



Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
GILLES CREEK BRIDGE REPLACEMENT
HIGHWAY 579
ASSIGNMENT No. 5013-E-0018
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 5368-11-00, SITE 39E-006
GEOCRES NO. 42H-60**

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PART A – FOUNDATION INVESTIGATION REPORT

**GILLES CREEK BRIDGE REPLACEMENT, SITE 39E-006
HIGHWAY 579
TOWNSHIP OF LEITCH, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5013-E-0018, G.W.P. 5368-11-00**



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 Gilles Creek Bridge, Site 39E-006 on Highway 579, Township of Leitch, District of Cochrane, Ontario.

2.0 SITE DESCRIPTION

The site (with coordinates of N 59,575; E 9,400) is located on Highway 579, approximately 26 km north of the highway's intersection with Highway 11 in the Township of Leitch, 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 five-span timber bridge that is 23± m long and 10± m wide, supported on timber piles. This bridge carries Highway 579 north bound and south bound traffic over Gilles Creek. Gilles Creek flows from west to east meandering within a well-defined flood plain, towards its confluence with the Abitibi River located 850± m east of Highway 579.

The terrain at the bridge site and surrounding area is generally flat to gently rolling. Vegetation within the flood plain area consists primarily of grass, shrubs and occasional small trees. Beyond the flood plain the area is vegetated with mature stands of deciduous and coniferous trees.

3.0 INVESTIGATION PROCEDURES

The field work for this project was carried out between July 14 and August 06, 2014 and consisted of drilling and sampling two boreholes to depths of 24.6 m and 26.3 m below ground surface. The approximate borehole locations are 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 241.4 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.	Local Coordinate System		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH1	59 596.25	9 398.01	239.1	24.6
BH2	59 561.73	9 402.90	239.2	26.3

The boreholes were drilled with a CME 55 truck-mounted drill rig supplied and operated by a specialist drilling contractor. 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 D1586¹. 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 both boreholes, cobbles and boulders were encountered within the till matrix and NQ-size diamond coring techniques were used to extend the boreholes below the cobbles and boulders. In Borehole 2 Dynamic Cone Penetration tests were also performed and the bedrock was cored by NQ-size diamond coring techniques. The field work was supervised on a full-time basis by a member of Terraprobe's staff who observed the drilling, sampling and in situ testing operations and logged the boreholes and rock cores.

Ground water conditions in the open boreholes were observed during the drilling operations and a standpipe piezometer was installed in Borehole 1 to permit longer term ground water level monitoring. The boreholes were backfilled 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 and Atterberg limits determinations in accordance with MTO and/or ASTM Standards as appropriate.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Surficial sediments in the study area were deposited during the Late Wisconsinan glaciation (Morris, Crabtree et al. 1998). The main overburden unit is the Matheson Till, consisting of substratified poorly sorted glacial debris and glaciofluvial deposits of cobbles, boulders, gravel, sands and silts. There is also a more recent deposit called the Cochrane Till which is an impervious clay-rich till that contains rounded pebbles (Bennett et al. 1967). Fresh water lacustrine deposits overlie the Cochrane Till.

A compilation of studies undertaken in the general area (Bennett et al. 1967; Wolfe et al. 1975) shows that the bedrock geology is dominated largely by metasedimentary gneissic rocks. Other rock types occurring within the study area include large batholiths of granitic intrusive rocks such as granodiorite and felsic volcanic rocks such as rhyolite.

4.2 Subsurface Conditions

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

The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic sections are inferred from non-continuous soil sampling and therefore represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations.

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

In summary, the ground surface is underlain by a flexible pavement, loose to dense sand fill material and stiff silty clay fill. The fill materials are underlain by deposits of firm to very stiff silty clay, stiff to hard silty clay till, and compact to very dense sand and silt till. The overburden soils are further underlain by rhyolite bedrock. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Flexible Pavement

Both boreholes encountered a flexible pavement consisting of 25 mm to 110 mm thick asphalt concrete underlain by granular fill consisting of gravelly sand and sand. The locations, thicknesses and base elevations of the granular base fill are summarized in the following table.

Pavement Granular Borehole Data

Borehole No.	Fill Thickness (mm)	Fill Base Elevation (m)
BH1	455	238.6
BH2	190	238.9

Standard Penetration tests carried out in the gravelly sand and sand fill gave SPT N-values of 13 blows for 0.3 m of penetration indicating a compact relative density. The natural water content of a sample of the granular fill is 2% by weight.

The grain size distribution curve of a sample of the sand fill is presented on Figure B1 in Appendix B. The results show a grain size distribution consisting of 19% gravel, 71% sand and 10% silt size particles.

4.2.2 Fill – Sand

Both boreholes 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	237.0
BH2	1.8	2.1	237.1

Standard Penetration tests performed in the fill measure SPT N-values ranging from 8 to 33 blows for 0.3 m of penetration indicating a loose to dense relative density. The natural water content of samples of the fill range from 2% to 5% 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 2% gravel, 92% sand and 6% silt size particles.

4.2.3 Fill – Silty Clay

Silty clay fill was encountered in both boreholes 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	2.3	4.4	234.7
BH2	1.3	3.4	235.8

Standard Penetration tests in the silty clay fill gave N-values ranging from 8 to 11 blows for 0.3 m of penetration indicating a stiff consistency. The natural water content of samples of the silty clay fill range from 9% to 23% by weight.

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 1% gravel, 13% sand, 50% silt and 36% clay size particles.

The silty clay fill was also subjected to an Atterberg Limits test and the results are presented on Figure B4 in Appendix B. These results indicate that the fill is a cohesive soil with low plasticity (CL). The results from the Atterberg limits tests are summarized below:

Liquid Limit:	29 %
Plastic Limit:	17 %
Plasticity Index:	12 %
Natural Moisture Content:	16 %

4.2.4 Silty Clay

Below the existing approach embankment fill there exists a layer of silty clay soil. Summarized below are the locations, thicknesses, depths and base elevations of the silty clay deposit.

Silty Clay Borehole Data

Borehole No.	Silty Clay Thickness (m)	Silty Clay Depth (m)	Silty Clay Base Elevation (m)
BH1	1.2	5.6	233.5
BH2	2.2	5.6	233.6

Standard Penetration tests in the silty clay measure SPT N-values of 5 and 7 blows per 0.3 m of penetration and, field vane tests measure in-situ undrained shear strengths ranging from 100 kPa to more than 100 kPa. Based on these tests the silty clay is described as having a firm to very stiff consistency. The sensitivity of the silty clay varies from 2.2 to 8.3, indicating a low sensitivity soil class (Canadian Foundation Engineering Manual [CFEM], 2006). The moisture contents of two samples of the silty clay are 22% and 28%.

The grain size distribution plots of two samples of the silty clay are depicted in Figure B5 in Appendix B. These results show a grain size distribution consisting of 0% gravel, 11% and 18% sand, 47% and 54% silt and, 28% and 42% clay sized particles.

Two samples of the silty clay deposit were also subjected to Atterberg limits tests and the results are presented in Figure B6 in Appendix B. These values indicate that the silty clay deposit is a cohesive soil of low plasticity (CL). The Atterberg limits test results are summarized below.

Liquid Limit:	31% and 33 %
Plastic Limit:	17% and 19 %
Plasticity Index:	12% and 16 %
Natural Moisture Content:	22% and 28 %

4.2.5 Silty Clay Till

In both boreholes the silty clay deposit is further underlain by silty clay till. The locations, thicknesses, depths and base elevations of the silty clay till deposit are summarized in the following table.

Silty Clay Till Borehole Data

Borehole No.	Silty Clay Till Thickness (m)	Silty Clay Till Depth (m)	Silty Clay Till Base Elevation (m)
BH1	3.8	9.4	229.7
BH2	1.6	7.2	232.0

The N-values of Standard Penetration tests carried out in the silty clay till deposit range from 10 to 54 blows per 0.3 m of penetration, suggesting a stiff to hard consistency and the moisture content of samples of this deposit range from 18% to 31% by weight.

Two samples of the silty clay till were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figure B7 in Appendix B. The test results show a grain size distribution consisting of 1% gravel, 5% and 20% sand, 18% and 57% silt and, 38% and 61% clay sized particles.

Atterberg limits tests were also carried out on two samples of the silty clay till and the results are plotted on the plasticity chart, Figure B8 in Appendix B. The results indicate that the till matrix generally consists of low to intermediate plasticity (CL to CI) silty clay soils. The Atterberg limits test results are summarized below.

Liquid Limit:	27% and 49 %
Plastic Limit:	14% and 21 %
Plasticity Index:	13% and 28 %
Natural Moisture Content:	18% and 31 %

4.2.6 Sand and Silt Till

A sand and silt till was encountered in both boreholes. Summarized in the following table are the locations, explored depths and base elevations of this deposit.

Sand and Silt Till Borehole Data

Borehole No.	Sand and Silt Till Depth of Deposit (m)	Sand and Silt Till Base Elevation (m)
BH1	24.6*	214.5
BH2	23.2	216.0

* Borehole termination depth.

Standard Penetration tests carried out in this deposit gave N-values that generally range from 52 to more than 100 blows per 0.3 m of penetration indicating a very dense relative density. In Borehole 1 an SPT N-value of 11 blows per 0.3 m of penetration was recorded in the upper 1 m± of this deposit indicating a compact relative density. The moisture content of samples from this stratum range from 7% to 11% by weight.

Two samples of this till deposit were subjected to grain size distribution tests and the results are presented in Figure B9 in Appendix B. These results show a grain size distribution consisting of 2% and 6% gravel, 48% and 51% sand, 35% and 39% silt and; 7% and 12% clay sized particles.

In Borehole 1 gravelly sand layers were encountered within the matrix of this till deposit and the grain size distribution curve of a sample of the gravelly sand is shown on Figure B10 in Appendix B. The grain size distribution of the gravelly sand consists of 34% gravel, 43% sand, 19% silt and 4% clay sized particles. The matrix of the sand and silt till contains frequent cobble and boulder inclusions and, NQ-size diamond coring techniques were necessary to extend the boreholes below the cobbles and boulders. Photographs of the cobbles and boulders are provided in Figure B11 in Appendix B.

4.2.7 Bedrock

In Borehole 2 the overburden soils are underlain by rhyolite bedrock that was encountered at a depth of 23.2 m below ground surface or elevation 216.0 m. Photographs of the bedrock core samples are provided in Figure B12 in Appendix B.

The rhyolite bedrock is described as unweathered, massive and its colour is grey with white (quartz) intrusions. The Rock Quality Designation (RQD) measured on the two rock core samples is 81% and 97%, indicating a rock mass of good to excellent quality. The Total Core Recovery (TCR) of the core samples is 97% and the Solid Core Recovery (SCR) of the core samples is 91% and 97%.

Point Load Index Tests were carried out on the bedrock core samples and the interpreted unconfined compressive strength (UCS) results range from 84 MPa to 147 MPa. These UCS results classify the tested portions of the bedrock as strong (R4 grade, 50 MPa to 100 MPa) to very strong (R5 grade, 100 MPa to 250 MPa) according to the rock strength classification in Table 3.5 of the *Canadian Foundation Engineering Manual 2006*.

4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling. A standpipe piezometer was installed in Borehole 1 and the measured ground water levels in the piezometer are summarized in the following table:

Ground Water Level Data

Borehole No.	Date	Water Levels	
		Depth (m)	Elevation (m)
BH1	September 16, 2014	2.6	236.5
	October 27, 2014	2.4	236.7

The ground water level at this site is estimated to be at an approximate elevation of $236.7 \pm$ m based on the soil moisture conditions, measured ground water levels and creek 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 supervised by Mr. Wen Zhu and the routine laboratory testing was carried out at Terraprobe's Brampton laboratory.

This report was prepared by Mr. Hussein Ahmed, P.Eng. and reviewed by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe; with assistance provided by Ms. Sepideh D-Monfared, MEng. and Mr. Ashkan Abouzar, P.Eng. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact conducted an independent quality control review.

