



# Terraprobe

*Consulting Geotechnical & Environmental Engineering  
Construction Materials Inspection & Testing*

**FOUNDATION INVESTIGATION & DESIGN REPORT  
MONTCALM CREEK BRIDGE REPLACEMENT  
HIGHWAY 11, 6.8 KM WEST OF OPASATIKA  
G.W.P. No. 5233-06-00, W.P. 5146-05-01, SITE 39W-058  
GEOCRES No. 42G-36  
MINISTRY OF TRANSPORTATION, ONTARIO  
NORTHEASTERN REGION**

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HIGHWAY 11  
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GEOCRES No. 42G-36  
PART 1: FACTUAL INFORMATION**

## **1 INTRODUCTION**

This report presents the factual findings obtained from foundation investigations conducted at the Montcalm Creek Bridge site where a bridge replacement and a detour structure are proposed. The site is located on Highway 11, 6.8 km west of Opatatika in the Township of McCrea; District of Cochrane, Ontario.

The purpose of this investigation was to explore the subsurface conditions at this site and, based on the data obtained, to provide borehole location plans, records of boreholes, stratigraphic profiles, laboratory test results and descriptions of the subsurface conditions. Models of the subsurface conditions were developed from the data obtained.

Terraprobe conducted the investigation as a sub-consultant to McCormick Rankin, a Member of MMM Group Ltd., (MRC) under the Ministry of Transportation Ontario (MTO) Northeastern Region Assignment Number 5009-E-0020.

The results of a preliminary foundation investigation carried out at the site were presented in the following report:

- Preliminary Foundation Investigation & Design Report, Montcalm Creek Bridge Replacement, Highway 11, 6.8 km West of Opatatika, G.W.P. No. 5233-06-00, W.P. 5146-05-01, Site 39W-058, Geocres No. 42G-32, dated March 02, 2011.

This report contains information from the above referenced report as well as additional subsurface information that has been subsequently obtained.

A Pavement Design Report which addressed pavement widening and the detour pavement requirements at this site is reported under separate cover.

## **2 SITE DESCRIPTION & PHYSIOGRAPHY**

Highway 11 crosses Montcalm Creek via an 11.5 m wide five span timber bridge measuring about 23 m in length. At this site Highway 11 is a two-lane highway with partially paved shoulders





carrying east and west bound traffic. An ONR (Ontario Northern Railway) track runs parallel to Highway 11 and is located approximately 45 m south of Highway 11 centreline.

Montcalm Creek flows from north to south meandering gently within a well-defined flood plain. The terrain is generally flat and within the flood plain area vegetation 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.

The study area is located in northeastern Ontario. Recent deposits consist of peat, gravel, sand, clay and till soils. The area is underlain by supracrustal rocks composed of metavolcanics, their intrusive equivalents and metasediments of Precambrian age.

### **3 SITE INVESTIGATION AND FIELD TESTING**

The site investigation and field testing for this project was carried out in two phases. Four boreholes, designated as MC1 to MC4 inclusive were drilled at the preliminary design stage between July 20 and August 10, 2010. The preliminary work plan consisted of two boreholes (MC1 & MC2) drilled at the existing bridge site and two boreholes (MC3 & MC4) drilled at the site of the proposed detour structure. The second phase of the investigation was carried out between October 17 and November 04, 2011 and consisted of drilling and sampling six additional boreholes, designated MCD1 to MCD6 inclusive. Boreholes MCD1 and MCD2 were drilled at the existing bridge and boreholes MCD3 and MCD4 were drilled in the approaches to the existing bridge. Boreholes MCD5 and MCD6 were drilled in the approaches to the temporary bridge. The locations of the boreholes are shown on the attached Borehole Locations and Soil Strata Drawing in Appendix C.

A series of shallow boreholes were also drilled at the site for the design of the roadway pavements for the detour. The results of the shallow boreholes are presented in the Pavement Design Report.

Samples of the overburden soils were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT), as specified in ASTM Method D1586. In the cohesive deposits the undrained shear strength of the soil was measured using an MTO type field vane. Relatively undisturbed soil samples were also collected with thin-walled Shelby Tube samplers. The boreholes at the abutments were also advanced into bedrock using NQ size diamond coring techniques.

Ground water conditions in the open boreholes were observed throughout the drilling operations. The boreholes were also instrumented with standpipe piezometers consisting of 25 mm diameter PVC pipe with a slotted screen enclosed in sand to permit longer term ground water level monitoring. The locations and completion details of the piezometers are outlined in Table 3.1. The piezometers were decommissioned between April 26 and 30, 2012.

The drilling, sampling and coring operations were observed on a full time basis by a member of Terraprobe's technical staff who logged the boreholes and rock cores and prepared the recovered





soil and rock samples for transport to Terraprobe's Brampton laboratory for further examination and testing.

**Table 3.1 – Piezometer Installation Details**

Piezometer Location	Piezometer Details	
	Tip Depth/ Elevation (m)	Completion Details
MC1	19.8/216.6	Piezometer with 1.5 m slotted screen installed with filter sand to 16.8 m, bentonite seal from 16.8 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
MC2	16.8/219.6	Piezometer with 1.5 m slotted screen installed with filter sand to 16.8 m, bentonite seal from 16.8 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
MC3	18.3/217.0	Piezometer with 1.5 m slotted screen installed with filter sand to 16.2 m and bentonite seal from 16.2 m to ground surface.
MC4	16.8/218.8	Piezometer with 1.5 m slotted screen installed with filter sand to 14.6 m and bentonite seal from 14.6 m to ground surface.
MCD1	18.0/218.5	Piezometer with 3.0 m slotted screen installed with filter sand to 14.0 m, bentonite seal from 14.0 m to 13.1 m, drill cuttings from 13.1 m to 0.9 m and a concrete encased flush mount cover from 0.9 m to ground surface.
MCD2	17.1/219.4	Piezometer with 3.0 m slotted screen installed with filter sand to 13.4 m, bentonite seal from 13.4 m to 12.5 m, drill cuttings from 12.5 m to 0.9 m and a concrete encased flush mount cover from 0.9 m to ground surface.
MCD3	7.9/228.5	Piezometer with 3.0 m slotted screen installed with filter sand to 4.3 m, bentonite seal from 4.3 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
MCD4	8.5/228.0	Piezometer with 3.0 m slotted screen installed with filter sand to 4.9 m, bentonite seal from 4.9 m to 4.3 m, drill cuttings from 4.3 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
MCD5	8.8/226.9	Piezometer with 3.0 m slotted screen installed with filter sand to 5.2 m, bentonite seal from 5.2 m to 4.6 m, drill cuttings from 4.6 m to 0.3 m and bentonite seal from 0.3 m to ground surface.
MCD6	10.0/225.7	Piezometer with 3.0 m slotted screen installed with filter sand to 6.1 m, bentonite seal from 6.1 m to 5.5 m, drill cuttings from 5.5 m to 0.6 m and bentonite seal from 0.6 m to ground surface.

#### 4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and water content determination. Selected samples were also subjected to a laboratory testing programme consisting of gradation analysis, Atterberg Limits tests, consolidation tests, unit weight and undrained shear strength testing with a laboratory vane. The results of this testing program are shown on the Record of Borehole sheets in Appendix A and on the figures in Appendix B.

#### 5 DESCRIPTION OF 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 Appendix A and on the "Borehole Locations and Soil Strata"





drawings in Appendix C. The stratigraphic boundaries shown have been inferred from non-continuous samples and observations of drilling resistance and typically represent a transition from one soil or rock type to another. These boundaries should not be interpreted to represent exact planes of geological change. The subsurface conditions are confirmed at the borehole locations only, and will vary between and beyond the locations investigated. The following discussion has been simplified in terms of the major soil strata.

## **5.1 Existing Bridge Site (Boreholes MC1, MC2, MCD1, MCD2, MCD3 & MCD4)**

In general, the site was underlain by flexible pavement (asphalt and sand and gravel), sand and silty sand fill, silty clay fill and deposits of silty clay to clayey silt, sandy silt till, clayey silt to silty clay till and cobbles and boulders. The overburden was underlain by bedrock consisting of igneous granitoid.

### **5.1.1 Flexible Pavement**

A flexible pavement comprising of 130 mm to 200 mm thick of asphaltic concrete underlain by a layer of sand and gravel ranging in thickness from 150 mm to 410 mm was encountered. The granular fill extended to elevations ranging from 235.9 m to 236.2 m below ground surface.

The grain size distribution plot of a sample of the granular fill is presented in Figure B1-1. These results show a grain size distribution consisting of 50% gravel, 45% sand and 5% silt and clay size particles.

'N' values in the range of 27 to 36 blows for 0.3 m were determined in the Standard Penetration Testing carried out in the granular fill, inferring a compact to dense relative density. The water content of samples of the granular fill ranged from 3% to 5% by weight.

### **5.1.2 Fill – Sand to Silty Sand**

Fill consisting of sand and silty sand with clay lumps was encountered beneath the pavement and to depths ranging from 1.9 m (Elev. 234.5 m) to 3.4 m (Elev. 233.1 m) below ground surface.

The grain size distribution plots of samples of the sand to silty sand fill are presented in Figure B1-2. These results show a grain size distribution consisting of 0-6% gravel, 81-88% sand and 6-19% silt and clay size particles.

N values in the range of 2 to 23 blows for 0.3 m were determined in the fill, indicating a very loose to compact relative density. The moisture content of samples of this fill ranged from about 5% to 28%.

### **5.1.3 Fill – Silty Clay**

Fill consisting of silty clay and organics was encountered in borehole MC2 extending to a depth of 3.7 m (Elev. 232.7 m) below ground surface.





A single N value of 14 blows per 0.3m was determined in the silty clay fill, inferring a relatively stiff consistency. The moisture content of the sample of silty clay fill recovered from the penetration testing was 51%.

#### 5.1.4 Silty Clay to Clayey Silt

A silty clay to clayey silt deposit was encountered beneath the fill and to depths ranging from 7.1 m (Elev. 229.4 m) to 7.5 m (Elev. 228.9 m) below ground surface.

The grain size distribution curves of samples of the silty clay to clayey silt are presented in Figures B1-3 and B1-4. These results show a grain size distribution consisting of 0-4% gravel, 4-24% sand, 36-62% silt and 21-60% clay size particles.

Samples were also subjected to Atterberg Limits tests and the results are shown on the plasticity chart, Figures B1-5 and B1-6. The index values from these tests are summarized below:

Liquid Limit:	19-38%
Plastic Limit:	13-21%
Plasticity Index:	6-17%
Natural Moisture Content:	12-30%

These values indicate that the deposit can be characterized as low plasticity silty clay to clayey silt.

The N values determined in the silty clay to clayey silt ranged from 4 to 24 blows for 0.3 m penetration. Field vane shear tests indicated undrained shear strengths ranging from 36 kPa to greater than 100 kPa. A laboratory vane test on a relatively undisturbed Shelby tube sample gave undrained shear strength of 72 kPa. Based on these results the silty clay to clayey silt was generally firm to stiff with some very stiff zones. The moisture content of samples of the silty clay to clayey silt ranged from 11% to 33% and the unit weight of a tested sample was 21.9 kN/m<sup>3</sup>.

A one dimensional consolidation test was carried out on a tube sample of the silty clay to clayey silt deposit from Borehole MCD4 and the results are presented on Figures B1-7 and B1-8. The consolidation characteristics listed in Table 5.1 were determined from the results of the consolidation testing.

**Table 5.1 - Summary of Consolidation Testing on Silty Clay**

Parameter	
Natural water content	14 %
Bulk Unit weight	21.9 kN/m <sup>3</sup>
Dry Unit weight	19.2 kN/m <sup>3</sup>
Compression index	0.114
Recompression index	0.015
Void ratio	0.40
Preconsolidation Pressure	70 kPa
Consolidation Coefficient	7.5 m <sup>2</sup> /yr





### **5.1.5 Sandy Silt Till**

A deposit of sandy silt till was encountered beneath the silty clay to clay silt. This deposit was fully penetrated in the deeper boreholes at depths ranging from 11.6 m to 11.7 m below ground surface or at elevations ranging from 224.8 m to 224.9 m. The approach boreholes were terminated in this deposit at depths of 8.7 m (Elev. 227.7 m) and 9.5 m (Elev. 227.0 m).

The results of grain size distribution tests carried out on samples obtained from this till deposit are shown in Figure B1-9. These results show grain size distributions consisting of 1-5% gravel, 25-32% sand, 55-65% silt and 4-13% clay size particles. The high penetration resistance and the resistance to auger advance observed in the boreholes were indications of the presence of cobbles and boulders in this soil matrix.

The N values in the sandy silt till ranged from 13 to more than 100 blows per 0.3 m penetration, indicating a compact to very dense relative density. The natural water content of samples from this stratum ranged from 7% to 12%.

