Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
DRIFTWOOD RIVER BRIDGE REPLACEMENT
HIGHWAY 577
ASSIGNMENT No. 5013-E-0018
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 417-91-00, SITE 39E-096
GEOCRES NO.

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ASSIGNMENT No. 5013-E-0018, G.W.P. 417-91-00

1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by MMM Group Limited (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of preliminary designs for the rehabilitation of structures identified in MTO's Request for Proposal (RFP) titled *"Preliminary Design, Rehabilitation/Replacement of Twelve Structures on Highway 11, 101, 577, 579, 634 & 668, in New Liskeard Area", Contract Number 5013-E-0018*.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's RFP, and in Section 5.7 of MMM's *Technical Proposal* for this assignment. 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 (with coordinates of N 5,379,260; E 328,350) 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.

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

The field work for this project was carried out between July 21 and 30, 2014. 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 at the approximate locations shown on Drawing 1. Terraprobe's staff staked out the borehole locations in the field relative to on-site features and, MMM 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 the boreholes. This data is summarized in the following table.

Borehole Details

Borehole No.	MTM NAD 83 Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH1	5 379 214.06	328 355.2	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

* River bed elevation.

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.

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 Borehole 1, 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 all of the boreholes, 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 accordance with MTO and/or ASTM Standards as appropriate.

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"

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

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.

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 compact sand and firm to 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 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 concrete 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.

Pavement Granular Borehole Data

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 size particles.

4.2.2 Fill – Sand

Boreholes 1 and 4 encountered a layer of sand fill below the flexible pavement. The locations, thicknesses, depths and base elevations of the sand fill are summarized in the following table.

Sand Fill Borehole Data

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

Standard Penetration tests performed in the sand fill measured SPT N-values ranging from 4 to 29 blows for 0.3 m of penetration indicating a very loose to compact relative density. The natural water content of samples of the sand fill range from 1% to 12% by weight.

The grain size distribution curve of a sample of this sand fill is shown on Figure B2 in Appendix B. The results show a grain size distribution consisting of 13% gravel, 80% sand and 7% silt size particles.

4.2.3 Fill – Silty Clay

Silty clay fill was encountered in Boreholes 1 and 4, and the locations, thicknesses, depths and base elevations of the silty clay fill are summarized in the following table.

Silty Clay Fill Borehole Data

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

Standard Penetration tests in the silty clay fill measured SPT N-values ranging from 5 to 11 blows for 0.3 m of penetration indicating a firm to stiff consistency. The natural water content (by weight) of a sample of the silty clay fill is 37%.

The grain size distribution curve of a sample of the silty clay fill is depicted on Figure B3 in Appendix B. These results show a grain size distribution consisting of 2% gravel, 7% sand, 39% silt and 52% clay size particles.

Because of insufficient sample quantities, Atterberg limits tests were not possible.

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.

Organic Silt Borehole Data

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 30 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.

Silty Clay to Clay Borehole Data

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

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 7.3, indicating a low sensitivity soil class (Canadian Foundation Engineering Manual [CFEM], 2006).

The variation of undrained shear strength with elevation plot depicted in Figure B7 generally illustrates low undrained shear strength values ranging from 12 kPa to 32 kPa. Below elevation 252.0 m, higher undrained shear strength values ranging from about 36 kPa to 88 kPa were measured.

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 B10 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 B11 to B13 in Appendix B. The results indicate a cohesive deposit of generally intermediate to high plasticity (CI to CH) with occasional low plasticity clay (CL). The Atterberg limits test results are summarized below.

Liquid Limit:	28% 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 B14. These results illustrate that the natural moisture contents of the tested samples are typically higher than the liquid limits. The moisture content of twenty seven samples of the silty clay to clay varies between 21% and 80% and the unit weight of a tested sample is 17.0 kN/m^3.

A one-dimensional consolidation test was performed on a sample of the silty clay to clay and the results are presented in Figures B15 to B18 in Appendix B. The results of the one-dimensional consolidation test are summarized below.

One-Dimensional Consolidation Test Results

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	89.4	88.0	0.547	0.074	1.35

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 pressure derived from the consolidation test data is approximately equal to the effective overburden pressure suggesting that the silty clay to clay deposit is normally consolidated.

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.

Sand Borehole Data

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

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.

Two samples of the sand deposit were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figure B19 in Appendix B. The test results show a grain size distribution consisting of 2% and 7% gravel, 89% and 93% sand and, 4% and 5% silt sized particles.

4.2.7 Silty Sand Till

In Borehole 1 the sand layer is further underlain by a silty sand till deposit that extends to a depth of 40.9 m or to elevation 222.9 m below ground surface.

Standard Penetration tests carried out in the silty sand till deposit gave N-values that range from 66 to more than 100 blows per 0.3 m of penetration indicating a very dense relative density. The moisture content of samples from this stratum range from 6% to 12% by weight.

A grain size distribution test was carried out on a sample from this deposit and the results illustrated in Figure B20, Appendix B; show a grain size distribution consisting of 0% gravel, 52% sand, 31% silt and 17% clay sized particles.

The matrix of the silty sand till contains cobble and boulder inclusions and, NQ-size diamond coring techniques were necessary to extend the boreholes below the cobbles and boulders. A photograph of the cobbles and boulders is provided in Figure B21 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.

Bedrock Borehole Data

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

The argillite bedrock is described as unweathered to slightly weathered, thickly bedded and its colour is greenish grey to dark grey. Photographs of the bedrock core samples are provided in Figures B22 to B24 in Appendix B. Summarized below are the Rock Quality Designation, Rock Mass Quality, Total Core Recovery and Solid Core Recovery.

Rock Core Sample Data

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%

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 all of the boreholes in the lower coarse-grained deposits (sand and silty sand till) overlying bedrock. The head of water is estimated to range from 0.9 m (Elevation 264.8± m) to 1.5 m above ground (Elevation 265.3± m) in Boreholes 4 and 1 respectively.

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. and the routine laboratory and one-dimensional consolidation testing was carried out at Terraprobe's Brampton laboratory.

This report was prepared by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe; with assistance provided by Ms. Sepideh D-Monfared, MESc and Mr. Hussein Ahmed, P.Eng. The report was reviewed by Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact.

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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. 5013-E-0018, G.W.P. 417-91-00

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This report presents interpretation of the geotechnical data in the factual report and presents preliminary geotechnical design recommendations to assist the design team to select a preferred alternative 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 for planning and preliminary design purposes only, as part of the assessment of the feasibility and constructability of potential alternatives.

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. Two span and three span bridge replacement alternatives are being considered on the existing alignment as well as on an alignment offset 13 m east of the existing highway centre line.

Replacement structures will require either raising the existing road profile or constructing the realigned highway to achieve an elevation of 265 m± at the bridge abutments. This geometry will require either raising the existing Highway 577 grade by 1 m± or, constructing new embankments up to 5 m± high.

6.2 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.

6.2.1 Spread Footings

At the abutment and pier locations the soft to stiff silty clay to clay deposit is unsuitable for supporting 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.

6.2.2 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 15± m to 30± m below original 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.

Under 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.

6.2.3 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 are a feasible and practical foundation scheme for supporting both abutments as well as the 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; 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.

6.2.3.1 Axial Resistance

The concentric axial factored geotechnical design resistance at ULS, the foundation load at SLS, and estimated pile tip elevations are tabulated below for two-span and three-span bridge configurations. The structural resistance of the pile should also be checked by the structural designer. For piles founded on bedrock an average UCS value of 200 MPa can be used for preliminary designs.

Axial Resistance of Driven Piles

Location	Reference Borehole	Pile Type	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)*
Two-span Bridge						
South Abutment	BH1	HP 310x110	225.5±	Silty Sand Till	1800	1500
		HP 360x132	225.0±	Silty Sand Till	2100	1600
Pier	BH2 & BH3	610mm Steel Tube	239.0±	Bedrock	6000	N/A
North Abutment	BH3 & BH4	HP 310x110	239.5±	Bedrock	2000	N/A
		HP 360x132	239.5±	Bedrock	2400	N/A
Three-span Bridge						
South Abutment	BH1	HP 310x110	225.5±	Silty Sand Till	1800	1500
		HP 360x132	225.0±	Silty Sand Till	2100	1600
South Pier	BH2	610mm Steel Tube	237.0±	Bedrock	6000	N/A
North Pier	BH3	610mm Steel Tube	241.0±	Bedrock	6000	N/A
North Abutment	BH3 & BH4	HP 310x110	239.5±	Bedrock	2000	N/A
		HP 360x132	239.5±	Bedrock	2400	N/A

* The bedrock is "unyielding" and the SLS condition will not govern.

Pile installation should be carried out in accordance with OPSS 903, November 2009. For piles driven to bedrock the Contractor shall adequately seat the pile on bedrock without damaging the pile as specified in OPSS 903 "Driving to Bedrock". The appropriate pile driving note is "Pile to be driven to bedrock". Steel tube sections shall be founded in rock sockets formed a minimum of 300 mm into the 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 preliminary design purposes an "R" value of 3600 kN is recommended for HP 310x110 piles.

The borehole data indicates that the existing bridge is supported on timber piles that are bearing on soils that are not prone to liquefaction, which implies that pile driving operations will not compromise the load bearing capacity of the existing piles. However, vibrations caused by pile driving will have to be controlled to reduce the risk of damage to the bridge's superstructure and; a pre-construction survey of the structure is recommended.

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.

6.2.3.2 Pile Tips

The tips of all piles 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 either be penetrating into soil containing cobbles and boulders or will be seated on bedrock, 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, increase the probability of achieving the desired penetration in the silty sand till and reduce the probability of sliding or skipping across the bedrock surface.

6.2.3.3 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. To provide the required flexibility in the piles, 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.

Integral Abutment Sand Grading

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%

6.2.3.4 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 preliminary design purposes a maximum horizontal passive resistance of 120 kN (ULS) is recommended.

Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation	Soil Type	Bulk Unit Weight (kN/m3)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m3)*
South Abutment BH 1	263.6 – 261.7	Fill – Gravelly Sand/Sand	20	30	–	6600
	261.7 – 260.1	Fill – Silty Clay	18	0	50	–
	260.1 – 237.5	Silty Clay to Clay	17	0	25	–
	237.5 – 235.4	Sand	19	29	–	1300
	235.4 – 222.9	Silty Sand Till	21	35	–	11000
North Abutment BH 4	263.7 – 261.0	Fill – Gravelly Sand/Sand	20	30	–	6600
	261.0 – 260.2	Fill – Silty Clay	18	0	50	–
	260.2 – 239.9	Silty Clay to Clay	17	0	25	–
	239.9 – 238.2	Sand	20	32	–	8500

* 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 6.8.9.2.

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.

6.2.4 Recommended Foundation Scheme

From a geotechnical point of view, it is recommended that the abutments be supported on steel H-piles and the piers be supported on closed end concrete filled steel tube piles. Based on the advantages, disadvantages, risks and consequences provided in Table 1, a driven H-pile foundation scheme at the abutments and steel tube piles at the piers are considered to be the most reliable, has the lowest risk associated with settlement and the highest probability of acceptable structural performance. This design concept also allows for the design of an integral abutment structure.

6.2.5 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.

6.3 Lateral Earth Pressure

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/m3)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC 2006 and according to Clause 6.9.3 of the CHBDC 2006; 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 be in accordance 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.

Lateral Earth Pressure Coefficients

Wall Condition	**Lateral Earth Pressure Coefficient (K)**					
	OPSS Granular A or OPSS Granular B Type II ϕ = 35°; γ = 22.8 kN/m³		OPSS Granular B Type I ϕ = 32°; γ = 21.2 kN/m³		Rock Fill ϕ = 42°; γ = 19.0 kN/m³	
	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, 2006.

6.4 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; 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.

6.5 Ground Water Control

Artesian conditions were encountered in all of the boreholes in the lower granular deposits but excavations are not expected to extend to this depth. Surface water and ground water control will be required to enable construction below the ground water table. The design, installation, operation and maintenance of the dewatering system is the Contractor's responsibility.

At the abutment locations, the excavation will extend through the existing embankment fill terminating 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 for an H-pile foundation scheme, 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 equipment is used and, adequately sized emergency "stand-by" equipment should also be considered in case of sudden failure of the primary dewatering equipment.

6.6 Approach Embankments

6.6.1 Settlement

To predict the magnitude and time rate of settlement of the underlying silty clay soils the commercially available program Settle 3D developed by Rocscience Inc. was used. The deformation parameters used for the analyses were established using data obtained from a consolidation test as well as empirical correlations of undrained shear strengths, laboratory index tests and soil moisture contents. These deformation parameters are provided in Figures C1, C2, C3 and C4, in Appendix C. The preconsolidation pressure (σ'_p) derived from the consolidation test data is approximately equal to the effective overburden pressure suggesting that the silty clay to clay deposit is normally consolidated.

The deformation parameters are summarized in the following table.

Silty Clay to Clay Deformation Parameters

Parameter	Upper Silty Clay (above elevation 249± m)	Lower Silty Clay (below elevation 249± m)
Overconsolidation Ratio	1.0	1.0
Compression Index - C_c	0.3	0.18
Recompression Index - C_r	0.05	0.035
Initial Void Ratio - e_o	1.50	1.10
Coefficient of Consolidation - C_v (m^2/s)	6.7×10^{-8}	6.7×10^{-8}

For a replacement bridge constructed on the existing highway alignment, a (1 m±) grade raise i.e. up to elevation 265± m is proposed. For this design geometry, it is estimated that the new embankment fill will induce approximately 285 mm of total consolidation settlement of the silty clay deposit in the footprint area of the existing Hwy. 577 embankment. Most of this settlement will be complete in about twelve months and, the remaining post construction settlement after the twelve month period will be less than 25 mm.

If a replacement bridge is constructed on a new alignment i.e. east of the existing highway alignment, the design grade at the abutment locations will be up to 264.5± m. For this design geometry, it is estimated that the embankment fill will induce about 880 mm of total consolidation settlement of the silty clay soils in the footprint area of 4± m high embankments. Most of this settlement will be complete in about twelve months and, the remaining post construction settlement after the twelve month period will be less than 25 mm.

If the replacement bridge is constructed on the existing highway alignment, a detour bridge will be required. The approach embankments of the detour bridge will be approximately 3± m high and, it is estimated that about 725 mm of total consolidation settlement of the silty clay soils will occur below these embankments. After a pre-load period of one year the remaining post construction settlement will be less than 25 mm.

Since the silty clay to clay soils are compressible, it is necessary to pre-load this deposit by constructing the approach embankments in advance of pile driving operations. Pre-loading will reduce the magnitude of post construction settlement, reduce downdrag loads on the piles, and reduce the potential for lateral squeeze and pile buckling. A pre-load period of one year is recommended and, settlement monitoring should be carried out to determine the constructing timing for pile driving and abutment construction i.e. to assess when most of the settlement is complete.

Embankments constructed with local 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.

6.6.2 Stability

The global, internal and surficial stability of the embankment will depend on the slope geometry and also to a large degree on the material used to construct the embankment. For the purpose of embankment stability analyses, the commercially available slope stability program Slide 6.0 developed by Rocscience Inc. was used.

The Morgenstern-Price and Spencer methods for stability analysis were employed and a minimum target factor of safety of 1.3 was established. The soil parameters used for the slope stability analyses and the factors of safety that were obtained are provided in the following table. The slope stability models depicting the corresponding factors of safety are provided in Figures D1 and D2 in Appendix D. The analyses indicate that the factors of safety will be equal to the target factor of safety of 1.3, provided that the embankment is constructed at a minimum side slope geometry of 2 Horizontal to 1 Vertical (2H:1V) or flatter.

Slope Stability Design Parameters and Results

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
Silty Clay to Clay (above elev. 252 m)	0	18	28	0	17
Silty Clay to Clay (below elev. 252 m)	0	2.67H+18*	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	1.3		1.3		-

* Refer to Figure C5 in Appendix C. H = clay layer thickness below elevation 252 m for the corresponding design Cu

6.6.3 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 must meet the requirements of OPSS.PROV 212 and bonding between existing fill and new fill should be carried out by benching in accordance with OPSD 208.010.

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

6.7 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.

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.

Temporary Protection System Design Parameters

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

6.8 Seismic Requirements

The site is treated as lying in Seismic Zone 1. Reference to Annex A3.1 of the CHBDC 2006 indicates that the following seismic parameters (Matheson) should be used for design:

- Velocity Related Seismic Zone 1
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Ground Acceleration 0.08 g (10% in 50 years)

The soil profile type at this site has been classified as Type IV and the Site Coefficient "S (ground motion amplification factor) that should be used in seismic design as per Clause 4.4.6.1, Table 4.4 of the CHBDC is 2.0.

6.9 Additional Studies

It is recommended that the following issues be considered during the future detail design studies.

- Carry out detail level field investigations for temporary and permanent structures and approach embankments;

- Confirm and further refine the preliminary geotechnical recommendations based on the preferred alternative; and
- Complete more rigorous assessments of foundation settlement, embankment stability and settlement for the preferred alternative and study methods for mitigating settlement.

7.0 CLOSURE

This report was prepared by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe; with assistance provided by Ms. Sepideh D-Monfared, MESc and Mr. Hussein Ahmed, P.Eng. The report was reviewed by Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact.

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REFERENCES

Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils*, Second Edition. McGraw Hill Book Company, New York.

Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06. 2006. CSA Special Publication, S6.1 06. Canadian Standard Association.

Chandler, R. J. 1988. *The In Situ Measurement of the Undrained Shear Strength of Clays Using the Field Vane, STP 1014, Vane Shear Strength Testing in Soils: Field and Laboratory Studies,* American Society for Testing and Materials.

Hough, B.K., 1957. *Basic Soils Engineering*. The Ronald Press Company, New York.

Kulhawy, F. H., and Mayne, P. W. 1990. *Manual on Estimating Soil Properties for Foundation Design, Report EL-6800*, Electric Power Research Institute Inc.

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.

Nagaraj, T. S., and Murthy, B. R. S. 1985. *Prediction of the Preconsolidation Pressure and Recompression Index of Soils*. Geotechnical Testing Journal, American Society for Testing and Materials, Vol. 8, No. 4.

Rendon-Herrero, O. 1983. *Universal Compression Index*, Journal of Geotechnical Engineering, Vol. 109, No. 10

Terzaghi, K., and Peck, R. B. (1967). *Soil Mechanics in Engineering Practice.* Wiley, New York.

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 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.

Ontario Provincial Standard Drawings (OPSD)

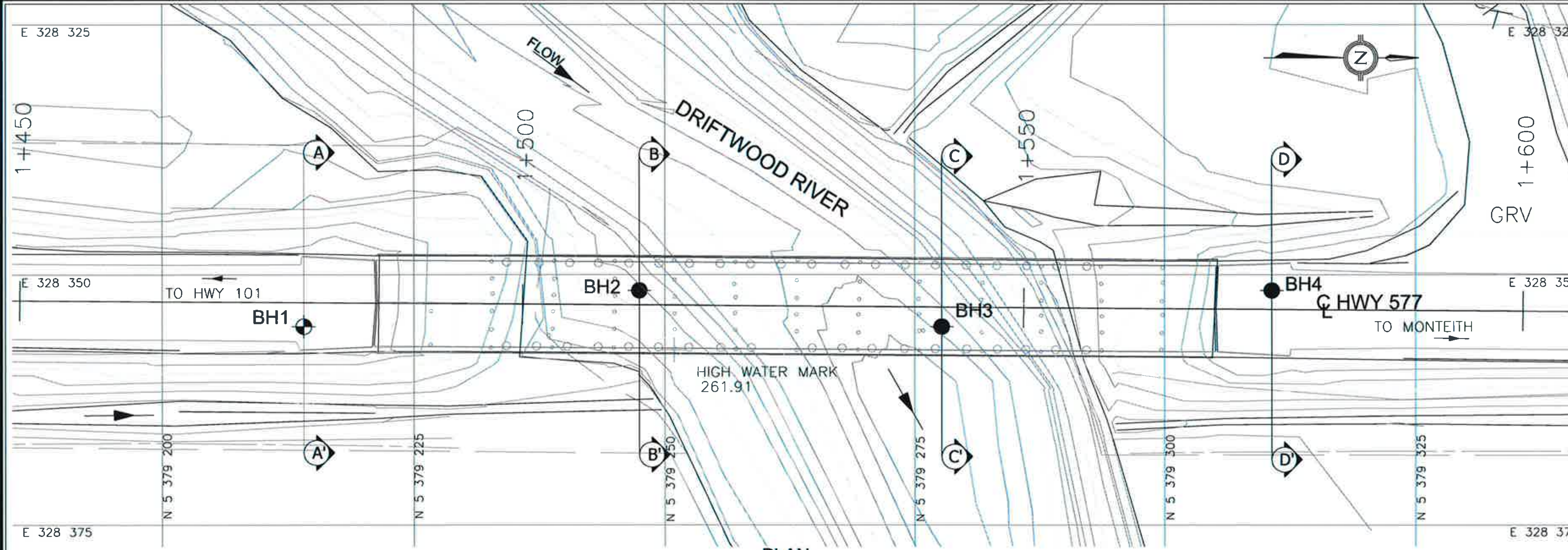
OPSD 208.010	Benching Of Earth Slopes.
OPSD 3090.100	Foundation, Frost Penetration Depths For Northern Ontario
OPSD 3101.150	Walls Abutment Backfill, Minimum Granular Requirement
OPSD 3101.200	Walls, Abutment, Backfill, Rock

TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES

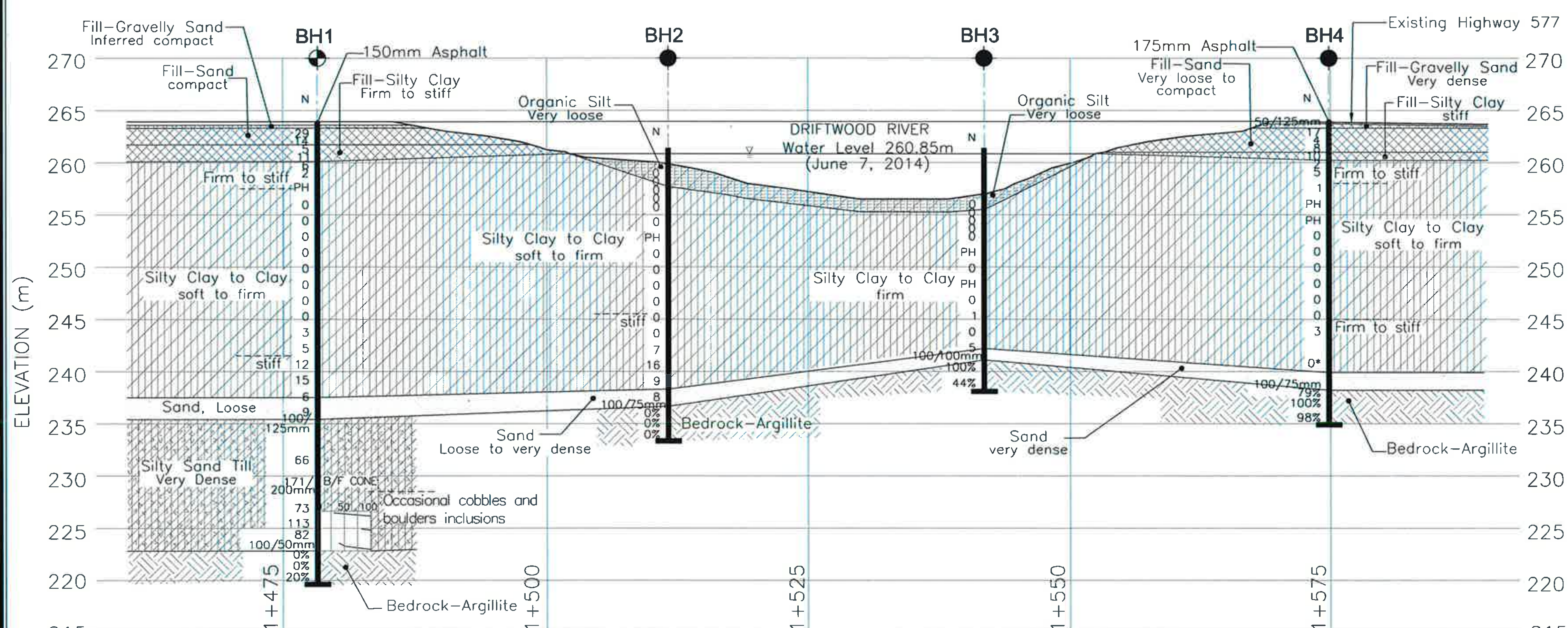
Foundation Element	Pile Foundations (H-Pile Sections)	Pile Foundations (Steel Tube Sections)	Augered Caissons
North and South Abutments	***Advantages:*** ▪ Reliable performance expected. ▪ High geotechnical resistances available by driving piles to refusal or driving piles to bedrock. ▪ Allows for the 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 socketing piles in 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 socketing piles in the bedrock. ▪ Cofferdam not required to facilitate steel tube pile installations. ▪ Pile cap not required compared to H-pile sections. ▪ 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 25mm. ▪ Artesian conditions will increase the level of construction effort.

