THE SPECIAL PROVISIONS.
4. THE MONITORING INSTRUMENTS SHALL BE INSTALLED AT THE TIMES SPECIFIED IN THE SPECIAL PROVISIONS.
5. THE CONTRACTOR SHALL ESTABLISH TEMPORARY BENCHMARKS OUTSIDE THE AREA OF CONSTRUCTION. THE CONTRACTOR SHALL SUBMIT THE PROPOSED BENCHMARK LOCATIONS TO THE CONTRACT ADMINISTRATOR FOR APPROVAL PRIOR TO CONSTRUCTION.
6. ALL POINTS SHALL BE SURVEYED AT THE FREQUENCIES SPECIFIED IN THE SPECIAL PROVISIONS.
7. THE FREQUENCY AT WHICH THE CONTRACTOR IS REQUIRED TO RECORD AND REPORT SETTLEMENT DATA IS SPECIFIED IN THE SPECIAL PROVISIONS.
8. WITHIN 24 HOURS OF COMPLETION OF ANY MEASUREMENT A COPY OF THE RESULTS SHALL BE MADE AVAILABLE TO THE CONTRACT ADMINISTRATOR..
9. THE CONTRACTOR SHALL MAKE ALL ARRANGEMENTS FOR TRAFFIC CONTROL IN ACCORDANCE WITH ONTARIO TRAFFIC MANUAL BOOK 7.

KEY PLAN

LEGEND

⊡	Bench Mark
⊙	Settlement Plate
●	Vibrating Wire Piezometer

INSTRUMENT LOCATIONS

I.D.	LOCATION	STATION	OFFSET FROM CENTRELINE(m)
SP1	HWY 577	1+485	0
SP2	HWY 577	1+475	0
VWP1	HWY 577	1+480	0
BM1	HWY 577	1+445.2	13.5

* TO BE DETERMINED IN THE FIELD

REFERENCE
Drawings provided in digital format by MMM Group Ltd. (Assignment 5013-E-0018 Preliminary Design for Rehab/Replacement of 12 Structures on Highways in New Liskeard Area) drawing files B8450577001, DTM8450577001, received September 11, 2014
And drawings provided in digital format by WSP via email on Nov. 26, 2017.

REVISIONS	DATE	BY	DESCRIPTION

HWY. 577	PROJECT No. 1-18-0689		GEOCRES No.: 42A-124
SUBM'D. SD	CHKD. RA	DATE: MARCH 2019	SITE: 39E-096
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 1

APPENDIX H

Operational Constraint and Special Provisions

GEOTECHNICAL ASSESSMENT - Item No.

Special Provision

1.0 SCOPE

The use of heavy construction equipment, including but not limited to heavy lift cranes, pile driving equipment etc. will be required during removal of the existing bridge and construction of the new bridge. The bearing, global stability and settlement impacts of the heavy equipment loads on the existing embankment, soft soils underlying the embankment, the river banks, the existing and proposed bridge foundations, and the RSS at the south bridge abutment must be taken into consideration when selecting the methodology and equipment employed for construction.

For bidding purposes:

a) Any excavation and/or material stockpiles, including excavated soils, construction materials and/or demolition debris, shall not be permitted anywhere between the crest of existing river valley slope and the edge of water on both sides of Driftwood River;
b) The pressure applied by the proposed construction equipment that requires analysis, including but not limited to pile driving equipment and heavy lift cranes; shall not exceed 50 kPa per metre width applied over a length of 5.0 m. This pressure applied at the north and south approaches shall take into consideration equipment type/crane pads and grade lowering if warranted. The leading edge of this applied pressure shall not be closer than 5.0 m behind the centre line of the proposed bridge abutments;
c) No pile driving equipment and heavy lift cranes are allowed on the river valley slopes; and
d) Minimum safe crane pad and heavy construction equipment setback distances from the existing timber pile bents must be assessed and established by the Contractor’s Geotechnical Consult to avoid any adverse stability and settlement effects on the new and existing abutment foundation piles during construction.

2.0 REFERENCES

The subsurface conditions at the site are described in the Foundation Investigation Report titled *Driftwood River Bridge Replacement, Highway 577, Assignment No. 5016-E-0038, WO #13, Ministry of Transportation, Ontario, G.W.P. No. 5104-18-00, Site 39E-096*.

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO’s RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – High Complexity.

The Contractor shall retain a Geotechnical Consultant to assess the impacts of the proposed equipment loads and construction methodology on the bearing capacity, stability and settlement of the subsurface soils. The Geotechnical Consultant shall determine the requirements and/or restrictions necessary to safely support the loads without a foundation or slope failure.

The geotechnical assessment shall include, but not be limited to the following:

a) Review of available geotechnical information and supplementing with additional subsurface information in the equipment pads and temporary support areas, as required.
b) Determining appropriate setbacks for heavy equipment from the crest of slopes (river valley forward slopes and embankment side slopes), the new bridge abutments and foundations, and the existing bridge foundations;
c) Determining the permissible ground pressure that may be applied to the foundation soils by the equipment, such as through a combination of equipment/crane pad design and grade lowering;
d) Providing recommendations for distributing equipment loads to limit the lateral deflections of foundation piles of the existing and new bridges including the RSS at the new bridge south abutment and also to maintain global stability of the river valley forward slopes and embankment side slopes; and
e) If use of a crane pad and sub-excavation is not feasible, an alternative pile-supported platform system may be considered. The Contractor shall provide recommendations for equipment/crane pad design to transfer the equipment/crane loads during lifting for construction of the new bridge or demolition of the existing bridge through the alternative pile-supported platform system, if necessary.

4.02 Submission Requirements

At least two weeks prior to the mobilization of heavy equipment, the Contractor shall submit to the Contract Administrator a geotechnical assessment report(s) containing details of the proposed construction equipment and methodology and details/findings of the geotechnical assessment.

The report shall be signed and sealed by two (2) Professional Engineers licensed by the Professional Engineers of Ontario, one (1) of whom shall be the RAQS Approved Key Personnel. The report shall include as a minimum:

a) Appropriate set back distances for heavy equipment from existing/new structures and river valley slopes as well as embankment side slopes;
b) Permissible ground pressures that may be applied to the foundation soils by heavy equipment;
c) Recommendations for the distribution of equipment loads to limit lateral deflections of existing and new foundation piles;
d) Recommendations for pile supported platform systems to support heavy construction equipment, if required.

5.0 MATERIALS – Not Used

6.0 EQUIPMENT – Not Used

7.0 CONSTRUCTION – Not Used

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour to do the work.

Payment for costs associated with heavy construction equipment necessary to complete the work, such as design and construction of temporary works, supply, mobilization/de-mobilization, and operation shall be made under the associated items.

<u>OPERATIONAL CONSTRAINT (GRADING)</u> - Preloading / Surcharging Embankments

Special Provision

The Contractor is advised that the timing for placement of preload/surcharge material is critical to the overall schedule. The Contractor shall schedule his/her operations such that all preload/surcharge material is in place for the entire period as indicated in the table below.