### **5.1.6 Clayey Silt to Silty Clay Till**

A clayey silt to silty clay till deposit was encountered beneath the sandy silt till in Boreholes MC1, MC2, MCD1 and MCD2. This till deposit extended to depths ranging from 17.7 m (Elev. 218.8 m) to 19.8 m (Elev. 216.7 m) below ground surface.

The grain size distribution plots of samples retrieved from this deposit are presented in Figure B1-10. These results show a grain size distribution consisting of 1-6% gravel, 18-34% sand, 45-61% silt and 17-26% clay size particles. Cobbles and boulders were also thought to have been encountered in the clayey silt to silty clay till.

The results of Atterberg Limits determinations on samples of the clayey silt to silty clay till are presented in Figure B1-11 and summarized below:

Liquid Limit:	19-23%
Plastic Limit:	12-14%
Plasticity Index:	6-11%
Natural Moisture Content:	9-11%

These values indicate that the clayey silt to silty clay till was of relatively low plasticity.

The N values in the clayey silt to silty clay till ranged from 34 to more than 100 blows for 0.3 m penetration indicating a hard consistency. The natural water content of samples of the clayey silt to silty clay till was in the range of 8% to 12%.

### **5.1.7 Cobbles and Boulders**

In Boreholes MC1 and MCD2, a formation comprised of mostly cobbles and boulders was encountered overlying the bedrock surface. This stratum extended to depths ranging from 20.3 m (Elev. 216.2 m) to 22.0 m (Elev. 214.5 m) below ground surface.





A Standard Penetration test in this layer gave an 'N' value of more than 100 blows for 0.3 m penetration, which indicates that this deposit had a very dense relative density.

#### 5.1.8 Bedrock

The overburden described above was underlain by igneous granitoid bedrock. Bedrock was proved by coring at the abutment locations and the bedrock depth and elevations to the top of bedrock are summarized in Table 5.2.

**Table 5.2 – Depth to Bedrock**

<b>BH No.</b>	<b>Depth to Bedrock (m)</b>	<b>Top of Bedrock Elevation (m)</b>
MC1	22.0	214.5
MC2	18.5	217.9
MCD1	18.0	218.5
MCD2	20.3	216.2

In Boreholes MC1, MC2 and MCD2, the bedrock is described as generally highly to moderately weathered at depths extending between 21.2 m (Elev. 215.2 m) and 24.5 m (Elev. 212.0 m). Below these depths the bedrock was described as slightly weathered to unweathered. In Borehole MCD1 the bedrock was described as unweathered and its colour is generally white to grey. Total core recovery in this bedrock ranged from 21% to 100%. The RQD values ranged widely from 0% to 84% but generally, most of the RQD values were below 50%. Based on these results the rock quality is considered to be very poor to poor with occasional zones of fair to good quality rock.

#### 5.2 Detour Alignment (Boreholes MC3, MC4, MCD5 & MCD6)

The alignment is located in an area that has experienced previous construction due to the proximity to the ONR tracks and the highway. In general, the site was underlain by topsoil, fill consisting of sand and gravel and silty clay, and native deposits of clayey silt to silty clay, sandy silt till, clayey silt to silty clay till and cobbles and boulders. The overburden was underlain by bedrock consisting of granite. Boreholes drilled as part of the pavement investigation encountered discontinuous layers of peat beneath the fill.

##### 5.2.1 Topsoil

Topsoil ranging from 130 mm to 150 mm thick was encountered at this site.

##### 5.2.2 Fill – Sand and Gravel/Silty Clay/

A layer of sand and gravel fill approximately 0.4m thick was encountered beneath the topsoil in Borehole MCD5. The sand and gravel fill was underlain by silty clay fill to a depth of 1.4 m (Elev. 234.3 m) below ground surface. Silty clay fill was encountered to depths of 0.7 m (Elev. 235.0 m) to 1.4 m (Elev. 233.9 m) below ground surface in Boreholes MC3, MC4 and MCD6.





The grain size distribution curve of a sample of silty clay fill is shown in Figure B2-1. These results show a grain size distribution consisting of 0% gravel, 6% sand, 38% silt and 56% clay size particles.

A sample was also subjected to an Atterberg Limits test and the results are presented in Figure B2-2. The index values from these tests are summarized below:

Liquid Limit:	51%
Plastic Limit:	25%
Plasticity Index:	26%
In-situ Moisture Content:	29%

N values in the range of 5 to 11 blows for 0.3 m were determined in the fill, indicating a firm to stiff consistency. The moisture content of samples of this fill ranged from about 15% to 29%.

### **5.2.3 Clayey Silt to Silty Clay**

Native clayey silt to silty clay deposits were encountered in all of the boreholes and to depths ranging from 6.4 m to 8.6 m below ground surface or to elevations ranging from 229.3 m to 227.1 m.

The grain size distribution plots of samples of the clayey silt to silty clay are presented in Figures B2-3 and B2-4. These results show a grain size distribution consisting of 0-4% gravel, 4-33% sand, 25-57% silt and 17-40% clay size particles.

Samples were also subjected to Atterberg Limits tests and the results are illustrated on the plasticity chart, Figures B2-5 and B2-6. The index values from these tests are summarized below:

Liquid Limit:	17-69%
Plastic Limit:	13-30%
Plasticity Index:	3-39%
Natural Moisture Content:	13-35%

These values indicate that the deposit generally consisted of low plasticity clayey silt to silty clay.

The N values determined in the clayey silt to silty clay ranged from 5 to 26 blows for 0.3 m penetration. A field vane shear test gave undrained shear strength of 52 kPa. These values indicate that the consistency of the clayey silt to silty clay was in the firm to very stiff range. The moisture content of samples of the clayey silt to silty clay ranged from 11% to 40%.

### **5.2.4 Sandy Silt Till**

Sandy silt till was encountered across this site extending to depths ranging from 10.1 m to 15.7 m below ground surface or to elevations ranging from 219.9 m to 225.2 m. The approach boreholes were terminated in this deposit at depths of 9.2 m (Elevation 226.5 m) and 10.5 m (Elevation 225.2 m).

The results of grain size distribution tests conducted on samples of this till are illustrated in Figure B2-7. These results show grain size distributions of 2-5% gravel, 25-36% sand, 48-55% silt





and 7-19% clay size particles. The high penetration resistance and the resistance to auger advance observed in the boreholes were indications of the presence of cobbles and boulders in this soil matrix.

The N values determined in the sandy silt till ranged from 13 to more than 100 blows per 0.3 m penetration indicating a compact to very dense relative density. The moisture content of samples from this stratum ranged from about 7 to 15%.

#### **5.2.5 Clayey Silt to Silty Clay Till**

Clayey silt to silty clay till was encountered beneath the sandy silt till in Borehole MC3 and to a depth of 15.5 m (Elev. 219.8 m) below ground surface.

The grain size distribution plots of samples of this till deposit are presented in Figure B2-8. These results show a grain size distribution consisting of 3-12% gravel, 25-27% sand, 46-58% silt and 12-17% clay size particles. The high penetration resistance and the resistance to auger advance observed in the boreholes were indications of the presence of cobbles and boulders in this soil matrix.

Samples of the clayey silt to silty clay till were also subjected to Atterberg Limits tests and the results are presented in Figure B2-9. The index values from these tests are summarized below:

Liquid Limit:	18-19%
Plastic Limit:	13-14%
Plasticity Index:	4-6%
Natural Moisture Content:	9%

These values indicate low plasticity clayey silt soils.

The N values determined in the clayey silt to silty clay till were more than 50 blows for 0.3 m penetration indicating a hard consistency. The moisture contents of samples of the clayey silt to silty clay till ranged from 9% to 11%.

#### **5.2.6 Cobbles and Boulders**

A stratum with frequent cobble and boulders was encountered in Boreholes MC3 and MC4 overlying the bedrock surface. This layer extended to depths ranging from 18.4 m to 19.0 m below ground surface or to elevations ranging from 216.3 m to 217.2 m. This stratum was inferred to have a very dense relative density.

#### **5.2.7 Bedrock**

The overburden was underlain by igneous granite bedrock. Bedrock was proved by coring in both abutment boreholes and the bedrock depths and top of bedrock elevations are summarized in Table 5.3.





**Table 5.3 – Depth to Bedrock**

BH No.	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
MC3	19.0	216.3
MC4	18.4	217.2

In Borehole MC3 the bedrock has been described as moderately weathered to a depth of 19.9 m (Elev. 215.4 m) and is unweathered below. In Borehole MC4 the bedrock was slightly weathered. The colour of the bedrock was white to grey. Total core recovery in the bedrock ranged from 89% to 100%. RQD values ranged from 0% to 70% but generally, the recorded RQD values ranged from 32% to 70%. Based on these results, the rock quality was considered to be generally poor to fair with occasional zones of very poor rock.

### 5.3 Water Levels

Standpipe piezometers were installed in the boreholes and the water level readings were measured on separate visits made after the completion of drilling. The water level records are presented in Table 5.4.

**Table 5.4 – Water Level Measurements**

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
Existing Bridge Site			
MC1	August 06, 2010	1.6	234.9
	August 10, 2010	1.6	234.9
	September 03, 2010	1.6	234.9
	April 26, 2012	1.3	235.2
MC2	August 06, 2010	1.4	235.0
	August 10, 2010	1.5	234.9
	September 03, 2010	1.5	234.9
	April 26, 2012	1.3	235.1
MCD1	November 08, 2011	1.4	235.1
	December 13, 2011	1.3	235.2
	April 26, 2012	1.1	235.4
MCD2	November 08, 2011	0.6	235.9
	December 13, 2011	0.8	235.7
	April 26, 2012	0.4 (frozen)	236.1
MCD3	November 03, 2011	1.4	235.0
	November 08, 2011	1.3	235.1
	December 13, 2011	1.2	235.2
	April 26, 2012	Damaged	
MCD4	November 03, 2011	1.5	235.0
	November 08, 2011	1.3	235.2
	December 13, 2011	1.2	235.3
Detour Alignment			
MC3	August 10, 2010	0.5	234.8
	September 03, 2010	0.5	234.8
	April 26, 2012	Damaged	-
MC4	August 10, 2010	0.4	235.2
	August 31, 2010	1.6	234.0
	September 01, 2010	1.6	234.0
	April 26, 2012	0.3	235.3





MCD5	November 03, 2011	0.8	234.9
	November 08, 2011	0.8	234.9
	December 13, 2011	0.5	235.2
	April 26, 2012	0.4	235.3
MCD6	November 03, 2011	0.5	235.2
	November 08, 2011	0.2	235.5
	December 13, 2011	0.4	235.3
	April 26, 2012	0.4	235.3

The free water level in the creek was recorded at Elevation 234.4 m in August, 2010 indicating that the ground water table exists just below the ground surface in the flood plain area.

At the existing bridge the piezometric head was estimated to range between Elevation  $\pm 234.9$  m and Elevation  $\pm 235.7$  m. Along the detour alignment the recorded water levels ranged between Elevations  $\pm 234.0$  and  $\pm 235.3$  m.

All groundwater observations at this site were short term and the levels are expected to fluctuate seasonally and with precipitation conditions. The ground water level may also be affected by the free water level in the creek.

#### 5.4 Miscellaneous

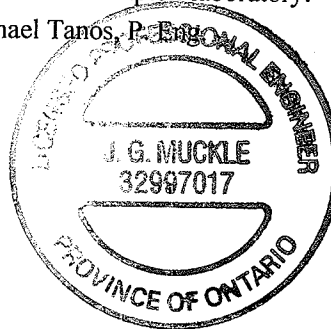
The borehole locations were marked in the field by surveyors from MRC who also provided Terraprobe with their coordinates and geodetic elevations. Terraprobe obtained utility clearances and permits prior to drilling.

The drilling, sampling and in-situ testing operations, piezometer installation and decommissioning were conducted with a track mounted drill rig owned and operated by Landcore Drilling of Chelmsford, Ontario.

The boreholes were advanced using hollow-stem augers and casing and washboring methods. Rock cores were retrieved by NQ size diamond coring techniques.

Ms. Pari Bofeshnavand, E.I.T., and Mr. Phil Khuu, B.A.T., carried out the field work and the laboratory testing was performed at Terraprobe's Brampton laboratory. The report was written by J. G. Muckle, P. Eng. and reviewed by Michael Tanos, P. Eng.

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**FOUNDATION DESIGN REPORT**  
**MONTCALM CREEK BRIDGE REPLACEMENT**  
**HIGHWAY 11**  
**6.8 KM WEST OF OPASATIKA**  
**MINISTRY OF TRANSPORTATION, ONTARIO**  
**G.W.P. No. 5233-06-00, W.P. 5146-05-01, SITE 39W-058**  
**GEOCRES No. 42G-36**  
**PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS**

**6 GENERAL**

This report presents interpretation of the geotechnical data in the factual report and provides geotechnical design recommendations for a replacement bridge and a temporary detour structure at Montcalm Creek located on Highway 11, 6.8 km west of Opasatika in the Township of McCrea; District of Cochrane, Ontario.