DRAWINGS

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



PLAN



℄ PROFILE HWY 577

HORIZ. SCALE
VERT. SCALE

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GWP No 417–91–00

HWY 577
DRIFTWOOD RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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KEY PLAN

	LEGEND
	Bore Hole
	Dynamic 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

No	ELEV.	COORDINATES	
		NORTHING	EASTING
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

NOTE

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

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

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of OPS General Conditions.

REFERENCE

Drawings provided in digital format by MMM Group Ltd. by CD (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

REVISIONS	DATE	BY	DESCRIPTION

HWY. 577	PROJECT No. 11–14–4066		GEOCRES No.: 42A–104
SUBM'D. HA	CHKD. RA	DATE: March 2016	SITE: 39E–096
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 1

METRIC
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GWP No 417-91-00

SHEET

HWY 577
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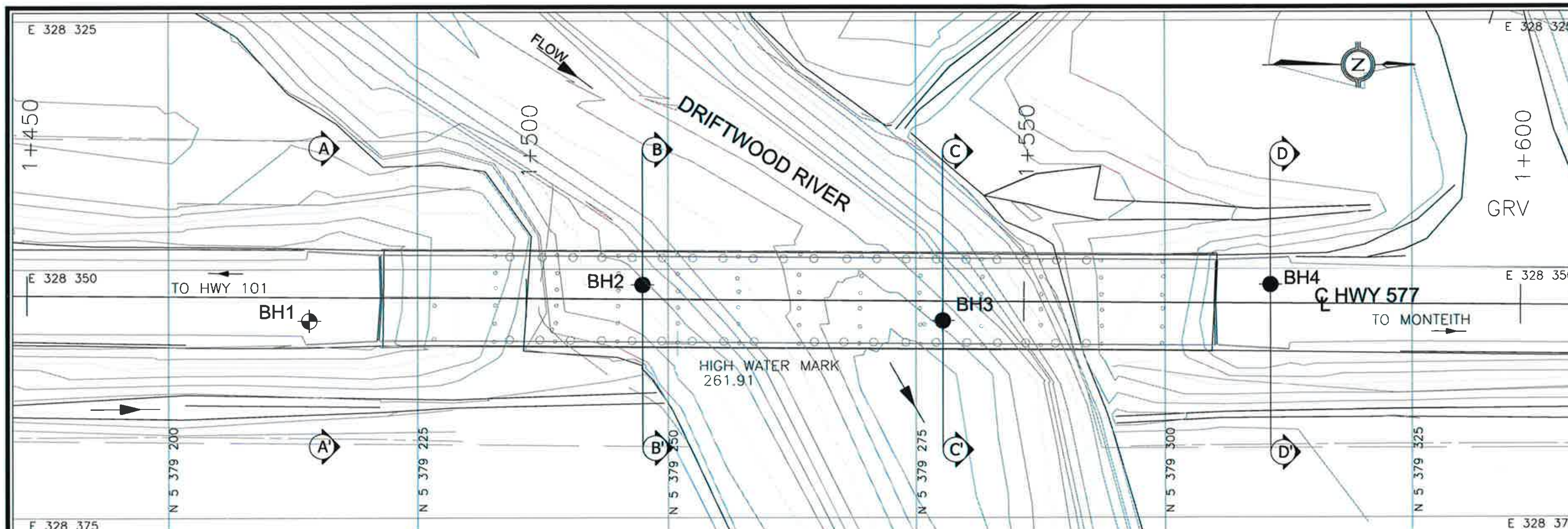

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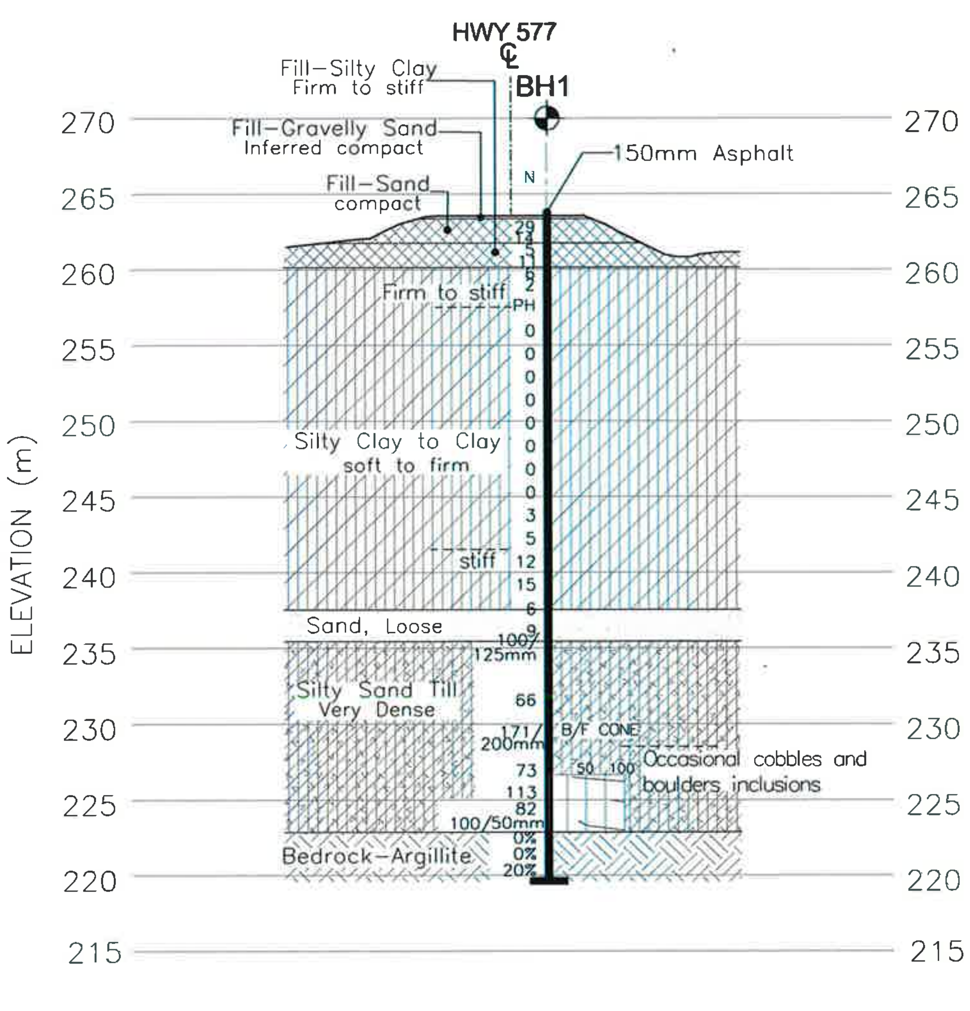

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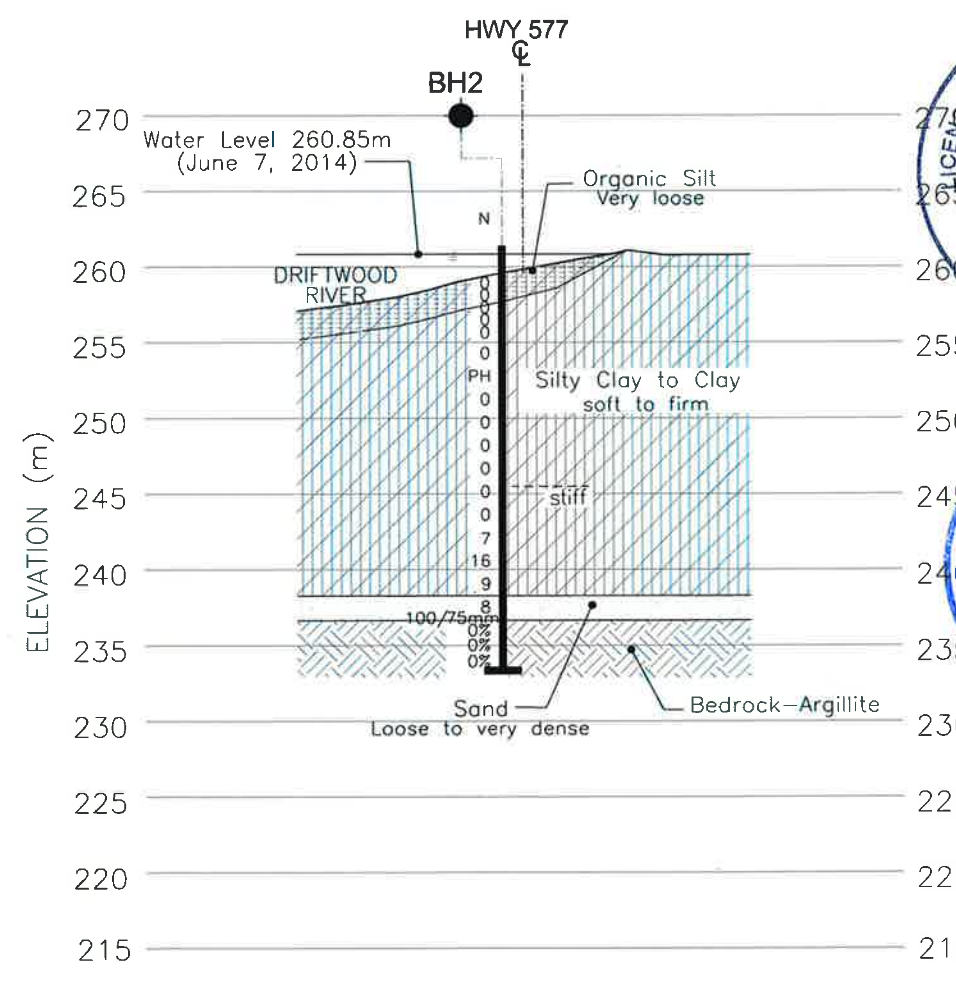

SECTION B-B'

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KEY PLAN

LEGEND

	Bore Hole
	Dynamic 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

No	ELEV.	COORDINATES	
		NORTHING	EASTING
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

NOTE

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

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

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of OPS General Conditions

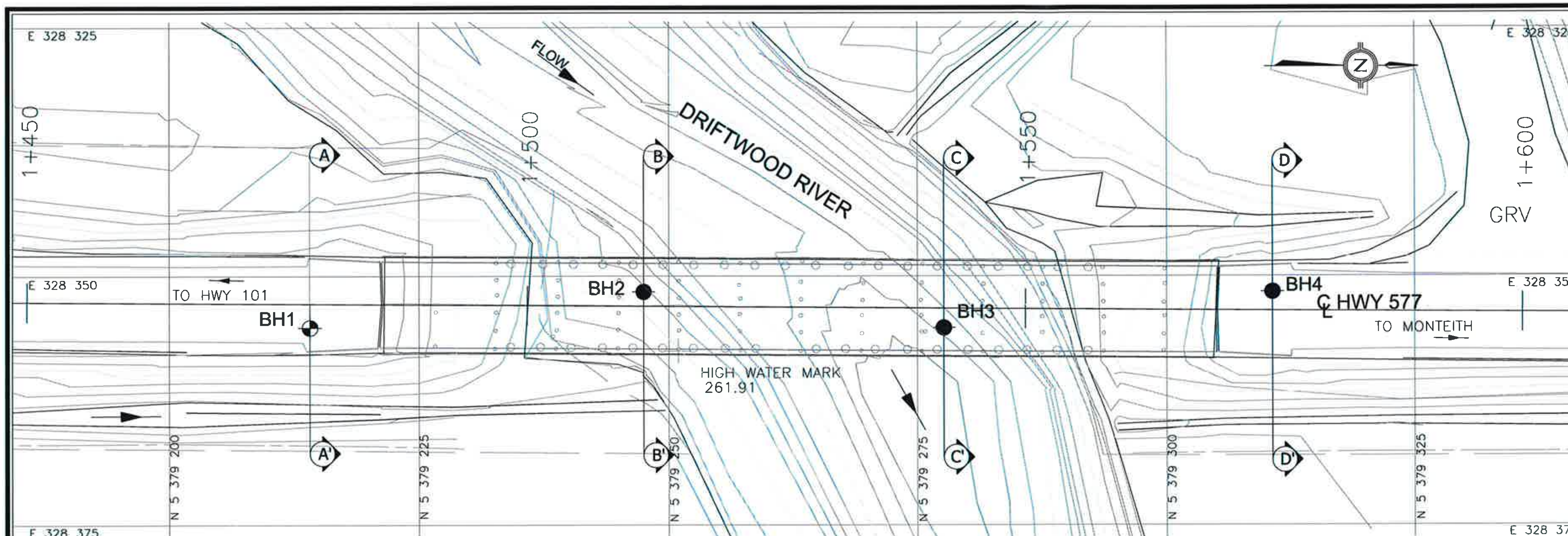
REFERENCE

Drawings provided in digital format by MMM Group Ltd. by CD (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

REVISIONS	DATE	BY	DESCRIPTION

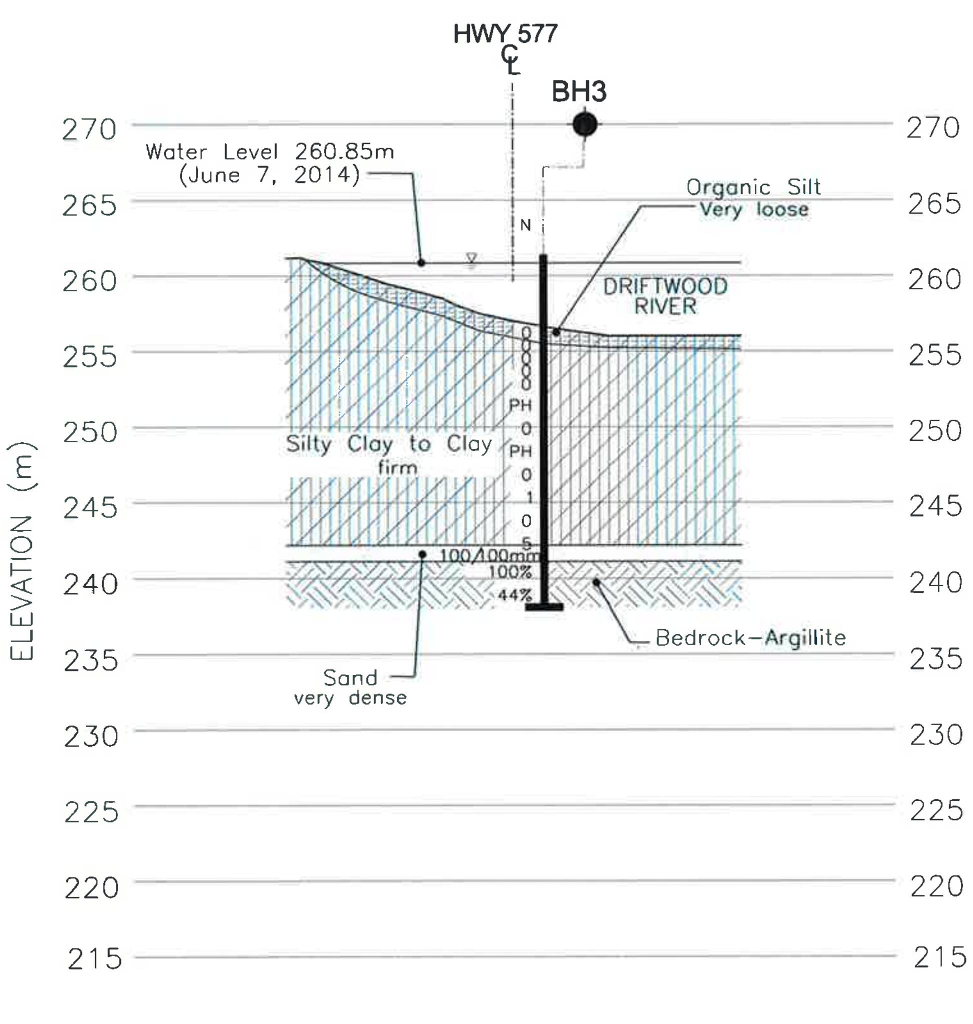
HWY. 577	PROJECT No. 11-14-4066		GEOCRES No.: 42A-104
SUBM'D. HA	CHKD. RA	DATE: March 2016	SITE: 39E-096
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 2

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



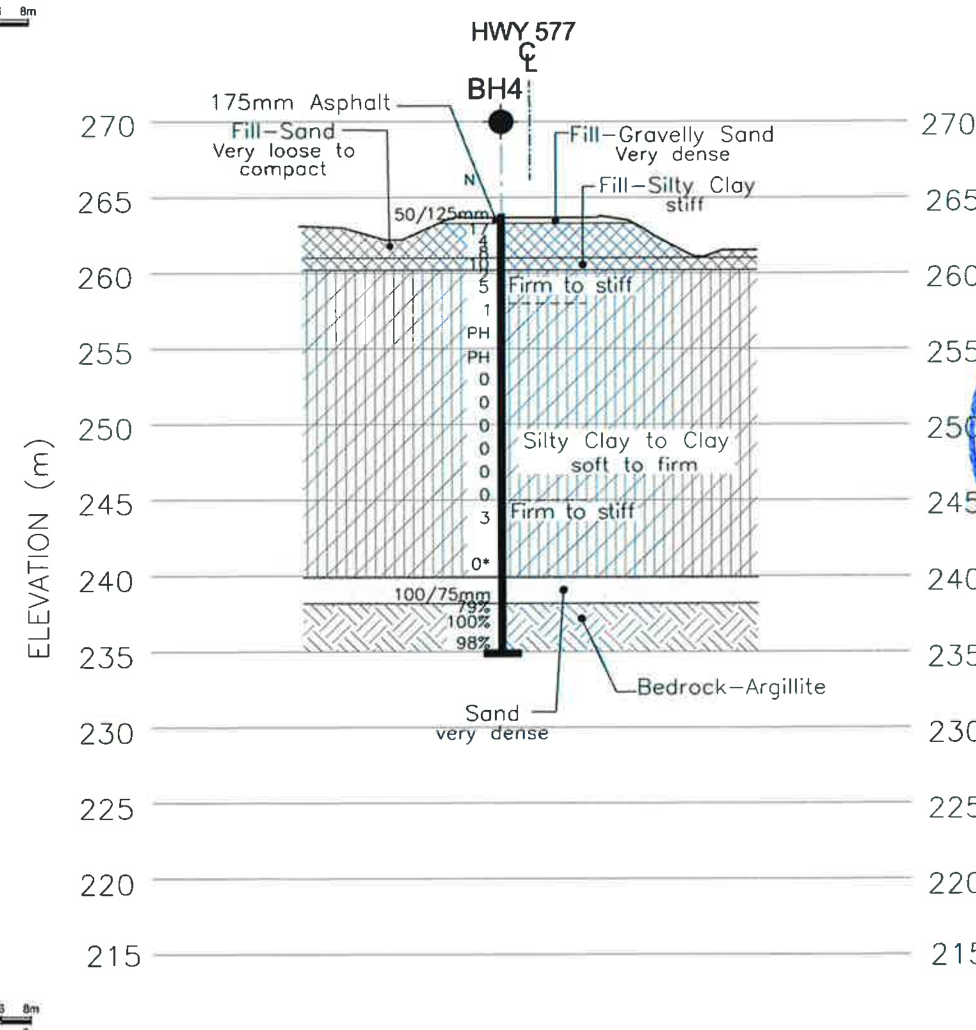
PLAN

SCALE 4 2 0 2 4 6 8m



SECTION C-C'

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



SECTION D-D'

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
March 31, 2016
PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
31-03-2016
PROVINCE OF ONTARIO

GWP No 417-91-00

SHEET

HWY 577
DRIFTWOOD RIVER BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

MMM Group Limited
2655 North Sheridan Way, Suite 300
Mississauga, ON Canada L5K 2P8
t: 905.823.8500, f: 905.823.8503

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

KEY PLAN

LEGEND

	Bore Hole
	Dynamic 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

No	ELEV.	COORDINATES NORTHING	EASTING
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

NOTE

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

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

The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of OPS General Conditions

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REVISIONS	DATE	BY	DESCRIPTION

HWY. 577	PROJECT No.	11-14-4066	GEOCRES No.: 42A-104
SUBM'D. HA	CHKD. RA	DATE: March 2016	SITE: 39E-096
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 3

APPENDIX A

Record of Borehole Sheets

LIMITATIONS AND RISK

Procedures

The soil conditions were confirmed at the borehole locations only and conditions may vary between and beyond the boreholes. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of stratigraphic change.