The Contractor shall place preload/surcharge material in the following areas;

Roadway/ Township	**Station to Station**	**Preload/ Surcharge Period (months)**
Hwy 577 / Township of Black River-Matheson	1+404 to 1+470	Surcharge 7 months
	1+475 to 1+495	Preload 7 months

Prior to placement of the Granular A base material and paving, the Contractor shall conduct a survey to determine the elevations of the top of the Granular B sub-base material, and shall place additional Granular B Type I material as and where required to achieve the pavement design sub-base elevation.

The Contractor shall not proceed with final granular placement and paving until approval has been given by the Contract Administrator.

<u>RIGID EXPANDED POLYSTYRENE EMBANKMENT FILL– Item No.</u>

Special Provision

1.0 SCOPE

This Special Provision covers the requirements for the supply and installation of Rigid Expanded Polystyrene (EPS) embankment fill and associated works as shown on the Contract Drawings.

2.0 REFERENCES

This Special Provision refers to the following standards, specifications or publications.

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 212	Construction Specification for Earth Borrow
OPSS.PORV 501	Construction Specification for Compacting
OPSS.PROV 517	Construction Specification for Dewatering
OPSS.PROV 904	Construction Specification for Concrete Structures

Ontario Provincial Standard Specifications, Materials

OPSS.PROV 1010	Aggregates – Base, Subbase, Select Subgrade, and Backfill Material

National Standards of Canada

CAN/ULC-S102-10	Standard Method of Test for Surface Burning Characteristics of Building Materials and Assemblies
CAN/ULC-S701-97	Thermal Insulation, Polystyrene, Boards and Pipe Covering

ASTM International

ASTM C177	Standard Test Method for Steady-State Heat Flux Measurements and Thermal Transmission Properties by Means of Guarded-Hot-Plate Apparatus
ASTM C203	Standard Test Method for Breaking Load and Flexural Properties of Block-Type Thermal Insulation
ASTM C518	Standard Test Method for Steady-State Thermal Transmission Properties by Means of the Heat Flow Meter Apparatus
ASTM D1621	Standard Test Method for Compressive Properties of Rigid Cellular Plastics
ASTM D2842	Standard Test Method for Water Absorption by Rigid Cellular Plastics
ASTM D2863	Standard Test Method for Measuring the Minimum Oxygen Content
ASTM D6817	Standard Specification for Rigid Cellular Polystyrene Geofoam

3.0 DEFINITIONS

For the purpose of this special provision, the following definitions apply:

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

Manufacturer: means the firm who supplies the Rigid Expanded Polystyrene

Rigid Expanded Polystyrene (EPS): means moulded rigid blocks produced by a process of pre-expansion, aging and forming of petroleum based raw material.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

4.01.01 Foundation Investigation Report

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

4.02 Submissions

4.02.01 Working Drawings

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator six copies of the Working Drawings and method statement that provides full details of materials and construction procedure. The submissions shall be signed and sealed by the Contractor's Engineer.

The Contractor shall submit full details of the following.

a) The method of foundation excavation and preparation.

b) The method of levelling pad construction.

c) The method of placement of Rigid Expanded Polystyrene blocks including temporary ballasting and protection of blocks during installation. The shop drawings shall indicate laying pattern and block dimensions on a layer-by-layer basis.

d) The method and limits of polyethylene sheeting placement.

e) The method of placement of the reinforced concrete top slab.

f) The method of placement of subbase material.

g) The method of placement of side slope cover.

4.02.02 Delivery, Storage, Handling, and Protection Procedure

At least three (3) weeks before the commencement of work, the Contractor shall submit to the Contract Administrator the method of delivery, storage, handling and protection from damage by weather, traffic, construction staging and other causes as per the Rigid Expanded Polystyrene manufacturer's requirement.

4.02.03 Rigid Expanded Polystyrene

At least two (2) weeks prior to commencement of the installation of the Rigid Expanded Polystyrene blocks, the following details shall be submitted in writing to the Contract Administrator:

1. A general statement as to the type, composition, and method of production of the material.

2. The manufacturer's name, address, phone number, identification of a contact person and description of experience background in the manufacturing of the Rigid Expanded Polystyrene.

3. An identification of the laboratory accredited by the Standards Council of Canada to conduct the testing of the physical and mechanical properties of the Rigid Expanded Polystyrene.

4. The physical and mechanical properties of the Rigid Expanded Polystyrene including:

    a) Geometry
    b) Nominal Density
    c) Compressive Strength
    d) Flexural Strength
    e) Thermal Resistance
    f) Flammability
    g) Water Absorption

5. Aging and durability characteristics of the Rigid Expanded Polystyrene including the chemical, biological and ultra-violet degradation resistance of the rigid polystyrene.

6. A sample of the Rigid Expanded Polystyrene material.

4.02.04 Quality Test Certificates

Prior to installation of the Rigid Expanded Polystyrene, the Contractor shall submit Quality test certification for each production lot supplied from a laboratory accredited by the Standards Council. The Quality test certificates shall demonstrate compliance with all requirements of this special provision.

4.02.05 Rigid Expanded Polystyrene embankment

For each Rigid Expanded Polystyrene embankment, a Request to Proceed shall be submitted to the Contract Administrator at each of the following milestones:

a) Following submission of the Quality Test Certificate and prior to construction.
b) Following foundation excavation and preparation and prior to installation of the leveling pad;
c) Following placement of Rigid Expanded Polystyrene blocks and polyethylene sheeting and prior to construction of the concrete top slab;

The next operation shall not proceed until a Notice to Proceed has been received from the Contract Administrator.

4.02.06 As-Built Drawings

As-built drawings shall be submitted to the Contract Administrator in a reproducible format prior to final acceptance of work.

The as-built drawings shall be signed and sealed by the Contractor's Engineer.

5.0 MATERIALS

5.01 Granular Levelling Pad

The levelling pad shall be as specified elsewhere in the contract documents and shall consist of Granular A material with gradation and physical requirements as specified in OPSS.PROV 1010.

5.02 Rigid Expanded Polystyrene

5.02.01 Production Lots

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

5.02.02 Detail Requirements

The Rigid Expanded Polystyrene shall meet the physical and mechanical properties requirements shown in Table 1 and as described below.

Table 1 – Material Properties

PROPERTY	UNIT	REQUIREMENTS	TEST PROCEDURE
Geometry - Linear Dimensions - Flatness - Squareness	mm (min)	1200 × 600 × 300 with tolerances ± 1% 10mm in 3m ± 0.5%	--
Compressive Strength at 5% Deformation	kPa (min)	115 (EPS Type 22) 170 (EPS Type 29)	ASTM D1621 (Procedure A)
Flexural Strength	kPa (min)	240 (EPS Type 22) 345 (EPS Type 29)	ASTM C203 (Method 1, Procedure B.2.7.4)
Thermal Resistance	$m^2 \cdot {}^oC/W$ (min for 25 mm thickness)	0.7	ASTM C177 or C518
Flammability	Limiting Oxygen Index (min)	24	ASTM D2863
Water Absorption	% by Volume (max)	3 (EPS Type 22) 2 (EPS Type 29)	ASTM D2842

5.03 Polyethylene Sheeting

The protective sheeting shall be at a minimum 6 mil thick polyethylene sheeting or better if specified elsewhere in the Contract Package and shall be free from defects.