An existing five span timber bridge measuring approximately 23 m in length and about 11.5 m wide carries Highway 11 east bound and west bound traffic over Montcalm Creek. This bridge will be replaced with a new single span bridge. A temporary detour will be constructed south of Highway 11 to maintain traffic during construction of the new Highway 11 bridge. A single lane single span temporary modular bridge will be constructed on Montcalm Creek on the detour alignment. Final construction will consist of removing the bridge and restoration work.

The new highway structure will consist of a single span bridge approximately 14.1 m wide and 19.3 m in length between abutments. The proposed finished grades at the structure will be Elevation 237.195 at the west abutment and 237.070m at the east abutment. At the east and west abutments the approaches will be about 0.5 and 0.4m respectively above the existing grades. Highway 11 will be widened slightly on both sides to accommodate the new structure

The detour structure will be a single span temporary modular bridge about 7m in width and 21m in length. The proposed finished grades on the temporary bridge will be 237.150 over both abutments. Approach fills of up to about 2.2m in height will be required to achieve these grades.

The following discussion and recommendations presented in this report are based on our understanding of the project and on the factual data obtained in the course of the preliminary and final investigations.





## **7 STRUCTURE FOUNDATIONS**

### ***Existing Bridge Site (Boreholes MC1, MC2, MCD1 & MCD2)***

The stratigraphy encountered at this site generally consisted of a flexible pavement (asphalt and sand and gravel), fill and native deposits of silty clay to clayey silt, sandy silt till, clayey silt to silty clay till and cobbles and boulders. These overburden soils extended to depths of 18.0 to 22.0m or between Elevations 214.5 and 218.5m. The boreholes were terminated in granite bedrock. The ground water level at this site was measured between Elevations 234.9 and 235.9m.

### ***Detour Alignment (Boreholes MC3 & MC4)***

The stratigraphy encountered at the detour bridge consisted of topsoil, fill and native deposits of clayey silt to silty clay, sandy silt till, clayey silt to silty clay till and cobbles and boulders. These overburden soils extended to depths of 19.0 and 18.4 m or to Elevations 216.3 and 217.2m in Boreholes MC3 and MC4 respectively. Both boreholes were terminated in granite bedrock. The ground water level at this site was measured at just below the flood plain level or at about Elevation 235.0m.

Consideration was given to the following foundation types:

- Spread footings
- Augered Caissons (drilled shafts)
- Driven piles

A comparison of the foundation alternatives based on advantages and disadvantages of each is included in Appendix D.

### **7.1 Spread Footings**

Spread footings were not considered to be a practical alternative for supporting either of the bridges. The geotechnical resistances of the underlying soils are low and foundation settlements will be high. Consequently the spread footing alternative was not recommended.

Consideration was also given to supporting spread footings on an engineered fill however this alternative was not considered feasible due to the depths of excavation required, combined with the potentially difficult ground water conditions.

### **7.2 Augered Caissons (Drilled Shafts)**

Augered caisson foundations were also considered for supporting the structure. However, the caissons would need to penetrate the very dense or hard till soils containing cobbles and boulders. It would be difficult to seal the bottom of the liner to exclude ground water due to the permeable nature of the overburden soils and the presence of cobbles and boulders. Unwatering the caissons and maintaining a sufficiently dry excavation to permit cleaning, inspection and high quality construction would also be challenging and impractical.

Given the foregoing, caisson foundations were not recommended for supporting the structures.





### 7.3 Driven Piles

The subsurface conditions at the site were considered suitable for the design of foundations supported on steel H-piles. Furthermore, the existing bridge is supported on pile foundations that have provided reliable performance. Therefore, a similar foundation scheme will have a high probability of providing reliable performance and the risk will be low.

High displacement piles such as close end steel tube piles were considered they were excluded in favour of low displacement H piles. Open ended steel pipe piles with toes strengthened using cruciform plates and/or pile points may be considered feasible for this site; however recognizing the high risk of damage due to the presence of cobbles and boulders in the till soils H-piles were preferred.

Steel H-piles will be driven to practical refusal in till soils or to bedrock at the foundation elements. However, the till matrix contains cobbles and boulders and piles may encounter effective refusal in this stratum without reaching the design tip elevations.

The existing structure is supported on approximately 10.5 m long timber piles that are likely bearing on the underlying sandy silt till which is not prone to liquefaction. Consequently, pile driving operations at the detour bridge are unlikely to 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 superstructure and a pre-construction survey of the structure and monitoring during construction is recommended.

#### 7.3.1 Axial Resistance

Two steel pile sections have been considered for use in the proposed foundations. Piles driven at the abutment locations and encountering effective refusal in the very dense sandy silt till, hard clayey silt to silty clay till, very dense cobbles and boulders and on bedrock should be designed on the basis of the concentric, axial geotechnical resistances given in Table 7.1. The structural resistance of the pile should be checked by the structural designer.

**Table 7.1 – Tip Elevations of Various Pile Sections**

Location	PILE TYPE - HP 310x110				
	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
Existing Bridge Site					
West Abutment	MC1	226.5±	Sandy Silt Till	1600	1200
	MCD2	226.5±			
East Abutment	MC2	226.5±	Sandy Silt Till		
	MCD1	226.5±			
Detour Alignment					
West Abutment	MC3	225.0±	Clayey Silt/Silty Clay Till	1600	1200
East Abutment	MC4	225.5±	Sandy Silt Till		





**Table 7.1 – Tip Elevations of Various Pile Sections**

Location	PILE TYPE – HP 360X132				
	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
Existing Bridge Site					
West Abutment	MC1	215.5±	Cobbles and Boulders	2100	1600*
	MCD2	219.0±			
East Abutment	MC2	217.9±	Bedrock - Granite		
	MCD1	219.0±			
Detour Alignment					
West Abutment	MC3	224.0±	Clayey Silt/Silty Clay Till	2100	1600
East Abutment	MC4	224.5±	Sandy Silt Till		

\* SLS Condition will not apply for piles driven to bedrock but a reduced geotechnical resistance of 1600 kN is recommended since some piles will encounter refusal in cobbles and boulders overlying bedrock.

The H-piles for the recommended foundation scheme will be driven to effective refusal in the overburden soils or to bedrock. Piles will penetrate till layers that contain cobbles and boulders. Given these aggressive driving conditions it is recommended that the pile tips be fitted with rock points to provide increased cutting ability and reinforcement to the pile section.

### 7.3.2 Downdrag

The grade raise at the existing bridge site on Highway 11 will be approximately 0.5 m. However, to accommodate the integral abutment construction a 3.0 m long CSP will surround the pile in the silty clay to clayey silt stratum. Consequently downdrag forces on the piles due to embankment reconstruction and the grade raise will be minimal.

Along the detour alignment, embankment construction will cause settlement of the underlying soils thereby imparting downdrag forces on piles that are installed before the embankments are constructed. Downdrag forces on piles were estimated based on compressible silty clay soils that extend to Elev. 228.5 m. Unfactored downdrag loads of 200 kN/pile (HP 310 x 110 section) and 235 kN/pile (HP 360 x 132 section) are recommended for design purposes.

### 7.3.3 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 the integral abutment design procedures.

After the pile is driven, the space between the pile and the CSP should be filled with sand. An NSSP should be included in the contract drawings specifying the gradation of the sand according to Table 7.2.





**Table 7.2 – 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%

### 7.3.4 Lateral Resistance

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

$$k_s = n_h \cdot z / D \text{ [cohesionless soils] (kN/m}^3\text{)}$$

$$k_s = 67 S_u / D \text{ [cohesive soils] (kN/m}^3\text{)}$$

$$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p \text{ [cohesionless soils] (kPa)}$$

$$p_{ult} = 9 S_u \text{ [cohesive soils] (kPa)}$$

where  $z$  = depth of embedment of pile (m)

$D$  = pile width (m)

$S_u$  = undrained shear strength (Table 7.3) (kPa)

$n_h$  = coefficient of horizontal subgrade reaction (Table 7.3) (kN/m<sup>3</sup>)

$\gamma$  = unit weight (Table 7.3) (kN/m<sup>3</sup>)

$K_p$  = passive earth pressure coefficient

The above equations and recommended parameters may be used to analyse 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. The horizontal passive resistance for the two pile sections under consideration are shown in Table 7.3.

**Table 7.3 – Passive Resistance of Pile Sections**

Pile Section	Passive Resistance ULS (kN)	Passive Resistance SLS (kN)
HP 310x110	120	50
HP 360x132	170	70

The spring constant,  $K$ , for analysis may be obtained by the expression,  $K = k_s \times L \times D$  (kN/m), where  $k_s$  is the coefficient of horizontal subgrade reaction (kN/m<sup>3</sup>),  $D$  is the pile width (m) and  $L$  is the length (m) of the pile segment or element used in the analysis. The ultimate lateral resistance,  $P_{ult}$ , may be obtained from the expression,  $P_{ult} = p_{ult} \times L \times D$ .





**Table 7.4 – Recommended Soil Parameters**

Area Reference Borehole No	Applicable Elevation	Soil Type	Bulk Unit Weight (kN/m <sup>3</sup> )	Angle of Internal Friction (φ) Degrees	Undrained Shear Strength (S <sub>u</sub> ) (kPa)	Recommended n <sub>h</sub> Value (kN/m <sup>3</sup> )*
<b>Existing Bridge Site</b>						
West Abutment BH MC1	235.9– 234.4	Fill – Sand	19	28	–	1300
	234.4 – 231.5	Silty Clay to Clayey Silt	19	0	40	–
	231.5 – 229.4	Silty Clay to Clayey Silt	19	0	75	–
	229.4 – 228.0	Sandy Silt Till	20	35	–	4400
	228.0 – 224.9	Sandy Silt Till	20	35	–	11000
	224.9 – 216.7	Clayey Silt/Silty Clay Till	20	0	225	–
	216.7 – 214.5	Cobbles and Boulders	19	35	–	11000
West Abutment BH MCD2	235.9 - 234.4	Fill – Sand	19	28	-	1300
	234.4 – 231.0	Silty Clay to Clayey Silt	19	0	40	-
	231.0 – 229.4	Silty Clay to Clayey Silt	19	0	75	-
	229.4 – 228.0	Sandy Silt Till	20	35	-	4400
	228.0 – 224.8	Sandy Silt Till	20	35	-	11000
	224.8 – 218.8	Clayey Silt Till	20	0	225	-
	218.8 – 216.2	Cobbles and Boulders	19	35	-	11000
East Abutment BH MC2	235.0 – 234.3	Fill – Sand	19	28	–	1300
	234.3 – 233.5	Fill – Silty Sand	18.5	27	–	1300
	233.5 – 232.7	Fill – Silty Clay	19	0	40	–
	232.7 – 231.0	Silty Clay	19	0	40	–
	231.0 – 229.3	Silty Clay	19	0	100	–
	229.3 – 227.5	Sandy Silt Till	20	35	–	4400
	227.5 – 224.8	Sandy Silt Till	20	35	–	11000
	224.8 – 217.9	Clayey Silt/Silty Clay Till	20	0	225	–
East Abutment BH MCD1	236.0 – 234.4	Fill – Sand	19	28	-	1300
	234.4 – 233.1	Fill – Sand	18.5	0	-	4400
	223.1 – 230.7	Silty Clay to Clayey Silt	19	0	50	-
	230.7 – 229.4	Silty Clay to Clayey Silt	19	0	75	-
	229.4 – 227.4	Sandy Silt Till	20	35	-	-
	227.4 – 224.8	Sandy Silt Till	20	35	-	4400
	224.8 – 218.5	Clayey Silt Till	19	0	225	11000
<b>Detour Alignment</b>						
West Abutment BH MC3	235.2 – 233.9	Fill – Silty Clay	19	0	30	–
	233.9 – 228.2	Clayey Silt/Silty Clay	19	0	50	–
	228.2 – 226.5	Sandy Silt Till	20	30	–	4400
	226.5 – 225.2	Sandy Silt Till	20	35	–	11000
	225.2 – 219.8	Clayey Silt Till	20	0	225	–
	219.8 – 216.3	Cobbles and Boulders	19	35	–	11000
East Abutment BH MC4	235.5 – 234.9	Fill – Silty Clay	19	0	50	–
	234.9 – 228.5	Silty Clay	19	0	75	–
	228.5 – 227.0	Sandy Silt Till	20	35	–	4400
	227.0 – 219.9	Sandy Silt Till	20	35	–	11000
	219.9 – 217.2	Cobbles and Boulders	19	35	–	11000

\* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3<sup>rd</sup> edition, 1992

Since the piles are end bearing, the 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<sub>s</sub> 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<sub>s</sub> 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 may be obtained by interpolation. For conventional abutments, the lateral resistance may be provided by battered piles.