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PART B – FOUNDATION DESIGN REPORT

**GILLES CREEK BRIDGE REPLACEMENT, SITE 39E-006
HIGHWAY 579
TOWNSHIP OF LEITCH, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5013-E-0018, G.W.P. 5368-11-00**



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 Gilles Creek 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 five span timber structure supported on timber pile foundations, with a length of $23\pm$ m and a width of $10\pm$ m. The bridge carries Highway 579 north bound and south bound traffic over Gilles Creek. Alternative replacement structures that are being considered include a new bridge and triple cell culverts that will be constructed either on the existing alignment or on new alignments. The offset distance (to the east relative to the existing highway centre line) will range from 2.8 m to 13 m for bridges and will be about 23.5 m for triple cell culverts.

Replacement structures will require either raising the existing road profile or constructing the realigned highway to a maximum elevation of $241\text{ m}\pm$ at the structure. This grade raise will require either raising the existing approach embankments or constructing new embankments with heights of up to $6\text{ m}\pm$.

6.2 Foundation Alternatives

The advantages, disadvantages, risks and consequences of practical foundation options for supporting either a bridge or a structural culvert 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

Based on the encountered subsurface stratigraphy, spread footings may be considered for supporting either the bridge abutments or culverts. The proposed Hwy. 579 grade raise of up to elevation $241\pm$ m may require approximately 7.5 m long abutment stems that may be impractical from a structural design perspective. Nevertheless, the recommended founding depths and geotechnical resistances for footings (minimum footing width of 2 m) founded on undisturbed competent natural soils are tabulated as follows:

Footing Depths and Geotechnical Resistance For Spread Footings

Borehole Location And Number	Existing Ground Surface Elev. (m)	Bottom of Footing Level Below Existing Ground Surface (m)	Founding Elevation (m)	Factored Geotech. Resistance at ULS (kPa)	Geotech. Reaction at SLS (kPa)	Subgrade Soil
North Abutment BH 1	239.1 \pm	Below 5.6 \pm	Below 233.5	250	175	Silty Clay Till
South Abutment BH 2	239.2 \pm	Below 5.6 \pm	Below 233.6	300	200	Silty Clay Till

Since the silty clay till is susceptible to disturbance when wet, it is recommended that a 75 mm thick layer of lean concrete (mud mat) be poured on the foundation bearing surfaces as soon as possible after excavation and approval.

If it is beneficial to the overall design the length of the bridge abutment stems can be reduced by founding spread footings on an engineered fill pad consisting of OPSS Granular 'A', compacted to 100% Standard Proctor Maximum Dry Density (SPMDD) at $\pm 2\%$ of optimum moisture content, as specified in OPSS.PROV 501. This foundation design concept i.e. an engineered fill pad, can also be used to support a triple cell concrete box culvert.

All topsoil and fill should be stripped from below the footprint of the engineered fill pad and the native soil should be excavated at least to the elevations provided in the table below (or deeper if required), to achieve the minimum thickness of engineered fill.

Recommended Base Elevations of Engineered Fill Pad

Borehole No.	Foundation Element	Elevation	Subgrade Soil
BH1	North Abutment	233.5	Silty Clay Till
BH2	South Abutment	233.6	Silty Clay Till

The engineered fill pad should extend at least 1 m horizontally beyond the spread footings or culvert sides, and should extend down at a 1 Horizontal to 1 Vertical (1H:1V) side slope to the recommended base elevations provided in the table above. For spread footings, the thickness of the engineered fill pad should be equal to or greater than the footing width, and should not be less than 2 m.

Spread footings or box culverts founded on a compacted Granular 'A' pad may be designed for the following concentric, vertical geotechnical resistances:

- Factored Geotechnical Resistance at ULS – 900 kPa; and
- Factored Geotechnical Reaction at SLS – 350 kPa

The ULS and SLS values provided herein are for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in the *Canadian Highway Bridge Design Code 2006* (CHBDC 2006), Clause 6.7.3 and Clause 6.7.4. The recommended SLS values correspond to a settlement of up to 25 mm, a significant portion of which will be complete by the end of construction.

6.2.1.1 Ultimate Coefficient of Friction

Resistance to lateral forces/sliding resistance between the concrete footing and the subgrade soils should be evaluated in accordance with the CHBDC 2006. The following ultimate coefficient of friction values are recommended between concrete and the bedding material or subgrade soils:

- OPSS Granular 'A' – ultimate coefficient of friction of 0.7; and
- Silty Clay till – ultimate coefficient of friction of 0.6.

6.2.2 Augered Caissons (Drilled Shafts)

Augered caisson foundations were considered as a foundation scheme. The caissons will have to be founded on the very dense sand and silt till which is located below the ground water table and 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 permeable nature of the sand and silt till and the presence of cobbles and boulders. Furthermore, 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 steel H-piles. Steel tube piles were considered but were excluded as a pile alternative because of the following reasons:

- Because of the cobbles and boulders in the sand and silt till, it would be 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; and
- Vibrations imparted to the surrounding soils and the existing bridge from pile driving operations will be higher if steel tube piles are used rather than “low displacement” H-pile sections.

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 pile sections. The structural resistance of the pile should also be checked by the structural designer. For piles founded on bedrock an average UCS value of 130 MPa can be used for preliminary designs.

Axial Resistance of Driven Piles

Existing Bridge Site and Detour Alignment					
Location	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
PILE TYPE - HP 310x110					
North Abutment	BH1	225.0±	Sand and Silt Till	1600	1200
South Abutment	BH2	222.5±	Sand and Silt Till		
PILE TYPE – HP 360X132					
North Abutment	BH1	224.0±	Sand and Silt Till	2100	1600
South Abutment	BH2	220.0±	Sand and Silt Till		

Pile installation should be carried out in accordance with OPSS 903, November 2009. Pile driving 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 “R” values of 3200 kN and 4200 kN are recommended for HP 310x110 and HP 360x132 pile sections respectively.

Steel H-piles will be driven to practical refusal in the sand and silt till. Since the till matrix contains cobbles and boulders, piles may encounter effective refusal in this stratum without reaching the predicted pile tip elevations.

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.

6.2.3.2 Pile Tips

Due to the presence of cobbles and boulders in the sand and silt till, the tips of all piles should be fitted with H-section rock points from an approved manufacturer such as Titus Steel Company (Standard “H” bearing pile point) or Associated Pile & Fitting Corp (APF Hard Bite).

The use of rock points is recommended for the following reasons:

- The piles will be penetrating into soil containing cobbles and boulders, which requires a higher level of protection; and
- Rock points will provide increased cutting ability to the pile sections, reduce the probability of misalignment and increase the probability of achieving the desired penetration in competent strata.

6.2.3.3 Integral Abutment Considerations (Bridge)

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 equations below:

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

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

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/m ³)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m ³)*
North Abutment BH 1	238.9 – 238.6	Fill – Gravelly Sand	20	30	–	6600
	238.6 – 237.0	Fill – Sand	19	30	–	4400
	237.0 – 234.7	Fill – Silty Clay	19	0	25	–
	234.7 – 233.5	Silty Clay	19	0	100	–
	233.5 – 229.7	Silty Clay Till	20	0	125	–
	229.7 – 214.5	Sand and Silt Till	21	33	–	11000
South Abutment BH 2	239.1 – 237.1	Fill – Sand	19	30	–	6600
	237.1 – 235.8	Fill – Silty Clay	19	0	25	–
	235.8 – 233.6	Silty Clay	19	0	100	–
	233.6 – 232.0	Silty Clay Till	20	0	200	–
	232.0 – 216.0	Sand and Silt Till	21	33	–	11000

* 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 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 a new bridge be supported on steel H-pile foundations. Based on the advantages, disadvantages, risks and consequences, a steel H-pile foundation scheme is reliable, has the lowest risk associated with settlement and the highest probability of acceptable structural performance.

If a triple cell box culvert is proposed, it is recommended that the box culverts be supported on an engineered fill pad. This foundation scheme is reliable and since the risk of excessive settlement is low, acceptable structural performance can be expected.

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 30° to 35° 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 35°. Compaction equipment including hand operated vibratory equipment should be in accordance with OPSS.PROV 501.

The backfill to the bridge abutments or culvert 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 $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ; \gamma = 19.0 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.30	0.46*	0.20	0.28*
At rest (Restrained Wall)	0.43	-	0.47	-	0.33	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	5.0	-

* For wing walls.

The lateral earth pressure coefficients provided in the table above are “ultimate” values that require certain structural movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the CHBDC, 2006.

6.4 Bedding and Backfill (Box Culverts)

The backfill should consist of free-draining, non-frost susceptible granular materials in accordance with OPSS.PROV 1010. All granular fill should be placed in loose lifts not exceeding 150 mm thick and should be compacted to at least 95 % of the materials Standard Proctor Maximum Dry Density (SPMDD).

Equal heights of backfill should be maintained on both sides of the structure during all stages of backfill placement, and backfilling operations should be carried out in accordance with OPSS 902. Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. Compaction equipment should be restricted in accordance with OPSS.PROV 501.

Bedding material should consist of OPSS Granular ‘A’ material. Additional bedding requirements that may be imposed by the supplier must also be followed. All bedding and cover material should be placed in 150 mm thick loose lifts and uniformly compacted to at least 95 % of the materials SPMDD. Any

organic soils or fill material found within the footprint area of the box culvert should be removed and replaced with OPSS Granular 'A' material.

The excavated soil can be used for backfilling purposes provided they are free of organics and other deleterious material. To achieve the specified compaction, soils must neither be too wet nor too dry of their optimum moisture content. Soils that are too wet (such as the silty clay) cannot be used immediately because the material will have to be dried to a moisture content of $2\pm$ % of optimum. If the construction operations are time sensitive, the use of imported granular material may be considered. Soils that are dry of optimum can be used immediately provided that the material is moisture conditioned (i.e. water added) to achieve a moisture content of $2\pm$ % of optimum.

6.5 Erosion Protection (Box Culverts)

Erosion protection should be provided at the culvert inlets and outlets (including the slopes and sides). At the inlet area a clay seal can be provided such that water is channelled through the culvert and does not seep through the backfill around and underneath the structure. The clay seal should extend to cover all the granular backfill materials, should be a continuous layer around the culvert, should have a minimum compacted thickness of 0.6 m, and should extend at least 1 m above the high water level. The clay seal should also be protected by a layer of rip-rap. Material used for the clay seal should conform to the requirements stipulated in OPSS 1205. Concrete cut-off and head walls can also be used as an alternative to a clay seal to protect the granular fill around the culvert from erosion.

Design of an erosion protection scheme for the stream bed in the inlet and outlet areas will depend on hydrologic, hydraulic and/or other concerns. Typically, rip-rap protection should be provided to these areas. The rip-rap layer should cover all surfaces on the embankment slopes with which creek water is likely to be in contact.

We recommend that a qualified Hydraulics Engineer be consulted to design the specifics of the channel, culvert outlet and inlet (i.e. thickness and extent of protection) and scour depth. Footings must also be placed below the scour depth.

6.6 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 233.5± m), the OHSA soil classifications are:

- Embankment fill – Type 3 soil; and
- Silty clay – Type 3 soil.

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



6.7 Ground Water Control

Surface water and ground water control will be necessary to enable construction below the ground water table. We recommend temporarily diverting the flow of creek water away from the construction area. Around the perimeter of the excavation, a cofferdam and an interceptor perimeter trench should also be installed to prevent surface water from entering the excavation.

The design, installation, operation and maintenance of the dewatering system is the Contractor's responsibility. Excavations will extend through the existing embankment fill and native silty clay deposit terminating at the surface of the silty clay till. A suitable dewatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

6.8 Approach Embankments

6.8.1 Settlement

The embankment settlement analysis was carried out using elastic deformation moduli established from predictions/empirical correlations using undrained shear strengths, Atterberg limits and SPT N-values, tempered with engineering judgement from our experience with similar soils in this region of Ontario. For a grade raise to elevation $241\pm$ m, it is estimated that the embankment fill will induce approximately 20 mm of total settlement in the footprint area of the existing Hwy. 579 embankment, and about 10 mm will be due to consolidation settlement of the silty clay deposit. Where the new embankments are located beyond the toe of the existing approach embankments, the estimated total settlement will be about 60 mm and about 30 mm will be due to consolidation settlement of the silty clay deposit. About 90% of the consolidation settlement will be complete in about 2 to 3 months.