5.04 Concrete Top Slab

The reinforced concrete top slab shall be as specified elsewhere in the contract documents.

6.0 EQUIPMENT

All cutting of Rigid Expanded Polystyrene materials shall be by electric equipment or by hand.

Heavy equipment shall be limited in weight and size and restricted in operation to avoid damaging the Rigid Expanded Polystyrene as per the manufacturer’s requirement.

7.0 CONSTRUCTION

7.01 General

7.01.01 Rigid Expanded Polystyrene Installation

The installation of the Rigid Expanded Polystyrene shall be undertaken under the supervision of the Contractor’s Engineer.

The Contractor inspection of the Rigid Expanded Polystyrene shall be carried out full-time.

The Contractor’s manufacturer representative shall be on site to oversee installation of the Rigid Expanded Polystyrene blocks at the commencement of the installation.

7.02 Delivery, Storage And Handling

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

The Contractor shall protect the Rigid Expanded Polystyrene from exposure to sunlight to avoid ultraviolet degradation as per manufacturer’s recommendation.

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

Rigid Expanded Polystyrene shall not be exposed to open flame or other ignition source. The contractor shall protect the Rigid Expanded Polystyrene blocks from petroleum based products such as gasoline and diesel fuel and organic solvents such as acetone, benzene and paint thinner.

7.03 Foundation Excavation

Foundation excavation shall be carried out to the design elevations shown on the drawings. Any softened, loosened or deleterious materials at the foundation footing elevation shall be subexcavated and replaced with OPSS.PROV 1010 Granular A material.

7.04 Leveling Pad

The Contractor shall place, level and compact a layer of OPSS.PROV 1010 Granular A material in accordance with OPSS PROV 501 to within ± 30 mm of the design elevation. The leveling pad shall not deviate by more than 10 mm at any place on a 3 m straight edge over the limits of the bottom course of blocks and shall be constructed parallel to the final pavement centre line design profile. The levelling pad shall not be placed on standing water, accumulated snow or ice or frozen ground. The levelling pad must be placed in-the-dry.

7.05 Installation of Blocks

The Contractor shall have on site at the commencement of the work, a representative of the supplier of the Rigid Expanded Polystyrene to advise on recommended construction procedure.

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

The Rigid Expanded Polystyrene embankment shall be installed to ensure that:

1 The individually marked blocks shall be placed on the prepared leveling pad. The top surface of the first layer of blocks is to be set plane and level. Local trimming of the blocks may be necessary.

2 Subsequent successive layers shall be oriented with the long axis of blocks positioned at 90º to the previous layer in order to avoid continuous joints. Block joints shall be offset and staggered between layers.

3 A continuous check shall be kept to ensure the evenness of the blocks is satisfactory in each layer. Blocks shall be laid with joints with maximum opening of 10 mm between blocks. Differences in heights between adjacent blocks in the same layer shall not exceed 5 mm.

4 Sloping end adjustments at the abutments shall be accomplished by leveling terraces in the subsoil in accordance with the block thickness.

5 Due to windy conditions temporary ballast shall be provided as necessary to prevent movement of Rigid Expanded Polystyrene both in storage and during placement. Timber fasteners or equivalent shall be used as necessary.

6 The Rigid Expanded Polystyrene embankment shall be protected from accidental ignition due to welding, smoking, grinding or cutting tools, etc. The Contractor shall take all necessary precautions to prevent ignition of the Rigid Expanded Polystyrene.

7 The Rigid Expanded Polystyrene shall be protected from organic solvents and other aggressive, harmful chemicals during construction.

8 Exposed blocks shall be covered immediately to avoid possible burrowing by animals.

9 Individually marked blocks shall be fabricated and placed to ensure the top surface matches the elevation and crossfall shown on the drawings.

10 The top surface and side surfaces of the Rigid Expanded Polystyrene shall be covered with 6 mil polyethylene sheeting extending onto adjacent work at the longitudinal ends of the embankment. All joints shall be lapped a minimum of 300 mm to provide a fully sealed enclosure.

7.06 Side Slope Cover

The side slopes of the Rigid Expanded Polystyrene embankment shall be covered with granular fill as detailed elsewhere in the Contract drawings.

8.0 MEASUREMENT FOR PAYMENT

8.01 Actual Measurement

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

9.0 BASIS OF PAYMENT

The concrete base pad and granular leveling pad shall be paid for with the appropriate tender items as detailed elsewhere in the contract.

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

SUPPLY AND INSTALLATION OF EMBANKMENT MONITORING EQUIPMENT - Item No.

Special Provision

1.0 SCOPE

1.01 Instrument Types

This special provision contains the requirements for the supply and installation of the following geotechnical instruments:

a) Settlement Plates/Rods (SP)
b) Vibrating Wire Piezometers (VWP)
c) Survey Benchmark/s or Targets (BM)

The purpose of these instruments is to monitor settlements and pore water pressures in the foundation soils under the south bridge approach embankment. The data will be used to plan the construction schedule and to determine when final paving operations can commence. Settlements will be measured by level surveying of the top of the settlement rods. The timing for the removal of the preload/surcharge and final paving operations shall be controlled by the instrument readings.

1.02 Or Equal

The term “or equal” shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration. Only one supplier shall be selected for the supply of the data acquisition system and the vibrating wire piezometer.

1.03 Equipment Operation and Weather Conditions

All installations and monitoring equipment and associated materials shall be capable of withstanding the range of temperatures possible for their location within and above the ground. The instruments shall be capable of operating within the manufacturer’s stated accuracy throughout the temperature range. Monitoring shall be conducted year round and the Contractor is advised that the equipment shall be accessible for monitoring throughout the duration of the Contract.

1.04 Specialist Qualifications

The Contractor (as referenced herein) shall be understood to refer to the Contractor and their Geotechnical Consultant.

The Contractor shall retain a Geotechnical Consultant registered in MTO’s consultant acquisition system (RAQS) for “Geotechnical (Structures and Embankments) – Medium Complexity”, to undertake the supply and installation of geotechnical instruments.

1.04 Notification

The Contract Administrator shall be notified a minimum of 15 working days in advance of commencing the installation of instruments.

1.05 References

1.05.1 Subsurface Conditions

The subsurface conditions at the site are described in the Foundation Investigation Report titled *Driftwood River Bridge Replacement, Highway 577, Assignment No. 5016-E-0038, WO #13, Ministry of Transportation, Ontario, G.W.P. No. 5104-18-00, Site 39E-096.*

The owner warrants that the information provided in the report can be relied upon with the following exceptions.

a) Any interpretations of the data or opinions expressed in the report are not warranted; and
b) Although the raw measured data presented is warranted, the Contractor must satisfy himself as to the sufficiency of the information presented and obtain any updated or additional information, and perform any studies, analysis or investigations the Contractor deems necessary in order to prepare his design, at no additional cost to the Owner.

1.05.2 Monitoring and Instrumentation Plan

Reference shall be made to the Monitoring and Instrumentation Plan included in the Contract Documents.