### 7.3.5 Pile Tips

Due to the presence of cobbles and boulders in the till layers, 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.
- This requirement will provide increased cutting ability to the pile sections and will increase the probability of achieving the desired penetration in competent strata or bedrock.

### 7.3.6 Pile Installation

Pile installation should be in accordance with OPSS 903, November 2009. The Contract Documents should contain a NSSP alerting the Contractor to the presence of cobbles and boulders in the till soils.

### 7.3.7 Pile Driving

Pile driving should be controlled by the Hiley Formula and an ultimate pile resistance to be specified by the designer 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 resistance of “R” kN per pile”. Piles should be driven with a suitable hammer capable of delivering a rated energy of at least 60 kJ/blow, but not more than 70 kJ/blow.





The Ultimate Geotechnical Resistance will be equal to 2 times the Design Load at ULS and must be given by the designer in the Pile Driving notes on the Contract drawings. Based on design pile loads of 280 kN and 950 kN for the detour and main structures, the corresponding ultimate geotechnical resistance values are in Table 7.5.

**Table 7.5 – Ultimate Geotechnical Resistance of Piles**

HP 310X110 Pile	Ultimate Resistance (R) (kN)
Detour Structure	560 kN
Main Bridge	1900 kN

#### **7.4 Recommended Foundation**

From a geotechnical point of view, it is recommended that the new bridge and the temporary modular bridge be supported on steel H-piles.

#### **7.5 Frost Cover**

Pile caps and footings should be provided with a minimum of 2.5 m of earth cover over the footing base (founding elevation). Alternatively the depth of frost cover can be reduced by insulating the pile caps with polystyrene foam insulation of a suitable R-value. For preliminary design purposes assume 25 mm of polystyrene insulation is equivalent to 300 mm of earth cover.

### **8 TEMPORARY SHORING**

The shape of the soil pressure distribution diagram behind a shoring system depends upon the type of soil to be encountered and the amount of movement that can be permitted. The shoring system can be restrained, fixed or flexible. The sequence of work may also 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 ( $K_a$ ). Where limited shoring movement (less than performance Level 1) 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 ( $K_p$ ).

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Shoring should be designed by a licensed Professional Engineer experienced in shoring design. Temporary shoring can be designed for a Performance Level 2, 25 mm maximum horizontal displacement.





The recommended unfactored values of the parameters for use in the design of structures subject to unbalanced earth pressures are given in Table 8.1.

**Table 8.1 - Earth Pressure Coefficients**

Soil	$\phi$ (deg)	$\gamma$ (kN/m <sup>3</sup> )	$K_a$	$K_o$	$K_p$
<b>Existing Bridge Site (Boreholes MC1 &amp; MC2)</b>					
Fill – Sand and Silty Clay	28	19	0.36	0.53	2.77
Fill – Silty Sand	27	18.5	0.38	0.55	2.66
Silty Clay to Clayey Silt	28	19	0.36	0.53	2.77
Sandy Silt Till	35	20	0.27	0.43	3.70
Clayey Silt to Silty Clay Till	28	20	0.36	0.53	2.77
Cobbles and Boulders	35	19	0.27	0.43	3.70
<b>Detour Alignment (Boreholes MC3 &amp; MC4)</b>					
Fill – Silty Clay	28	19	0.36	0.53	2.77
Clayey Silt to Silty Clay	28	19	0.36	0.53	2.77
Sandy Silt Till	35	20	0.27	0.43	3.70
Clayey Silt to Silty Clay Till	28	20	0.36	0.53	2.77
Cobbles and Boulders	35	19	0.27	0.43	3.70

It is envisaged that the shoring could consist of steel sheet piles. Due to the very dense nature of the sandy silt till and the presence of cobbles and boulders, sheet piles will encounter effective refusal in this stratum.

The shoring piles can be designed as cantilevered structures or supported by struts and wales in the case of closed cell cofferdams, or supported by employing a soil anchor system depending on the depth of soil to be retained and the performance criteria. For a soil anchor system the anchors should be grouted in place and should have their bond length formed entirely within the sand and silt till. Temporary soil anchors can be designed based on an unfactored tentative bond resistance (soil to concrete bond value) of 50 kPa in the very dense sand and silt till. Anchor testing, installation and post-grouting should be undertaken in accordance with SP999S26.

For the design of temporary shoring in cohesive silty clay soils, the ultimate horizontal resistance can be estimated as  $4c_u$ , where  $c_u$  is the undrained shear strength of the silty clay in this zone. The undrained shear strength can be taken as a nominal value of 50 kPa.

## 9 EXCAVATION AND BACKFILL

All excavations must be carried out in accordance with the Occupational Health and Safety Act (OHSA). For the purposes of the OHSA, the soils at this site may be classified as follows:

- Fill (Sand, Silty Sand and Silty Clay) – Type 3 soils above the water table and Type 4 soils below the water table.
- Silty Clay – Type 4 soils below the water table.
- Sandy Silt Till – Type 4 soils below the water table.
- Clayey Silt to Silty Clay Till – Type 3 soils below the water table.





Excavation below the ground water level is not recommended without prior dewatering. Provided dewatering is carried out as described below, unsupported excavations may be sloped at 2.5H:1V or flatter.

## **10 GROUND WATER CONTROL**

The free water level in the creek was recorded at Elevation 234.4 m in August, 2010 indicating that the ground water table was generally just below the ground surface in the flood plain area. The recorded water levels in the standpipe piezometers indicate a water level that ranged from Elevation 234 m to Elevation 235.9 m at the time of the field investigations. High hydrostatic pressures were measured in the glacial till that underlies the site.

Excavations at the bridge site may extend into sands below the ground water level. These soils will be easily disturbed by construction activity and will also yield water due to their relatively high permeability. To alleviate construction related problems it is recommended that the ground water table be lowered and maintained at least 1 m below the base of the excavation. Excavations adjacent to the creek may have to be undertaken within a system of closed shoring such as sheet piles in order to minimize the extent of excavation required and to control ground water seepage.

It is anticipated that the sheet piles will penetrate several metres into the firm to very stiff silty clay unit. Significant penetration into the underlying till strata is not considered feasible without damaging the piles. It may be feasible to manage the ground water by pumping from a series of property filtered sumps located as required within the sheeted excavation or cofferdam.

The existing structure is supported on approximately 10.5 m long timber piles that are likely bearing in the underlying sand and silt till. Dewatering of the overlying surficial soils is not expected to cause settlement of the existing structure foundations.

## **11 APPROACH EMBANKMENTS**

### **11.1 Stability**

#### **11.1.1 Highway 11**

The global, internal and surficial stability of the approach embankment fill will depend on the slope geometry and also to a large degree on the material used to construct the embankment. Embankments constructed using non-cohesive earth fill will have stable side slopes at inclinations of up to 2H:1V. If the embankments are constructed with rock fill, it may be assumed that the side slopes will be stable at inclinations up to 1.25H:1V.

For the purpose of embankment stability analyses, the commercially available slope stability program Slide 5.0 developed by Rocscience Inc. was used. The Spencer, Morgenstern-Price and Bishop's simplified method for stability analysis were employed.

Global stability analyses were conducted for 2H:1V earth fill embankments and for 1.25H:1V rock fill embankments and a target factor of safety of 1.3 was set. For earth or rock fill embankments up





to 4.5 m high, factors of safety against global failure of 1.3 and greater were obtained for both long term and short term conditions. Therefore no embankment stability problems are expected. The slope stability models are included in Appendix E.

It is envisaged that mid-height berms would not be incorporated in the designs since the embankments are not expected to reach heights of 8 m (earth fill) or 10 m (rock fill).

#### **11.1.2 Detour Embankments**

The detour will be constructed on an alignment on the south side of the existing bridge. Boreholes drilled along the detour alignment encountered traces of organic material within the fill and in the underlying near surface subgrade soils. Boreholes drilled as part of the investigation for the design of the detour pavement encountered discontinuous layers of peat beneath fill which also contained peat. Traces of peat were also observed in some of the samples recovered from boreholes drilled under the existing highway embankment. It was considered that traces of peat may be remnants of peat that was removed during the original highway construction. It was also apparent that the near surface soils removed during the original construction were deposited adjacent to the highway in the area where the detour will be constructed. Settlement of the detour due to the consolidation of the near surface subgrade soils was an important design consideration for the detour road embankments.

The design approach for the detour embankments developed with due consideration to settlement is outlined in Section 11.2.2 of this report. The recommended approach for constructing the temporary embankment involves leaving the fill and underlying peat in place and using staged construction techniques to manage the settlement. An embankment over-build (i.e. preloading) has been recommended in order that the settlements of the detour pavements will be within a tolerable range when the detour is in service. The results of slope stability analyses carried out for this approach indicated that that a geotextile and a geogrid will be needed to achieve a Factor of Safety of 1.3 or greater for 2H:1V side slopes for embankments constructed on the existing ground. The results of the stability analyses are provided in the attached Appendix F. Further details on the treatment of the detour embankments and the construction staging for this aspect of the work are also presented in the Pavement Design Report. The recommended cross section for the embankment construction is shown on Figure G1, Appendix G. An NSSP for the geotextile and geogrid is provided in Appendix H.

### **11.2 Settlement**

#### **11.2.1 Highway 11**

On Highway 11 a grade raise of up to about 0.5m is required to accommodate the new structure. Significant embankment widening at the structure is not expected.

Settlement analyses were carried out using the consolidation characteristics of the silty clay interpreted from the results of the one dimensional consolidation testing shown on Figure B1-7 and shown on Table 5.1. The results of the analyses indicated that the settlement due to the grade raise





at the structure is expected to be less than 20mm. Settlements of this magnitude are expected to be within a tolerable range. The embankment widening required at the structure is minor and is not expected to have a significant impact on the performance of the approach slabs.

### **11.2.2 Detour**

A large portion of the detour is underlain by weak/compressible soil consisting of peat interspersed with sand, silt and clay. Potentially large settlements would result from the construction of the temporary detour embankments on the existing ground. Consideration was given to the following alternatives:

- The peat could be excavated and replaced with suitable soils;
- The detour could be relocated to another alignment;
- The embankment could be floated over the poor soils and the construction staging designed to allow the primary settlement to occur.

The first alternative was not considered due to cost and the temporary nature of the work and the second alternative was not considered since any alternative alignment will still be located in the flood plain and similar conditions would be expected.

With the floating method, substantial deformations are expected due to the consolidation of the peat under the embankment loading. A large proportion of the consolidation will take place during the months immediately following embankment construction; however, the rate of settlement would be expected to decrease after the primary consolidation phase. For this reason staged construction of the detour will need to be considered. This would involve construction of the embankment (and possibly a surcharge) in the fall and resumption of construction of the detour pavement and the bridge in the following spring. Some settlement would be expected to continue during the serviceable life of the detour and it is likely that some maintenance of the pavement will be required.

Settlement analyses were carried out to provide estimates of the range of settlement to be expected. A 1 m thick layer of peat and a 1.5 m high embankment over existing ground were selected for analysis. This combination is typical of the locations where the greatest settlement is expected. Three cases were considered using embankments initially built 0.25m, 0.5 m and 0.75 m higher than final grade to determine the anticipated settlements. The consolidation characteristics of the peat that were used in the analyses were adopted from the results of consolidation testing carried out on samples of the peat obtained at the nearby Crow Creek bridge and are summarized below.





**Table 11.1 – Consolidation Characteristics of Peat**

Parameter	
Natural water content	586 %
Bulk Unit weight	9.8 kN/m <sup>3</sup>
Dry Unit weight	5.4 kN/m <sup>3</sup>
Compression index	1.67
Recompression index	0.426
Void ratio	2.8
Preconsolidation Pressure	20 kPa
Consolidation Coefficient	0.146 m <sup>2</sup> /yr

Table 11.2 below shows the anticipated consolidation that will take place in the first 6 months and the consolidation that can be expected while the detour is in operation between 6 and 10 months after initial construction for each of the three cases.

**Table 11.2 – Consolidation Estimates**

	<b>Case 1 0.25 m Surcharge</b>	<b>Case 2 0.50 m Surcharge</b>	<b>Case 3 0.75 m Surcharge</b>
Consolidation 0 - 6 months (mm)	220-250	250-300	280-350
Consolidation 6 -10 months (mm)	< 30	< 30	< 30

The results of the analyses indicate that for Case 1, where the embankment is initially overbuilt by 0.25 m, over the initial six months the embankment will settle approximately to the profile grade and up to 30 mm additional settlement could be expected during the serviceable life of the detour (i.e. between six and ten months following embankment construction). The use of greater surcharges to try to accelerate the consolidation and minimize settlement during the service life of the detour was found to be not effective.