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities.

Changes In Site And Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The design advice is based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, or there is any additional information relevant to the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

This report was prepared for the express use of the Ministry of Transportation, its retained design consultants and MMM Group Limited. It is not for use by others. This report is copyright of Terraprobe Inc. and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe Inc. The Ministry of Transportation, its retained design consultants and MMM Group Limited, are authorized users.

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.

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

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

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

N (BLOWS/0.3m)	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.

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
r_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
r	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
r_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
r_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
r'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL

e	1,%	VOID RATIO
n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX = (w_L - w_P)
I_L	1	LIQUIDITY INDEX = (w - w_P)/I_P
I_C	1	CONSISTENCY INDEX = (w_L – w)/I_P
e_{max}	1,%	VOID RATIO IN LOOSEST STATE

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

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	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
263.8	**GROUND SURFACE**							
263.6 0.2	150mm **ASPHALTIC CONCRETE**							
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		263	
			3	SS	14		262	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		261	2 7 39 52
			5	SS	11		260	
260.1 3.7	**SILTY CLAY TO CLAY** (varved) containing 1mm to 10mm thick silt layers, soft to firm, grey, wet		6	SS	6			
	trace sand, trace organics, firm to stiff, moist to wet		7	SS	2		259	0 0 58 42
							258	
			8	TW	PH		257	
			9	SS	0*		256	
							255	
			10	SS	0*		254	0 0 21 79
			11	SS	0*		253	July 21, 2014 July 22, 2014
							252	
			12	SS	0*		251	
			13	SS	0*		250	0 1 11 88
							249	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivities plotted (top to bottom): 7.3, 3.0, 3.3, 2.8, 4.6, 2.7, 2.0, 1.6, 2.0, 2.0, 3.0, 2.4, 2.7

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

WATER CONTENT (%): 10 20 30

Water content annotations: 44, LL=41; 57; 65, LL=67

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

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 (continued)

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY TO CLAY (varved) containing 1mm to 10mm thick silt layers, soft to firm, grey, wet	14	SS	0*	
		15	SS	0*	
		16	SS	0*	
		17	SS	3	
		18	SS	5	0 1 51 48
	frequent silt layers, stiff	19	SS	12	
		20	SS	15	
		21	SS	6	
237.5 / 26.3	**SAND**, trace silt, trace gravel, loose, grey, wet	22	SS	9	2 93 (5)
235.4 / 28.4	**SILTY SAND**, some clay, containing cobbles and boulders, very dense, grey, moist to wet (GLACIAL TILL)	23	SS	100 / 125mm	

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

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values plotted: 2.5, 3.3, 4.0, 3.4, 1.6, 1.6, 1.6, 2.1, 3.4, 4.1, 2.5, 2.1, 1.8

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

WATER CONTENT (%): 10 20 30 (value 52 plotted off scale)

UNIT WEIGHT γ kN/m³

Continued Next Page

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

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)	DESCRIPTION	SAMPLE 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		○	0 52 31 17
					231		
					230		
					229		
		25	SS	171 / 200mm			
		26	RC		228	○	
		27	SS	73	227		
					226		
		28	SS	113		○	
					225		
		29	SS	82			
					224		
222.9 / 40.9		30	SS	100 / 50mm	223		July 23, 2014 ------------------ July 24, 2014 NQ Coring
	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.	1	RUN	NQ			RUN# 1 TCR=100% SCR=86% RQD=0%
		2	RUN	NQ	222 / 221		RUN# 2 TCR=90% SCR=75% RQD=0%
		3	RUN	NQ	220		RUN# 3 TCR=48% SCR=40% RQD=20%
219.6 / 44.2	**END OF BOREHOLE**						

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS; 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/m³

>>

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

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			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) ○ 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.

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.

+ 3, × 3: Numbers refer to Sensitivity ○ 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

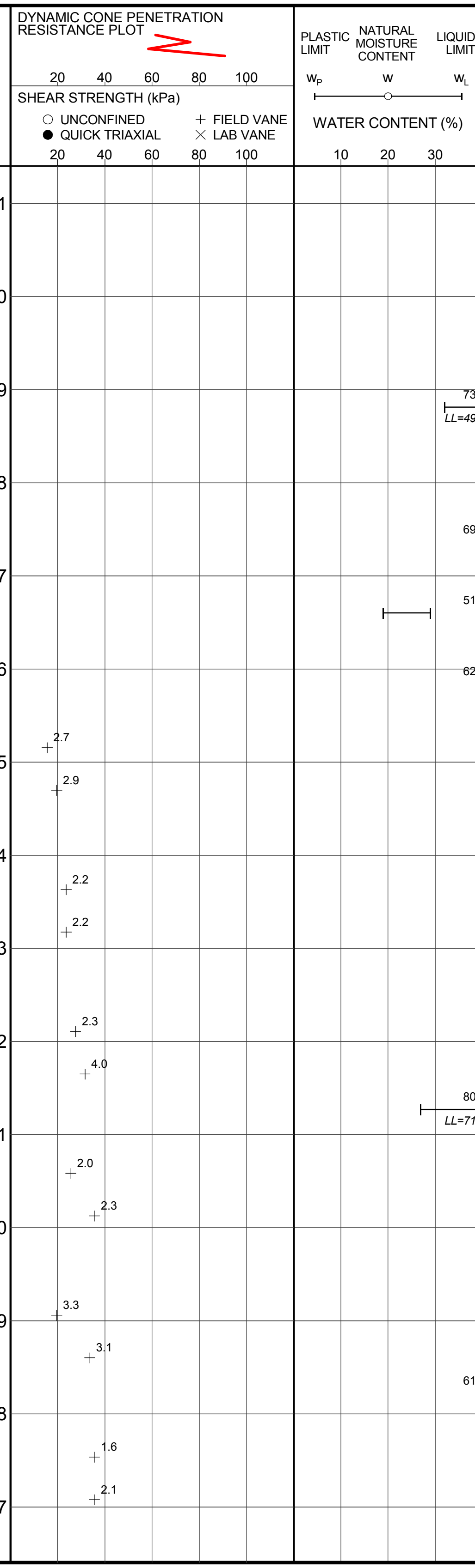
DATUM GEODETIC DATE 2014-7-28 - 2014-7-30 CHECKED BY R.A

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
1	SS	0*
2	SS	0*
3	SS	0*
4	SS	0*
5	SS	0*
6	SS	0*
7	TW	PH
8	SS	0*
9	SS	0*
10	SS	0*
11	SS	0*



UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	0 5 74 21
	0 0 74 26
	0 0 17 83
	July 28, 2014 ----------------- July 29, 2014

Continued Next Page

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

ELEV DEPTH (m)	DESCRIPTION (continued)	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	DCPT / SHEAR STRENGTH (kPa)	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	Field vane 2.6; 2.9		
		12	SS	0*	245		60	
					244	Field vane 2.5; 2.7		
		13	SS	0*	243	Field vane 3.6; 1.9		
		14	SS	7	242		w_P, w	0 0 64 36
					241	Field vane 3.5; 2.6		July 29, 2014 / July 30, 2014
		15	SS	16	240			
		16	SS	9	239		w	
238.3 / 23.1	**SAND**, trace silt, trace gravel, loose to very dense, grey, wet				238			
		17	SS	8	237			
236.7 / 24.7		18	SS	100 / 75mm				NQ Coring
	BEDROCK-ARGILLITE, slightly weathered, thickly bedded, greenish grey, inferred very strong to extremely strong. Fragmented from 24.7m to 28.1m.	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

*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

Ministry of Transportation Ontario

Terraprobe

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	NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
261.4	**WATER SURFACE**				
	RIVER BED				
256.2 5.2	**ORGANIC SILT**, some clay, trace sand, very loose, dark brown, wet	1	SS	0*	
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*	0 0 7 93
		4	SS	0*	
		5	SS	0*	
		6	TW	PH	
		7	SS	0*	0 0 29 71
		8	TW	PH	
		9	SS	0*	0 0 29 71

SOIL PROFILE | SAMPLES | STRAT PLOT | GROUND WATER CONDITIONS | ELEVATION SCALE

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): ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE: 20 40 60 80 100

Field vane sensitivity: 2.3, 2.9, 1.2, 1.6, 4.0, 1.7, 2.1, 3.1

PLASTIC LIMIT w_P | NATURAL MOISTURE CONTENT w | LIQUID LIMIT w_L

WATER CONTENT (%): 10 20 30

Moisture contents: 90; 70 (LL=58); 68; 51 (LL=43)

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

Ministry of Transportation Ontario

Terraprobe

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	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	Shear strength (Field vane, kPa) / Water content	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued) **SILTY CLAY to CLAY**, (varved) occasional silt layers up to 10mm thick, firm, grey, wet				246	+ 3.0; + 2.5	
		10	SS	1	245		July 21, 2014 ------------------ July 22, 2014
	containing frequent 20mm to 30mm thick silt layers					+ 2.2; + 3.2	
		11	SS	0*	244		0 0 54 46
					243	+ 3.8; + 3.2	
242.2		12	SS	5			
19.2	**SAND**, trace silt, trace gravel, very dense, grey, wet				242		
241.1		13	SS	100 / 100mm			7 89 (4)
20.3	**BEDROCK-ARGILLITE**, unweathered, thickly bedded, dark grey, very strong to extremely strong. Mechanically fragmented below 22.5m.	1	RUN	NQ	241 / 240		NQ Coring RUN# 1 TCR=100% SCR=100% RQD=100% UCS**= 165 - 333 (MPa) July 22, 2014 ------------------ July 23, 2014
		2	RUN	NQ	239		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

Ministry of Transportation Ontario

Terraprobe

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 **ASPHALTIC CONCRETE**
263.3 / 0.6	430mm **FILL**, gravelly sand, trace silt, very dense, brown, dry
261.0 / 2.9	**FILL**, sand, some gravel, trace silt, very loose to compact, brown, wet
260.2 / 3.7	**FILL**, silty clay, trace sand, trace gravel, containing organics, stiff, dark brown, wet
	SILTY CLAY TO CLAY (varved) firm to stiff, brown — soft to firm, grey, wet

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	WATER CONTENT / LIMITS	UNIT WEIGHT γ (kN/m³)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	50 / 125mm			21 71 (8)
2	SS	17			
3	SS	4			commence casing and wash boring
4	SS	8			
5	SS	10			
6	SS	2			0 1 53 46
7	SS	5			
8	SS	1	LL=50		0 0 26 74
9	TW	PH			
10	TW	PH	48, LL=46	17.0	July 22, 2014 / July 23, 2014; 0 0 29 71
11	SS	0*			
12	SS	0*	58, LL=56		0 1 14 85
13	SS	0*			

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, ● QUICK TRIAXIAL, + FIELD VANE, × LAB VANE; 20 40 60 80 100

Field vane sensitivity values (top to bottom): 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

Continued Next Page

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

library: library - terraprobe gint.glb report: mto-terraprobe soil file: 11-14-4066 (39e-096) driftwood river bridge.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

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

Column headings: STRAT PLOT; GROUND WATER CONDITIONS; ELEVATION SCALE; 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/m³

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

Field vane sensitivity values plotted: 3.8, 2.7, 4.0, 1.9, 2.2, 2.2, 2.1, 2.7, 2.0, 3.2

Water content value: 51

Continued Next Page

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

Ministry of Transportation Ontario

Terraprobe

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				GROUND WATER CONDITIONS	ELEVATION SCALE	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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE						

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

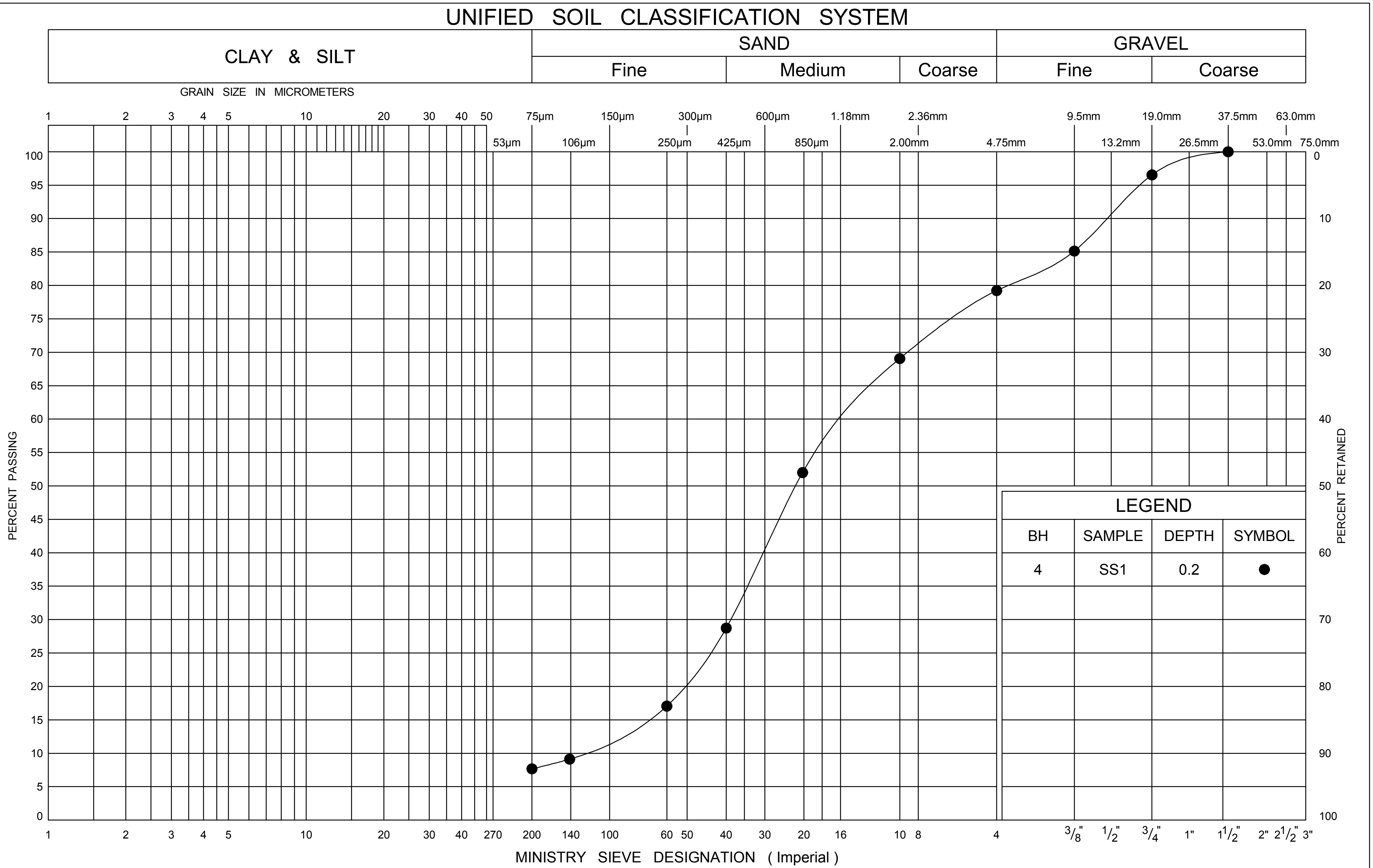
APPENDIX B

Field & Laboratory Test Results

&

Photographs

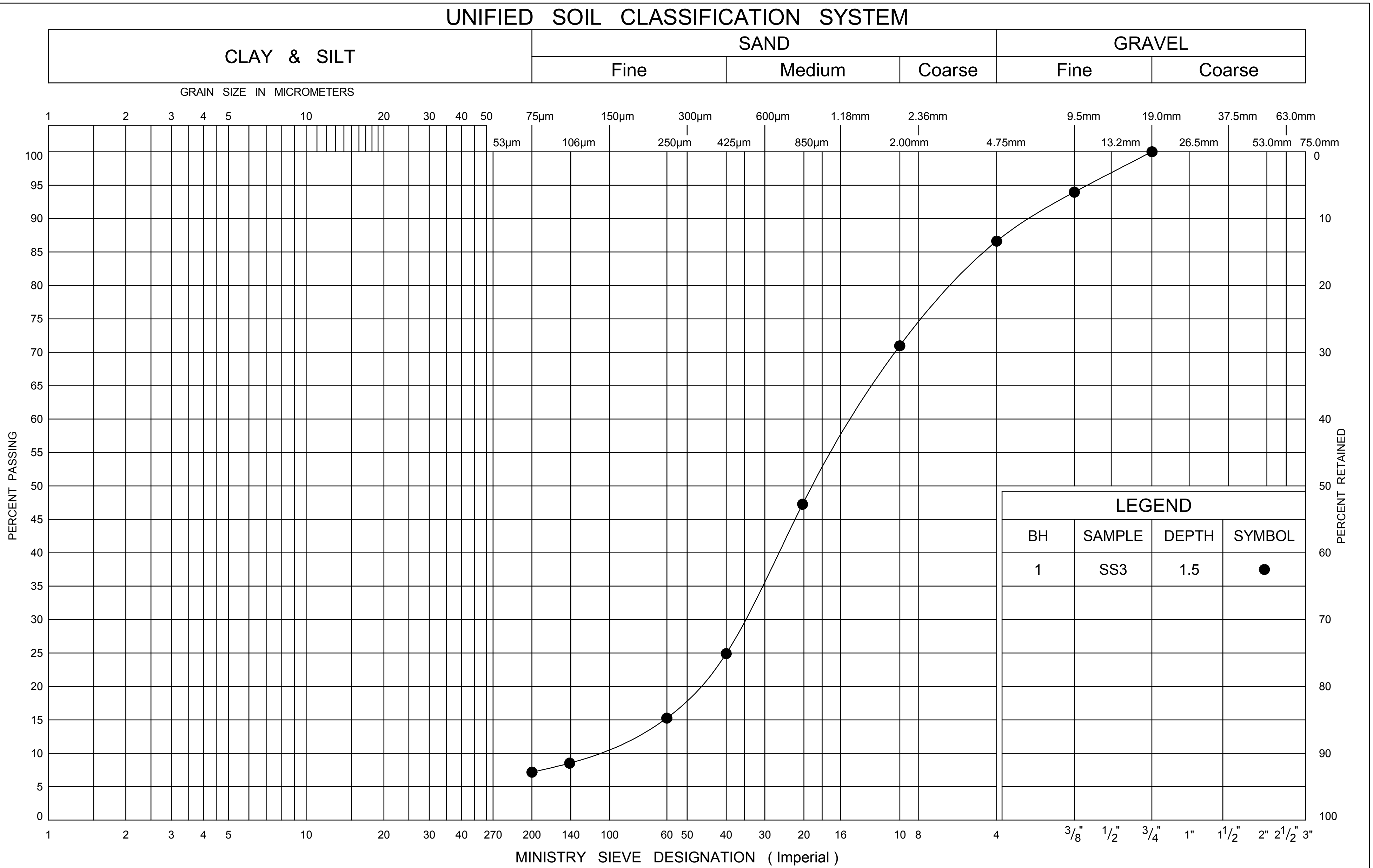
Terraprobe



BH	SAMPLE	DEPTH	SYMBOL
4	SS1	0.2	●

Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION FILL-GRAVELLY SAND	FIG No B1 G W P 417-91-00 Driftwood River Bridge Site 39E-096