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.

6.8.2 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 Figure C1 in Appendix C. The analyses indicate that the factors of safety will be greater than 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	30	0	30	0	19
Silty Clay	0	25	28	0	19
Silty Clay Till	0	100	30	0	20
Silty Sand Till	33	0	33	0	21
Design Factors of Safety	1.4		1.4		-

6.8.3 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 SPMD. 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 OPSS 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.

6.9 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)	Active Earth Pressure Coefficient	At - Rest Earth Pressure Coefficient	Passive Earth Pressure Coefficient
			K_a	K_o	K_p
Existing Fill Soils	30	19	0.33	0.50	3.00
Silty Clay	28	19	0.36	0.53	2.77
Silty Clay Till	30	20	0.33	0.50	3.00
Sand and Silt Till	33	21	0.29	0.46	3.39

6.10 Seismic Requirements

The site is treated as lying in Seismic Zone 0. Reference to Annex A3.1 of the CHBDC 2006 indicates that the following seismic parameters (Cochrane) should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 1
- Zonal Acceleration Ratio 0.05
- Peak Horizontal Ground Acceleration 0.06 g (10% in 50 years)

The soil profile type at this site has been classified as Type I 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 1.0.

Culverts should be designed in accordance with Clause 7.5.5 of the CHBDC for a seismic event having a 10% probability of being exceed in 50 years. The vertical component of the earthquake acceleration ratio (A_v) shall be two-thirds of the horizontal ground acceleration ratio (A_h) and A_h shall be set equal to the zonal acceleration ratio.

6.11 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;
- Complete more rigorous assessments of foundation settlement as well as embankment stability and settlement for the preferred alternative; and

- Prepare a settlement monitoring and instrumentation plan to monitor the existing bridge if it remains open to traffic during pile driving operations.

7.0 CLOSURE

This report was prepared by Mr. Hussein Ahmed, P.Eng. and reviewed by Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe; with assistance provided by Ms. Sepideh D-Monfared, MEng. and Mr. Ashkan Abouzar, P.Eng. Mr. Michael Tanos, P.Eng., Terraprobe's Designated MTO Contact conducted an independent quality control review.

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Michael Tanos, P.Eng.
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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.
- Ontario Geological Survey 2001. *Results of modern alluvium sampling, Kapuskasing-Fraserdale area, northeastern Ontario: Operation Treasure Hunt-Kapuskasing Structural Zone.* Ontario Geological Survey, Open File Report 6044, 146p..

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.
OPSS.PROV 1004	Material Specification For Aggregates – Miscellaneous.
OPSS.PROV 1010	Material Specification For Aggregates – Base, Subbase, Select Subgrade and Backfill Material.
OPSS 1205	Material Specification For Clay Seal.

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

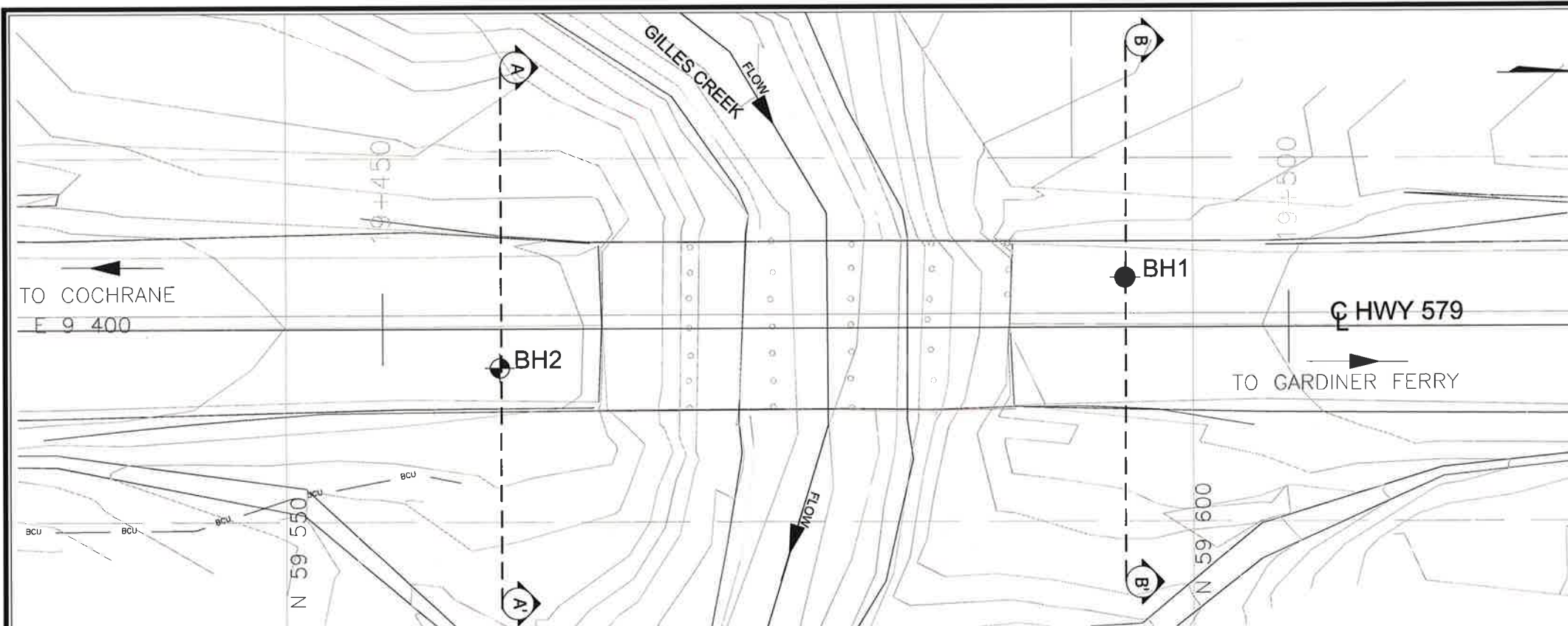


TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES

Structure Type	Spread Footings on Native Soils	Spread Footings on an Engineered Fill Pad	Pile Foundations
Bridge	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Eliminates the cost of an expensive Granular A engineered fill pad.No construction concerns associated with cobbles and boulders being an impediment during pile driving. <p>Disadvantages:</p> <ul style="list-style-type: none">Relative low geotechnical resistance compared to an engineered fill pad.Does not allow for the design of an integral abutment.Relative long abutment stems required.Dewatering required and a working mat is necessary to prevent disturbance of the subgrade soils.Relatively large and deep excavation required.	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Higher geotechnical resistance compared to constructing spread footings on native soils.Shorter abutment stems can be provided compared to constructing spread footings on native soils.Does not require a working mat at the base of the engineered fill pad.No construction concerns associated with cobbles and boulders being an impediment during pile driving. <p>Disadvantages:</p> <ul style="list-style-type: none">Does not allow for the design of an integral abutment.Relatively large and deep excavation required.Requires importing and stockpiling a relatively large amount of Granular A material which is expensive.	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.High geotechnical resistances available by driving piles to refusal.Allows for the design of an integral or semi integral abutment.Shallower excavation depth, reduced excavation volume and reduced dewatering requirements compared to the spread footing options.Eliminates the cost of an expensive Granular A engineered fill pad. <p>Disadvantages:</p> <ul style="list-style-type: none">Construction concerns related to the possibility of piles being obstructed by boulders during driving.A working mat is required to support pile driving equipment in areas where weak soils exist.
Culvert	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Precast culvert units can be used facilitating easy transportation, handling and placement.Eliminates the cost of an expensive Granular A engineered fill pad.No construction concerns associated with cobbles and boulders being an impediment during pile driving. <p>Disadvantages:</p> <ul style="list-style-type: none">Relative low geotechnical resistance compared to an engineered fill pad.Dewatering required and a working mat is necessary to prevent disturbance of the subgrade soils.Relatively large and deep excavation required.	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Precast culvert units can be used facilitating easy transportation, handling and placement.Higher geotechnical resistance compared to constructing spread footings on native soils.Does not require a working mat at the base of the engineered fill pad.No construction concerns associated with cobbles and boulders being an impediment during pile driving. <p>Disadvantages:</p> <ul style="list-style-type: none">Relatively large and deep excavation required.Requires importing and stockpiling a relatively large amount of Granular A material which is expensive.	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Precast culvert units can be used facilitating easy transportation, handling and placement.High geotechnical resistances available by driving piles to refusal.Shallower excavation depth, reduced excavation volume and reduced dewatering requirements compared to the spread footing options.Eliminates the cost of an expensive Granular A engineered fill pad. <p>Disadvantages:</p> <ul style="list-style-type: none">Construction concerns related to the possibility of piles being obstructed by boulders during driving.A working mat is required to support pile driving equipment in areas where weak soils exist.
Bridge & Culvert	<p>Risks/Consequences</p> <ul style="list-style-type: none">Very Low risk of bearing capacity failure.Very low risk that total settlement will exceed 25 mm.Culvert construction requires significant in-stream work.	<p>Risks/Consequences</p> <ul style="list-style-type: none">Very low risk of bearing capacity failure.Very low risk that total settlement will exceed 25 mm.Culvert construction requires significant in-stream work.	<p>Risks/Consequences</p> <ul style="list-style-type: none">Very low risk of bearing capacity failure.Very low risk that total settlement will exceed 25 mm.Culvert construction requires significant in-stream work.

DRAWINGS





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

GWP No 5368-11-00

HWY 579
GILLES CREEK BRIDGE
BOREHOLE LOCATIONS AND SOIL STRATA

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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 (October 2014)
 - ⊕ Piezometer
 - 90% Rock Quality Designation
 - A/R Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	239.1	59 596.25	9 398.01
2	239.2	59 561.73	9 402.90

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.

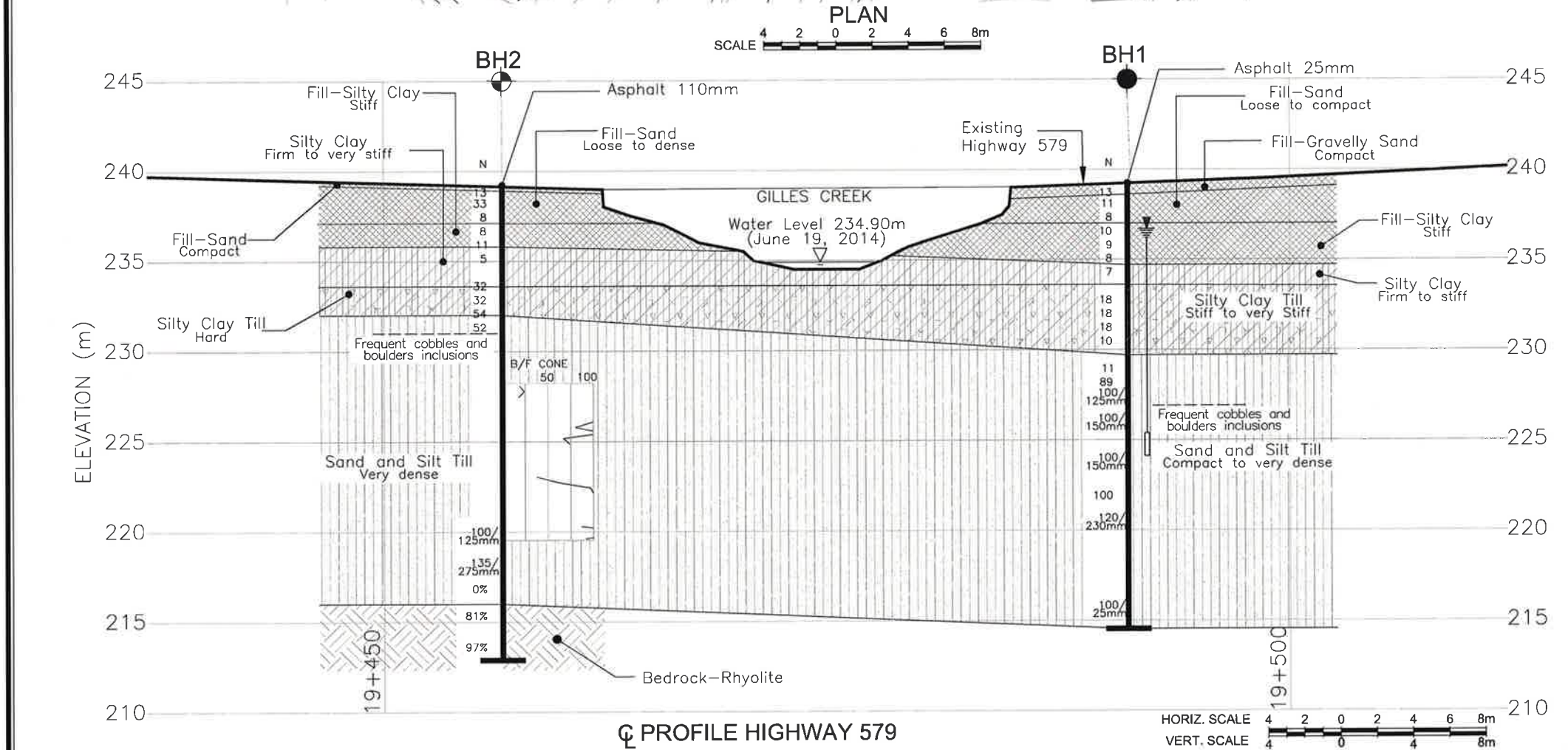
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of 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 B13420579001, DT13420579001, received September 11, 2014.