Based on the above analysis it has been recommended that the existing fill and peat be left in place and that a staged construction approach be used together with a surcharge height of 0.25m.

### **11.2.3 Other Considerations –Detour**

The approach ramps provide a transition from the roadway to the structure. The approach ramps for the temporary modular bridge are essentially hinged at the structure and will be supported on a pad of Granular A constructed for this purpose. The granular pad (and the approach ramps) will also experience some settlement due to consolidation of the underlying silty clay and peat unless the peat is sub-excavated and replaced with an engineered fill material. Sub-excavation and removal of the peat on the east side of the structure will necessitate excavations of greater than 2m below the flood plain elevation. This will be a difficult, if not a costly task due to the difficult ground water conditions at the site.





As an alternative to the excavation and replacement, consideration could be given to leaving the peat in place and constructing the pads using the methodology that has been proposed for the detour embankment and pavements. Settlements of less than 30mm have been estimated for the service period of the detour pavements using the methodology that has been proposed and settlement of a similar magnitude would be expected for the pads. In the event that the construction period is extended, and the settlements are approaching the upper limit of the expected range, it may be necessary to make adjustments to the approach ramps and to pad the pavements to maintain the serviceability.

The latter approach is recommended due to the difficulty of excavating and replacing the compressible soil within the flood plain and since the range of settlement expected is a range that can be managed. It is understood that the ramps can tolerate a small degree of rotation that would result from settlement. Consideration could be given to building the approaches slightly higher than the design grade in order that as much movement as possible can be accommodated during the service life of the bridge.

### **11.3 Embankment Construction**

Embankment construction should be in accordance with OPSS 206, November 2009 and the approach fills should be constructed in advance of pile driving operations. Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles will be driven.

Bonding between the embankment fill and the existing soils should be established by benching as per OPSD 208.010. The requirements for erosion protection on the embankment slopes are provided in Section 14 of this report

## **12 BACKFILL TO ABUTMENTS**

For a conventional abutment, granular backfill is recommended but rock backfill can be permitted. A NSSP is required to specify grading limits for the rock fill. The rock fill used as backfill to the abutment should be limited to fragments no greater than 250 mm and should include adequate spalls to fill voids in the rock fill.

In all cases where the approach embankment consists of rock fill and granular backfill to the abutment wall is used, the granular backfill must consist of OPSS Granular B Type II.

The backfill to the abutment walls 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.

All granular material should meet the specifications of Special Provision 110S13 “Amendment to OPSS 1010, April 2004”.

Compaction equipment to be used adjacent to retaining structures should be restricted in accordance with Special Provision 105S10 “Amendment to OPSS 501, February 1996”.





The design of the abutment should incorporate a subdrain as shown in OPSD 3101.150 or OPSD 3101.200, as applicable.

### **13 EARTH PRESSURE**

For cases where backfill to the abutment is placed in accordance with OPSD 3101.150 or OPSD 3102.200 as recommended, the lateral earth pressure will be governed by the properties of the material within the backfill limits shown in the respective OPSD, i.e. a line projected up at 1.5H:1V for granular backfill and 1.25H:1V for rock backfill.

If the support system allows yielding of the wall (unrestrained system) then active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system) then at-rest horizontal earth pressures should be used. The amount of wall movement required for the development of active, passive and at-rest earth pressures may be interpreted using Figure C6.9.1(a) in the Commentary to the CHBDC.

Earth pressures acting on the structure should be computed in accordance with Clause 6.9 of the CHBDC but generally are given by the expression:

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

$P_h$  = horizontal pressure on the wall (kPa)

$K$  = earth pressure coefficient (see table 14.1)

$\gamma$  = unit weight of retained soil (see table 14.1)

$h$  = depth below top of fill where pressure is computed (m)

$q$  = value of any surcharge (kPa)

In accordance with Clause 6.9.3 of the CHBDC, a compaction surcharge should be added. The magnitude should be 12 kPa at the top of fill and decreasing to 0 kPa at a depth of 1.7 m for Granular B Type I or at a depth of 2.0 m for Granular A or Granular B Type II.

Earth pressure coefficients for backfill to the abutment wall are dependent on the material used as backfill. Typical values are given in Table 13.1.





**Table 13.1 – Earth Pressure Coefficients**

Wall Condition	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	0.40*	0.31	0.48*	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.

In conventional design, the use of a material with a high friction angle and low active pressure coefficient (e.g. Granular A, Granular B Type II) might be preferred as it results in lower earth pressures acting on the wall.

The factors in the table above are “ultimate” values and require certain movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.9.1 (a) in the Commentary to the CHBDC, 2006.

## 14 EROSION PROTECTION

It is understood that the actual flow velocities at the structure sites are relatively low and technically require only minimal erosion protection. However as a minimum the forward slopes at the new HWY 11 bridge are to be provided with rip rap/rock protection in accordance with OPSS 511, November 2008. Portions of the slopes above the design high water levels may be vegetated.

As presently proposed, the detour embankment will be constructed in late fall 2012. Erosion control blankets are recommended for the embankment slopes because the seeding is unlikely to provide sufficient vegetation cover to control erosion during the following spring–thaw period.

It is recommended that the new highway embankments and detour fills that will remain after construction, be treated with seed and mulch to control erosion. The application of seed and mulch to the detour slopes should be delayed until the optimal germination conditions in the spring of 2014.

## 15 SEISMIC CONSIDERATIONS

### 15.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 0. The following seismic parameters (Kapuskasing) should be used for design:





---

• Velocity Related Seismic Zone	0
• Zonal Velocity Ratio	0
• Acceleration Related Seismic Zone	0
• Zonal Acceleration Ratio	0.00
• Peak Horizontal Acceleration	0.091

The soil profile type at this site has been classified as Type I. Therefore, according to Table 4.4.6.1 of the CHBDC, a Site Coefficient “S” (ground motion amplification factor) of 1.0 should be used in seismic design.

## **15.2 Potential for Liquefaction**

The piles supporting the new bridge will develop resistance in the dense sandy till or in the underlying hard clayey silt till. A preliminary assessment of the potential for liquefaction to occur can be made by considering the geologic age and origin of the deposits, grain size and plasticity characteristics, degree of saturation, depth and soil penetration resistance.

Liquefaction is most likely to occur in fluvial, lacustrine and Aeolian deposits and least likely to occur in older deposits like glacial till.

The potential for liquefaction is greater for soils having less than 15 % finer than 5 microns with a liquid limit of less than 35 % and an in-situ water content approaching the liquid limit.

A high degree of saturation is generally required for liquefaction.

Liquefaction is more likely to occur in soil deposits within about 15m of the ground surface.

Liquefaction has been known to occur in soils with normalized N values of less than 22 blows per 0.3m and a threshold N value of 30 blows per 0.3m has been considered.

Although the grain size and plasticity characteristics, and degree of saturation of the sandy silt till are within respective ranges for which a higher potential for liquefaction can be predicted, the geological nature of the deposit together with the very high penetration resistance are not consistent with such behaviour. In addition the site is not located in a seismically active area and there is no history of liquefaction failure in the area of the site.

On the basis of the above, the potential for liquefaction failure at the site is very low and does not warrant further investigation.





### 15.3 Retaining Wall Dynamic Earth Pressures

In accordance with Clause 4.6.4 of the CHBDC, retaining structures should be designed using active ( $K_{AE}$ ) and passive ( $K_{PE}$ ) earth pressure coefficients that incorporate the effects of earthquake loading.

In calculating the active, passive and at rest earth pressure coefficients the angle of friction between the wall and backfill material is assumed to be  $0.5\phi$ . For the design of retaining walls, the coefficients of horizontal earth pressure in Table 15.1 may be used:

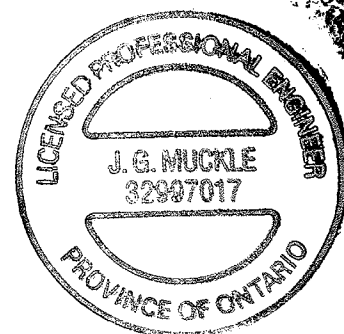
**Table 15.1 – Earth Pressure Coefficient for Earthquake Loading**

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35^\circ$ ; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$ ; $\delta = 16^\circ$ $\gamma = 21.2 \text{ kN/m}^3$		Rock Fill $\phi = 42^\circ$ ; $\delta = 21^\circ$ $\gamma = 19.0 \text{ kN/m}^3$	
	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 ( $K_{AE}$ )*	0.30	0.40	0.30	0.50	0.20	0.30
Passive ( $K_{PE}$ )	3.69	-	3.26	-	5.05	-
At Rest ( $K_{OE}$ )**	0.50	-	0.50	-	0.40	-

\* After Mononobe and Okabe, passive case assumes a horizontal surface in front of the wall.

\*\* After Woods

Engineering Analysis and Report Preparation by:  
J. G. Muckle, P. Eng.,  
Associate, Senior Geotechnical Engineer



*Michael Tanos*

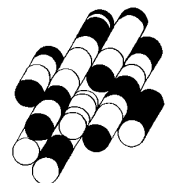
Report Reviewed by:  
Michael Tanos, P. Eng.,  
Review Principal





# TABLE

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**TABLE 1**

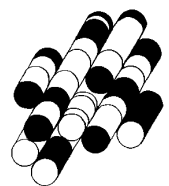
<b>DOCUMENT</b>	<b>TITLE</b>
OPSS 206	Construction Specification for Grading.
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting
OPSS 571	Construction Specification for Sodding.
OPSS 572	Construction Specification for Seed and Cover.
OPSS 902	Construction Specification for Excavation & Backfilling of Structures
OPSS 1010	Material Specifications for Aggregates, Select Subgrade, Backfill
OPSD 208.010	Benching of Earth Slopes
OPSD 3101.150	Walls, Abutment Backfill – Min. Granular Requirement
OPSD 3101.200	Walls, Abutment Backfill – Rock
SP105S10	Amendment to OPSS 501
SP110S13	Amendment to OPSS 1010
SP999S26	Construction Specifications for Design, Installation and Testing of Temporary and Permanent Pre-stressed Anchors





# APPENDICES

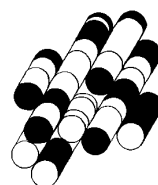
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# APPENDIX A

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## 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 McCormick Rankin, a Member of MMM Group Ltd. 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 McCormick Rankin, a Member of MMM Group Ltd., 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  $\bar{N}$ .

**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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$C_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vm}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_c$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_s$	kPa	REMOULDED SHEAR STRENGTH
$S_i$	1	SENSITIVITY = $c_u / \tau_s$

## PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	e	1.0	VOID RATIO	$e_{max}$	1.0	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	n	1.0	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	w	1.0	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(w_L - w_p)$	v	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	L	1	LIQUIDITY INDEX = $(w - w_p) / I_p$	i	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_c$	1	CONSISTENCY INDEX = $(w - w_p) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1.0	VOID RATIO IN LOOSEST STATE	j	kN/m <sup>2</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						



## EXPLANATORY SHEET FOR CORE LOG

### Column Number

1. Elevation of borehole collar.
2. Depth of geotechnical boundary in borehole
3. Geologic symbol for rock or soil material
4. General description of geotechnical unit - qualitative description, including rock type(s), percentage rock types, frequency and sizes of interbeds, colour, texture.

### Joint (discontinuity) Characteristics

5. Number of joint sets: a rock mass can be intersected by a number of joint sets of varying orientations.
6. Joint type: B = Bedding joint C = Cross joint
7. Orientation: only variations in dip can be identified in core; dip direction is from field mapping or oriented core:  
F = Flat = 0 - 20° D = Dipping = 20 - 50° V = Vertical = 50 - 90°
8. Joint spacing: this is an approximate measure of spacing between joints in specific joint sets.