2.0 DESIGN AND SUBMISSION REQUIREMENTS

2.01 Submission Requirements

The Contractor shall submit details of proposed installations including:

a) Design and construction drawings, including equipment layout;
b) Installation methodology and timing; monitoring enclosure details;
c Equipment and material specifications, data sheets;
d) Location and types of survey benchmarks; and
e) Installation schedule.

2.02 Instrument Quantities and Locations

A summary of instrumentation requirements is given below in Table 2.0. Details and specific material requirements are presented elsewhere in this special provision.

Table 2.0 – Instrument & Benchmark Quantities and Locations

INSTRUMENT I.D.	Hwy. 577 STATION	OFFSET FROM CENTRELINE (m)	NO. OF INSTRUMENTS		
			SP	VWP	BM
SP1	1+485	0	1		
SP2	1+475	0	1		
VWP1	1+480	0		1	
BM1	Southwest of Hwy. 577	Outside of construction area			1
Total Instruments			**2**	**1**	**1**

2.03 Instrument Location

Prior to the installation of instruments, the Contractor shall accurately survey and stake the location of each instrument and obtain a ground surface elevation at each instrument location.

2.04 Survey Benchmark (BM)

The Contractor shall provide as a minimum stable non-settling targets on existing hydro poles within the area subject to approval by the Contract Administrator. The Contractor shall accurately survey and stake the benchmark points/targets and obtain their geodetic elevations.

The location of the benchmarks shall be such that direct sighting is possible from all settlement rods to at least one benchmark.

2.05 Accuracy of Surveying for Elevations

Elevations shall be surveyed and referenced to Geodetic datum to an accuracy of ±2 mm or better.

2.06 Monitoring Instrument Location

The MTM NAD 83 coordinate system shall be used to establish northing and easting coordinates of all monitoring instruments.

2.07 Materials and Equipment

The Contractor shall supply all materials and equipment required for the installation of instrumentation unless noted otherwise.

2.08 Underground Utilities

The Contractor shall be responsible for locating and protecting all underground utilities prior to drilling boreholes for installing instruments. Any damage to underground utilities caused by the Contractor's work shall be repaired by the Contractor, at no cost to the Ministry.

2.09 Marking and Labelling

The location of any above ground monitoring fixture shall be made clearly visible to nearby traffic before, during and after embankment construction. Marking shall be of sufficient size to be visible from a reversing vehicle and after heavy snow falls.

Instruments or their data cables shall be clearly labelled in the field, each instrument having a unique identifier. The labelling shall remain legible for at least 1 year.

2.10 Protection of Instruments

All instruments shall be adequately protected by the Contractor such that they are not damaged during construction. Any instrument damaged by the Contractor's work shall be immediately replaced at no cost to the Ministry.

2.11 Boreholes

The Contractor shall make a basic stratigraphic log of the borehole that is drilled to install the vibrating wire piezometer. In situ or laboratory testing is not required.

The borehole shall be advanced using conventional drilling methods and shall be as straight and vertical as practical.

2.12 Installation Program

Instrument installation shall be completed before any embankment construction. Table 2.1 summarizes the installation schedule requirements.

Table 2.1 – Installation Program

TYPE	START INSTALLATION	FINISH INSTALLATION
SP	After excavating to the embankment base elevation	Plates - Before embankment construction Rods - On completion of embankment construction
VWP	Before embankment construction	Before embankment construction
BM	Before embankment construction	Before embankment construction

3.0 BENCHMARK (BM) – SUPPLY & INSTALLATION

3.01 Scope

This section contains the requirements for the supply and installation of benchmarks (BM). The purpose of the benchmark is to provide non-settling references for the surveying of settlement rods.

3.02 Location

Benchmarks shall be located and installed outside of the area of construction activity.

Table 3.1 – Approximate Bench Mark Location

Location	No. of BM
Southwest of Bridge	
Outside of Construction Area on Existing Hydro Pole	BM1

3.03 Materials

The Contractor shall supply all materials and equipment required for the installation of reflective targets as benchmarks.

3.04 Installation

The elevation, easting and northing of the benchmark target shall be surveyed.

3.05 Coordination With Monitoring/Notification

The Contractor shall notify the Contract Administrator no later than 3 days after installing a benchmark. At this time the Contractor shall also supply the following information to the Contract Administrator.

a) Description of benchmark
b) Date of installation;
c) Installation notes/sketches
d) Location of the proposed target including Easting, Northing in MTM NAD83 coordinates; and
e) Geodetic Elevation

3.06 Monitoring

Monitoring of settlements with reference to the benchmark shall be done by others. Monitoring shall be conducted during and following the embankment construction. The Contractor shall provide access to the benchmark for monitoring including, but not limited to snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed.

4.0 SETTLEMENT RODS (SR) – SUPPLY & INSTALLATION

4.01 Scope

This Section contains the requirements for the supply and installation of settlement rods. The purpose of the settlement rods is to monitor settlements of the foundation soils below the embankment base. The settlement readings shall help to establish the timing for the removal of preload/surcharge fill, as well as final paving operations. Settlement is measured by surveying the top of the rod with reference to stable, benchmarks.

4.02 Location

The locations of the settlement rods are shown on the Contract Drawings and are given below in Table 4.1.

Table 4.1 – Approximate Settlement Rod Locations

INSTRUMENT I.D.	Hwy. 577 STATION	OFFSET FROM CENTRELINE (m)	NO. OF SETTLEMENT RODS	ESTIMATED THICKNESS OF EMBANKMENT (m)*
SP1	1+485	CL	1	3.5
SP2	1+475	CL	1	3.5

Notes:* Embankment thickness based on the settlement plate base elevation and does not include any preload/surcharge height.

4.03 Materials

The settlement rods shall be attached to a plate placed on the embankment subgrade. As embankment construction proceeds the rods shall be extended above the new top of embankment. Sleeves around the rods shall be installed to reduce friction and allow uninhibited movement of the rod with the plate. A protective surround shall be extended with the rods as embankment construction proceeds.

4.03.1 Plate

The Contractor shall supply steel plates with thickness of at least 6.35 mm. The plates shall be at least 0.5 m by 0.5 m.

4.03.2 Rod

The Contractor shall supply steel pipe Schedule 40 with an outside diameter not less than 25.4 mm (1”), supplied in lengths as required to complete the installation as described in Section 4.3.

The top end of each length of rod shall be threaded to receive a cap. A rounded cap shall be installed at the top of the rod in such a way that a single survey point can be clearly identified and returned to.

4.03.3 Friction Reducing Sleeve

The Contractor shall supply a friction reducing sleeve consisting of Schedule 40 – 50.8mm (2”) O.D. PVC pipe cut perpendicular to the axis of the pipe.

4.03.4 Protective Surround

The Contractor shall supply a protective surround for the portion of the rod within the embankment. The surround shall consist of 250 mm diameter corrugated steel pipe (CSP – OPSS 1801) with the ends cut perpendicular to the axis of the pipe and free of burrs and sharp edges. The space between the CSP and the Friction Reduction Sleeve (PVC pipe) shall be filled with medium to coarse sand.

4.04 Installation

The Contractor shall install settlement rods as per the Contract Drawings provided in addition to what is stated or emphasized below.