Terraprobe

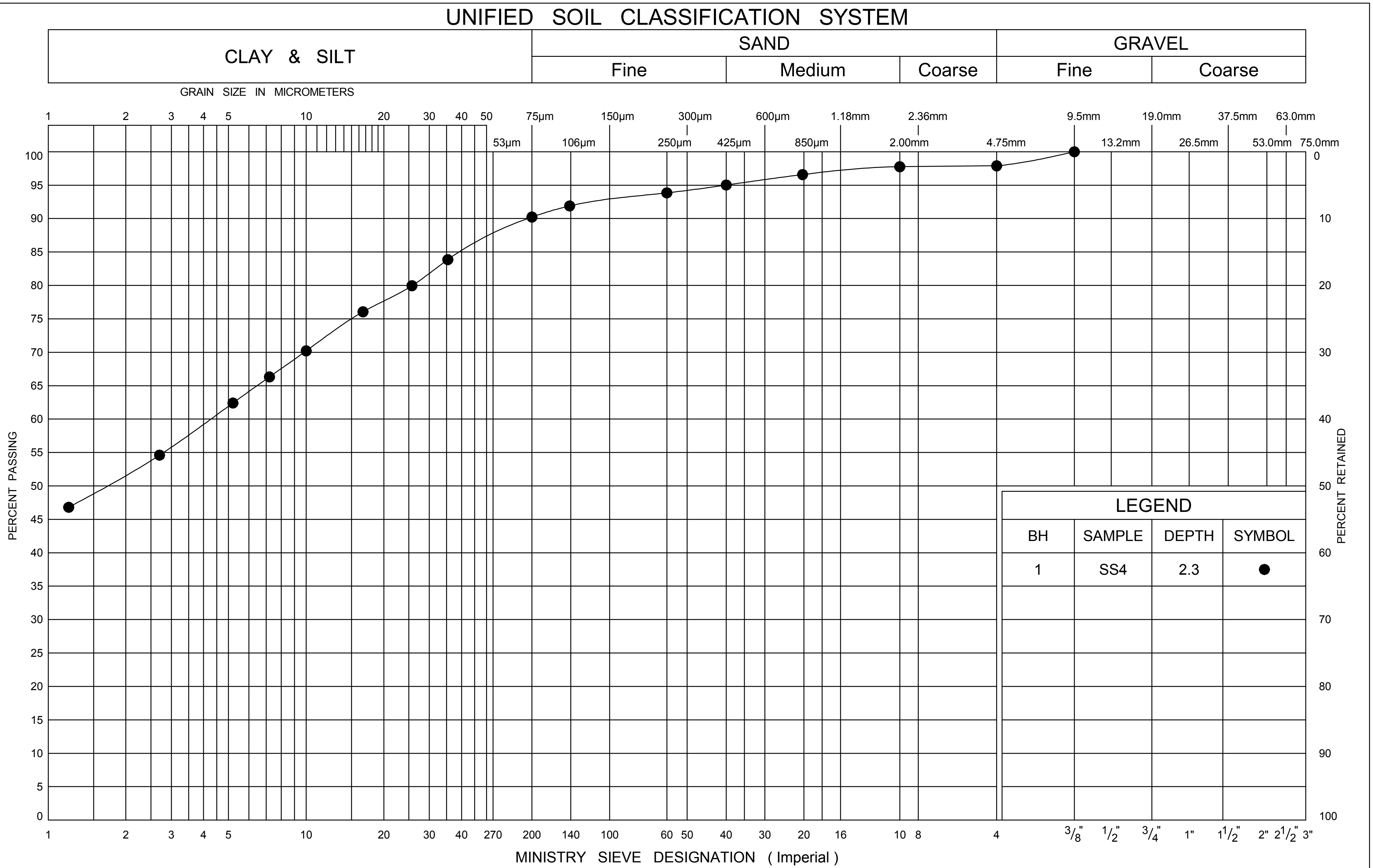


Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
FILL-SAND

FIG No B2	
G W P 417-91-00	
Driftwood River Bridge Site 39E-096	

Terraprobe



BH	SAMPLE	DEPTH	SYMBOL
1	SS4	2.3	●

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Ontario

GRAIN SIZE DISTRIBUTION

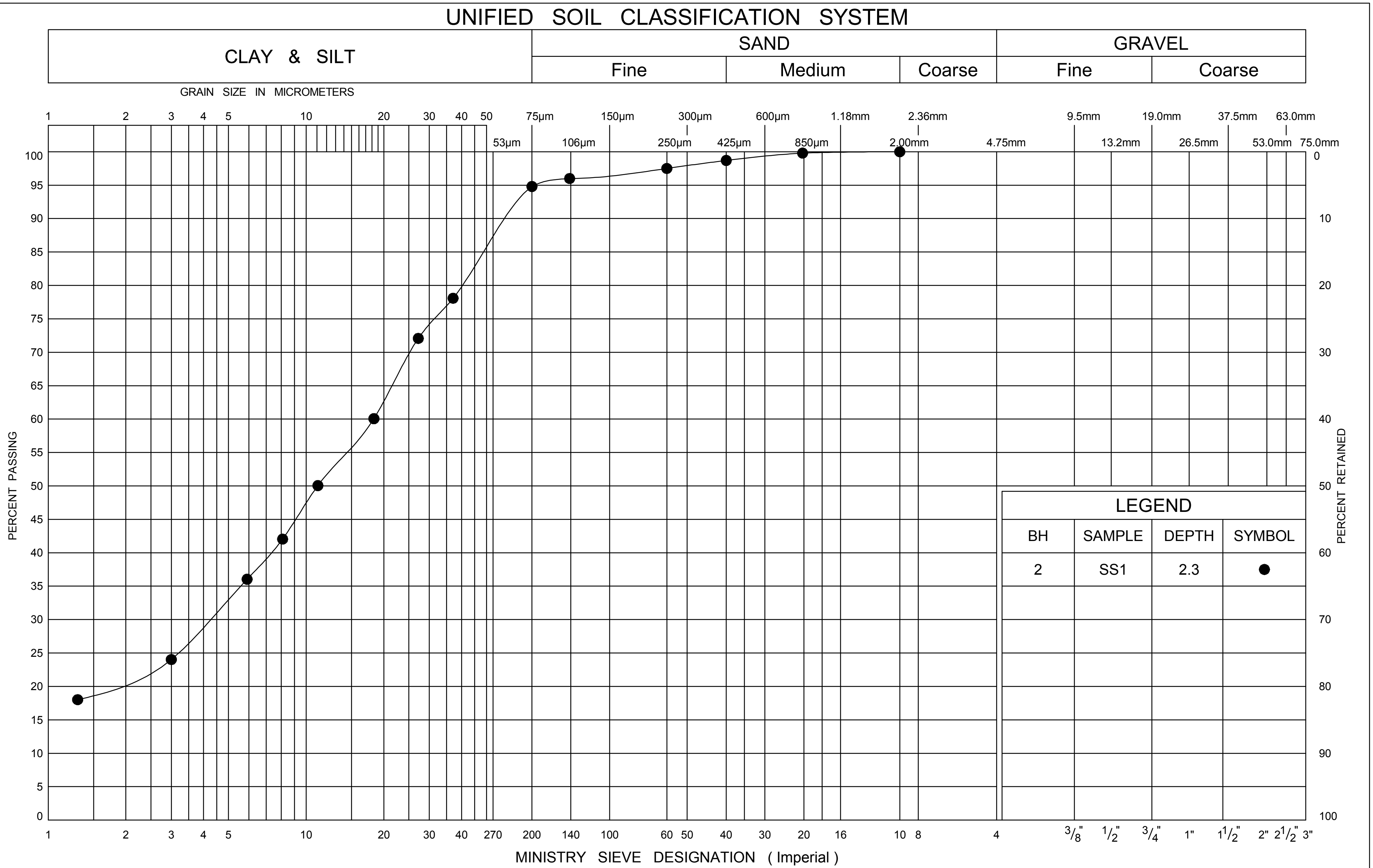
FILL-SILTY CLAY

FIG No B3

G W P 417-91-00

Driftwood River Bridge Site 39E-096

Terraprobe



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Ontario

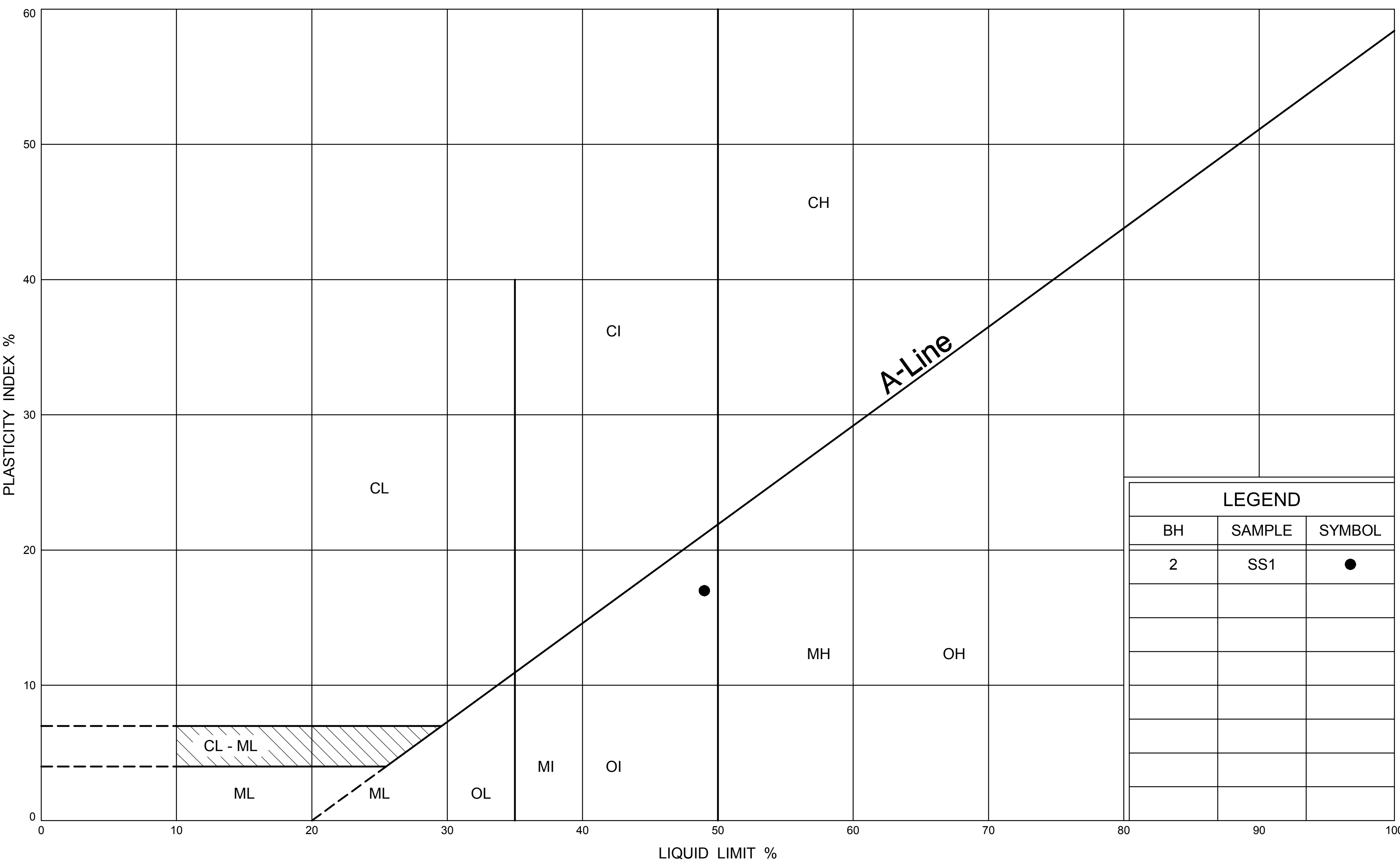
GRAIN SIZE DISTRIBUTION
ORGANIC SILT

FIG No B4

G W P 417-91-00

Driftwood River Bridge Site 39E-096

Terraprobe



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PLASTICITY CHART

ORGANIC SILT

FIG No B5

G W P 417-91-00

Driftwood River Bridge Site 39E-096

VARVED SILTY CLAY TO CLAY SAMPLES	FIGURE B6

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Silty Clay to Clay

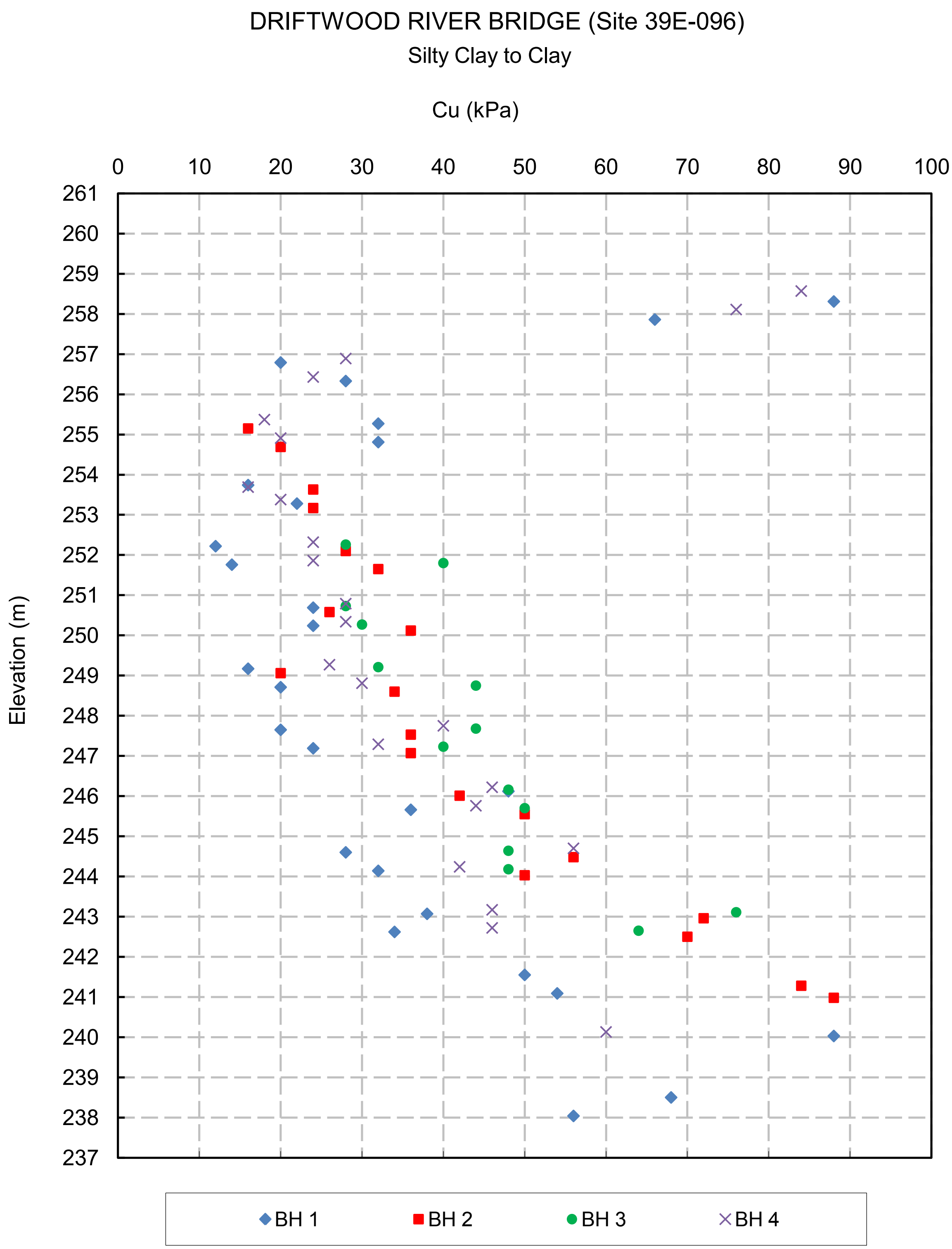
BH1 SS18

BH2 SS13

Project No. :	11-14-4066	Terraprobe Inc.	Prepared by :	SD
Date :	February, 2015		Checked by :	RA

Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\6- Driftwood River Bridge Hwy 577 (39E-096)\Eng. Analysis\Spread Sheets\e0-P'c-Cc-Cr-Cu.xls

UNDRAINED SHEAR STRENGTH	FIGURE B7



Project No. : 11-14-4066

Date : February, 2015

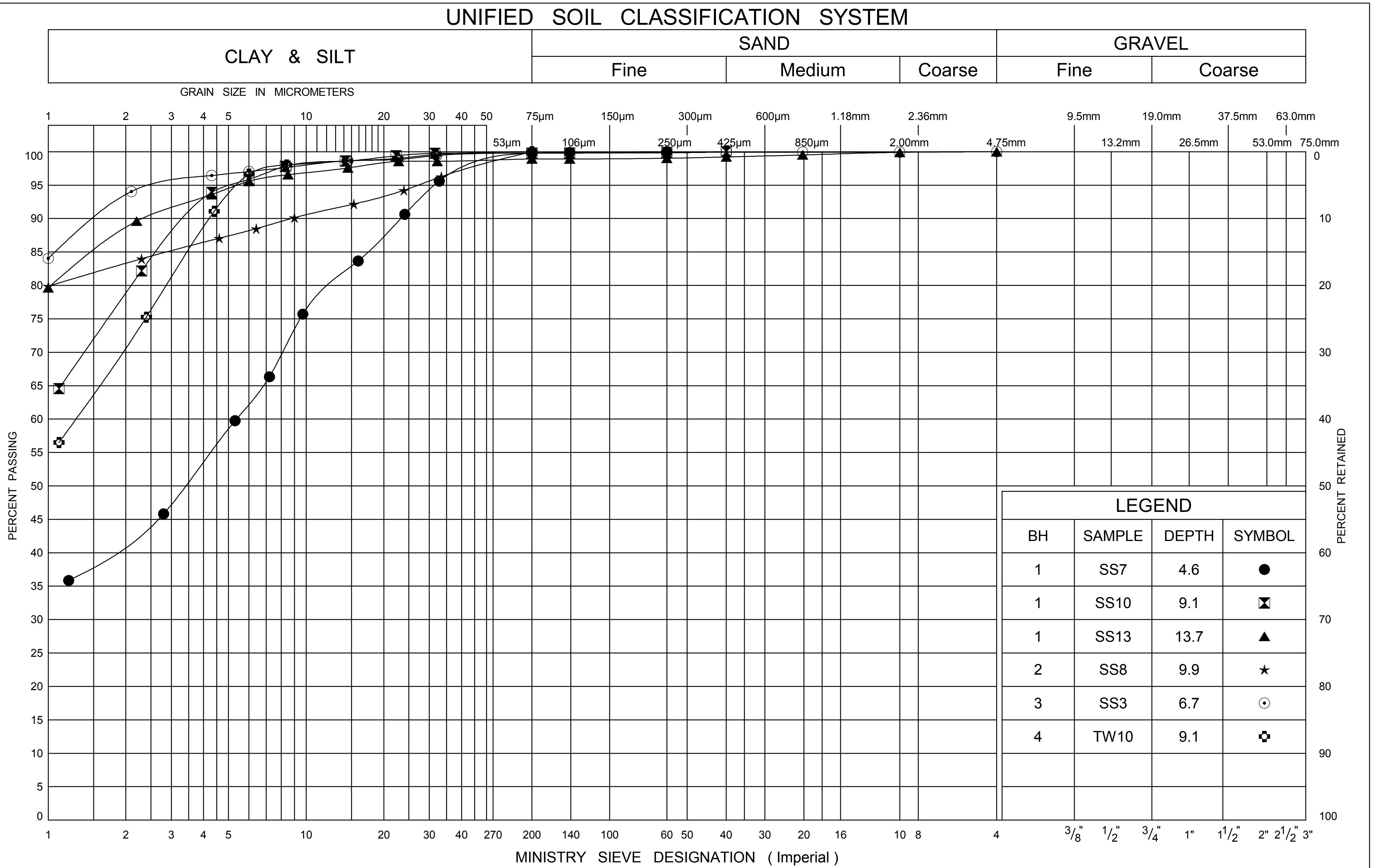
Terraprobe Inc.

Prepared by : SD

Checked by : RA

Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\6- Driftwood River Bridge Hwy 577 (39E-096)\Eng. Analysis\Spread Sheets\e0-P'c-Cc-Cr-Cu.xls