SPACING	> 3 m	1 m - 3 m	0.3 m - 1 m	50 mm - 300 mm	< 50 mm
	VERY WIDE	WIDE	MODERATE	CLOSE	VERY CLOSE

9. Roughness:

RU = Rough Undulating  
SU = Smooth Undulating  
LU = Slickensided Undulating

RP = Rough Planar  
SP = Smooth Planar  
LP = Slickensided Planar

10. Filling:

T = Tight, hard, non-softened  
O = Oxidation surface staining only  
SA = Slightly altered; clay-free  
S = Sandy particles; clay-free  
Si = Sandy and silty, minor clay  
NC = Non-softening Clays; 5mm  
SC = Swelling Clay fillings; 5mm

### Approximate $\phi$

25 - 35  
25 - 30  
25 - 30  
20 - 25  
16 - 24  
6 - 12

11. Aperture: estimated size of joint opening.

12. Degree of weathered rock material:

DEGREE	DESCRIPTION
UNWEATHERED	NO SIGNS OF DISCOLOURATION OR OXIDIZATION
SLIGHTLY WEATHERED	PARTIAL DISCOLOURATION; FRACTURES (JOINTS), TYPICALLY OXIDIZED
MODERATELY WEATHERED	TOTAL DISCOLOURATION
HIGHLY WEATHERED	TOTAL DISCOLOURATION; TYPICALLY FRIABLE AND PITTED
COMPLETELY WEATHERED	RESEMBLE A SOIL; ROCK STRUCTURE - USUALLY PRESERVED

13. Strength of rock material:

VERY HIGH STRENGTH	SPECIMEN CAN ONLY BE CHIPPED BY GEOLOGICAL HAMMER	MPa
HIGH STRENGTH	SPECIMEN REQUIRES A NUMBER OF BLOWS OF A GEOLOGICAL HAMMER TO FRACTURE IT; CANNOT BE SCRAPPED WITH POCKET KNIFE	> 200
MEDIUM STRENGTH	SPECIMEN CANNOT BE FRACTURED BY A SINGLE, FIRM BLOW OF GEOLOGICAL HAMMER; CAN BE SCRAPPED WITH POCKET KNIFE, NOT PEELED	50 - 200
LOW STRENGTH	SHALLOW INDENTATIONS MADE BY FIRM BLOW WITH POINT OF GEOLOGICAL HAMMER; CAN BE PEELED WITH POCKET KNIFE WITH DIFFICULTY	15 - 50
VERY LOW STRENGTH	CRUMBLES UNDER FIRM BLOW WITH POINT OF GEOLOGICAL HAMMER; CAN BE PEELED	4 - 15
		1 - 4

14. Fracture frequency: number of natural joints occurring over a meter length of core. All natural joints are counted irrespective of the number of joint sets.

FRACTURE FREQUENCY	JOINT SPACING	LENGTH
0.3 m	VERY WIDE	> 3 m
0.3 - 1 m	WIDE	1 m - 3 m
1 - 3 m	MODERATE	0.03 m - 1 m
3 - 20 m	CLOSE	0.005 m - 0.03 m
20 m	VERY CLOSE	< 0.005 m

15. Run number and Core Recovery

- (i) Drill run number

- (ii) Total Core Recovery is the total length of core pieces, irrespective of their individual lengths obtained in a core run, and expressed as a percentage of the length of that core run.

16. Rock Quantity Designation (RQD): The total length of those pieces of sound core which are 0.01 metres or greater in length in a core run, expressed as a percentage of the total length of that core run. Sound pieces of rock are those pieces separated by natural breaks and not machine breaks or subsequent artificial breaks.

Rock Mass Classification (after Deare)					
RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
DESCRIPTION	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

17. Core and Casing sizes: changes of core and casing sizes are indicated.

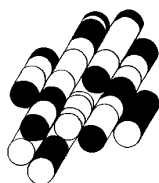
18. Water recovery, level and tests:

- (i) percentage drill water recovery

- (ii) water level depth

- (iii) positions and results of tests, e.g., permeability and packer tests





## **Terraprobe Inc.**

*Consulting Geotechnical & Environmental Engineering  
Construction Materials Engineering & Testing*

### **ABBREVIATIONS, TERMINOLOGY, & GENERAL INFORMATION**

#### **Sampling Method**

SS - split spoon  
ST - Shelby tube  
AS - auger sample  
RC - rock core

#### **Penetration Resistance**

Standard Penetration Resistance ('N' values) is defined as the number of blows by a hammer of 63.5kg mass (140lbs) falling freely for a distance of 0.76m (30 inches) required to advance a standard 50mm (2inch) diameter split spoon sampler for a distance of 0.3m (12 inches).

Dynamic Cone Penetration Resistance is defined as the number of blows by a hammer of 63.5kg mass (140 lbs) falling freely for a distance of 0.76m (30 inches) required to advance a conical steel point to 50mm diameter and with 60 degrees sides of 'A' size drill rods for a distance of 0.3m (12 inches).

#### **Soil Description**

##### **Cohesionless Soils**

##### **Relative Density**

very loose  
loose  
compact  
dense  
very dense

##### **'N' Value**

<4  
4 - 10  
10 - 30  
30 - 50  
>50

##### **Cohesive Soils**

##### **Consistency**

very soft  
soft  
firm  
stiff  
very stiff  
hard

##### **Undrained Shear Strength (kPa)**

<12  
12 - 25  
25 - 50  
50 - 100  
100 - 200  
>200

#### **Soil Composition**

'trace' (eg. trace silt)  
'some' (eg. some gravel)  
'adjective' (eg. sandy)  
'and' (eg. sand and gravel)  
'occasional'

#### **% By Weight**

<10  
10 - 20  
20 - 35  
35 - 50  
applies to coarse particles that are randomly and  
irregularly distributed in soil matrix

#### **General Information**

The recommendations provided in this report are based on the factual information obtained from the boreholes and on the general information provided for the proposed project.

Site investigations by means of boreholes and/or test pit identifies subsurface conditions at the location and time of sampling only. Ground conditions at locations away from the boreholes and test pit may vary.

Recommendations are made by interpretation of this factual data for specific conditions such as size, configuration and location of the proposed project. Changes in project conditions should be reviewed by the Geotechnical consultant as they may affect the recommendations provided.

In order to identify possible changes in ground conditions between the sample locations and their effect on the project, it is recommended that site inspections be carried out during construction by qualified Geotechnical personnel.



**METRIC**

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE



**RECORD OF BOREHOLE No MCD1**

2 of 2

**METRIC**

G.W.P. 5233-06-00 LOCATION Coords: E:381087.5 N:5489894.5 ORIGINATED BY PB  
 DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS / WASH BORING COMPILED BY DB  
 DATUM GEODETIC DATE 10.27.11 CHECKED BY HA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT      NATURAL MOISTURE CONTENT      LIQUID LIMIT		UNIT WEIGHT $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR   SA   SI   CL						
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20    40    60    80    100						W <sub>p</sub> W                      W <sub>L</sub>					
								SHEAR STRENGTH (kPa)						WATER CONTENT (%)					
								○ UNCONFINED ● QUICK TRIAXIAL		+ FIELD VANE X LAB VANE									
	(continued)																		
	CLAYEY SILT to SILTY CLAY, sandy, trace gravel, occasional cobbles and boulders, hard, grey, moist (GLACIAL TILL) (continued)																		

**END OF BOREHOLE**

Borehole was filled with drill water  
upon completion of drilling.


25mm piezometer installed.

**WATER LEVEL READINGS**

Date	Water Depth (m)	Elevation (m)
November 8, 2011	1.4	235.1
December 13, 2011	1.3	235.2
April 26, 2012	1.1	235.4




Y:\1-Project Files\11-Geotechnical\2010\11-10-5076\A Dwg, Log\AutoCAD\11-10-5076 CORE.dwg, DB

CORE LOG													 Terraprobe						
ProjectCrow Creek Bridge Replacement				OrientationVertical			Ground Elevation236.5m			DatumGeodetic				Borehole No.MCD1					
LocationHwy 11, Township of McCrea, Ontario				Date StartedNovember 2, 2011			CompletedNovember 2, 2011			Logged ByB.R.				Sheet 1 of 1					
ClientMTO				Drilling AgencyLandcore Drilling			Drill TypeCME55			Core Barrel & Bit DesignNQ				Project No.11-10-5076					
ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m³)	
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE									
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
219.5	17.0		Overburden, see Borehole Log MCD1  <b>BEDROCK – Granitic Gneiss</b> (biotite-quartz-feldspar gneiss to hornblende-quartz-feldspar gneiss) unweathered, massive, white to grey, medium to high strength, medium to coarse grained.																
218.5	18.0			2	CC	DV	VC	SP	SA	0 to 1					#1 TCR 95 SCR 32	32	NQ		
217.5	19.0			2	CC	DV	VC	SP	O	0 to 1					#2 TCR 91 SCR 83	57	NQ		
216.5	20.0			2	CC	DV	C	SP	O	0 to 1					#3 TCR 100 SCR 100	74	NQ		
215.5	21.0		End of Core Log																
214.8	21.7																		
214.5	22.0																		
213.5	23.0																		
212.5	24.0																		
211.5	25.0																		
210.5	26.0																		
209.5	27.0																		
208.4	28.0																		
207.3	29.0																		

Remarks:

LEGEND:

 Bedrock



## RECORD OF BOREHOLE No MCD2

1 of 2

METRIC

G.W.P.	5233-06-00	LOCATION	Coords: E:381058.3 N:5489913.2	ORIGINATED BY	PB
DIST	-	HWY	Hwy 11	BOREHOLE TYPE	HOLLOW STEM AUGERS / WASH BORING
DATUM	GEODETIC	DATE	11.2.11	CHECKED BY	HA

[illegible]

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity      ○<sup>3%</sup> STRAIN AT FAILURE









**RECORD OF BOREHOLE No MCD2**

2 of 2

**METRIC**

G.W.P. 5233-06-00 LOCATION Coords: E:381058.3 N:5489913.2 ORIGINATED BY PB  
DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS / WASH BORING COMPILED BY DB  
DATUM GEODETIC DATE 11.2.11 CHECKED BY HA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)								WATER CONTENT (%)
	(continued)															
218.8 17.7	CLAYEY SILT to SILTY CLAY, some sand to sandy, trace gravel, occasional cobbles and boulders, hard, grey, moist (GLACIAL TILL) (continued)		15	SS	52		221									
							220									
			16	SS	123 / 225mm											
216.2 20.3	COBBLES and BOULDERS, inferred very dense, grey		1	RUN			219									
							218									
			2	RUN												
210.2	For details see rock core log mcd2 (BEDROCK)		3	RUN			216									
							215									
							214									
							213									
							212									
			6	RUN			211									

**END OF BOREHOLE**

Borehole was filled with drill water upon completion of drilling.

25mm piezometer installed.

**WATER LEVEL READINGS**

Date	Water Depth (m)	Elevation (m)
November 8, 2011	0.6	235.9
December 13, 2011	0.8	235.7
April 26, 2012	0.4	236.1



Project	Crow Creek Bridge Replacement	Orientation Vertical	Ground Elevation 236.5m	Datum Geodetic	Borehole No. MCD2
Location	Hwy 11, Township of McCrea, Ontario	Date Started November 3, 2011	Completed November 4, 2011	Logged By B.R.	Sheet 1 of 1
Client	MTO	Drilling Agency Landcore Drilling	Drill Type CME55	Core Barrel & Bit Design NQ	Project No. 11-10-5076

[illegible]

Remarks:

LEGEND:



Bedrock



**RECORD OF BOREHOLE No MCD3**

1 of 1

**METRIC**

G.W.P. 5233-06-00 LOCATION Coords: E:381033.2 N:5489910.4 ORIGINATED BY PB  
DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY DB  
DATUM GEODETIC DATE 10.17.11 CHECKED BY HA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)									
								20	40	60	80	100					
236.4	GROUND SURFACE																
236.1	130mm ASPHALTIC CONCRETE		1A	AS													
0.3	180mm FILL - sand and gravel, trace silt, brown, damp		1B														
	FILL, sand, trace silt, trace gravel; loose to compact, brown, wet		2	SS	20												
234.5	Occasional peat inclusions		3A	SS	7												
1.9			3B														
233.8			4	SS	7												
2.6	SILTY CLAY to CLAYEY SILT, trace to some sand, trace gravel, firm to stiff, grey, moist		5	SS	6												
			6	AS													
			7	AS													
			8	SS	14												
			9	SS	10												
			10	ST													
228.9	SANDY SILT, trace clay, trace gravel, occasional cobbles and boulders, compact to very dense, grey, moist (GLACIAL TILL)		11	SS	26												
7.5			12	SS	100 / 50mm												
227.7																	

**END OF BOREHOLE**

Auger refusal

Unstabilized water level measured at 7.0m and borehole caved to 7.9m below grade upon completion of drilling

25mm piezometer installed.