4.04.1 Settlement Plate

The settlement plate shall be installed horizontally after embankment subgrade preparation is completed and prior to fill placement. Settlement plates shall be placed on undisturbed soils at the base of the excavation.

The elevation of the base of the plate shall be surveyed before backfilling.

4.04.2 Rod

The rod shall be fixed to the center of the plate and installed perpendicular to the plate. The coupling of the rods shall be such that all sections have the same axis and no separation or contraction will occur at the couplings.

The settlement rods shall be extended upwards as the embankment is constructed so that the top of the rod is always at least 0.3 m but not more than 2 m above the surrounding fill.

4.04.3 Friction Reducing Sleeve

The friction reducing sleeve shall be over the entire length of the rod that is below ground and within the embankment fill except that the cap on top of the settlement rod shall extend 25 mm above the top of the friction sleeve at all times.

4.04.4 Protective Surround

The CSP, Friction Reducing Sleeve and sand protective surround shall be extended with the rods. The settlement rod shall be in the center of the CSP and friction-reducing sleeve.

The annulus between the CSP and the friction-reducing sleeve shall be filled with sand to a level not higher than the top of the sleeve.

4.04.5 Coordination With Monitoring/Notification

The Contractor shall notify the Contract Administrator no later than 3 days after installing a settlement rod. At this time the Contractor shall also supply the following information to the Contract Administrator.

a) Dates of installation;
b) Installation notes/sketches;
c) Location of the instrument including Easting, Northing in MTM NAD83 coordinates of the centre of the base plate;
d) Elevation of plate and top of rod referenced to Geodetic datum.

Adjustments in the length of any settlement rod shall be coordinated with the Contract Administrator to allow surveying of the elevation of the top of the rod immediately before and immediately after adjustment. This surveying is necessary to accurately track the settlement data.

4.05 Monitoring

Monitoring of the settlement rods shall be done by others. Monitoring shall be conducted during the embankment construction and preload period.

The Contractor shall provide access to the settlement rods for monitoring including, but not limited to a level scaffolding platform and ladder, if required and snow clearing in the winter. The Contractor shall provide electric power and general area lighting as needed for reading the instruments.

5.0 VIBRATING WIRE PIEZOMETER (VWP) – SUPPLY & INSTALLATION

5.01 Scope

This Section contains the requirements for the supply and installation of a vibrating wire (VW) piezometer. The purpose of the piezometer is to monitor the piezometric head at depth within the foundation soil below the embankment. The piezometer readings shall help to establish the timing and sequence of the removal of the embankment preload, and final paving operations.

5.02 Location

The Contractor shall install the VW sensor at the location and depth given in Table 5.1.

Table 5.1 – VW Piezometer Location

INSTRUMENT I.D.	Hwy. 577 STATION	OFFSET (m)	No. OF VWP	APPROX. ELEV. OF GROUND SURFACE (m)	TIP ELEV. (m)
VWP1	1+480	CL	1	263.5	250.0

The vibrating wire piezometer shall be installed in a borehole prior to the start of any embankment construction, any preload fill construction, and any piling. Prior to installation of the instrument adjacent to new construction features (including limit of pile cap, edge of unwatering system, extent of sub-excavation and backfilling), the construction features shall be laid out in the field to ensure there are no conflicts with the instrument.

The VW signal cable for the VWP shall be extended out of the embankment and preload footprint area (where applicable) and away from the construction area through a metal or plastic conduit buried in trenches.
The conduit for the VW signal cable for the VWP shall be routed to be connected to a single data acquisition system (data-logger).

5.03 Materials

5.03.1 VW Piezometers

The Contractor shall supply a VW borehole piezometer by Slope Indicator model 52611020 (-5 to 50 psi), RST model VW2100-0.35 – or equal; compatible with the Slope Indicator CR1000 data-logger, RST model ELGL1200 – or equal.

The piezometer shall be calibrated prior to installation and the calibration data for the piezometer shall be provided to the Contract Administrator.

5.03.2 Signal Cable

The Contractor shall supply Slope Indicator model 50613524 cable, RST model EL380004 cable – or equal. The length of cable for the piezometer shall be estimated to ensure that there is enough signal cable for the piezometer to provide enough slack in the borehole and along the trench until the cable is out of the construction area where the cable shall also be protected from construction equipment.

5.03.3 Bentonite

The Contractor shall supply bentonite (OPSS.PROV 1205) in pellet form in sufficient quantity to form borehole plugs as required.

5.03.4 Filter Sand

The Contractor shall supply clean washed sand for filter around VWP sensors. The sand shall be Sakcrete washed general-purpose sand – or equal.

5.03.5 Grout

The Contractor shall supply cement-bentonite grout. A suitable grout mix design consists of 23 kg of bentonite (OPSS.PROV 1205), 143 litres of water and 40 kg of cement (Type G.U. – OPSS.MUNI 1301).

5.03.6 Trench Burial and Conduit

The signal cable for the piezometer shall be buried in a shallow trench and taken out of the construction area. The Contractor shall supply a suitable conduit (e.g. Schedule 40 – 75 mm (3”) – steel pipe or Schedule 80 - 75 mm (3”) – rigid PVC pipe) to protect the signal cable in the trench and above ground surface.

The signal cable and conduit shall be routed such that future grading works do not interfere with the cable or conduit.

5.03.7 Data Acquisition System (Data-Logger)

The signal cable from the vibrating wire piezometer shall be connected to a data-logger (to be located away from the approach embankment), Slope Indicator model 56701000 (CR1000), RST model ELGL1200 – or equal. The data-logger shall consist of the following:

a) ENC 16/18 Water-proof Enclosure model 56705020, model ELF0638 – or equal;
b) SC32A Serial Interface (with RS232 transfer cable) model 56704010, model CS-SC32A – or equal;
c) VW Interface model 56701510 or 56701500, model CS-AVW200 – or equal;
d) AM16/32 Multiplexer model 56702110, model ELGL2042 – or equal;
e) A suitable power supply which shall be able to last for 1 year (i.e. large capacity rechargeable battery); and
f) LoggerNet Software model 56708020, model CS-Loggernet – or equal.

A minimum of one data logger is required. The Contractor shall submit a detailed proposal on the setup of the data-logging system (i.e. location of the data-logging unit) to the Contract Administrator for review, prior to ordering the data-logger. The Contractor shall program the data-logger according to the following:

Recording Software: VWP data shall be recorded four times daily (one reading every 6 hours)
Test Software: once this program is transferred to the data-logger, one shall be able to test the system and record data manually on site

The real-time data shall be retrieved on site by direct wire (i.e. RS232 Cable) with a portable laptop computer as specified in the next section.

5.03.8 Portable Laptop Computer

The Contractor shall supply:

a) A New Portable Laptop Computer (with a Three year warranty): Intel Core i5 or better (1.6 GHz or above) with Windows 10 Operating System, minimum 4GB memory, a minimum of 250GB hard drive storage, a DVD+/-RW and Microsoft Office 2010, to retrieve, read and store the VW piezometer readings;
b) Extra battery for the laptop computer and a vehicle adaptor for the computer charger.

The portable laptop computer will become property of the MTO and shall be handed to the Contract Administrator after the installation of instruments for the Monitoring program and the contents shall include all data-logging software and hardware, operation instructions and calibration constants.