Terraprobe



LEGEND			
BH	SAMPLE	DEPTH	SYMBOL
1	SS7	4.6	●
1	SS10	9.1	⧗
1	SS13	13.7	▲
2	SS8	9.9	★
3	SS3	6.7	⊙
4	TW10	9.1	✿

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Ontario

GRAIN SIZE DISTRIBUTION

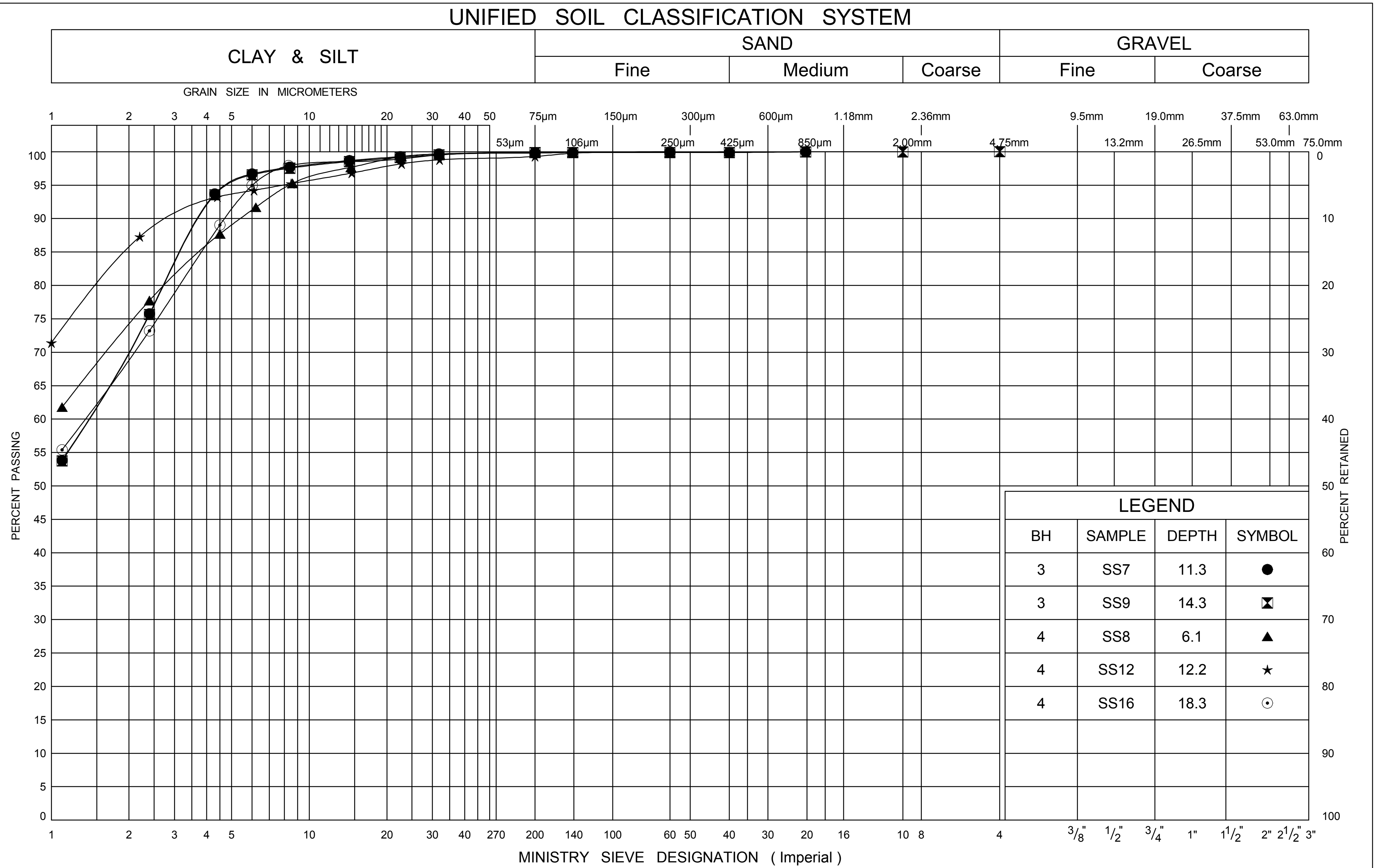
SILTY CLAY TO CLAY

FIG No B8

G W P 417-91-00

Driftwood River Bridge Site 39E-096

Terraprobe



Ministry of Transportation
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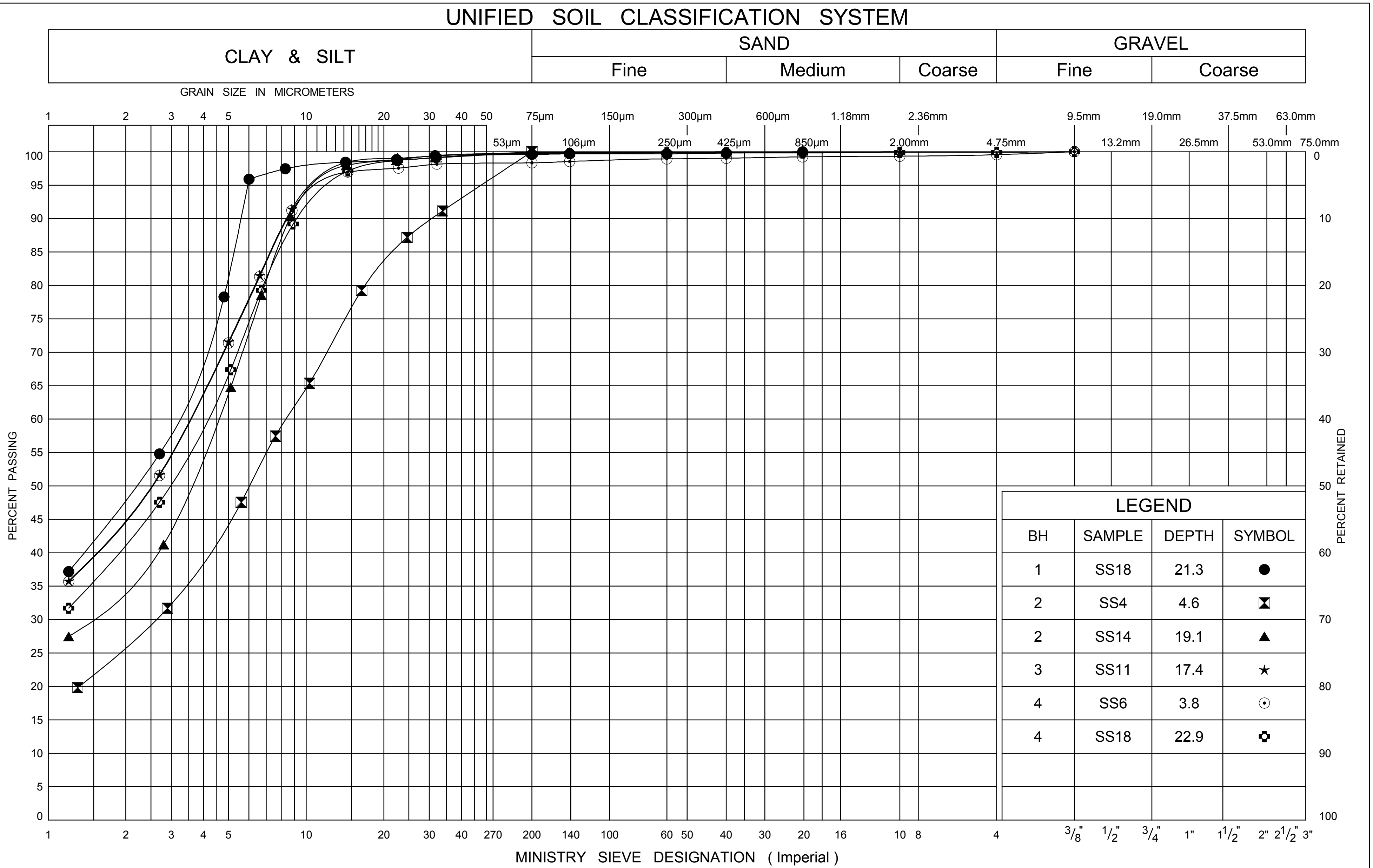
GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

FIG No	B9
G W P	417-91-00
Driftwood River Bridge Site 39E-096	

Terraprobe



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	✿

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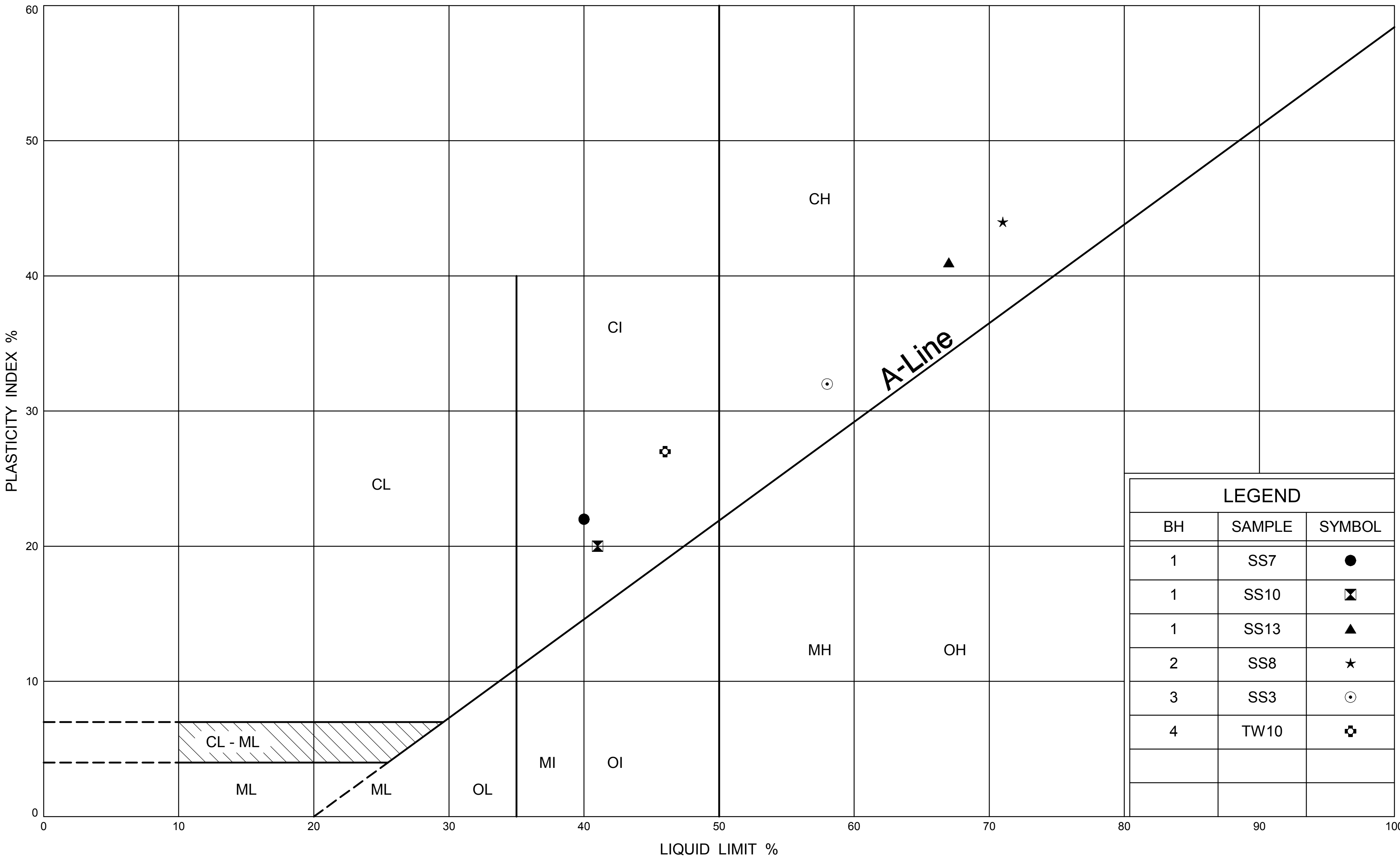
GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

FIG No B10

G W P 417-91-00

Driftwood River Bridge Site 39E-096



LEGEND

BH	SAMPLE	SYMBOL
1	SS7	●
1	SS10	⧗
1	SS13	▲
2	SS8	★
3	SS3	☉
4	TW10	✿

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Ontario

PLASTICITY CHART

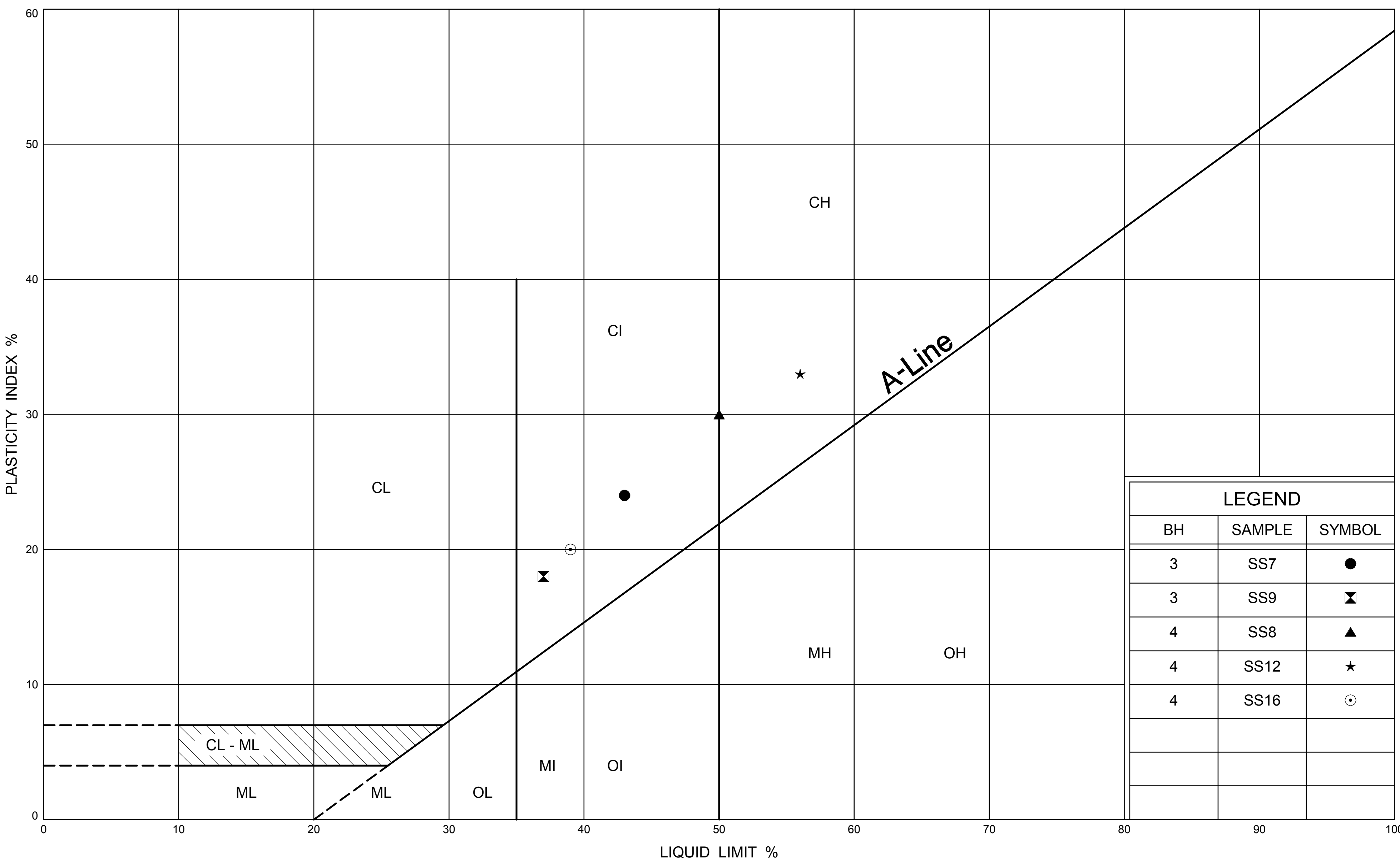
SILTY CLAY TO CLAY

FIG No B11

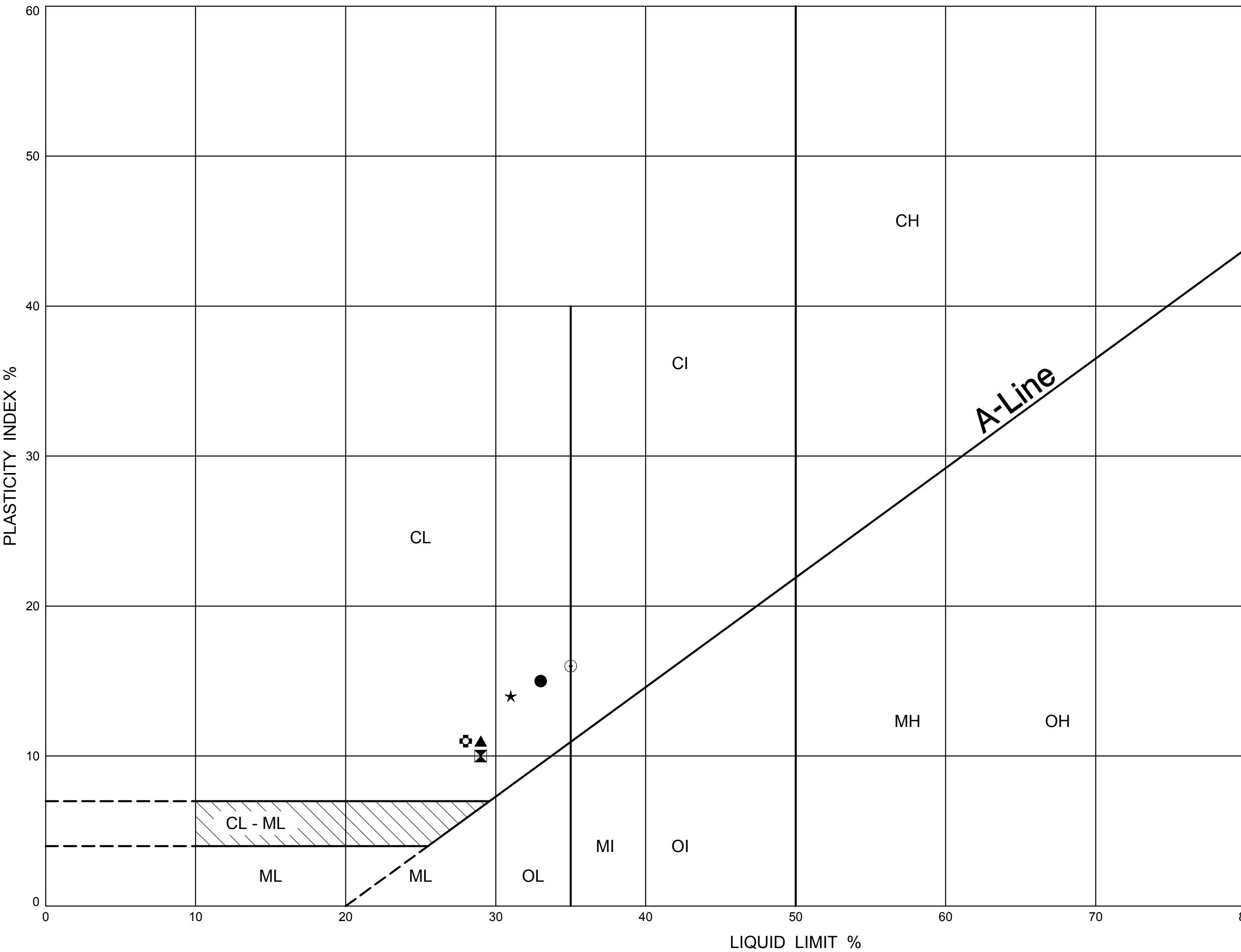
G W P 417-91-00

Driftwood River Bridge Site 39E-096

Terraprobe



Ministry of Transportation Ontario	PLASTICITY CHART SILTY CLAY TO CLAY	FIG No B12
		G W P 417-91-00
		Driftwood River Bridge Site 39E-096



LEGEND

BH	SAMPLE	SYMBOL
1	SS18	●
2	SS4	⧗
2	SS14	▲
3	SS11	★
4	SS6	☉
4	SS18	✿

Ministry of Transportation
Ontario

PLASTICITY CHART

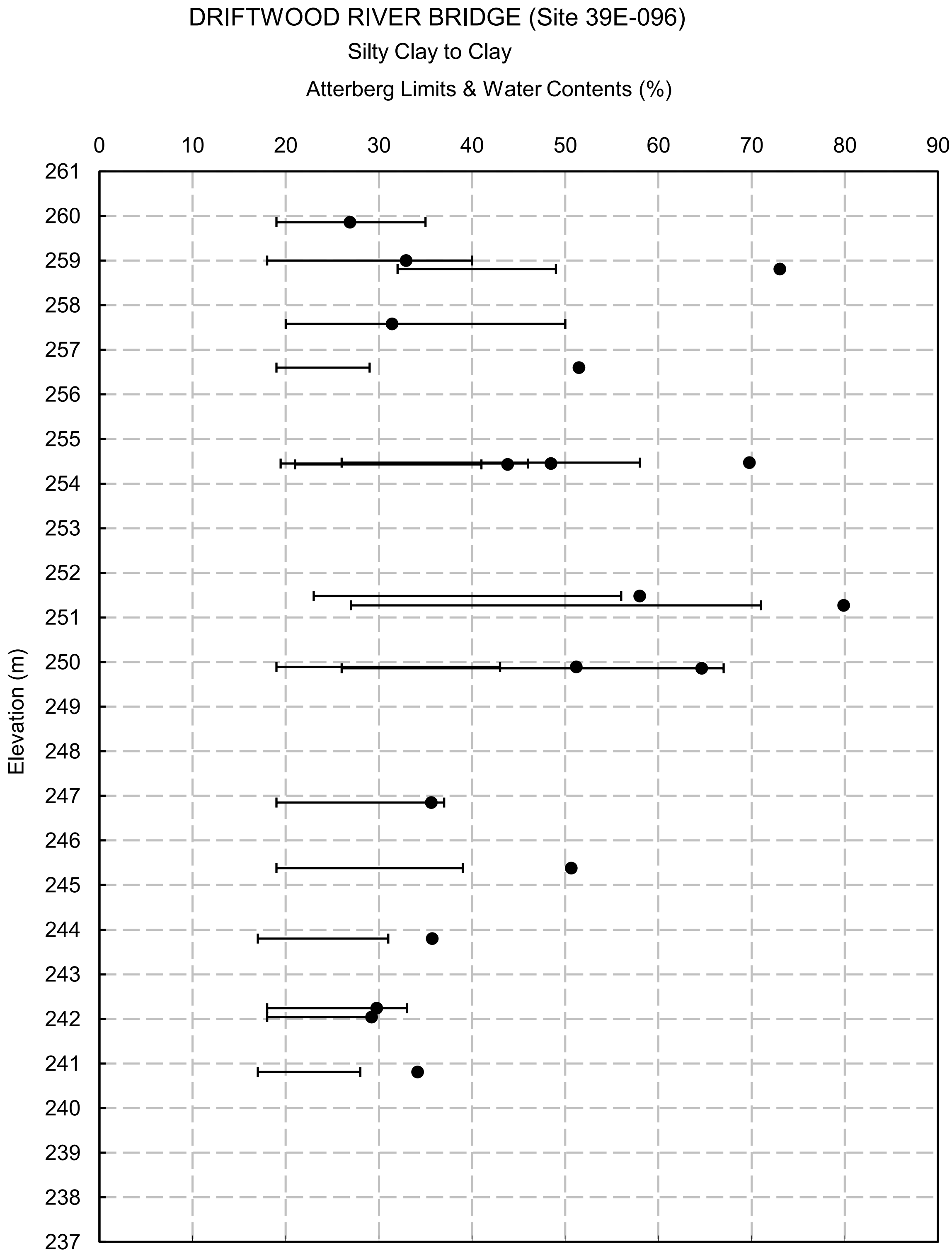
SILTY CLAY TO CLAY

FIG No B13

G W P 417-91-00

Driftwood River Bridge Site 39E-096

ATTERBERG LIMITS AND WATER CONTENTS	FIGURE B14



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Project No. :	11-14-4066	Terraprobe Inc.	Prepared by :	SD
Date :	February, 2015		Checked by :	RA

CONSOLIDATION TEST SUMMARY	FIGURE B15

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

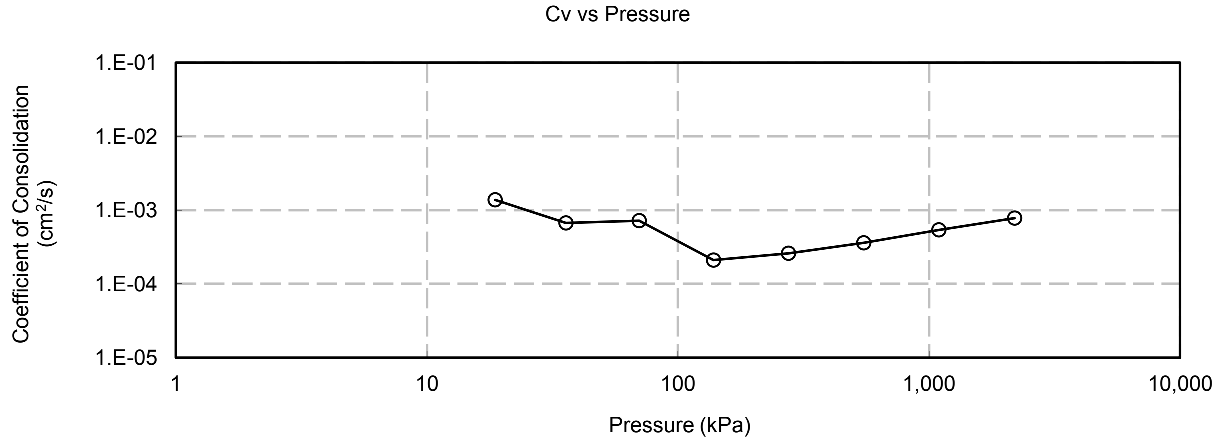
Sample Height (mm) :	13.91	Unit Weight (kN/m^3) :	19.74
Sample Diameter (mm) :	63.44	Dry Unit Weight (kN/m^3) :	15.29
Area (cm^2) :	31.61	Specific Gravity :	2.73
Volume (cm^3) :	43.97	Solid Height (mm) :	8.09
Water Content (%) :	29.14	Volume of Solids (cm^3) :	25.15
Wet Mass (g) :	88.51	Volume of Voids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