**WATER LEVEL READINGS**

Date	Water Depth (m)	Elevation (m)
November 3, 2011	1.4	235.0
November 8, 2011	1.3	235.1
December 13, 2011	1.2	235.2
April 26, 2012	n/a (damaged)	n/a



**RECORD OF BOREHOLE No MCD4**

1 of 1

**METRIC**

G.W.P. 5233-06-00 LOCATION Coords: E:381110.4 N:5489889.2 ORIGINATED BY PB  
DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY DB  
DATUM GEODETIC DATE 10.18.11 CHECKED BY HA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	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 <sub>p</sub> W W <sub>L</sub>				
								○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE											
236.5	GROUND SURFACE																		
236.3	170mm ASPHALTIC CONCRETE		1A							○									
236.2	150mm FILL - sand and gravel, trace silt, brown, damp		1B	AS						○									
236.0	FILL, sand, some silt, trace gravel; compact, brown, damp		2	SS	15					○									
234.4	...at 1.5m, wet		3	SS	12						○			0 81 (19)					
234.2	Trace organics		4	SS	12						○			0 23 53 24					
233.2	CLAYEY SILT to SILTY CLAY, some sand to sandy, trace gravel, firm to very stiff, grey, moist		5	SS	6							○							
233.0			6	AS								○							
232.0			7	ST										4 20 53 23					
231.0			8	AS															
230.0			9	SS	16														
229.4	SANDY SILT, trace clay, trace gravel, occasional cobbles and boulders, compact to very dense, grey, moist (GLACIAL TILL)		10	SS	13							○		5 30 58 7					
227.0			11	SS	194 / 225mm							○							
227.0																			

**END OF BOREHOLE**

Auger refusal

Unstabilized water level measured at 7.0m and borehole caved to 8.5m below grade upon completion of drilling

25mm piezometer installed.

**WATER LEVEL READINGS**

Date	Water Depth (m)	Elevation (m)
November 3, 2011	1.5	235.0
November 8, 2011	1.3	235.2
December 13, 2011	1.2	235.3



**RECORD OF BOREHOLE No MCD5**

1 of 1

**METRIC**

G.W.P. 5233-06-00 LOCATION Coords: E:381029.9 N:5489898.5 ORIGINATED BY PB  
DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY DB  
DATUM GEODETIC DATE 10.25.11 CHECKED BY HA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT N' VALUE			SHEAR STRENGTH (kPa)									
								20 40 60 80 100									
235.7	GROUND SURFACE																
235.6	150mm TOPSOIL		1A	SS	13												
0.2			1B														
235.2	380mm FILL - sand and gravel, some silt, brown, moist		1C														
0.5			2	SS	9												
	FILL, silty clay, trace sand, trace gravel; stiff, brown, wet																
234.3			3	SS	5												
1.4	CLAYEY SILT to SILTY CLAY, trace to some sand, trace gravel, firm to very stiff, grey, moist																
			4	SS	7												
			5	AS													
			6	SS	9												
			7	SS	24												
			8A	SS	35												
229.3			8B														
6.4	SANDY SILT, some clay, trace gravel, occasional cobbles and boulders, compact to very dense, grey, moist (GLACIAL TILL)		9	SS	19												
			10	SS	185 / 225mm												
226.5																	

**END OF BOREHOLE**

Auger refusal

Borehole was dry and caved to 7.3m below grade upon completion of drilling.

25mm piezometer installed.

**WATER LEVEL READINGS**

Date	Water Depth (m)	Elevation (m)
November 3, 2011	0.8	234.9
November 8, 2011	0.8	234.9
December 13, 2011	0.5	235.2
April 26, 2012	0.4	235.3



**RECORD OF BOREHOLE No MCD6**

1 of 1

**METRIC**

G.W.P. 5233-06-00 LOCATION Coords: E:381107.1 N:5489877.4 ORIGINATED BY PB  
DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY DB  
DATUM GEODETIC DATE 10.25.11 CHECKED BY HA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100									
								SHEAR STRENGTH (kPa)									
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE	WATER CONTENT (%)					
235.7	GROUND SURFACE																GR SA SI CL
235.6 0.2	150mm TOPSOIL		1	SS	11								○				
235.0 0.7	FILL, silty clay, sandy, trace gravel, trace organics; stiff, grey, moist Occasional peat inclusions		2	SS	7											○	
233.9 1.8	CLAYEY SILT to SILTY CLAY, some sand to sandy, trace gravel, firm to very stiff, grey, moist		3	SS	6											○	
			4	SS	7								○				2 33 48 17
			5	SS	7								○				
			6	SS	6								○				1 23 55 21
			7	SS	18								○				
			8	SS	10								○				4 25 48 23
			9	SS	8								○				
227.1 8.6	SANDY SILT, some clay, trace gravel, occasional cobbles and boulders, very dense, grey, damp (GLACIAL TILL)		10	SS	100 / 150mm								○				
225.2			11	SS	147								○				2 25 54 19

**END OF BOREHOLE**  
Auger refusal

Unstabilized water level measured at  
5.5m and borehole caved to 7.0m  
below grade upon completion of  
drilling

25mm piezometer installed.

**WATER LEVEL READINGS**

Date	Water Depth (m)	Elevation (m)
November 3, 2011	0.5	235.2
November 8, 2011	0.2	235.5
December 13, 2011	0.4	235.3
April 26, 2012	0.4	235.3



# RECORD OF BOREHOLE No MC-1

1 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489903.6 E:381055.6 ORIGINATED BY PK  
 DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
 DATUM Geodetic DATE 7.21.10 - 7.22.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																	
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236.5	Ground Surface							20	40	60	80	100																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																				</

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

ON\_MOT\_OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON\_MOT\_GDT 5/22/12



# RECORD OF BOREHOLE No MC-1

2 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489903.6 E:381055.6 ORIGINATED BY PK  
DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
DATUM Geodetic DATE 7.21.10 - 7.22.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		
							20	40	60	80	100						
			15	SS	39												
			16	SS	66												
216.7 19.8	COBBLES AND BOULDERS very dense, grey																
			17	SS	100/ 13cm												
			1	RUN	NQ												
214.5 22.0	BEDROCK - GRANITE unweathered below 24.5m, massive, white to grey, medium to high strength.																
			2	RUN	NQ												
			3	RUN	NQ												
210.6 25.9	End of Borehole  Borehole open to full depth and filled with drill water upon completion of drilling.  Piezometer installation consists of a 25mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.  Water Level Readings: Date Depth(m) Elevation(m) Aug.06.10 1.6 234.9 Aug.10.10 1.6 234.9 Sep.03.10 1.6 234.9 Apr.26.12 1.3 235.2																

ONL\_MOT\_OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON\_MOT.GDT 5/22/12



CORE LOG



Terraprobe

Project	Montcalm Creek Bridge Replacement	Orientation	Vertical	Ground Elevation	236.5m	Datum	Geodetic	Borehole No.	MC-1
Location	Hwy 11, Township of McCrea, Ontario	Date Started	July 22, 2010	Completed	July 22, 2010	Logged By	A.W.	Sheet	1 of 1
Client	MTO	Drilling Agency	Landcore Drilling	Drill Type	CME55	Core Barrel & Bit Design	NQ	Project No.	1-10-5076

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m³)
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE								
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
215.5	21.0		Overburden, see Borehole Log MC-1															
214.5	22.0													#1 TCR 21 SCR 0	0	NQ		
213.5	23.0		<b>BEDROCK – GRANITE</b>  Unweathered below 24.5m, massive, white to grey, medium to high strength.  Highly weathered from 22.0m to 22.3m.  Moderately to slightly weathered from 22.3m to 24.5m.  Highly fractured zones from 22.0m to 22.4m and 25.4m to 25.7m.	2	CC	DV	VC	SP	SA	0 to 1								
					CC	DV	VC	SP	O	0 to 3								
				2										#2 TCR 72 SCR 41	18	NQ		
					CC	DV	C	SP	O	0 to 1								
212.5	24.0																	
				2	CC	DV	C	SP	O	0 to 1				#3 TCR 98 SCR 56	43	NQ		
					CC	DV	VC	SP	O									
210.6	25.9			CC	DV	C	SP	O										
			End of Core Log															

Remarks:	<div>LEGEND:  Bedrock</div>
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# RECORD OF BOREHOLE No MC-2

1 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489904.1 E:381090.2 ORIGINATED BY PK  
DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
DATUM Geodetic DATE 7.20.10 - 7.21.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							
							○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE								
236.4	Ground Surface														
236.2	200mm ASPHALT														
0.2 235.9 0.5	310mm FILL - Sand and Gravel, trace silt, compact, brown, dry		1	SS	27		236								
	FILL - Sand, trace silt, trace gravel, compact, damp		2	SS	16		235								6 88 (6)
	----- loose, wet		3	SS	4		234								
234.3							233								
2.1	FILL - Silty Sand, frequent clay lumps, very loose, grey, wet		4	SS	2		232								
233.5							231								
2.9	FILL - Silty Clay and Organics, sandy, stiff, grey, moist		5	SS	14		230								
232.7							229								
3.7	SILTY CLAY some sand, trace gravel, firm to very stiff, grey, moist		6	SS	6		228								1 12 62 25
			7	SS	9		227								
			8	SS	21		226								
			9	SS	24		225								
229.3							224								
7.1	SANDY SILT trace to some clay, trace gravel, occasional cobbles and boulders, compact to very dense, grey, damp to moist  (GLACIAL TILL)		10	SS	28		223								
			11	SS	100/ 8cm		222								1 32 55 12 commence casing and washboring
			12	SS	175/ 25cm										
224.8															
11.6	CLAYEY SILT TO SILTY CLAY sandy, trace gravel, hard, grey, damp to moist  (GLACIAL TILL)		13	SS	34										6 25 45 24
			14	SS	51										

Continued Next Page

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

ONL\_MOT\_OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON\_MOT\_GDT 5/22/12



# RECORD OF BOREHOLE No MC-2

2 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489904.1 E:381090.2 ORIGINATED BY PK  
 DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
 DATUM Geodetic DATE 7.20.10 - 7.21.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		
							20	40	60	80	100						
	----- occasional cobbles and boulders		15	SS	49												
217.9			16	SS	100/ 8cm												
18.5	BEDROCK - GRANITE unweathered below 21.2m, massive, white to grey, medium to high strength.		1	RUN	NQ												
			2	RUN	NQ												
			3	RUN	NQ												
214.4	End of Borehole																
22.0	Borehole open to full depth and filled with drill water upon completion of drilling.  No sample recovery at SS15. Sampler redriven and disturbed sample collected.  Piezometer installation consists of a 25mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.  Water Level Readings: Date Depth(m) Aug.06.10 1.4 235.0 Aug.10.10 1.5 234.9 Sep.03.10 1.5 234.9 Apr.26.12 1.3 235.1																

ON\_MOT\_OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON\_MOT.GDT 5/22/12







# RECORD OF BOREHOLE No MC-3

1 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489892.0 E:381057.3 ORIGINATED BY PK  
 DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
 DATUM Geodetic DATE 8.7.10 - 8.8.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							W <sub>p</sub> W W <sub>L</sub>
								20 40 60 80 100							
								○ UNCONFINED + FIELD VANE							
								● QUICK TRIAXIAL × LAB VANE							
	20 40 60 80 100					WATER CONTENT (%)									
235.3	Ground Surface														
235.2	130mm TOPSOIL														
0.1			1	SS	5		235								
	FILL - Silty Clay, trace sand, trace organics, firm, brown, moist		2	SS	5								51	0 6 38 56	
233.9							234								
1.4	CLAYEY SILT TO SILTY CLAY sandy, trace gravel, stiff, grey, moist		3	SS	9										
			4	SS	8										
			5	SS	8		232							1 24 57 18	
			6	SS	10										
			7	SS	14		231								
			8	SS	14		230								
			9	SS	8		229							2 22 50 26	
228.2															
7.1	SANDY SILT some clay, trace gravel, frequent cobbles, compact to very dense, damp to moist  (GLACIAL TILL)		10	SS	13		228								
							227								
			11	SS	100/ 13cm		226							5 36 48 11	
225.2														commence casing and washboring	
10.1	CLAYEY SILT TO SILTY CLAY sandy, trace to some gravel, frequent cobbles and boulders below 15.7m, hard, grey, damp to moist  (GLACIAL TILL)		12	SS	152		225							12 25 46 17	
							224								
			13	SS	169		223							3 27 58 12	
							222								
			14	SS	89		221								

Continued Next Page

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

ON\_MOT\_OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON\_MOT.GDT 5/22/12



# RECORD OF BOREHOLE No MC-3

2 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489892.0 E:381057.3 ORIGINATED BY PK  
 DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
 DATUM Geodetic DATE 8.7.10 - 8.8.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT  w <sub>p</sub>	NATURAL MOISTURE CONTENT  w	LIQUID LIMIT  w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE												
							20	40	60	80	100									
219.8	COBBLES AND BOULDERS inferred very dense, grey, moist															Commence NQ Coring				
15.5			15	WS	-															
			16	WS	-															
			17	WS	-															
216.3	BEDROCK - GRANITE unweathered below 19.9m, massive, white to grey, medium to high strength.		1	RUN	NQ											RUN#1 TCR=100% SCR=99% RQD=32%				
19.0			2	RUN	NQ											RUN#2 TCR=94% SCR=86% RQD=49%				
			3	RUN	NQ											RUN#3 TCR=100% SCR=80% RQD=70%				
213.1	End of Borehole																			
22.3	Borehole filled with drill water upon completion of drilling.  Continous soil core sample collected from 15.2m to 19.0m.  Unable to push vane beyond 7.0m.  Piezometer installation consists of a 25mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.  Water Level Readings: Date      Depth(m) Elevation(m) Aug.10.10      0.5      234.8 Sep.03.10      0.5      234.8 Apr.26. 12      Damaged																			