REVISIONS	DATE	BY	DESCRIPTION

HWY: 579	PROJECT No: 11-14-4056	GEOCRES No.: 42H-60
SUB'D: HA	CHKD: RA	DATE: October 2015
DRAWN: KC	CHKD: RA	APPD: MT



V:\PDS\Server\Project\Plan\11-14-4056\New Liskeard Area\2-Gilles Creek Bridge Hwy 579\B13420579001.dwg, 11-14-4056 Gilles Creek Bridge Hwy 579, Figure-005, 11-14-4056

APPENDIX A

Record of Borehole Sheets



EXPLANATION OF TERMS USED IN REPORT

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

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
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1.0%	VOID RATIO	e_{min}	1.0%	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1.0%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1.0%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_S	%	SHRINKAGE LIMIT	q	m ² /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $(w - w_p)/I_p$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $(w_L - w)/I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1.0%	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1

1 of 2

METRIC

G.W.P. 5368-11-00 LOCATION Coords: E:9398.01 N:59596.25 ORIGINATED BY W.Z
 DIST HWY 579 BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING COMPILED BY H.A
 DATUM GEODETIC DATE 2014-7-14 - 2014-8-6 CHECKED BY R.A

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE		SHEAR STRENGTH (kPa)					W _p	W	W _L		
239.1	GROUND SURFACE						20	40	60	80	100					
238.6 0.5	25mm ASPHALTIC CONCRETE		1	SS	13											
	455mm FILL-GRAVELLY SAND, trace silt, compact, brown, drv		2	SS	11											2 92 (6)
	FILL, sand, trace gravel, trace silt, loose to compact, brown, dry															
237.0 2.1	FILL, silty clay, trace sand, trace gravel, containing wood fragments, stiff, brown, moist to wet		3	SS	8											
			4	SS	10											
			5	SS	9											
			6	SS	8											
234.7 4.4	SILTY CLAY, some sand, firm to stiff, grey, moist to wet		7	SS	7											0 11 47 42
233.5 5.6	SILTY CLAY, trace sand, stiff to very stiff, grey, moist to wet (GLACIAL TILL)		8	SS	18											
			9	SS	18											0 5 57 38
			10	SS	18											
			11	SS	10											
229.7 9.4	SAND AND SILT, trace to some clay, trace gravel, containing cobbles and boulders below 12.2m, compact to very dense, grey, moist (GLACIAL TILL)		12	SS	11											sampler wet at 9.9m
			13	SS	89											6 48 39 7
			14	SS	100 / 125mm											July 14, 2014
			15	RC												July 31, 2014 NQ Coring
			16	SS	100 / 150mm											
			17	RC												

Continued Next Page

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

library: library - terraprobe gint - md.gib report: mto-terraprobe soil file: 11-14-4066 bh logs.gilles creek bridge-rav 1-gpi

RECORD OF BOREHOLE No 1

2 of 2

METRIC

G.W.P. 5368-11-00 LOCATION Coords: E:9398.01 N:59596.25 ORIGINATED BY W.Z
 DIST HWY 579 BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING COMPILED BY H.A
 DATUM GEODETIC DATE 2014-7-14 - 2014-8-6 CHECKED BY R.A

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SPT 'N' VALUE	SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
								20	40	60	80	100	w _p	w			w _L
	(continued)																
	SAND AND SILT, trace to some clay, trace gravel, containing cobbles and boulders below 12.2m, compact to very dense, grey, moist (GLACIAL TILL)		17	RC													
			18	SS	100 / 150mm											34 43 19 4	
	containing gravelly sand layers below 15.8m																
			19	SS	100												
			20	SS	120/ 230mm											July 31, 2014	
																Aug. 05, 2014	
			21	RC													
			22	RC													
											</						

END OF BOREHOLE

Piezometer installation consists of a 19mm diameter schedule 40PVC pipe with a 1.52m slotted screen.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 16, 2014	2.6	236.5
Oct 27, 2014	2.4	236.7

RECORD OF BOREHOLE No 2

1 of 2

METRIC

G.W.P. 5368-11-00 LOCATION Coords: E:9402.9 N:59561.73 ORIGINATED BY W.Z
DIST HWY 579 BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING/NQ CORING COMPILED BY H.A
DATUM GEODETIC DATE 2014-7-15 - 2014-7-16 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)							WATER CONTENT (%)				
								20	40						60	80	100	20	40
239.2	GROUND SURFACE																		
238.9	110mm ASPHALTIC CONCRETE						239												
0.3	190mm FILL-SAND, some gravel, some silt, compact, brown, dry		1	SS	13									19 71 (10)					
	FILL, sand, trace to some gravel, trace silt, loose to dense, brown, dry		2	SS	33														
			3	SS	8		238												
237.1																			
2.1	FILL, silty clay, some sand, trace gravel, stiff, brown, moist		4	SS	8		237							1 13 50 36					
	50mm amorphous peat layer		5	SS	11		236												
235.8																			
3.4	SILTY CLAY, some sand, trace gravel, trace organics, firm to very stiff, grey, moist to wet		6	SS	5		235							0 18 54 28					
	50mm sand and silt layer, grey, wet																		
233.6			7	SS	32		234												
5.6	SILTY CLAY, some sand, trace gravel, occasional silty sand to sand and gravel layers, hard, grey, moist to wet (GLACIAL TILL)		8	SS	32		233							sampler wet at 6.1m					
			9	SS	54		232							1 20 18 61					
232.0														commence casing and washboring					
7.2	SAND AND SILT, some clay, trace gravel, containing cobbles and boulders, very dense, grey, moist to wet (GLACIAL TILL)		10	SS	52		231							NQ Coring					
			11	RC			230												
							229												
							228												
			12	RC			227												
							226												
							225												

Continued Next Page

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

library: library - terraprobe gint - md.gib report: mto-terraprobe soil file: 11-14-4066 bh logs_gilles_creek_bridge-rav 1-gpi

METRIC

[illegible]

<p>SAND AND SILT, some clay, trace gravel, containing cobbles and boulders, very dense, grey, moist to wet (GLACIAL TILL)</p>	13	RC			<p>July 15, 2014 July 16, 2014</p>
	14	RC			
	15	RC			
	16	RC			
	17	SS			
	18	SS	135 / 275mm	<p>2 51 35 12</p>	
	19	RC			
	1	RUN	NQ		<p>RUN# 1 TCR=97% SCR=91% RQD=81% UCS*= 111 - 147 (MPa)</p>
	2	RUN	NQ		
	212.9				213

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

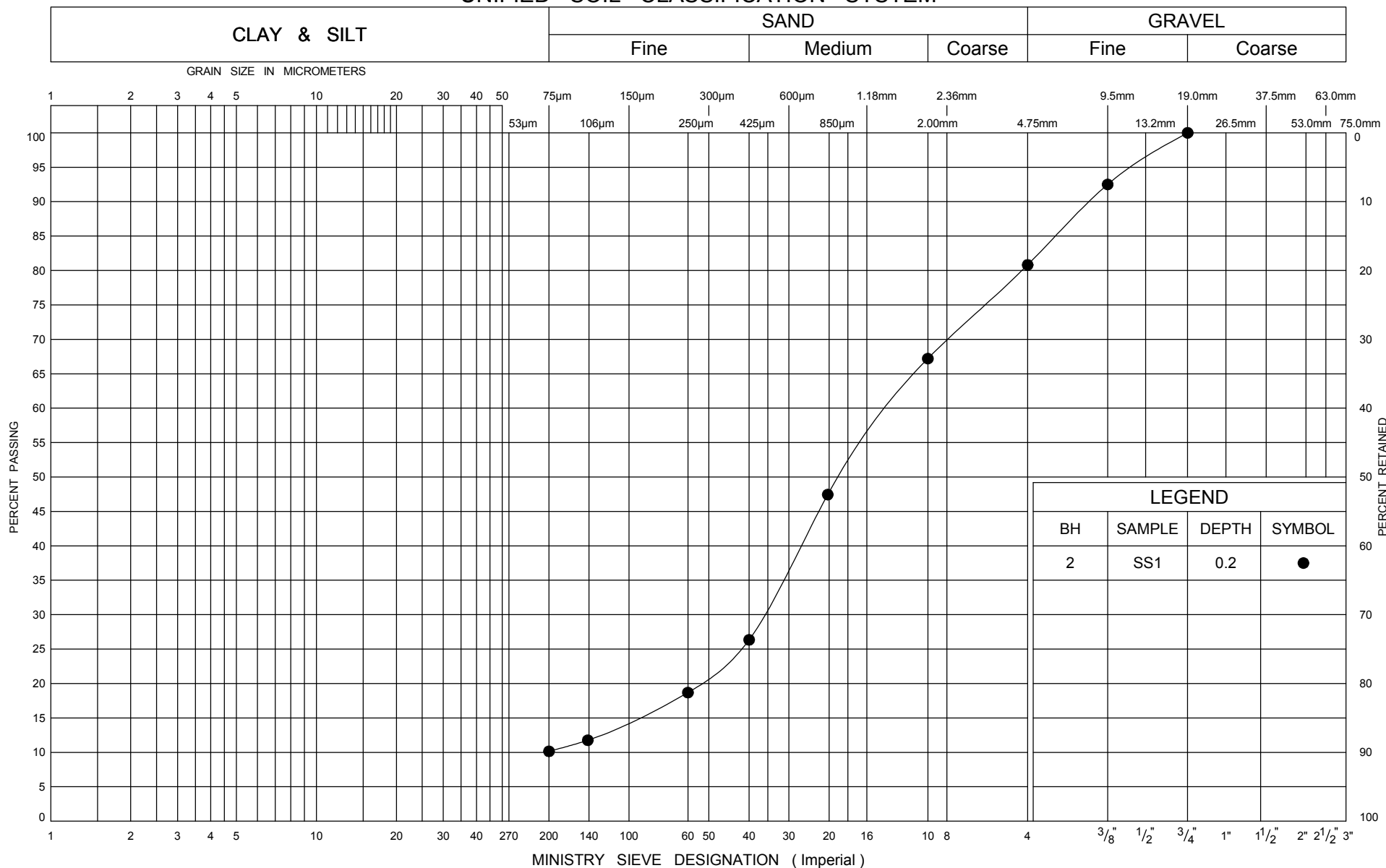
+³, ×³: Numbers refer to Sensitivity **○³%** STRAIN AT FAILURE