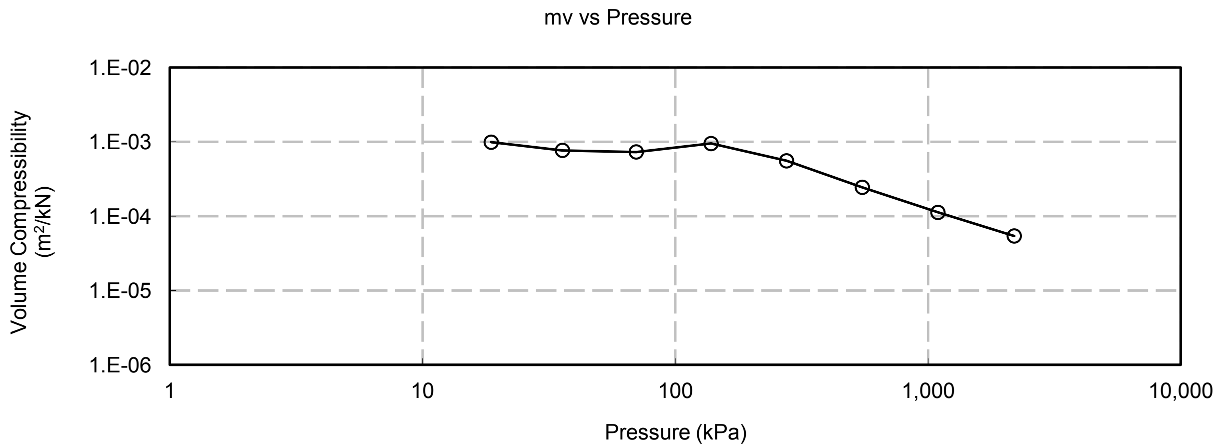
Project No. :	11-14-4066	Terraprobe Inc.	Prepared By :	SD
Date :	February 2015		Checked By :	RA

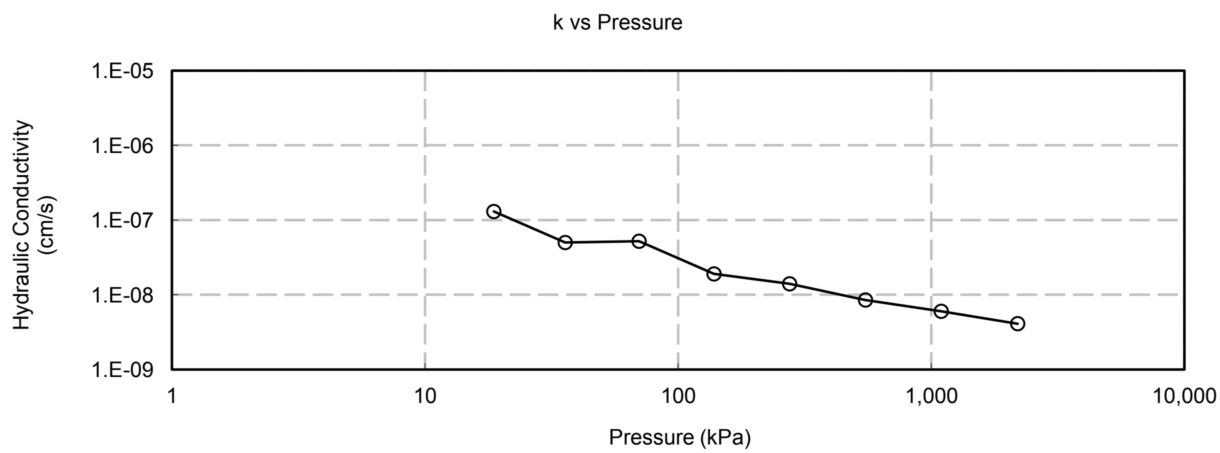
CONSOLIDATION TEST	FIGURE B16

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

BH 4, TW 10







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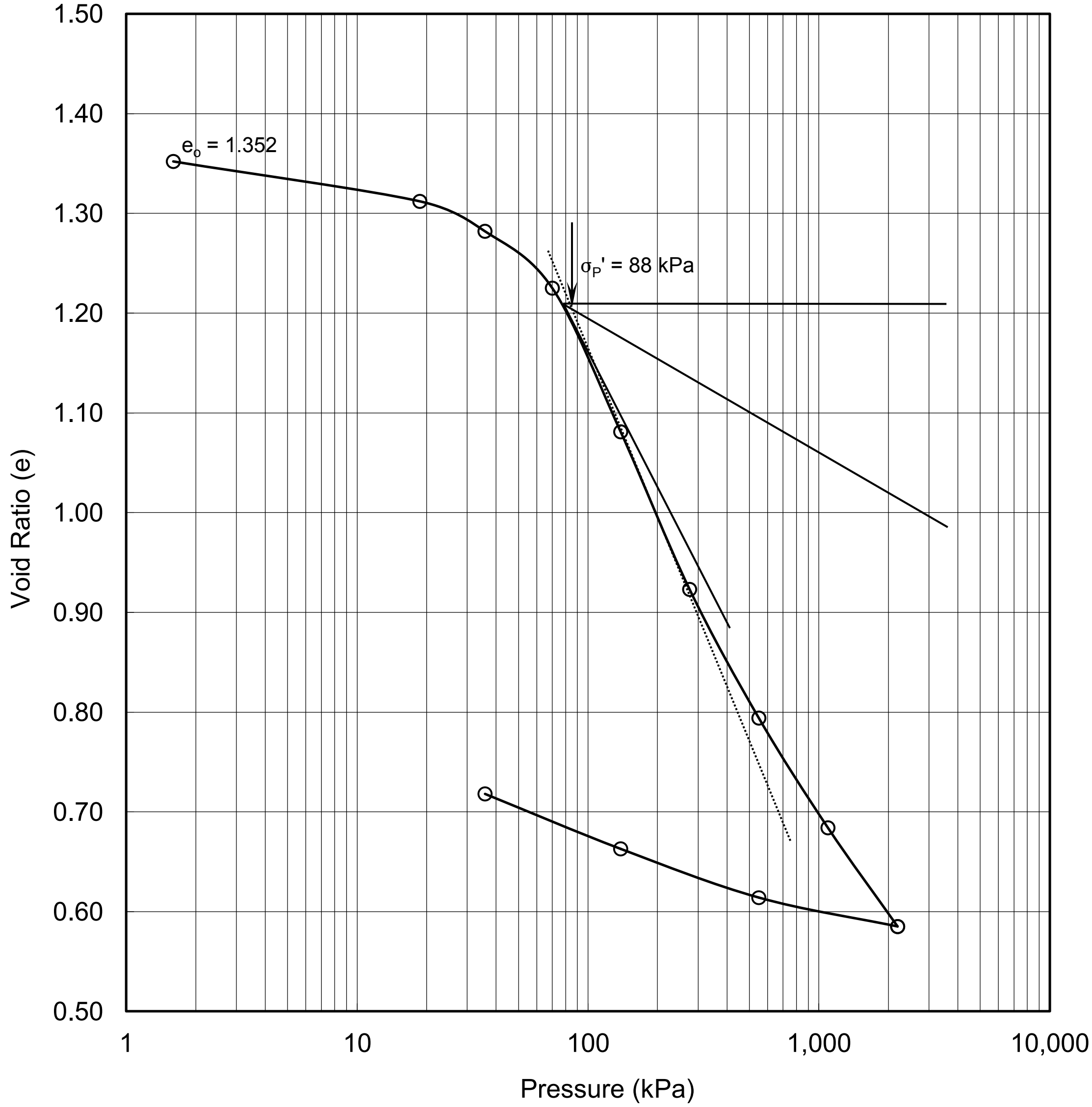
Project No. :	11-14-4066	Terraprobe Inc.	Prepared By :	SD
Date :	February 2015		Checked By :	RA

CONSOLIDATION TEST	FIGURE B17

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}' =	89.4 kPa
ω =	48%	ω_P =	19%	σ_P' =	88.0 kPa
γ =	17.0 kN/m^3	PI =	27%		
Gs =	2.73				

Project No. : 11-14-4066
Date : February 2015

Terraprobe Inc.

Prepared By : SD
Checked By : RA

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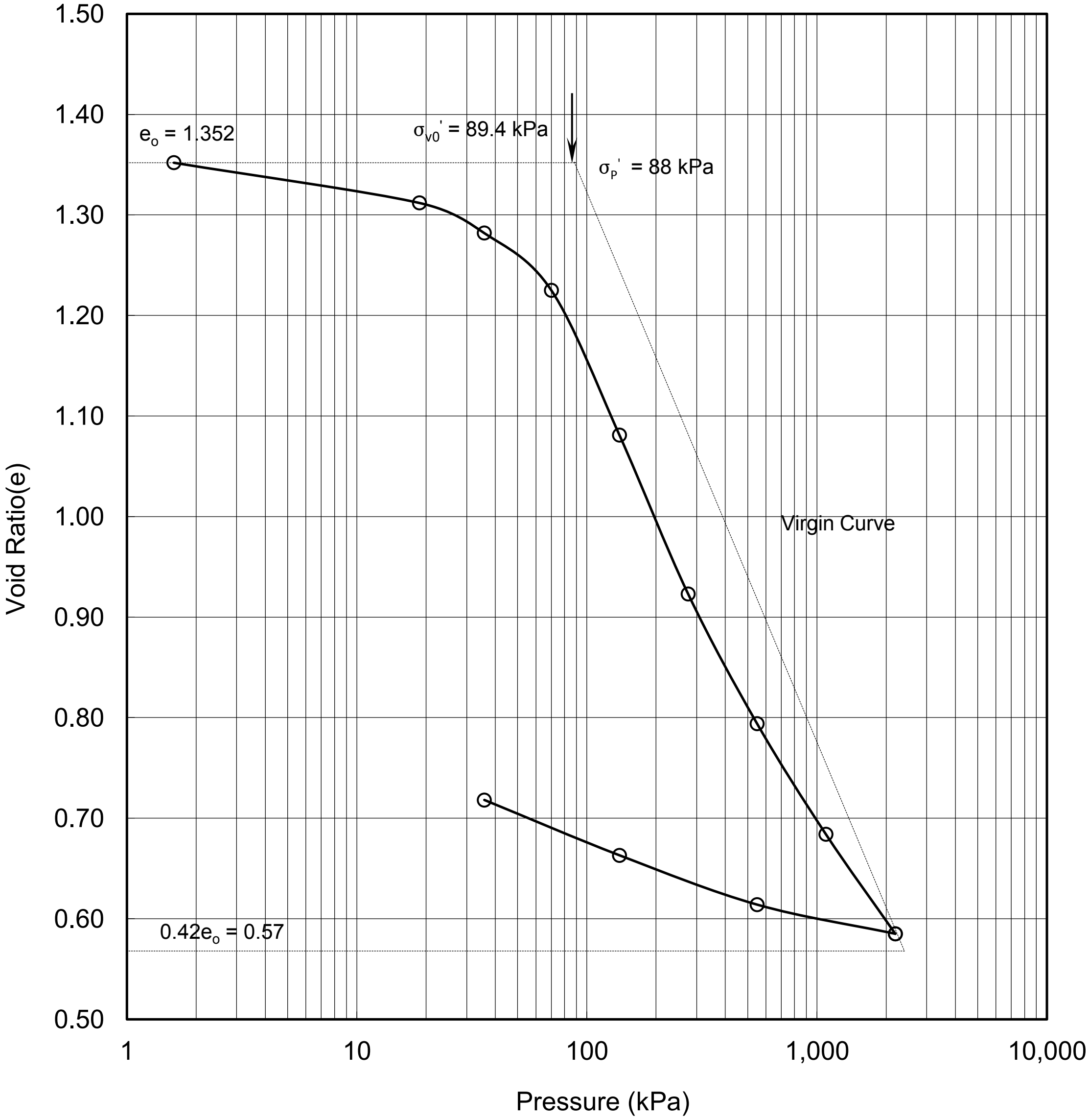
CONSOLIDATION TEST	FIGURE B18

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}' =	89.4 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

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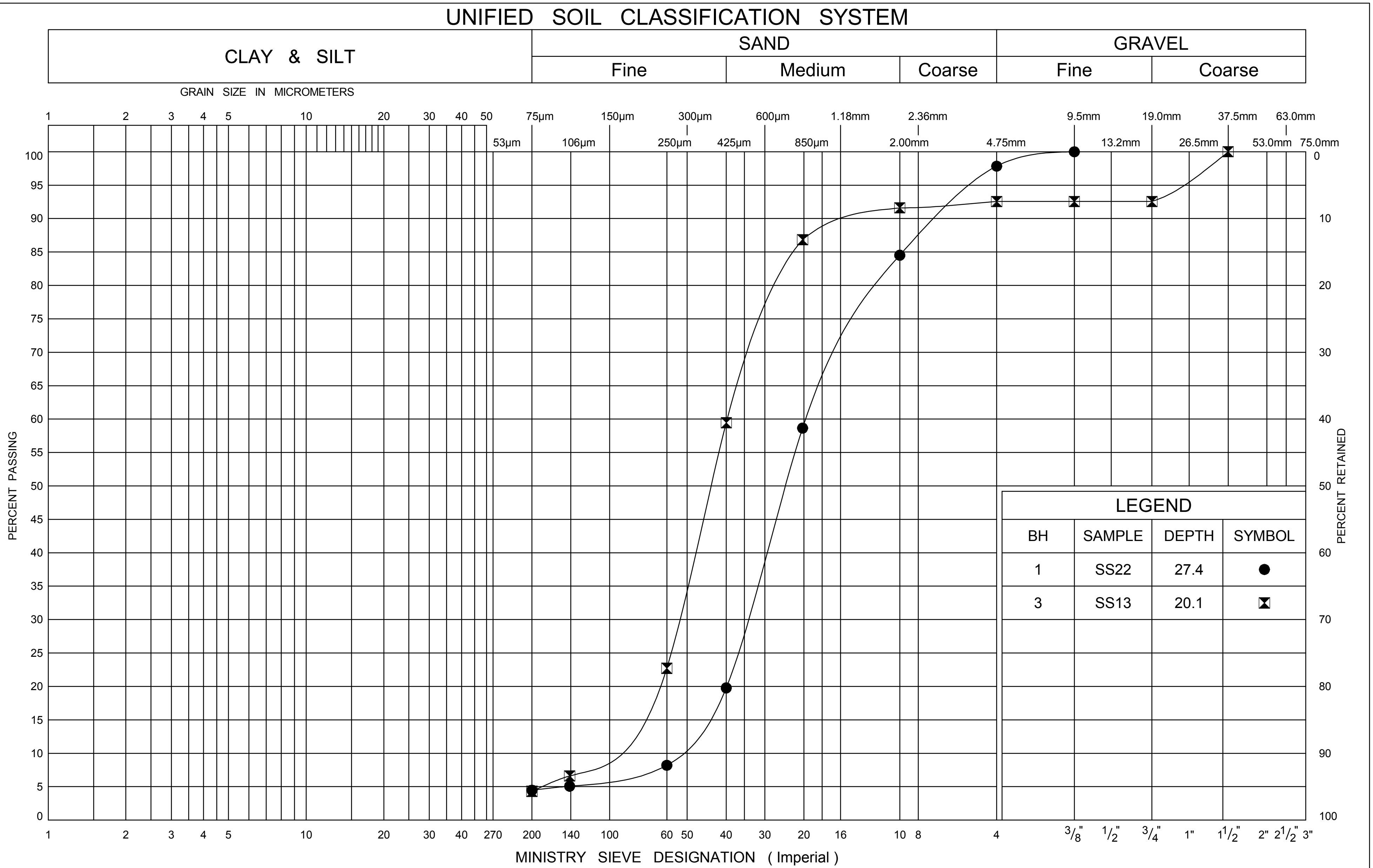
Date :	February 2015		Checked By :	RA



Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

SAND

FIG No B19

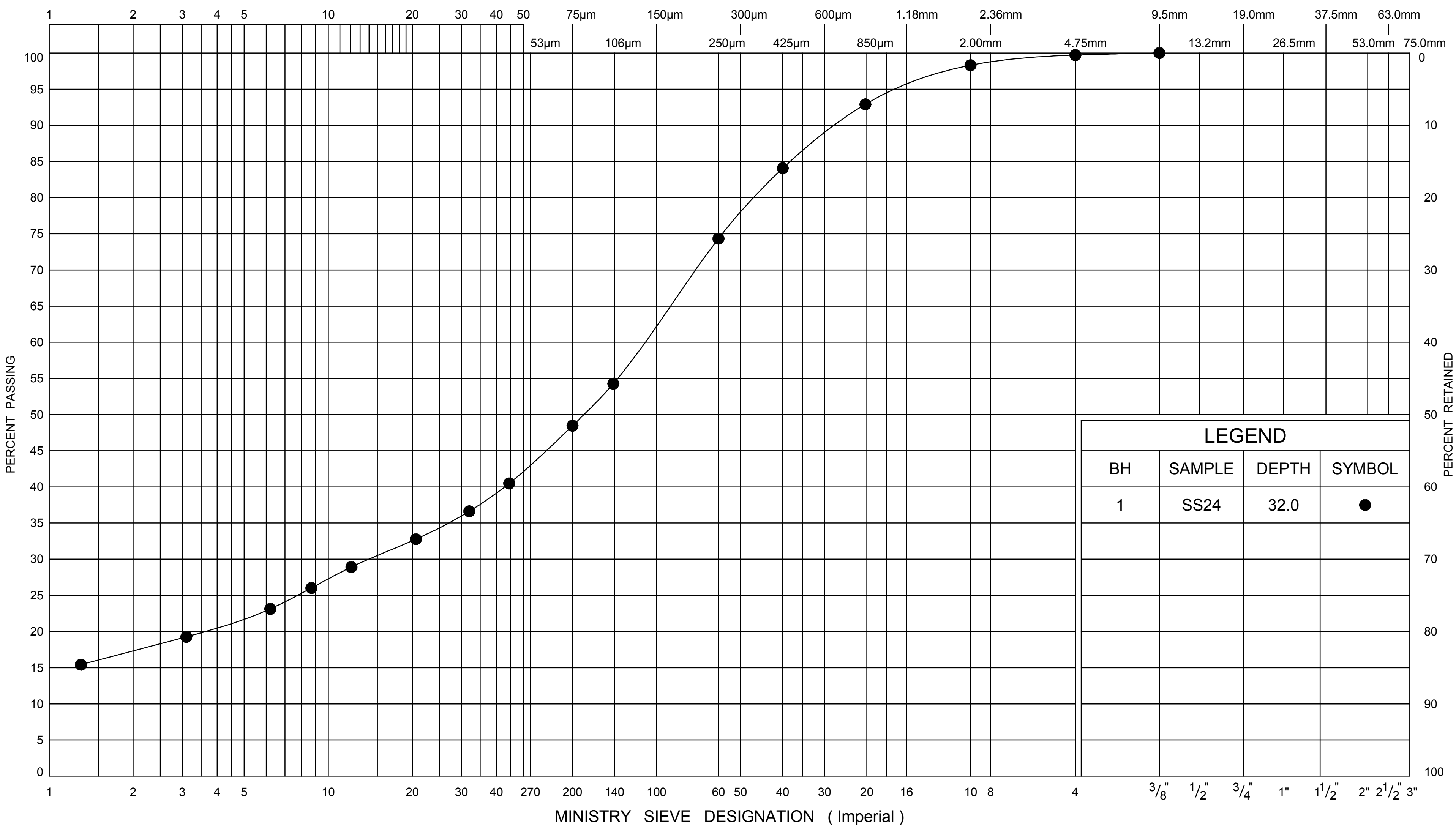
G W P 417-91-00

Driftwood River Bridge Site 39E-096

Terraprobe

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
1	SS24	32.0	●

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILTY SAND TILL

FIG No B20

G W P 417-91-00

Driftwood River Bridge Site 39E-096

PHOTOGRAPH OF COBBLES AND BOULDERS	FIGURE B21

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\6- Driftwood River Bridge Hwy 577 (39E-096)\Eng. Analysis\Spread Sheets\e0-P'c-Cc-Cr-Cu.xls

Project No. :	11-14-4066	Terraprobe Inc.	Prepared by :	SD
Date :	February, 2015		Checked by :	RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B22

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

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Project No. :	11-14-4066	Terraprobe Inc.	Prepared by :	SD
Date :	February, 2015		Checked by :	RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B23

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

Project No. :	11-14-4066	Terraprobe Inc.	Prepared by :	SD
Date :	February, 2015		Checked by :	RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES	FIGURE B24

DRIFTWOOD RIVER BRIDGE (Site 39E-096)