The calibration factor for the vibrating wire piezometer shall be entered in the portable laptop computer by the Contractor for initialization of the instrument.

5.03.9 Long-term Monitoring (Monitoring Enclosure)

The Data-logger shall be installed in an enclosure to prevent vandalism and prolonged wear-out of the data-logger against extreme weather. The Monitoring Enclosure shall be a lockable and weather proof unit that is fabricated and attached to wooden post(s). Wooden posts: 100 mm x 100 mm (4”x4”), minimum 3 m (10”) long shall be used to support the data acquisition system and to affix the Monitoring Enclosure around the data acquisition unit.

The Contractor shall submit details of the Monitoring Enclosure (i.e. materials and location(s) etc.) to the Contract Administrator for review, prior to construction.

The Contractor shall ensure access to the Monitoring Enclosure at all times, including but not limited to snow clearing in the winter. The Contractor shall also transfer the key for the lock of the Monitoring Enclosure to the Contract Administrator.

5.04 Installation

Installation of the VW piezometer shall be as per the manufacturer's recommendations in addition to what is stated or emphasized below.

The VWP shall not be installed closer than 1.5 m to the nearest adjacent edge of shoring or unwatering system. The exact location of the VWP installation shall be determined in the field after sub-excavation and backfilling to original ground surface.

It is known that the process of installing VW piezometers can temporarily alter the pore water pressure acting on the piezometer tip. The installation of a VW piezometer shall not be considered to be complete until the pore pressure acting on the piezometer has returned to and stabilized at the value prevailing in the surrounding, unaffected soil mass. The Contractor shall take daily reading of the pore pressures until the value has stabilized. Stabilization shall be deemed to have occurred:

a) When no change in the measured value has occurred over a period of 5 days and the measured value is within 10% of the anticipated hydrostatic value.
b) When the daily rate of change is less than four (4) kPa per day for three consecutive days and the measured value is within 5% of the anticipated hydrostatic value.

The Contractor shall be prepared to wait for a period of 10 to 15 days after completion of installation of the VWP for the baseline readings to stabilize prior to the commencement of the construction works.

The Contractor shall notify the Contract Administrator no later than 3 days after installing a VW piezometer. At this time, the Contractor shall also supply the following information to the Contract Administrator.

a) Date of installation;
b) Installation notes/sketches;
c) VW piezometer location, easting, northing referenced to MTM NAD83 coordinates;
d) Elevation of VW sensor referenced to Geodetic datum;
e) Stratigraphic log of subsurface conditions, including drilling method notes;
f) Model, make and serial numbers of VW sensor, readout unit and signal cable; and
g) Calibration details of VW sensor.

5.05 Coordination With Monitoring/Notification

Monitoring of the VW piezometer shall be done by others. Monitoring shall be conducted during embankment fill construction and during the preload/surcharge period.

The Contractor shall be available for one site meeting with the Contract Administrator to transfer equipment and data and to answer any questions from the Contract Administrator regarding the monitoring instruments baseline data and software.

6.0 DECOMMISSING OF INSTRUMENTS

The Contractor shall decommission all the Settlement Rods (SRs), VW piezometer (VWP), and Benchmark (BM) at the end of the monitoring program following construction unless advised otherwise by the Contract Administrator. Decommissioning of instrumentation shall be carried out according to the Ontario Water Resources act, R.R.O. 1990, Regulation 903 (as amended by Ontario Reg. 372).

7.0 PAYMENT

7.1 Basis Of Payment

Payment at the Lump Sum price for this tender item shall be full compensation for all labour, monitoring equipment and material to do the work.

1.0 GENERAL

Requirements specified for Specialist Qualifications, Services, Deliverables and Records; and the Foundation Monitoring Plan apply to all Instrumentation Monitoring. Instrumentation monitoring is required for the following items:

a) Settlement Rods (SR)
b) Vibrating Wire Piezometers (VWP)

The instrumentation monitoring services include:

a) Data collection, data reduction and reporting;
b) Adherence to criteria used to assess the embankment performance based on the monitoring data collected from the instruments installed by others;
c) Interpretation of instrument readings for the purpose of providing geotechnical input for the rate of fill placement, timing for paving and start of pile driving for bridge abutments.

1.01 Or equal

The term, “or equal”, shall be understood to indicate that the equal product is the same or better than the specified product in function, performance, reliability, quality and general configuration.

1.02 Specialist Qualifications

The Foundation Engineering Consultant services required for this assignment have been categorized ***Geotechnical specialty – Medium Complexity.***

The Foundation Engineering Consultants that are registered in MTO's consultant acquisition system (RAQS) at complexity ratings in the required specialty that meet or exceed the identified complexity requirement for this assignment are eligible to provide Foundation Engineering services for this project. The Foundation Monitoring Consultant shall not be the same Geotechnical Consultant retained by the Contractor for the supply and installation of embankment monitoring equipment. The Foundation Monitoring Consultant shall be directly retained by MTO through the CA assignment.

1.03 Services, Deliverables and Records

The Foundation Monitoring Consultant shall:

a) Review the Monitoring Programme and, if deemed necessary, submit in writing to the Contract Administrator (CA) recommendations for modifications to the Monitoring Programme;
b) Review the proposal for installation of VW instruments and data logger setup by the Contractor;
c) Review the reading frequency (i.e. number of readings taken per day per instrument) to be programmed and saved in the data loggers based on the available data logger storage capacity;
d) Meet with the Contractor to receive the VW Data Recorder, Portable Laptop Computer and associated software used for monitoring vibrating wire instruments and to receive reports with installation details of instruments installed by the Contractor, as specified in special provision entitled “Supply and

Installation of Embankment Monitoring Equipment", included in the contract documents. Contractor's reports shall include all calibration certificates;

e) The Foundation Monitoring Consultant is required on site to establish the baseline readings. The CA staff may take all other required readings and immediately forward the readings to the Foundation Monitoring Consultant. The Foundation Monitoring Consultant shall train CA staff on how to obtain and download monitoring readings;

f) Calibrate and maintain monitoring equipment;

g) Reduce monitoring data and prepare monitoring reports;

h) Provide transmittal of instrument readings and reports to the CA;

i) Interpret instrument readings as needed for the purposes of ongoing construction;

j) Notify the CA of required modifications to the construction procedures accordingly, if necessary. Interpretation shall include making correlations between monitoring data and specific construction activities;

k) Notify the CA within 24 hours if critical instrument readings (i.e. review/alert levels), as specified herein, for any instrument have been reached;

l) Discuss within 48 hours with the CA response action(s), and submit a plan of actions, to prevent the critical instrument readings (i.e. review/alert levels) from being exceeded.

A monthly progress report shall be submitted to the CA, MTO Contract Services Administrator and MTO Foundations Office. Monthly reports shall be issued from the beginning of construction monitoring to the end of waiting period after the top of preload has been reached. The progress report shall discuss the Contractor's operations with respect to the installation of instruments and/or a summary of the monitoring that has been completed for the month.