APPENDIX B

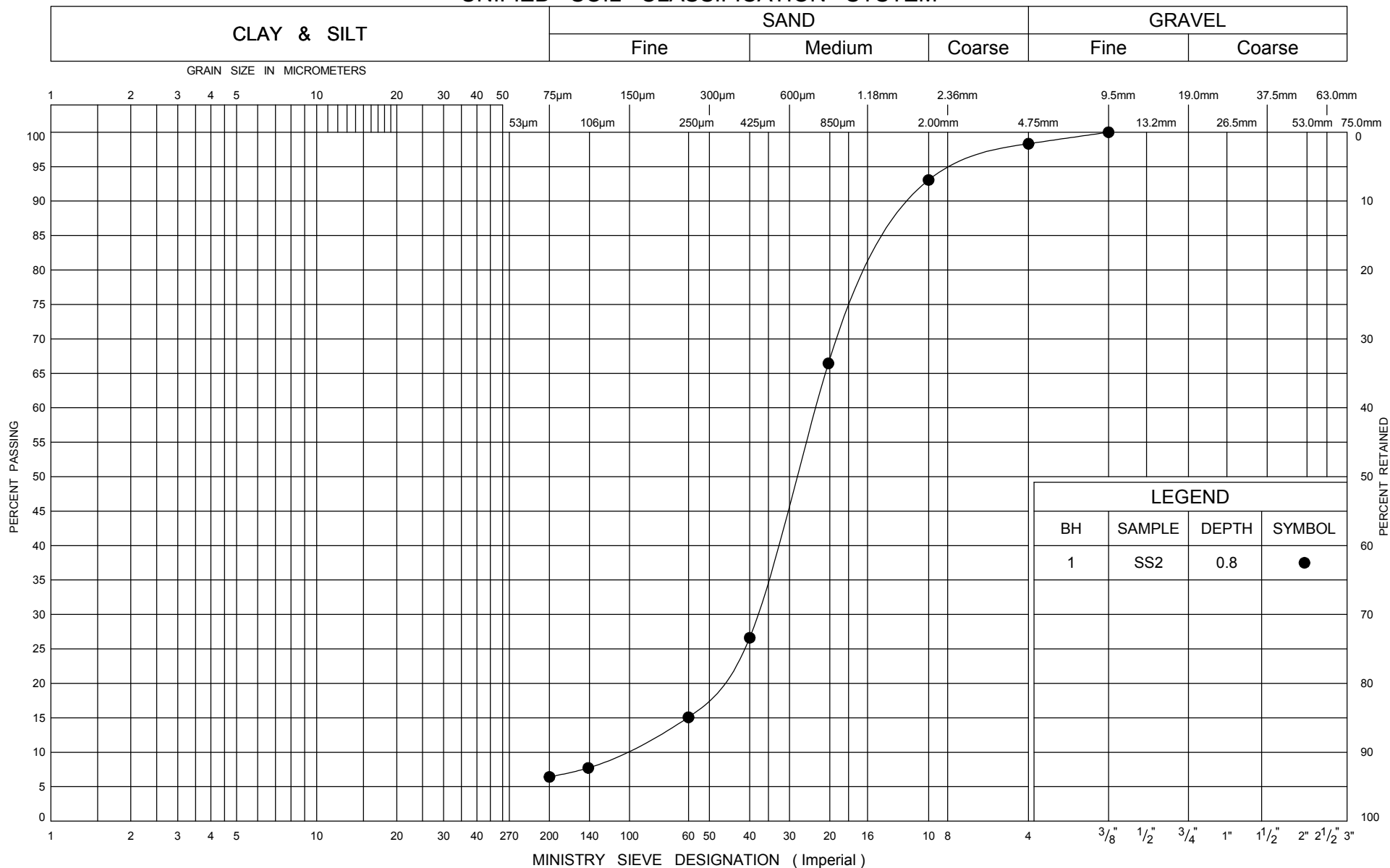
Laboratory Test Results



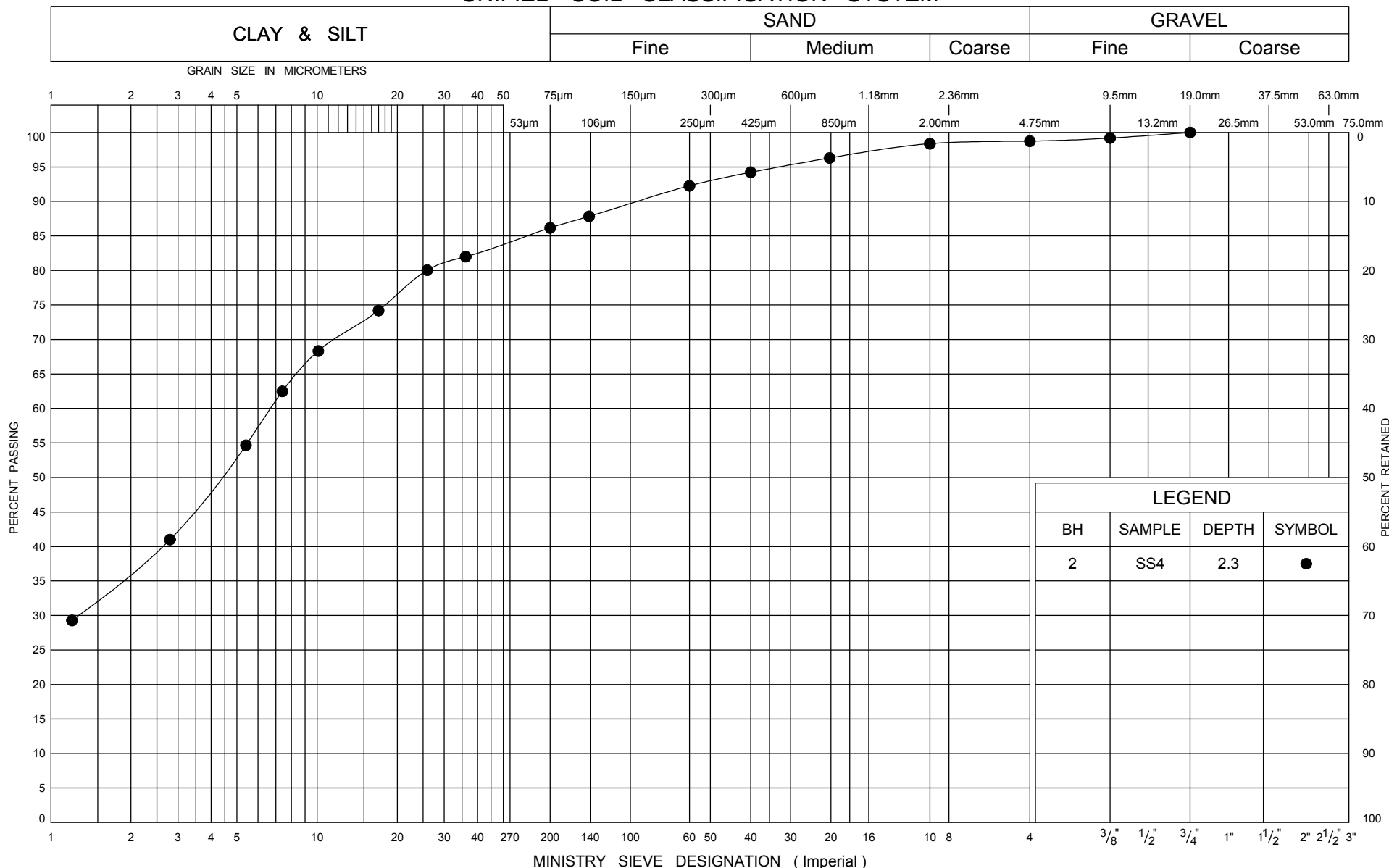
UNIFIED SOIL CLASSIFICATION SYSTEM



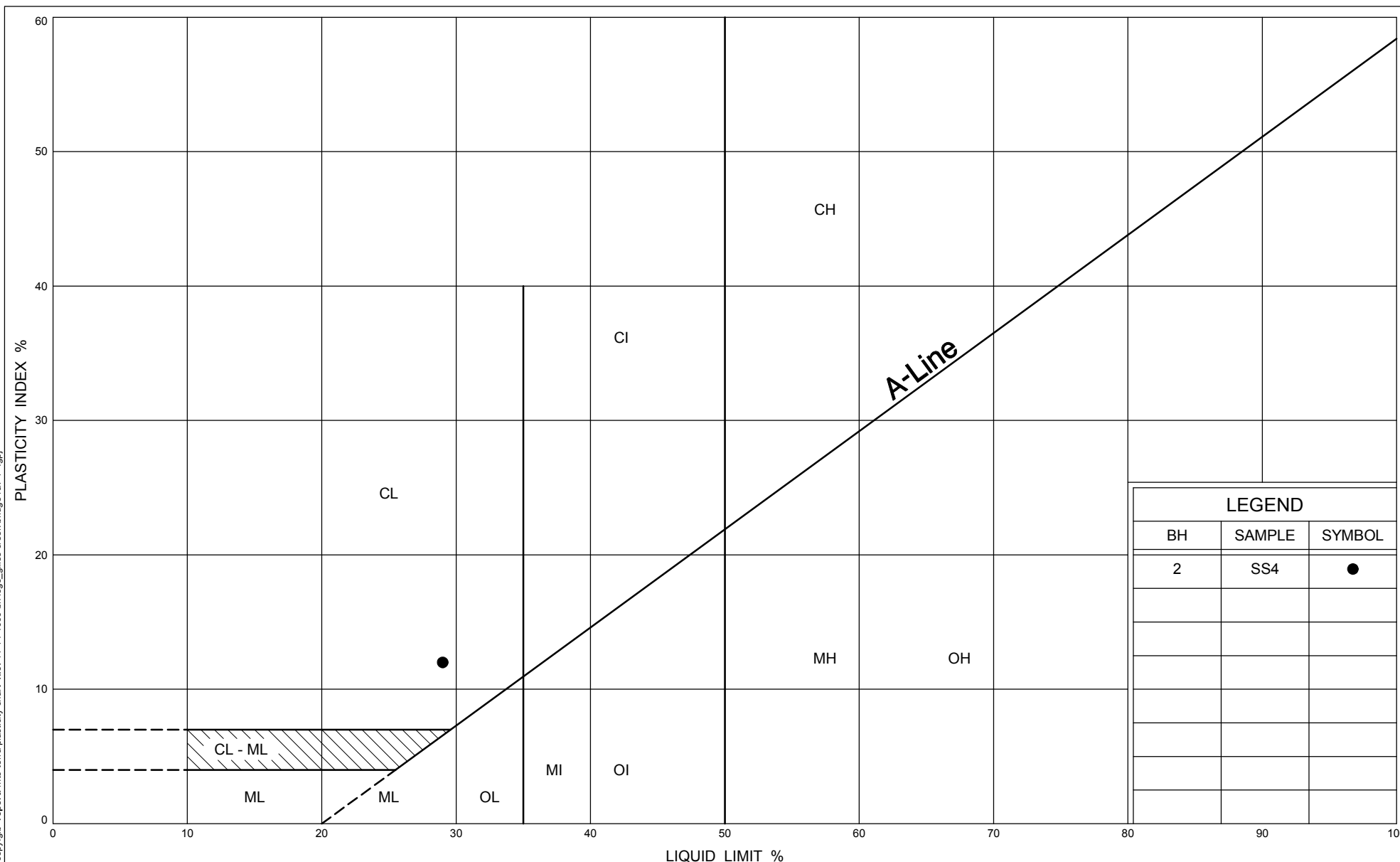
UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM

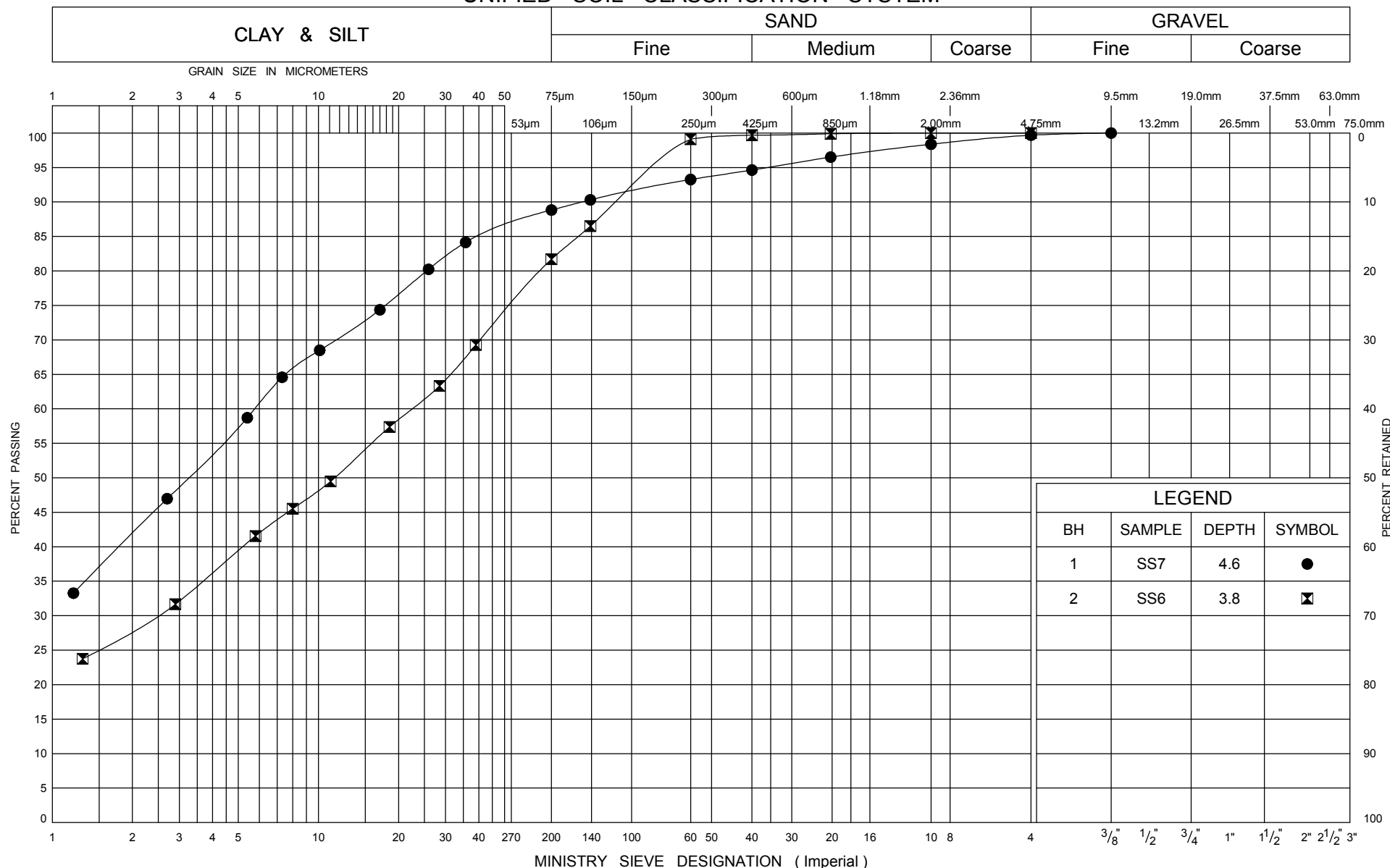


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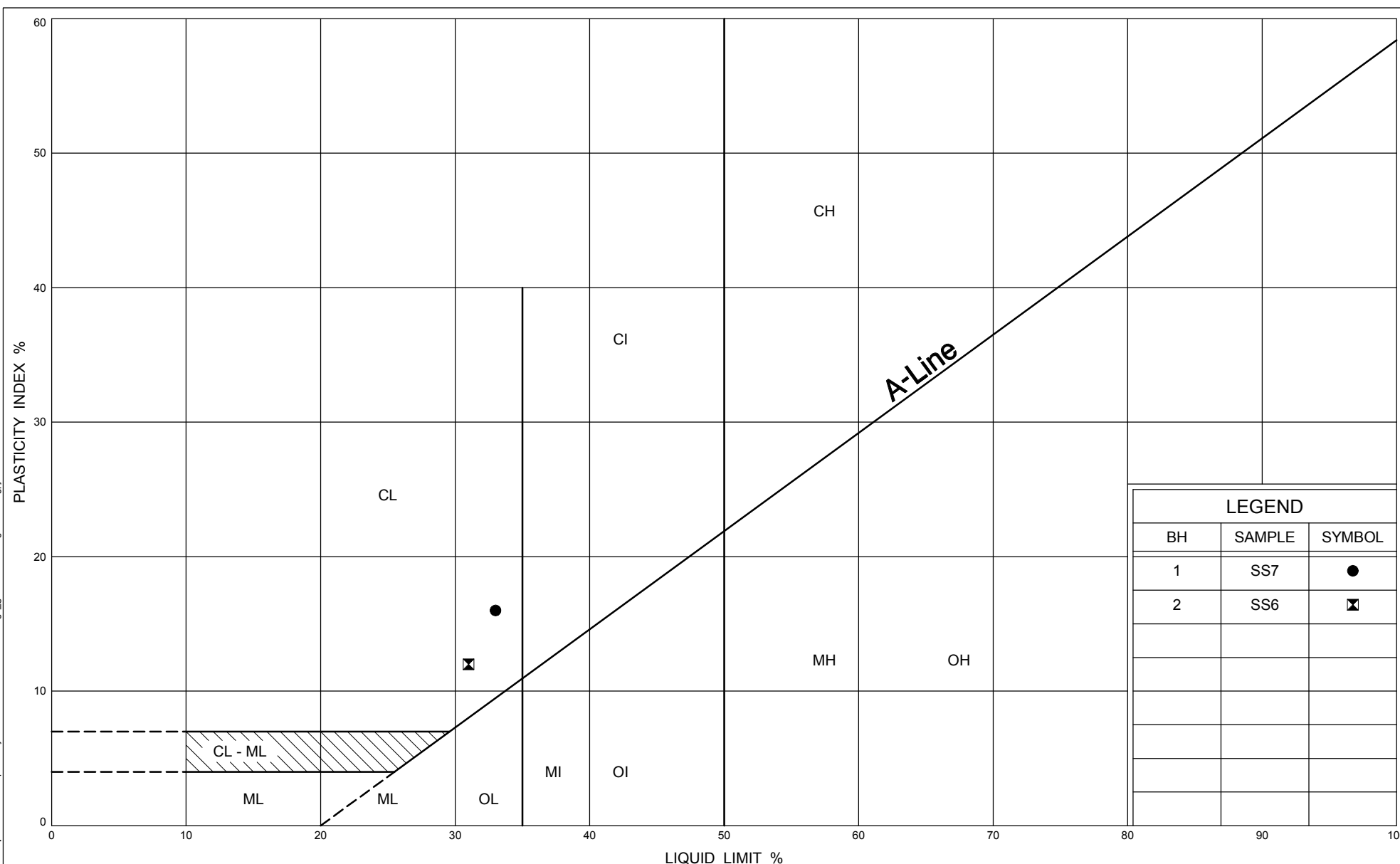


LEGEND		
BH	SAMPLE	SYMBOL
2	SS4	●

UNIFIED SOIL CLASSIFICATION SYSTEM



library: library - terraprobe.gint.glb report: mto-terra-plasticity chart file: 11-14-4066 bh logs_gilles creek bridge-rav 1--.ggl



Ministry of
Transportation

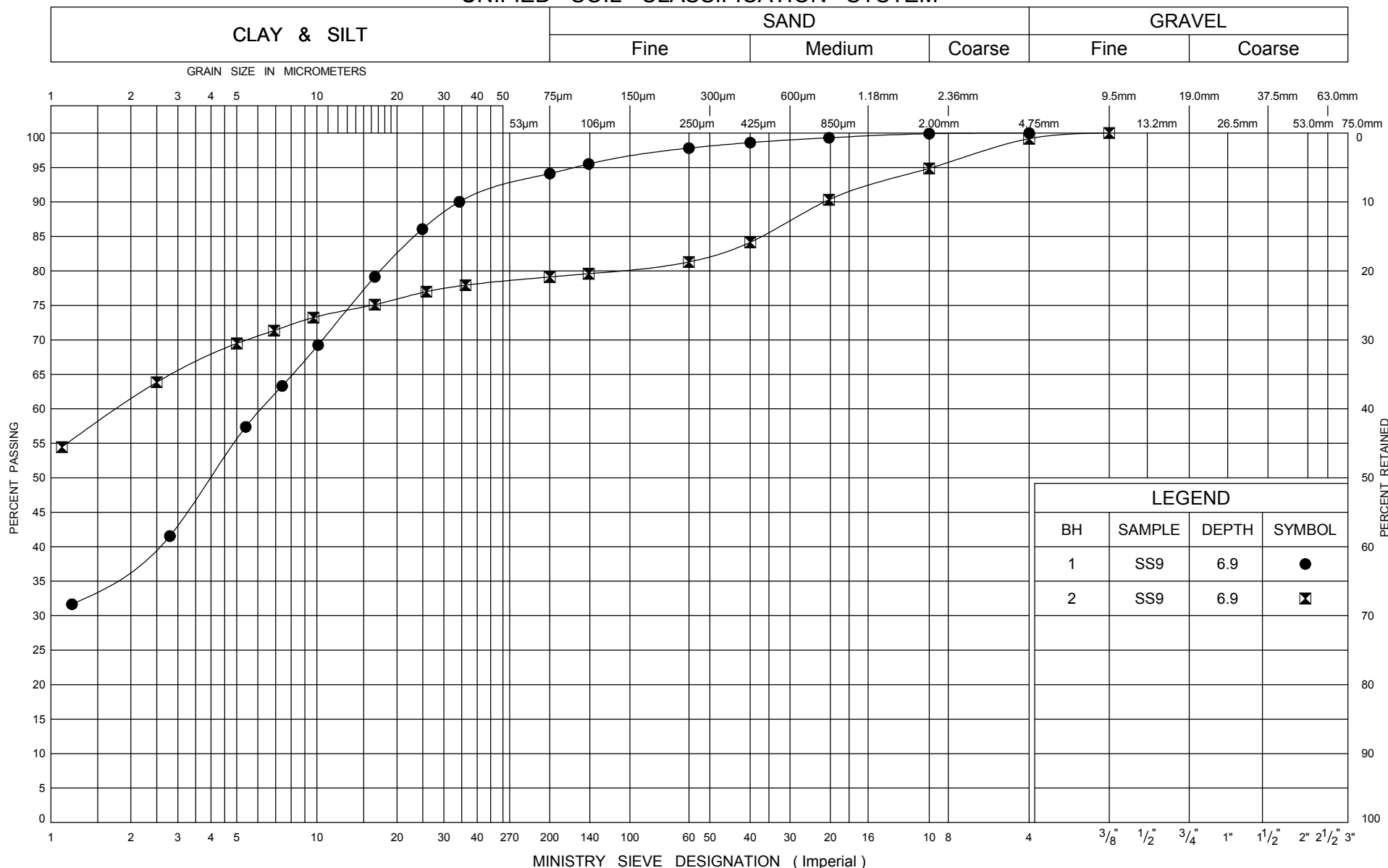
PLASTICITY CHART SILTY CLAY

FIG No B6

G W P 5368-11-00

Gilles Creek Bridge Site (39E-006)

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND			
BH	SAMPLE	DEPTH	SYMBOL
1	SS9	6.9	●
2	SS9	6.9	⊠