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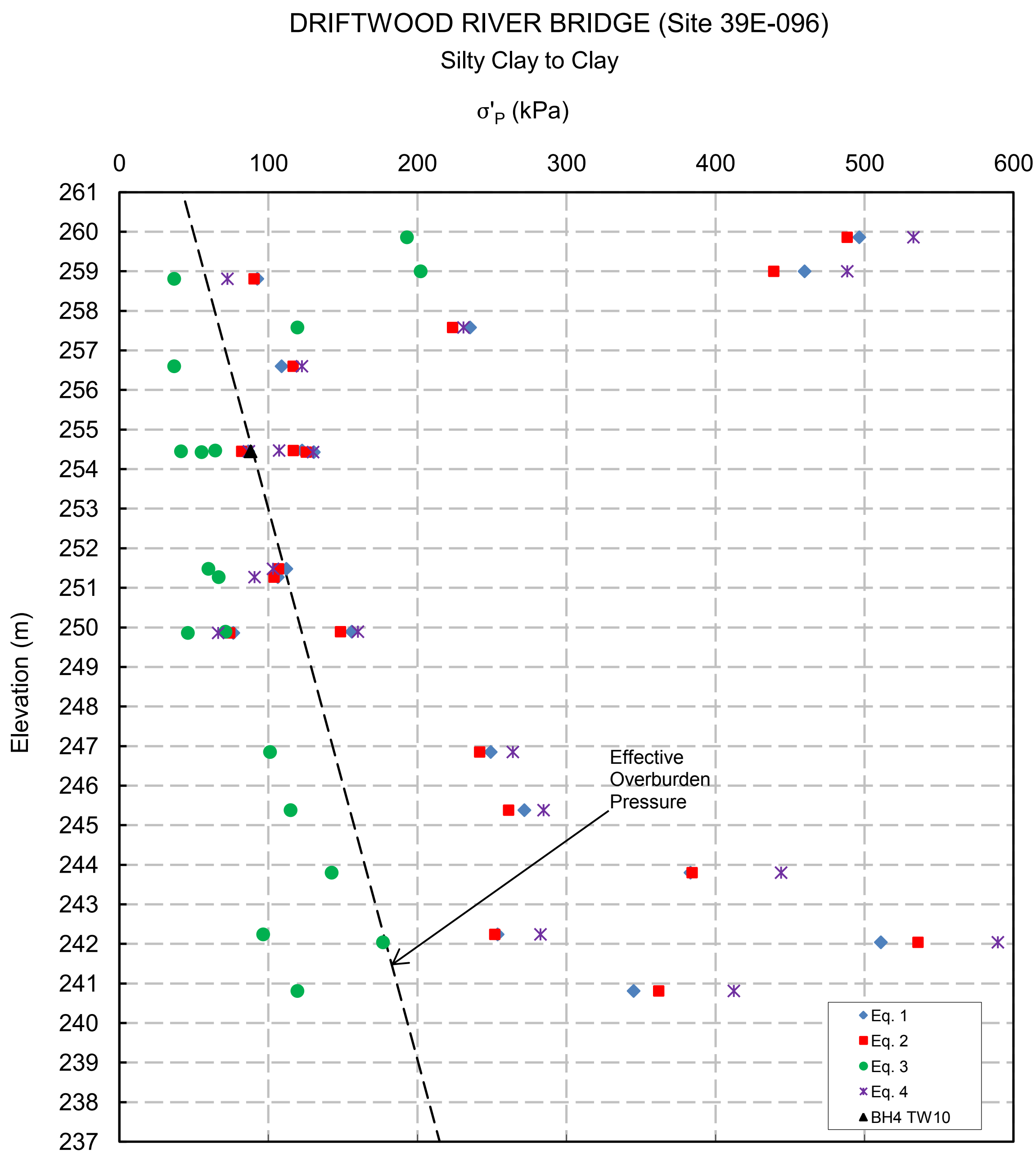
Project No. :	11-14-4066	Terraprobe Inc.	Prepared by :	SD
Date :	February, 2015		Checked by :	RA

APPENDIX C

Soil Design Parameters

PREDICTED AND MEASURED PRECONSOLIDATION STRESSES	FIGURE C1



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)

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Checked by : RA

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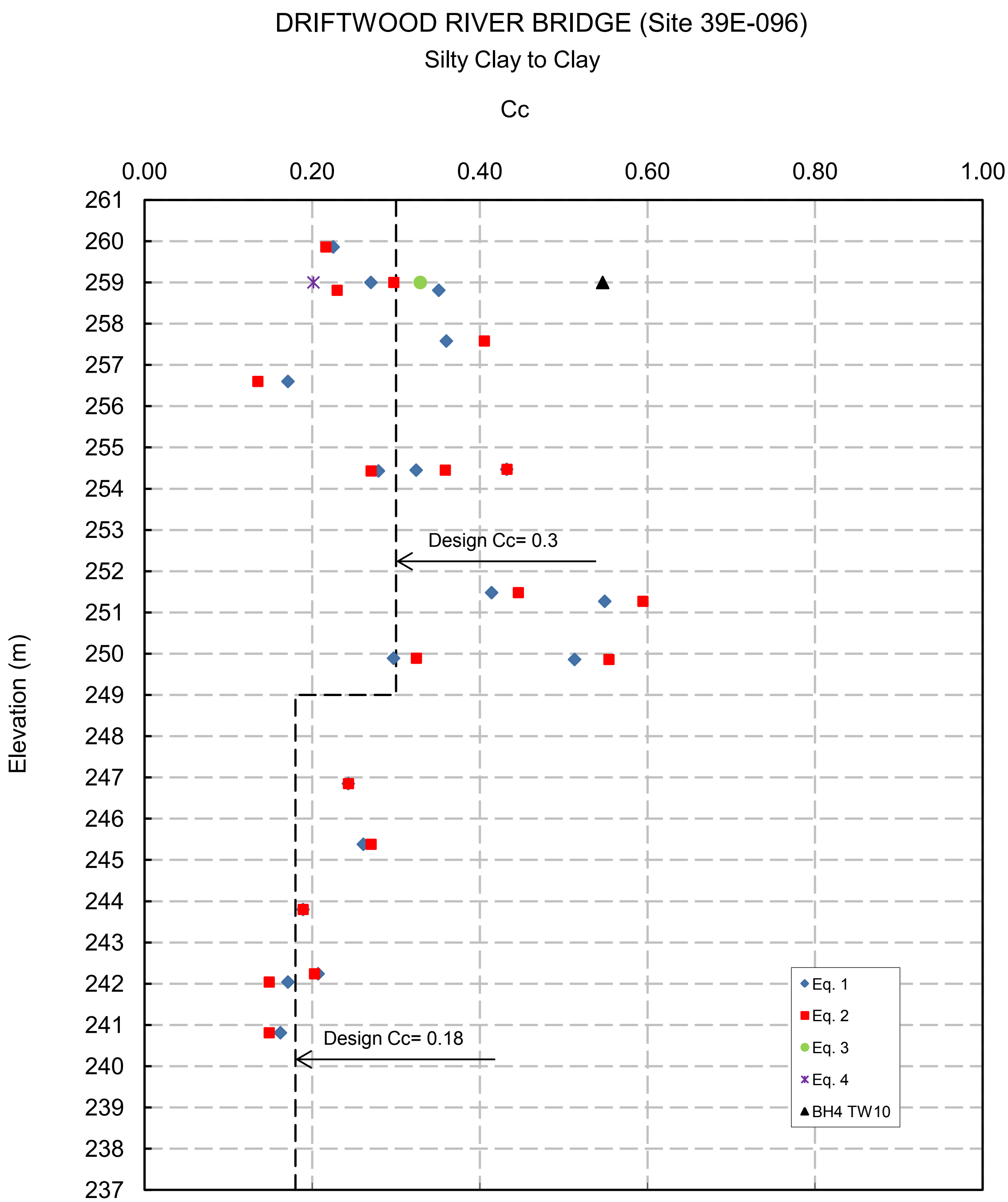
PREDICTED AND MEASURED COMPRESSION INDICES	FIGURE C2



Eq. 1 Cc = 0.009 * (LL - 10) Terzaghi & Peck (1967)

Eq. 2 $Cc = I_p / 74$ Kulhaway & Mayne (1990)

Eq. 3 Cc = 0.141 * Gs1.2 * ((1 + eo) / Gs)2.38 Rendon - Herrero (1983)

Eq. 4 $Cc = 0.141 * Gs * (\gamma_w / \gamma_d)^{2.4}$ Herrero (1983)

Project No. : 11-14-4066

Date : January, 2015

Terraprobe Inc.

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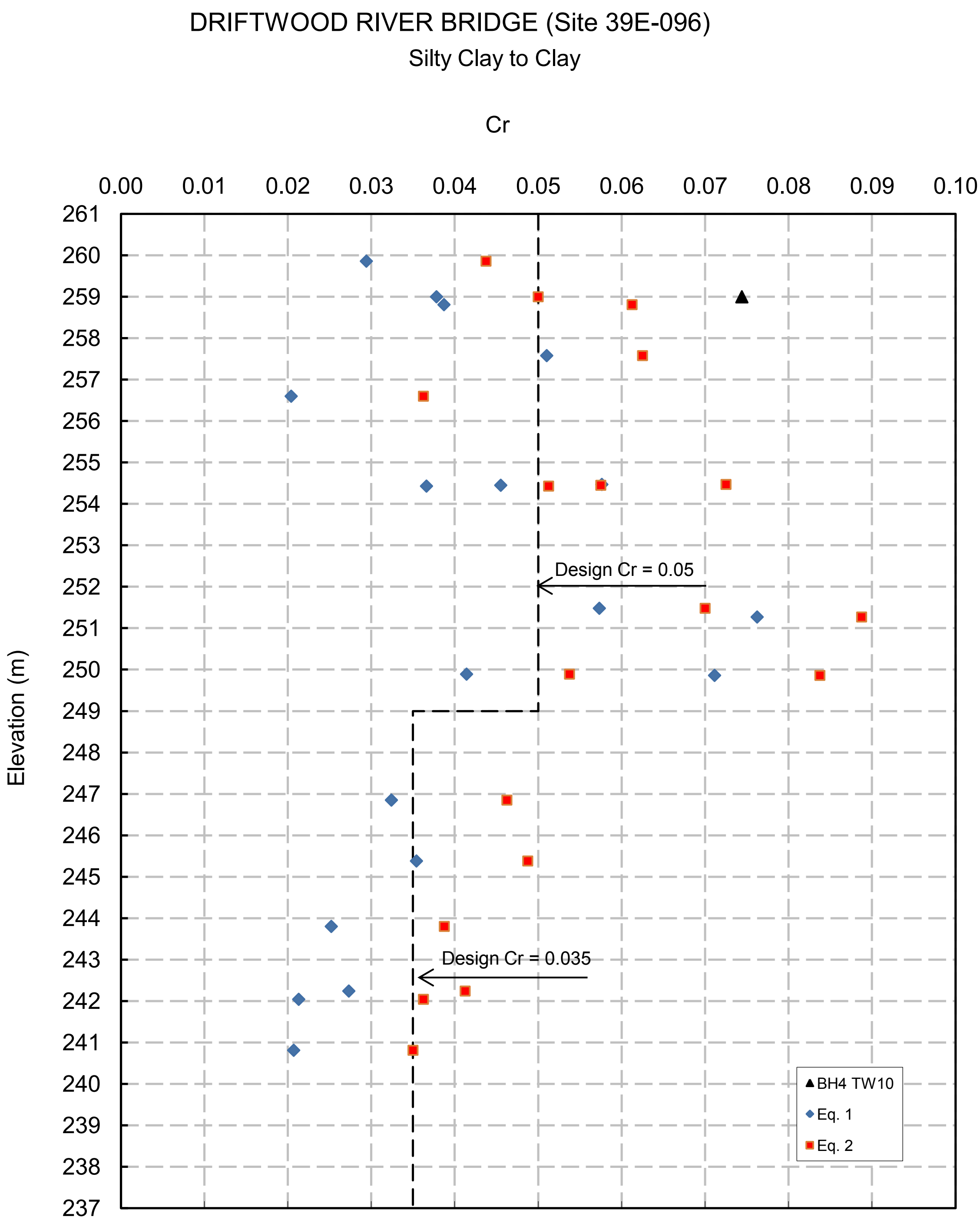
Checked by : RA

Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\6- Driftwood River Bridge Hwy 577 (39E-096)\Eng. Analysis\Spread Sheets\Consolidation Results.xls

PREDICTED AND MEASURED RECOMPRESSION INDICES	FIGURE C3



Eq. 1 Cr = Cc / 5 ~ Cc / 10 Das (1993)

Eq. 2 Cr = 0.000463*LL*Gs Nagaraj & Murty (1985)

Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\6- Driftwood River Bridge Hwy 577 (39E-096)\Eng. Analysis\Spread Sheets\Consolidation Results.xls

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Date :	January, 2015	Checked by :	RA

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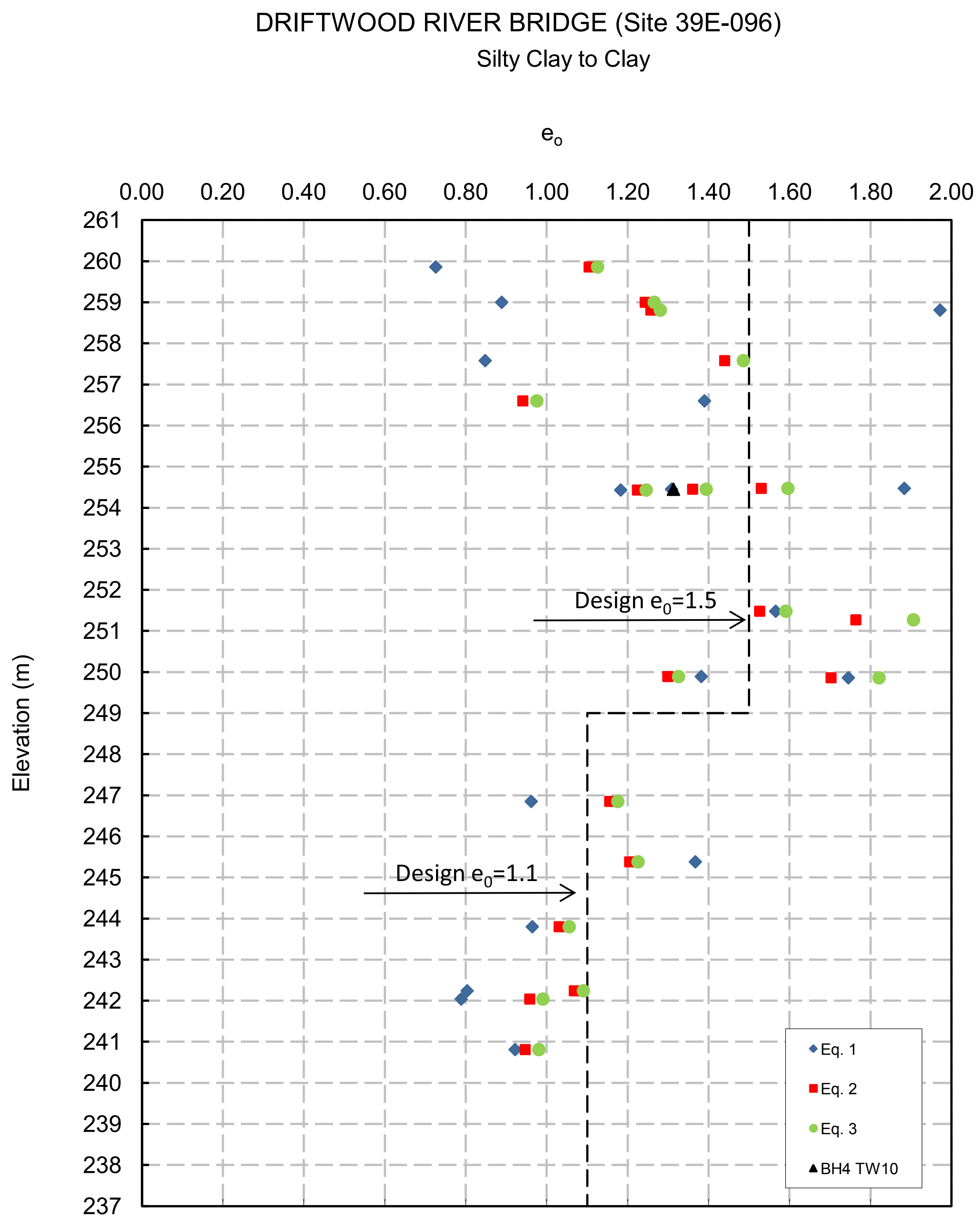
PREDICTED AND MEASURED VOID RATIOS	FIGURE C4



Eq. 1 eo = 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)

Project No. : 11-14-4066

Date : January, 2015

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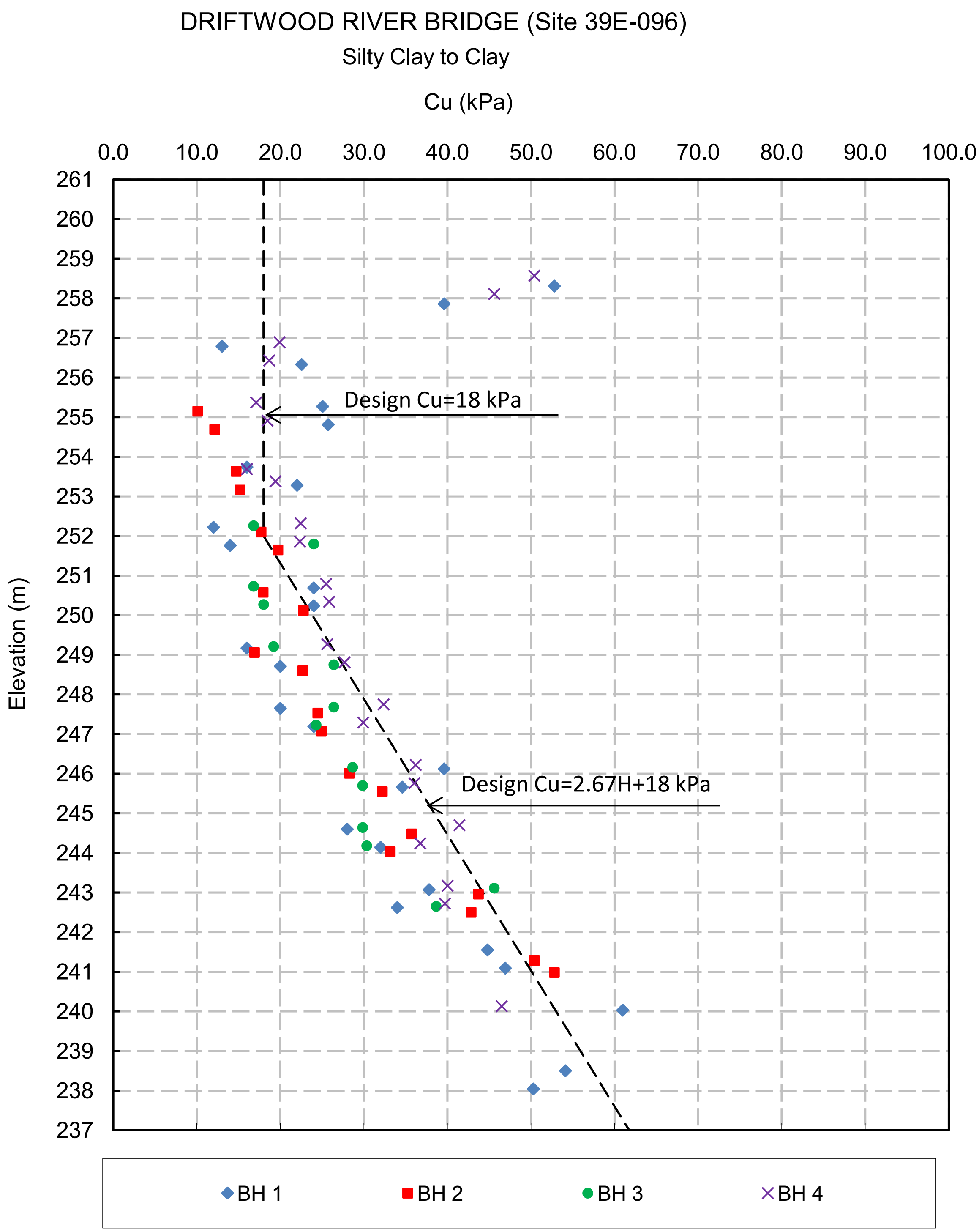
UNDRAINED SHEAR STRENGTH	FIGURE C5



H = Clay layer thickness below elevation 252m for the corresponding Design Cu.

Field vane shear strengthes were corrected based on Aas, et al. (1986)

Project No. : 11-14-4066

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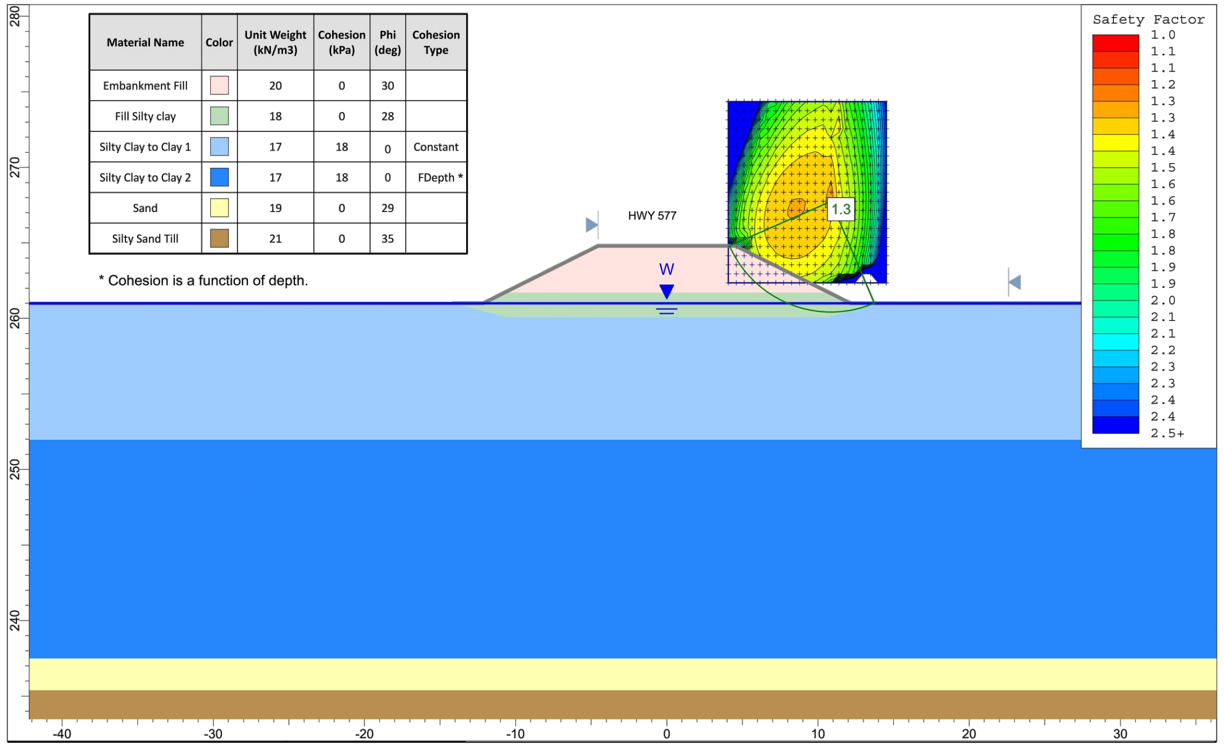
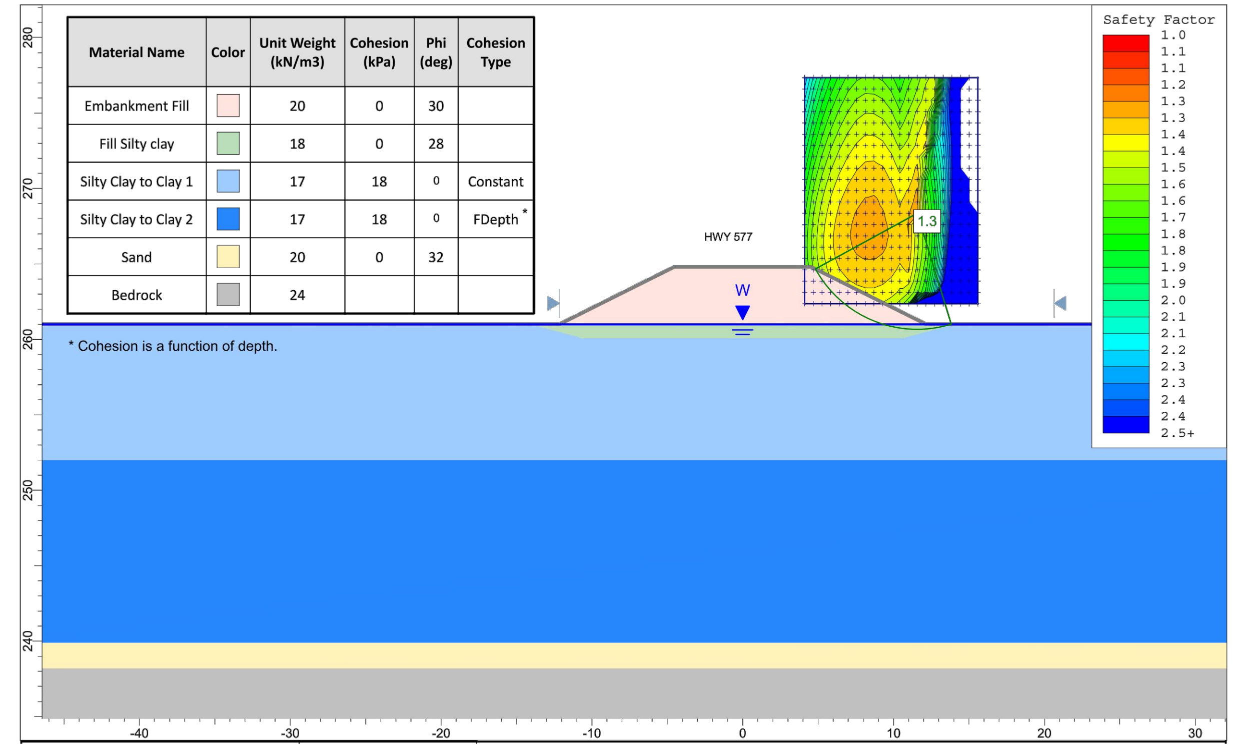
APPENDIX D