The CA shall maintain a Foundations Monitoring diary and shall provide a copy of this diary to the Foundation Monitoring Consultant. The diary shall document original conditions, work in progress, including extent and height of fill placement, any unusual or problem situations that arise, record of actions taken by the Contractor to rectify the situation, and restored conditions. The diary shall be supported by photographs of these conditions.

1.04 Submission of Foundation Monitoring Plan

The Foundation Monitoring Consultant shall, in a brief narrative, discuss the applicable experience and qualifications of specialist staff, the role that each will play in administration of the monitoring tasks, the authority to be assumed, and the reporting relationship with the construction administration staff.

The Foundation Monitoring Consultant shall also complete the Foundation Monitoring Plan table in the format provided below.

Foundation Monitoring Plan		
Major Monitoring Tasks	**Level of Monitoring**	**Deliverable Record(s)**
List major monitoring tasks associated with foundation monitoring.	State frequency/level of monitoring.	List associated Deliverable Records for each task.

2.0 PURPOSE

The purpose of these instruments is to monitor settlements and excess pore water pressures in the foundation soils at selected locations during construction of the south approach embankment of the proposed Driftwood River Replacement Bridge on Highway 577 in the Township of Taylor, District of Cochrane, Ontario.

The rate of fill placement, timing for paving and construction shall be controlled by the instrument readings.

The instruments shall not be decommissioned unless instructed by the CA after discussion with and concurrence from MTO.

3.0 DRAWINGS

Reference shall be made to the following drawing included in the Contract Documents:

a) Hwy. 577, Driftwood River Bridge, Monitoring and Instrumentation Plan

4.0 SUBSURFACE CONDITIONS

The subsurface conditions at the site are described in the Foundation Investigation Report titled *Driftwood River Bridge Replacement, Highway 577, Assignment No. 5016-E-0038, WO #13, Ministry of Transportation, Ontario, G.W.P. No. 5104-18-00, Site 39E-096.*

5.0 READING SCHEDULE AND FREQUENCY

The Foundation Monitoring Consultant shall save and archive raw data in electronic and hard copy format.

Monitoring shall commence immediately after the installation of an instrument. Monitoring is to continue during a period from the date of instrument installation to at least seven (7) months following completion of embankment.

The minimum monitoring frequencies along with the anticipated number of readings for the approach embankments are given in the following sections. The monitoring frequency is the same for each individual instrument. Instruments shall be read more or less frequently if determined to be required by the CA.

It should be noted that the number of readings given in the following sections are approximate and may vary depending upon the embankment performance.

5.01 Minimum Monitoring Frequency

The minimum monitoring frequency for each instrument is summarized in Table 1.

Table 1 – Minimum Monitoring Frequency

STAGE	FREQUENCY	ANTICIPATED NUMBER OF READINGS PER INSTRUMENT (*)
Baseline Reading (**)	3 readings on 3 consecutive days, no sooner than 5 days following installation	3
Immediately prior to start of embankment construction	Once	1
During embankment construction	Once every 1 m lift and following placement of last lift	6 to 8 (South Approach)
During preload period (anticipated duration: 7 months)	- Weekly for Months 1 & 2 - Bi-weekly for Months 3 to 5 - Monthly for Months 6 to 7	8 6 2

Note: (*) Due to the uncertainty in the construction schedule, the number of readings may vary from those shown.

(**) Baseline Readings: Value of instrument readings taken prior to construction to provide a baseline against which all subsequent readings are compared to assess movements of ground and changes in piezometric head.

6.0 INSTRUMENT SPECIFIC REQUIREMENTS

6.01 Settlement Rods (SR)

6.01.01 Surveying

The elevations of the survey target of the settlement rods (SR) shall be surveyed to an accuracy of plus/minus two (±2) mm or better and shall be reported to the nearest millimetre.

During each reading of the SRs, the elevations of the top of embankment at the SR location shall be surveyed to an accuracy of plus/minus ten (±10) mm or better and shall be reported to the nearest 10 millimetre.

During the embankment construction, the Contractor will extend all settlement rods, friction reducing sleeves and CSP protective surrounds simultaneously prior to the placement of the next lift of fill. The Contractor will notify the CA no less than 3 days prior to extending any settlement rod. Surveying of the elevations of the top of rod immediately before and immediately after the extension of SRs to an accuracy of ±2 mm is necessary to accurately track the settlement data. The survey of SR length adjustments shall be coordinated with the CA and the Contractor.

Surveying for settlement monitoring shall be conducted by a registered surveyor with appropriate equipment and experience. The surveyor shall be retained by the CA.

6.01.02 Reporting

The CA shall be notified within 24 hours if critical readings are reached and a brief interpretation of the updated monitoring data shall be reported to the CA within five (5) working days after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical forms in the monthly progress report.

As a minimum, the following shall be submitted to the CA in the monthly progress report based on the readings collected from SR instruments:

a) A plot of settlement of the base of the embankment (SRs) versus time;
b) Fill height/top of fill elevation within 5 m of the instruments versus time;
c) Plan view, cross section and profile sketches showing the top of fill location of the embankment when the instrument data is collected.

6.01.03 Review and Alert Levels

A target settlement of 115 mm is specified and a minimum preload period of 7 months is required. An accelerated/sudden settlement of 250 mm is set as the Review Level and an accelerated/sudden settlement of 350 mm is set as the Alert Level.

If the maximum settlement measured exceeds the Review Level, the Foundation Monitoring Consultant shall immediately inform the CA and the CA will request the Contractor for plan of action(s). The Foundation Monitoring Consultant will review the plan of action(s) submitted by the Contractor to prevent the alert level from being reached and provide recommendations to the CA. All construction work shall be continued such that instrument alert levels are not reached.

If the maximum settlement measured exceeds the Alert Level, the Foundation Monitoring Consultant shall immediately inform the CA and the CA shall instruct the Contractor to stop all construction activities on and within the embankment. No construction shall take place on the affected embankment until all the following conditions are satisfied:

a) The cause of the accelerated/sudden settlement has been identified and analyzed by the Foundation Monitoring Consultant;
b) CA to request the Contractor to submit a plan of corrective action(s);
c) Foundation Monitoring Consultant to review Contractor’s plan of corrective action(s) and provide recommendations to the CA;
d) Any corrective action deemed necessary by the CA and the Foundation Monitoring Consultant has been implemented;
e) The CA deems that it is safe to proceed with the construction of the remainder of the embankment.

6.02 Vibrating Wire Piezometer (VWP)

6.02.01 Data Logger and Readout Unit

The VWP shall be read using the VW Data Logger installed in the monitoring enclosure and supplied by the Contractor.

The data logger and readout unit shall be tested prior to taking any baseline readings to ensure functionality and repeatability.

6.02.02 Coordination of Readings

The VWP data reduction or calculation of excess pore pressure (EPP: pore pressure in excess of hydrostatic) requires the hydrostatic groundwater level elevation at the time the VWPs are read. Excess pore pressure should be calculated based on a hydrostatic groundwater level of 261.0 m.

6.02.03 Reporting

The CA shall be notified within 24 hours if critical readings are reached and a brief interpretation of the updated monitoring data shall be reported to the CA within five (5) working days after each set of readings is obtained. A full set of up-to-date and processed monitoring data shall be presented in tabular and graphical forms in the monthly progress report.