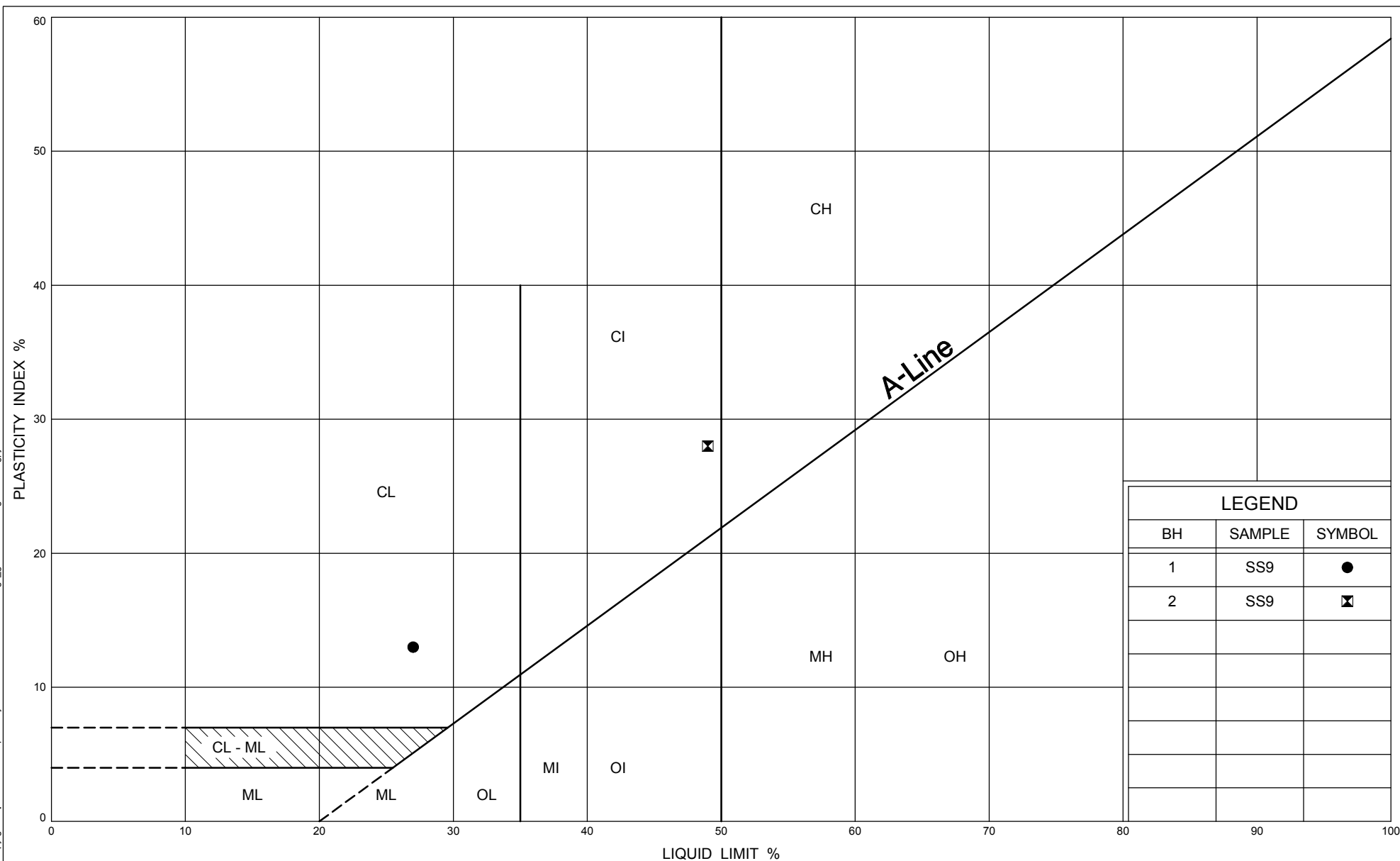
GRAIN SIZE DISTRIBUTION SILTY CLAY TILL

FIG No B7

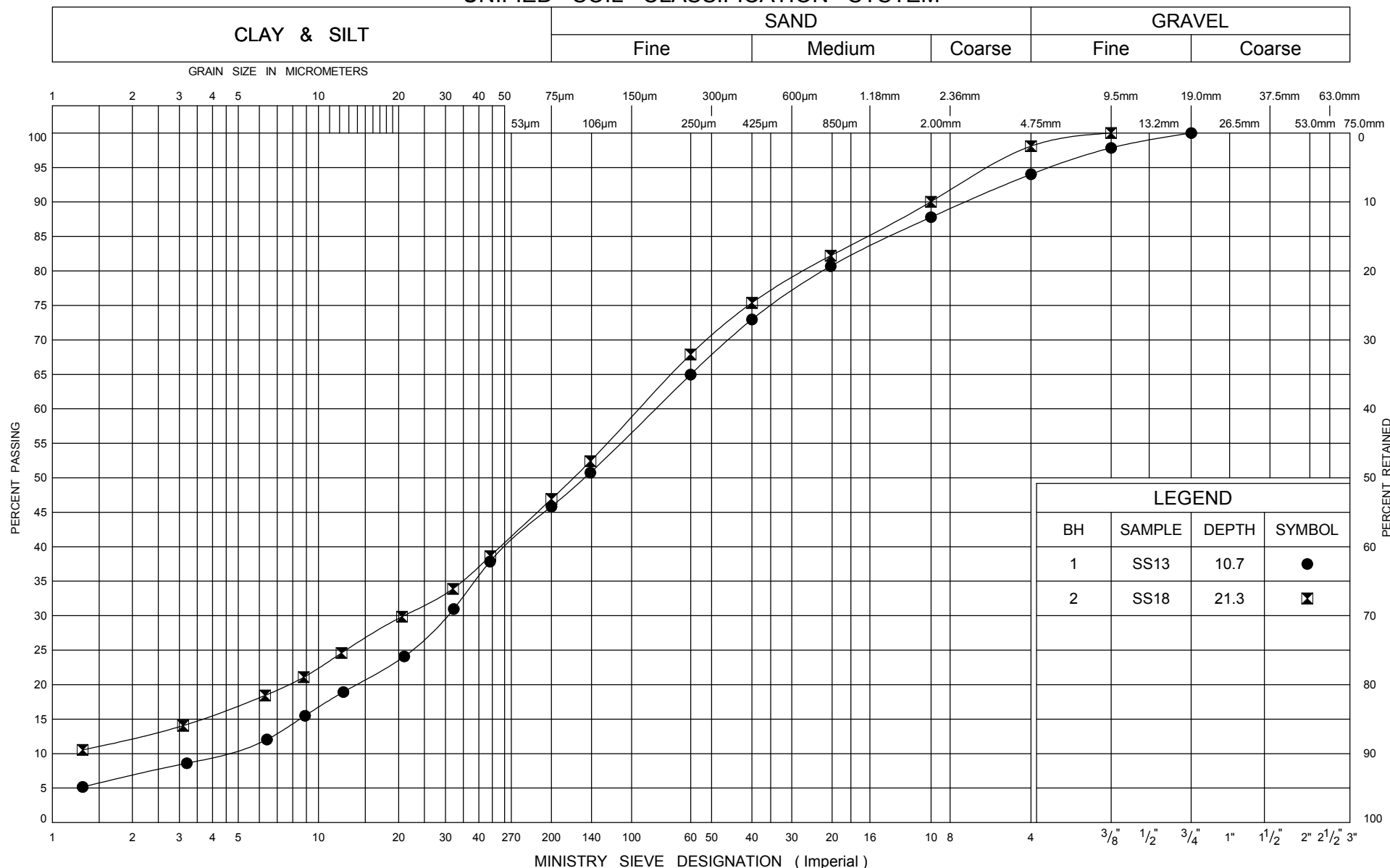
G W P 5368-11-00

Gilles Creek Bridge Site (39E-006)

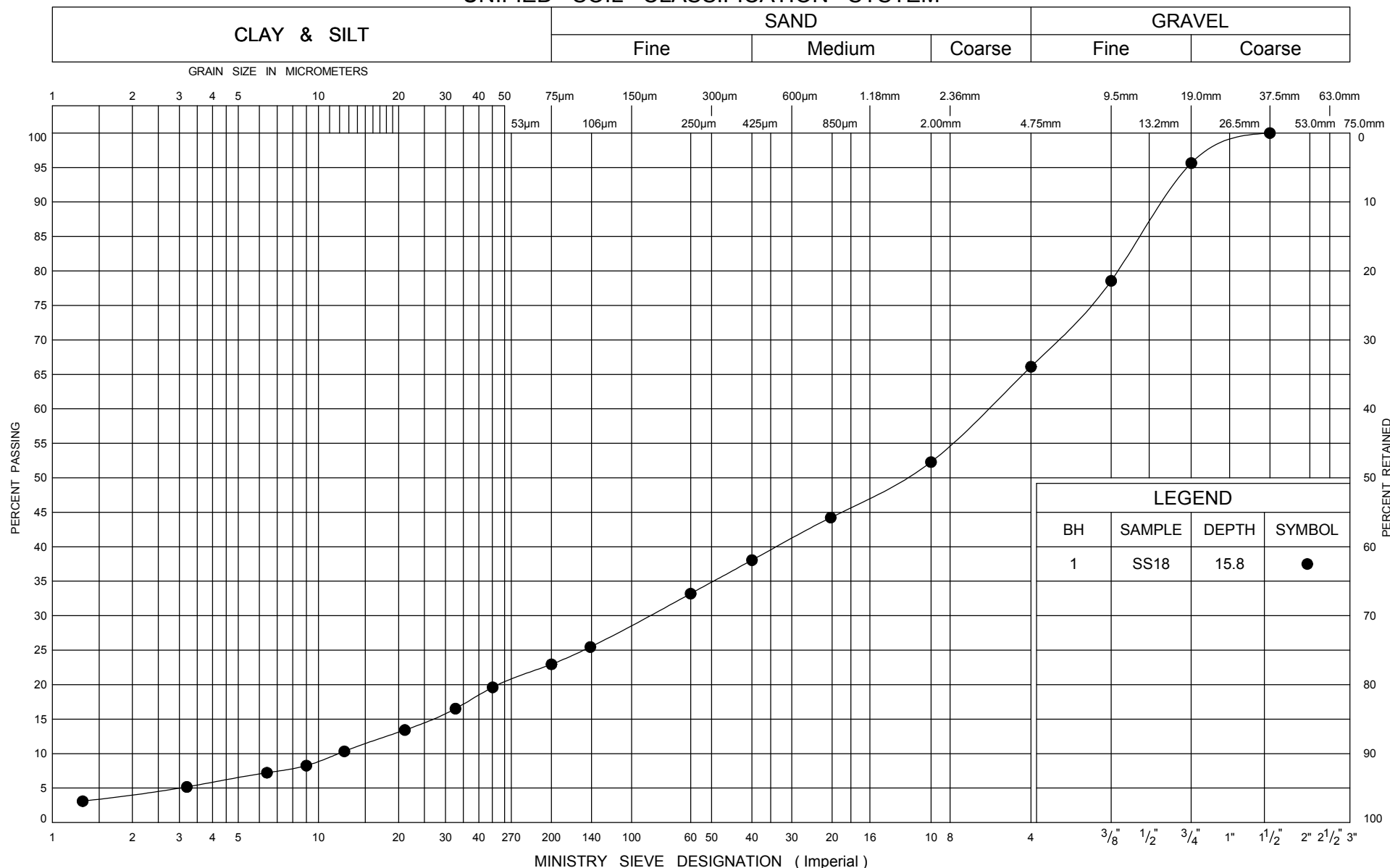
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UNIFIED SOIL CLASSIFICATION SYSTEM



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PHOTOGRAPHS OF COBBLES AND BOULDERS

FIGURE B11

GILLES CREEK BRIDGE (Site 39E-006)



Project No. : 11-14-4076

Date : December, 2014



Terraprobe Inc.