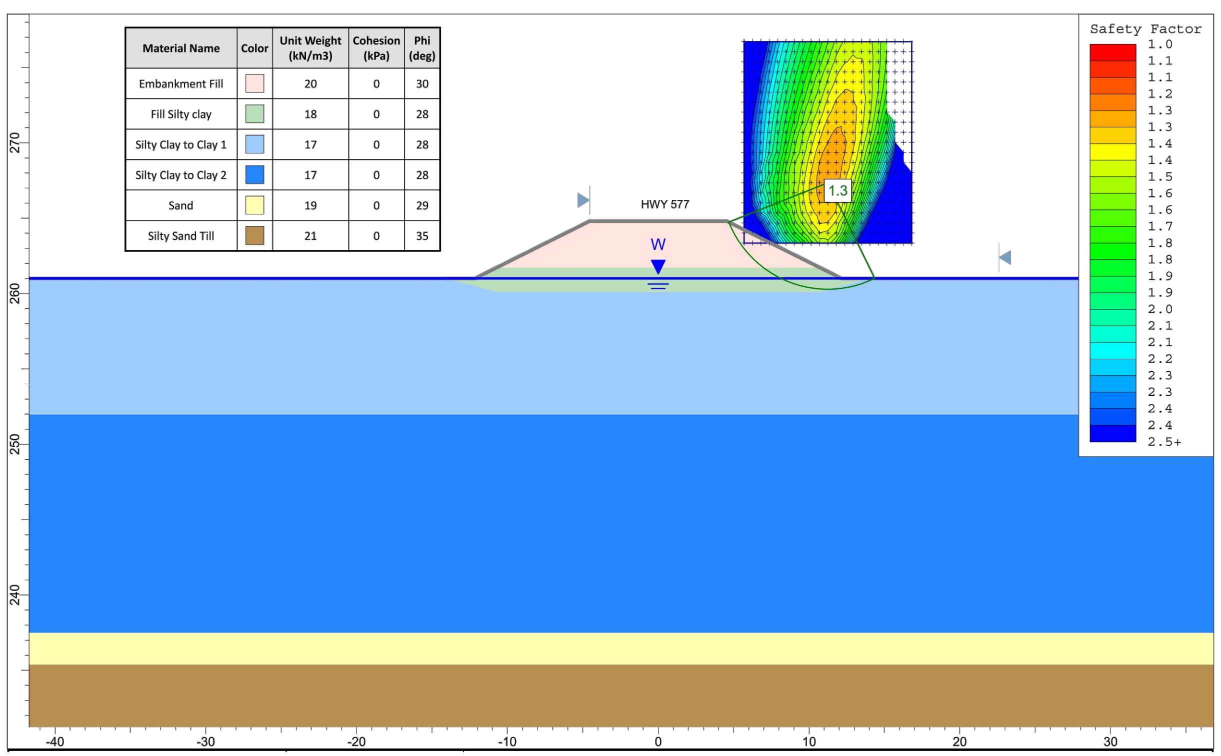
Slope Stability Models & Results



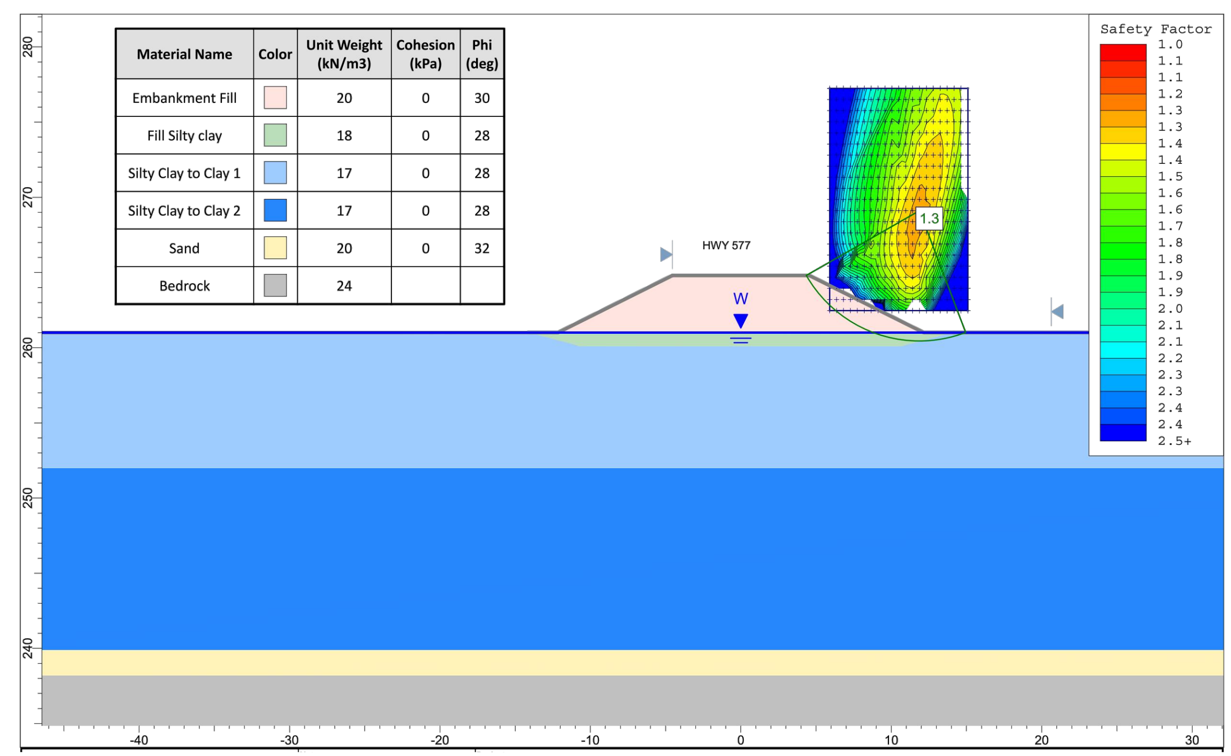
South Approach Embankment-Existing Hwy 577 - Total Stress Analysis



North Approach Embankment-Existing Hwy 577 - Total Stress Analysis

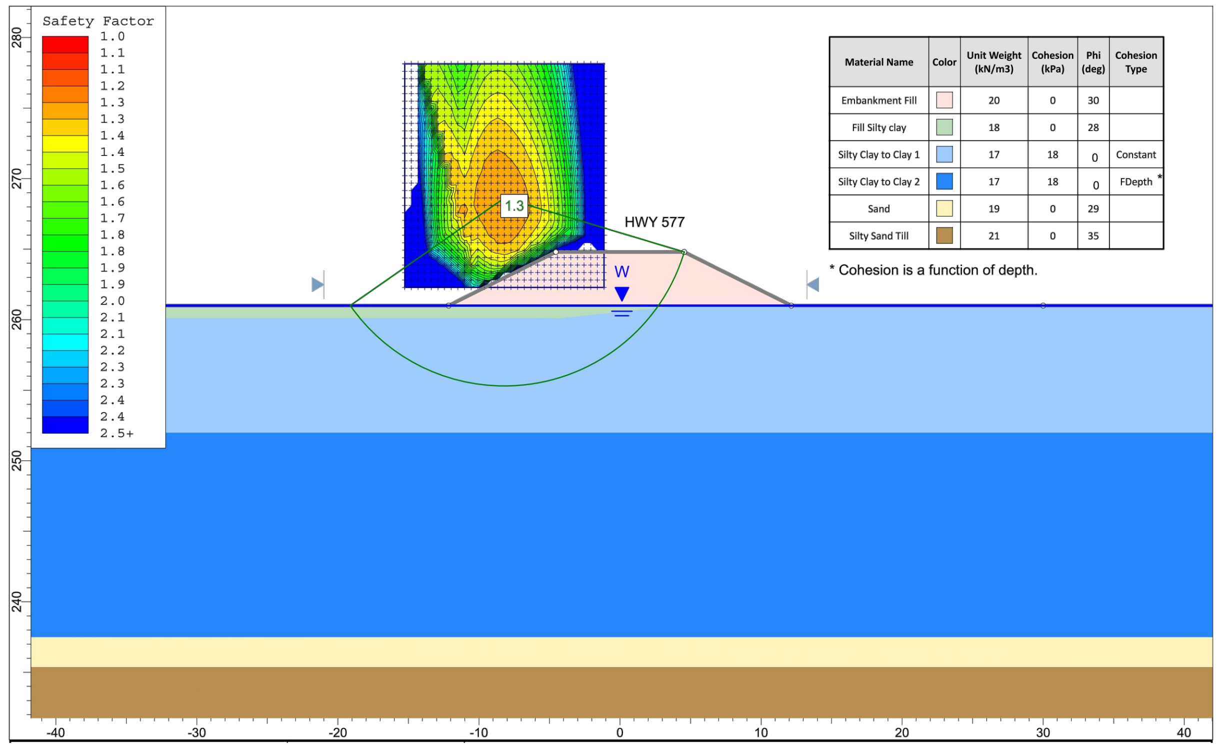


South Approach Embankment-Existing Hwy 577 - Effective Stress Analysis



North Approach Embankment-Existing Hwy 577 - Effective Stress Analysis

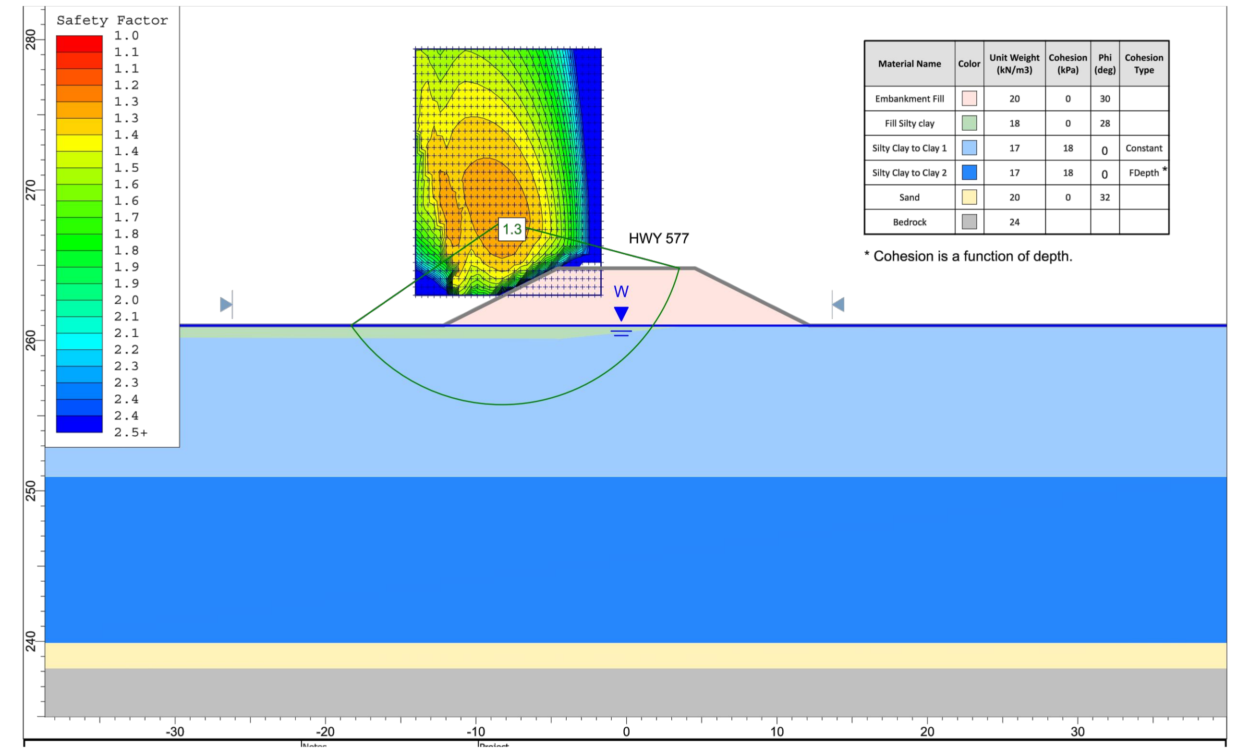
Terraprobe Inc. Consulting Geotechnical & Environmental Engineering, Construction Materials, Inspection & Testing. 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	HWY 577 DRIFTWOOD RIVER BRIDGE, SITE 39E-096		
G.W.P 417-91-00		DATE: MARCH 2015	
SUBM'D. HA	CHKD. RA		APPD: MT
Project No: 11-14-4066		Figure D1	



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)	Cohesion Type
Embankment Fill		20	0	30	
Fill Silty clay		18	0	28	
Silty Clay to Clay 1		17	18	0	Constant
Silty Clay to Clay 2		17	18	0	FDepth *
Sand		19	0	29	
Silty Sand Till		21	0	35	

* Cohesion is a function of depth.

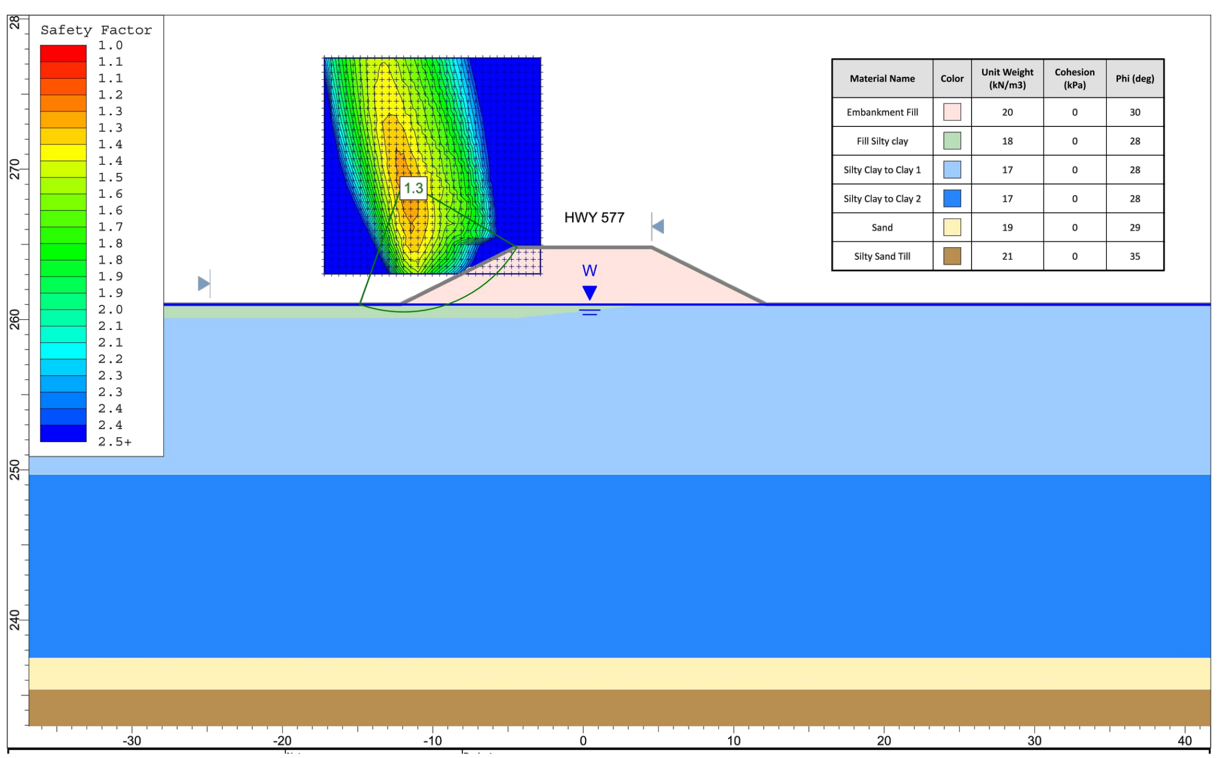
South Approach Embankment-Realigned Hwy 577 - Total Stress Analysis



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)	Cohesion Type
Embankment Fill		20	0	30	
Fill Silty clay		18	0	28	
Silty Clay to Clay 1		17	18	0	Constant
Silty Clay to Clay 2		17	18	0	FDepth *
Sand		20	0	32	
Bedrock		24			

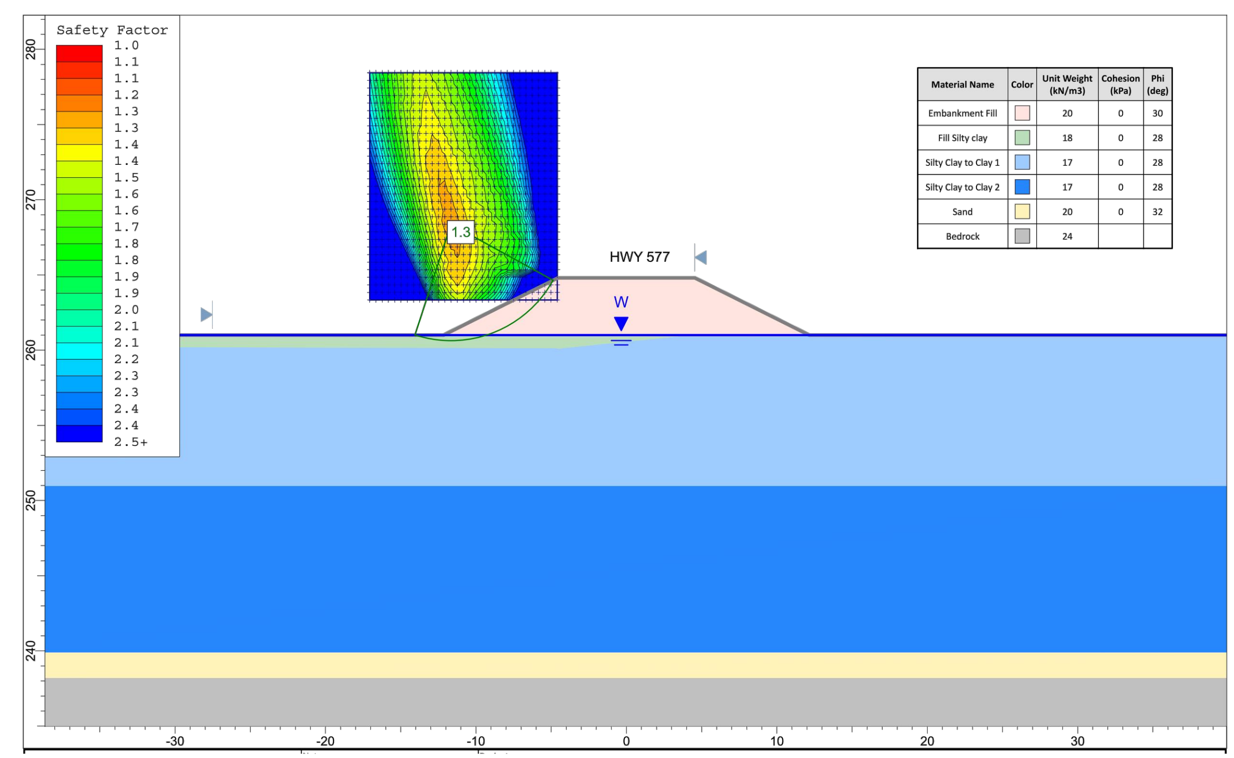
* Cohesion is a function of depth.

North Approach Embankment-Realigned 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
Silty Clay to Clay 1		17	0	28
Silty Clay to Clay 2		17	0	28
Sand		19	0	29
Silty Sand Till		21	0	35

South Approach Embankment-Realigned Hwy 577 - Effective Stress Analysis



Material Name	Color	Unit Weight (kN/m3)	Cohesion (kPa)	Phi (deg)
Embankment Fill		20	0	30
Fill Silty clay		18	0	28
Silty Clay to Clay 1		17	0	28
Silty Clay to Clay 2		17	0	28
Sand		20	0	32
Bedrock		24		

North Approach Embankment-Realigned Hwy 577 - Effective Stress Analysis

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HWY 577
DRIFTWOOD RIVER BRIDGE, SITE 39E-096

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Project No: 11-14-4066		Figure	D2