As a minimum the following shall be submitted to the CA in the monthly progress report based on the readings collected from the VWP:

a) Plot of excess pore pressure (EPP) for the corresponding VWP versus time;
b) Plot of hydrostatic groundwater elevation versus time;
c) Fill elevation at the VWP location versus time.

6.02.04 Review and Alert Levels

An excess pore water pressure value of 80 kPa is specified for the Review Level and an excess pore water pressure value of 100 kPa is specified for the Alert Level.

If the measured excess pore water pressure exceeds the Review Level, the Foundation Monitoring Consultant shall immediately inform the CA and the CA will request the Contractor to provide a response action(s). The Contractor shall submit a plan of action(s) to prevent the alert level from being reached. This will be reviewed by the Foundation Monitoring Consultant who will recommend a course of action to the CA. All construction work shall be continued such that instrument alert levels are not reached.

If the measured excess pore water pressure exceeds the Alert Level, the Foundation Monitoring Consultant shall immediately inform the CA and the CA shall instruct the Contractor to stop all construction activities on and within the embankment. No construction shall take place on or nearby the affected embankment until all of the following conditions are satisfied:

a) The cause of the excess pore pressure exceedance has been identified and analyzed by the Foundation Monitoring Consultant;
b) CA to request the Contractor to submit a plan of corrective action(s);
c) Foundation Monitoring Consultant to review Contractor’s plan of corrective action(s) and provide recommendations for corrective action(s) to the CA;
d) Any corrective action deemed necessary by CA and the Foundation Monitoring Consultant has been implemented;
e) The CA deems that it is safe to proceed with the construction of the remainder of the embankment.

7.0 CONTROL MONITORING LEVELS

7.01 General

The monitoring programme will provide input for the control of the rate of fill placement and timing for paving.

7.02 Stabilization of Settlements due to Primary Consolidation

Settlement data monitored at SRs and excess pore water pressure in the Vibrating Wire Piezometer will allow an approximate assessment of the total settlement due to primary consolidation and the approximate time required for settlements due to primary consolidation to stabilize.

The anticipated amount of total settlement and the required time for settlements due to primary consolidation to stabilize shall be assessed for each SR using an appropriate analytical method.

8.0 FINAL REPORT

At the completion of the monitoring programme, a final monitoring report shall be issued to the CA. The monitoring results shall be presented in tabular and graphical forms as described above for each instrument type. Interpretation of the monitoring readings shall be included in the report.

<u>GEOGRID</u> - Item No.

Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and placement of a bi-axial geogrid (geogrid) at the locations specified in the Contract. The geogrid is intended to provide support for construction equipment travelling on soft/weak soils while constructing approach embankments.

2.0 REFERENCES

This special provision references the following standards, specifications or publications where applicable:

ASTM International

D4355 - 07	Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus
D4759 - 02(2007)	Standard Practice for Determining the Specification Conformance of Geosynthetics
D5818 - 11	Standard Practice for Exposure and Retrieval of Samples to Evaluate Installation Damage of Geosynthetics
D6637 - 10	Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 206	Construction Specification for Grading;
OPSS.PROV 209	Construction Specification for Embankments Over Swamps And Compressible Soils;
OPSS.PROV 501	Construction Specification for Compacting;
OPSS.PROV 510	Construction Specification for Removal; and
OPSS.PROV 1860	Material Specification for Geotextiles.

3.0 DEFINITIONS

For the purposes of this specification the following definitions apply:

Aperture means an opening, such as a hole, gap, or slit.

Geogrid means a sheet-like woven or non-woven geosynthetic having a regular network of apertures that function as reinforcement by allowing interlocking of soils, rock or similar material. Used to assist in the engineered creation of monolithic structures to resist forces and loads.

Geosynthetic means a synthetic material used in geotechnical engineering applications. Geosynthetics may include such items as geotextiles, geomembranes, geocells, geogrids, geonets, and geocomposites.

MD means machine direction.

Quality Assurance (QA) means a system or series of activities carried out by the Owner to ensure that materials received from the Contractor meet the specified requirements.

Quality Control (QC) means a system or series of activities carried out by the Contractor, Subcontractor, supplier, and manufacturer to ensure that materials supplied to the Owner meet the specified requirements.

XMD means cross-machine direction.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

4.01.01 Foundation Investigation Report

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

4.01.02 Submission Requirements

At least three (3) weeks before commencement of work, the Contractor shall submit to the Contract Administrator six copies with information on the geogrid installation including:

a) Equipment to be used and methodology to deal with obstructions that might be encountered;
b) Methodology of laying geogrid on the geotextile, backfill placement and compaction;
c) Designs verifying that the selected geogrid will support the weight of construction equipment;
d) Drawings illustrating equipment layout;
e) Geogrid material specifications, data sheets; and
f) Installation schedule.

The Contract Administrator shall be notified a minimum of 10 working days in advance of commencing the installation.

The Contractor is advised that the undrained shear strength of the silty clay soils at the design geogrid elevation may vary from 20 kPa to 40 kPa.

5.0 MATERIALS, DELIVERY AND STORAGE

Non-woven geotextile fabric has been specified elsewhere in the contract. The Contractor shall supply and install a geogrid designed to support the proposed construction equipment intended to be used for excavation and construction of the approach embankments.

All geogrid materials supplied shall be free of defects, rips, holes or flaws. During shipment the geogrid shall be protected from damage. During on-site storage, the storage area shall be such that the geogrid is protected from sunlight, dirt, dust, mud, debris and any other detrimental substances.

6.0 CONSTRUCTION

6.01 General

The geogrid shall be installed as specified in the Contract Documents and as per the manufacturer's specifications. No changes to the layout, shall be made without the prior written consent of the Contract Administrator

The area shall be cleared of sharp objects that may damage the geogrid.

The geogrid shall be placed with a minimum overlap longitudinally of 0.3 m, and a minimum overlap of 0.3 m transversely between rolls, or greater as specified by the manufacturer. As part of the installation, the geogrid shall be pulled taut to remove any slack prior to placement of granular fill. The geogrid shall be temporarily secured in place with staples, pins, sand bags or backfill material to prevent movement during backfill placement.

The Contractor shall be responsible for any damage to the geogrid during construction, including on-site storage and installation. If the geogrid is damaged it shall be replaced at no additional cost to the Owner.

Should a discrepancy exist between the Contract Documents and the manufacturer's specifications regarding the installation procedure, then the manufacturer's specifications shall take precedence.

6.02 Operational Constraints

Vehicular and/or construction equipment shall not be allowed to operate directly on the geogrid. A minimum of 150 mm or granular material shall be placed on top of the geogrid prior to allowing any vehicular and/or construction equipment traffic over the area. Sudden braking and sharp turns shall be avoided.

The Contractor is advised that the undrained shear strength of the silty clay soils at the design geogrid elevation may vary from 20 kPa to 40 kPa.

6.03 Management of Excess Materials

Management of excess material shall be according to the Contract Documents.

7.0 MEASUREMENT FOR PAYMENT

7.01 Geogrid

Measurement will be in square metres with no allowance made for overlap.

8.0 BASIS OF PAYMENT

8.01 Geogrid - Item

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