Prepared by : SD

Checked by : RA

PHOTOGRAPHS OF BEDROCK CORE SAMPLES

FIGURE B12

GILLES CREEK BRIDGE (Site 39E-006)



Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\2- Gilles Creek Bridge Hwy 579 (39E-006)\Eng. Analysis\Spread Sheets\0-P'c-Cc-Cr-Cu.xls

Project No. : 11-14-4076
Date : December, 2014



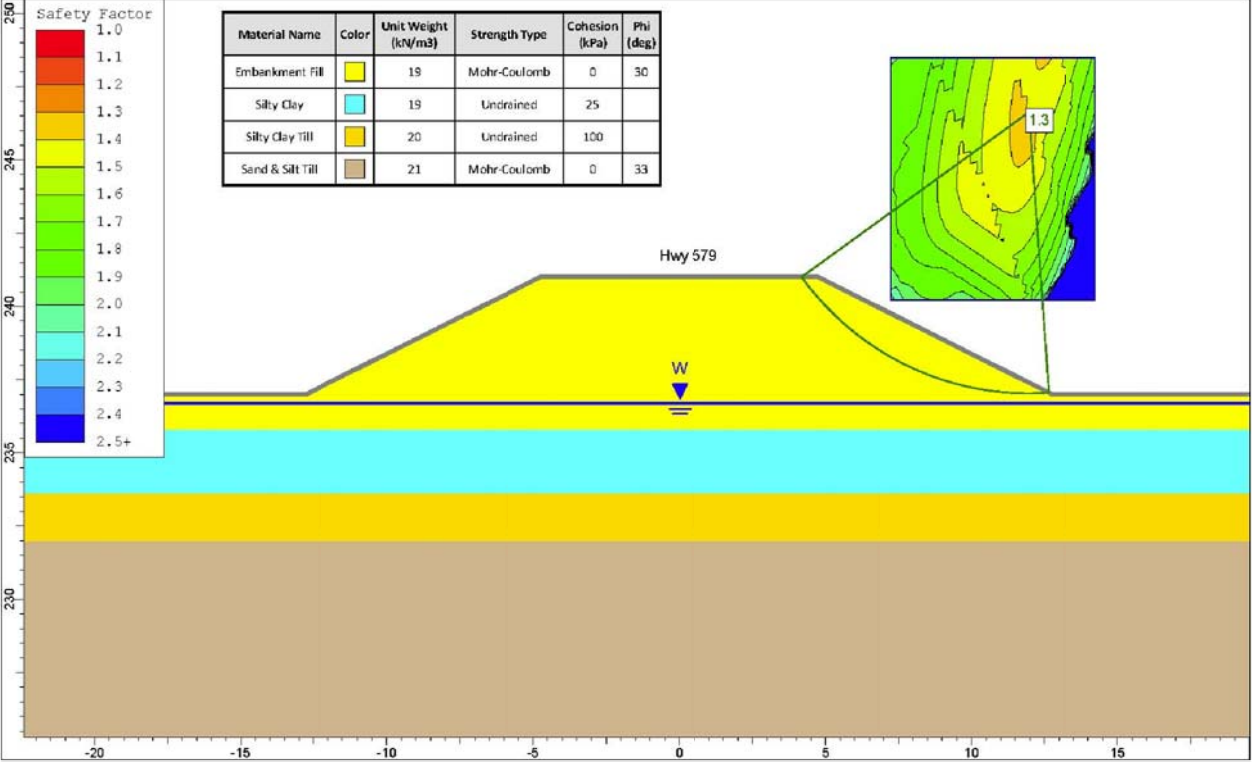
Terraprobe Inc.

Prepared by : SD
Checked by : RA

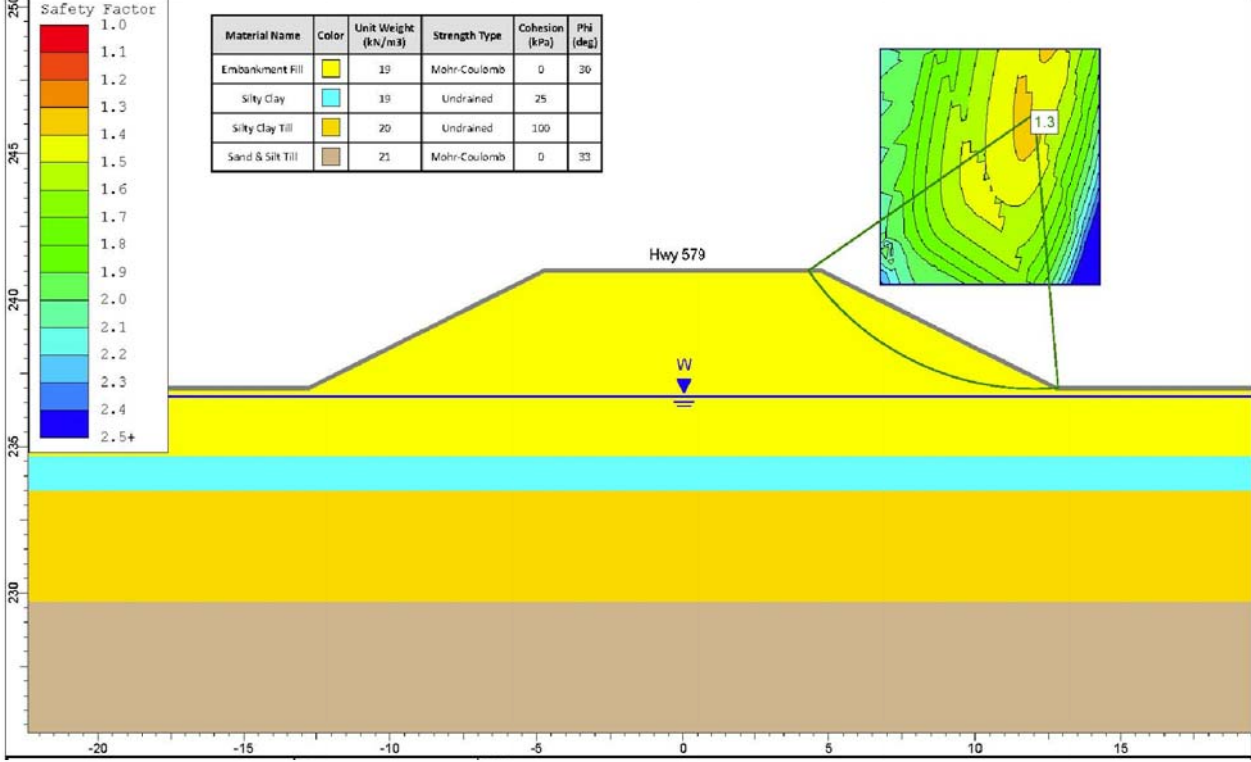
APPENDIX C

Slope Stability Analysis

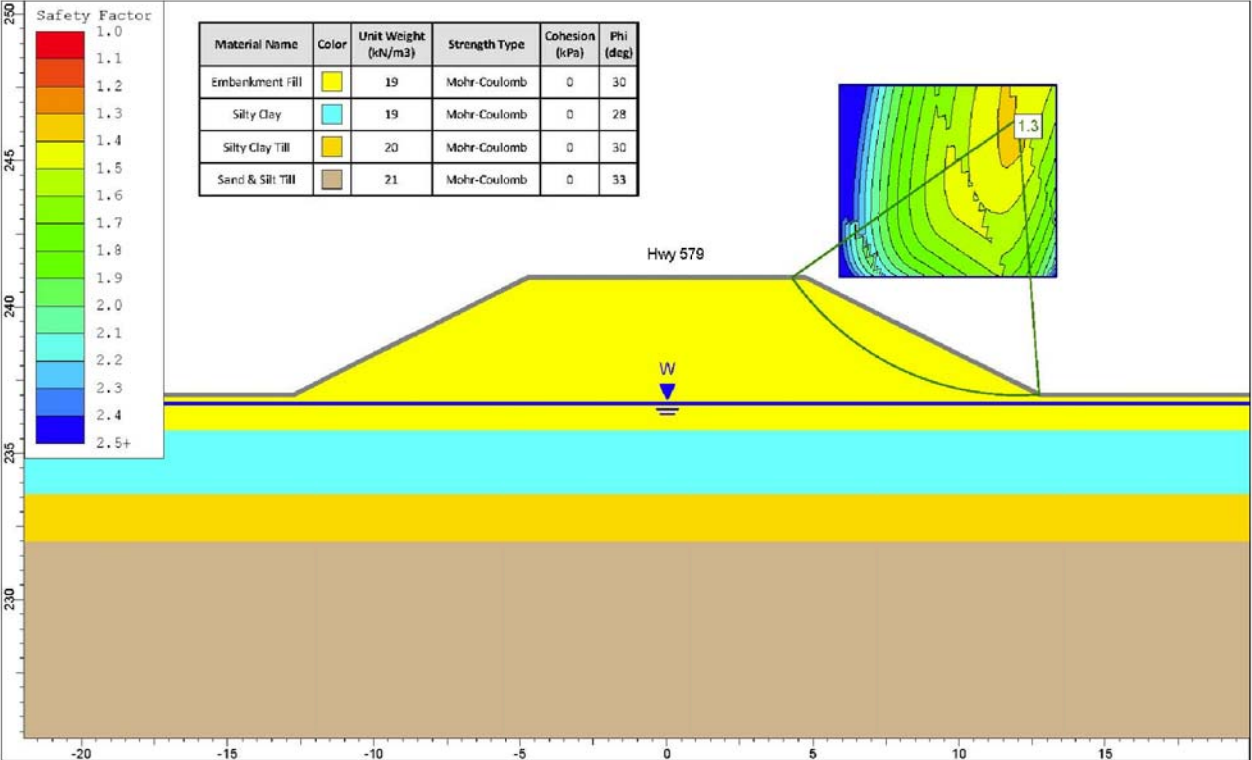




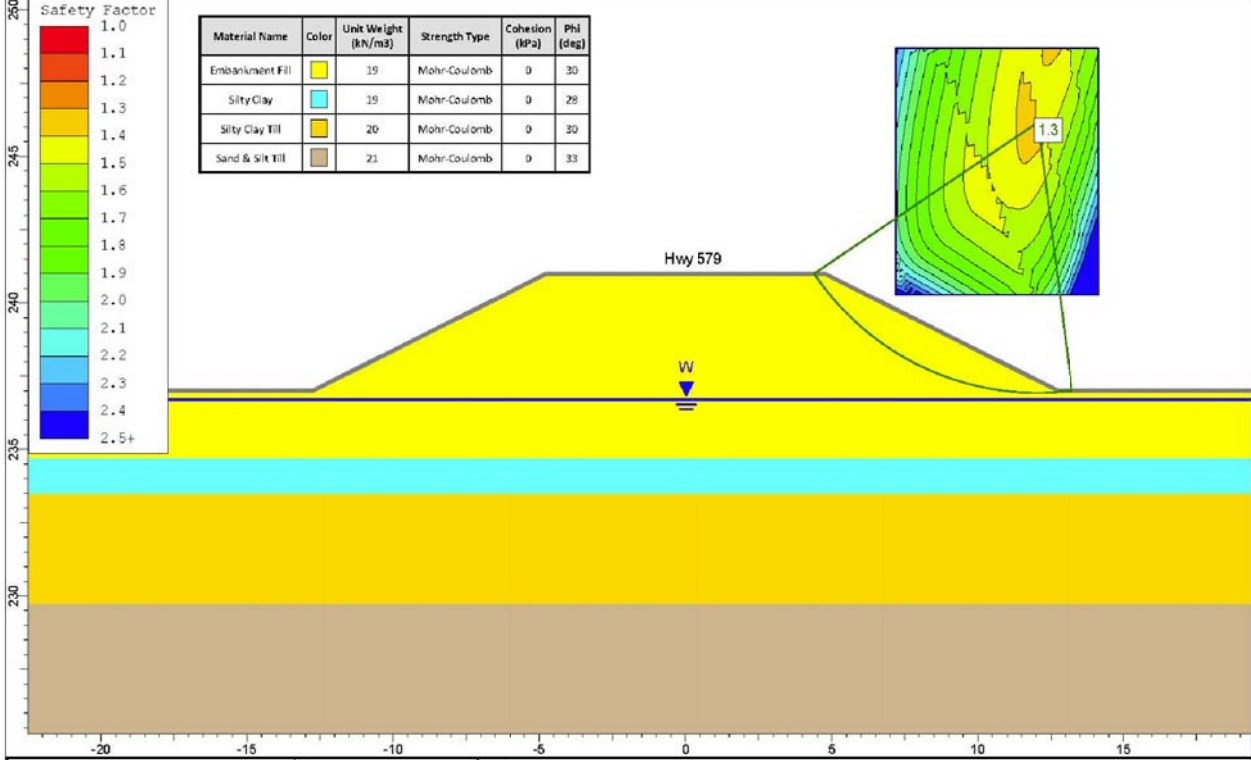
South Approach Embankment, Section A-A'-Total Stress Analysis




North Approach Embankment, Section B-B'-Total Stress Analysis



South Approach Embankment, Section A-A'-Effective Stress Analysis



North Approach Embankment, Section B-B'-Effective Stress Analysis



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HWY 579
GILLES CREEK BRIDGE, SITE '39E-006

G.W.P 5368-11-00	DATE: December 2014
SUBM'D. AA	CHKD. RA
Project No: 11-14-4066	Figure C1