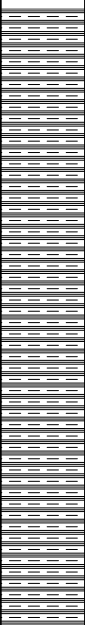


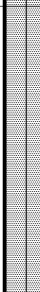



CORE LOG



Terraprobe

Project	Montcalm Creek Bridge Replacement	Orientation	Vertical	Ground Elevation	235.3m	Datum	Geodetic	Borehole No.	MC-3
Location	Hwy 11, Township of McCrea, Ontario	Date Started	August 8, 2010	Completed	August 8, 2010	Logged By	A.W.	Sheet	1 of 1
Client	MTO	Drilling Agency	Landcore Drilling	Drill Type	CME55	Core Barrel & Bit Design	NQ	Project No.	1-10-5076

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics								WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (KN/m³)
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE									
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	
217.3 216.3 215.3 214.3 213.3 213.0	18.0		Overburden, see Borehole Log MC-3																
	19.0		<u>BEDROCK – GRANITE</u>  Unweathered below 19.9m, massive, white to grey, medium to high strength.  Moderately to slightly weathered from 19.0m to 19.9m.  Slightly rubbilized zone from 21.5m to 21.6m.  Weathered zones are typically slightly friable and pitted.	1	C	F	C	RP	S	0 to 1					#1 TCR 100 SCR 99	32	NQ		
	20.0			2	CC	DV	C	RP	SA	0 to 1				#2 TCR 94 SCR 86	49	NQ			
	21.0				CC	DV	C	RP	O										
	22.0			1	C	D	C	RP	T	0 to 1				#3 TCR 100 SCR 80	70	NQ			
	22.3																		
										</									

Remarks:

LEGEND:

 Bedrock



# RECORD OF BOREHOLE No MC-4

1 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489882.6 E:381079.1 ORIGINATED BY PK  
DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
DATUM Geodetic DATE 8.9.10 - 8.10.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL						× LAB VANE		
235.6	Ground Surface																	
235.5	150mm TOPSOIL																	
0.2	FILL - Silty Clay, trace sand, trace organics, stiff, brown, moist		1	SS	10		235						○					
234.9																		
0.7	SILTY CLAY some sand to sandy, trace gravel, stiff to very stiff, grey, moist		2	SS	12		234						○					
			3	SS	11									69	0 4 25 71			
			4	SS	13		233						○					
			5	SS	9		232						○					
			6	SS	9		231						○					
			7	SS	26								○		1 19 53 27			
			8	SS	24		230						○					
			9	SS	16		229						○		2 25 51 22			
228.5	compact		10	SS	20		228						○		5 33 55 7			
7.1	----						227						○		commence casing and washboring			
	SANDY SILT trace clay, trace gravel, frequent cobbles and boulders below 13.7m, very dense, grey, damp to moist  (GLACIAL TILL)		11	SS	100/ 15cm		226						○					
			12	SS	100/ 10cm		225						○					
			13	SS	155/ 25cm		223						○		4 33 53 10			
			14	SS	100/ 8cm		222						○					
							221											

Continued Next Page

+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONL\_MOT\_OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON\_MOT.GDT 5/22/12







# RECORD OF BOREHOLE No MC-4

2 OF 2

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5489882.6 E:381079.1 ORIGINATED BY PK  
 DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB  
 DATUM Geodetic DATE 8.9.10 - 8.10.10 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+	FIELD VANE								
						● QUICK TRIAXIAL	×	LAB VANE										
						20	40	60	80	100	10	20	30					
219.9	COBBLES AND BOULDERS inferred very dense, grey, dry		15	SS	100/ 13cm		220									Aug.09 ----- Aug.10		
15.7																		
217.2	BEDROCK - GRANITE slightly weathered, massive, white to grey, high strength.		1	RUN	NQ		217									RUN#1 TCR=96% SCR=58% RQD=16%		
18.4																	RUN#2 TCR=89% SCR=63% RQD=56%	
																		RUN#3 TCR=100% SCR=77% RQD=0%
213.8	End of Borehole		3	RUN	NQ		214											
21.8	Borehole filled with drill water upon completion of drilling.  Piezometer installation consists of a 25mm diameter, Schedule 40 PVC pipe with a 1.52m slotted screen.  Water Level Readings: Date            Depth(m)            Elevation(m) Aug.10.10      0.4                    235.2 Aug.31.10      1.6                    234.0 Sep.01.10      1.6                    234.0 Apr.26. 12     0.3                    235.3																	

ON\_MOT\_OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON\_MOT.GDT 5/22/12



CORE LOG



Terraprobe

Project	Montcalm Creek Bridge Replacement	Orientation	Vertical	Ground Elevation	235.6m	Datum	Geodetic	Borehole No.	MC-4
Location	Hwy 11, Township of McCrea, Ontario	Date Started	August 10, 2010	Completed	August 10, 2010	Logged By	A.W.	Sheet	1 of 1
Client	MTO	Drilling Agency	Landcore Drilling	Drill Type	CME55	Core Barrel & Bit Design	NQ	Project No.	1-10-5076

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics							WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (kN/m³)
				No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE								
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
217.6	18.0		Overburden, see Borehole Log MC-4															
217.2	18.4																	
216.6	19.0		BEDROCK – GRANITE Slightly weathered, massive, white to grey, high strength.	2	CC	FV	C	RP	SA	0 to 1				#1 TCR 96 SCR 58	16	NQ		
215.6	20.0		Highly fractured zones from 20.0m to 20.5m.		CC	FV	VC	RP	O	0 to 1				#2 TCR 89 SCR 63	56	NQ		
214.6	21.0				CC	FV	C	RP	O									
213.8	21.8			2	CC	FV	C	RP	O	0 to 1				#3 TCR 100 SCR 77	0	NQ		
			End of Core Log															

Remarks:

LEGEND:

 Bedrock



Foundation Investigation Report  
Montcalm Creek Bridge Replacement  
G.W.P. No.: 5233-06-00; W.P. 5146-05-01

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**Bedrock Core Sample**

Borehole: MC-1

Runs: 1, 2 & 3

Depth: 21.0m – 25.9m





Foundation Investigation Report  
Montcalm Creek Bridge Replacement  
G.W.P. No.: 5233-06-00; W.P. 5146-05-01

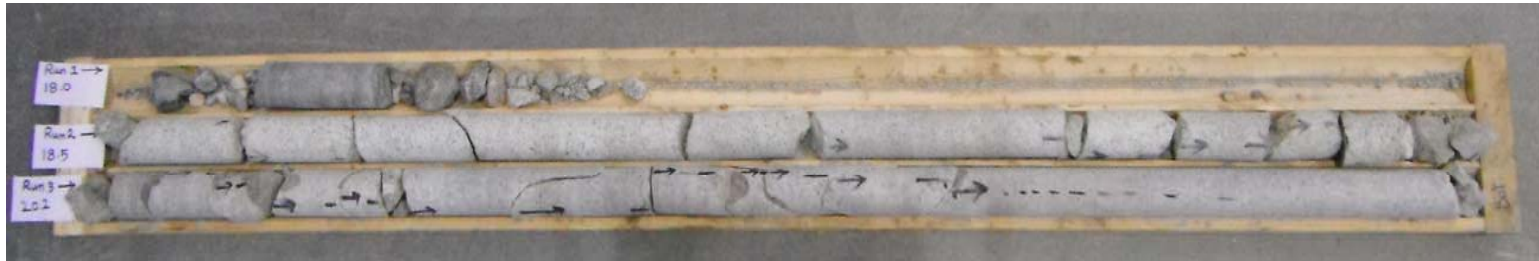
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**Bedrock Core Sample**  
Borehole: MC-2  
Runs: 1, 2 & 3  
Depth: 18.5m – 22.0m







### **Bedrock Core Sample**

Borehole: MCD-1; Runs: 1 to 3; Depth: 18.0m – 21.7m



### **Soil/Bedrock Core Sample**

Borehole: MCD-2; Runs: 1 to 6; Depth: 17.7m – 26.3m





Foundation Investigation Report  
Montcalm Creek Bridge Replacement  
G.W.P. No.: 5233-06-00; W.P. 5146-05-01

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**Bedrock Core Sample**

Borehole: MC-3

Runs: 1, 2 & 3

Depth: 19.0m – 22.3m





Foundation Investigation Report  
Montcalm Creek Bridge Replacement  
G.W.P. No.: 5233-06-00; W.P. 5146-05-01

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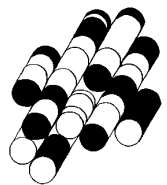
**Bedrock Core Sample**  
Borehole: MC-4  
Runs: 1, 2 & 3  
Depth: 18.4m – 21.8m





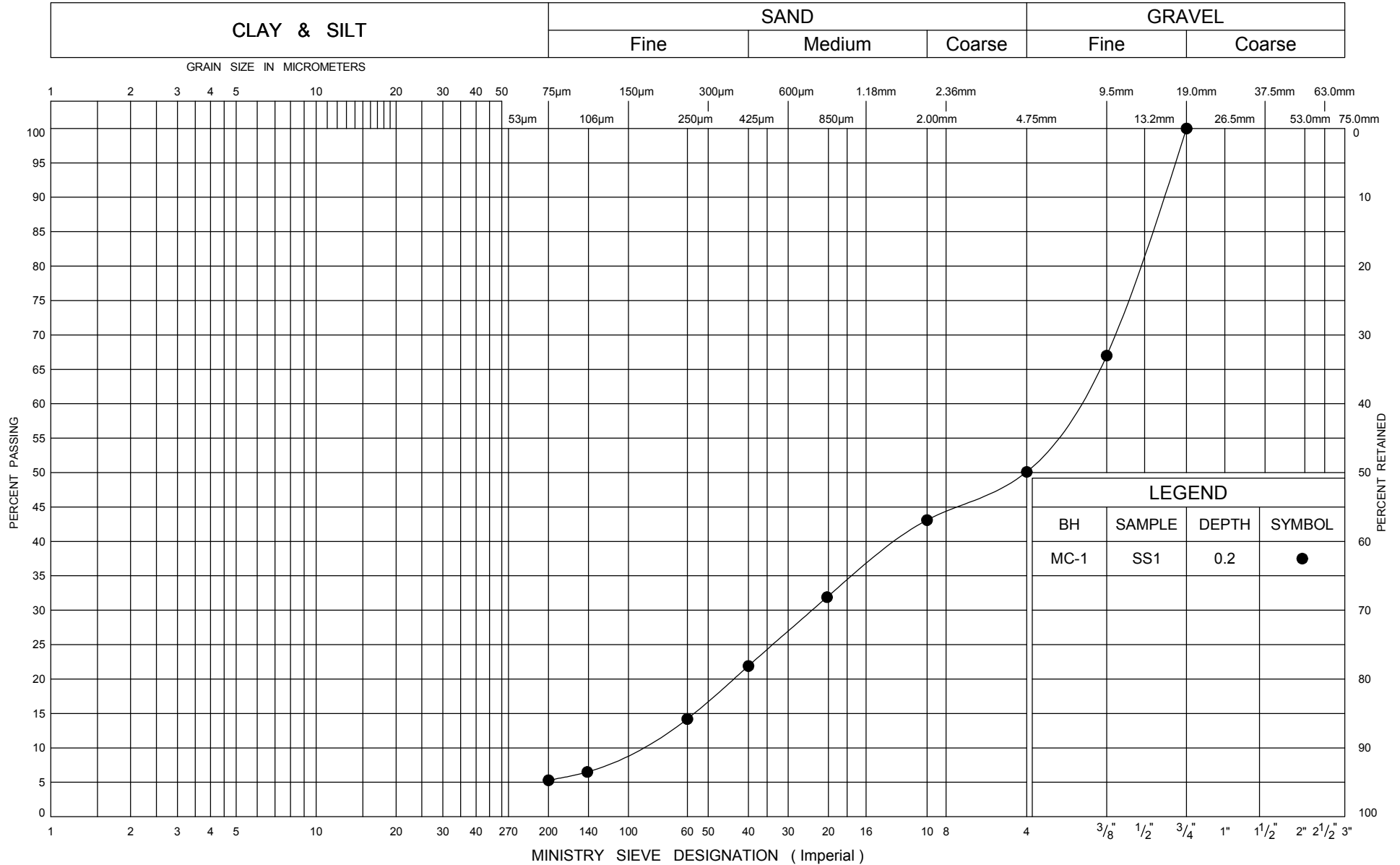
# APPENDIX B

**TERRAPROBE INC.**



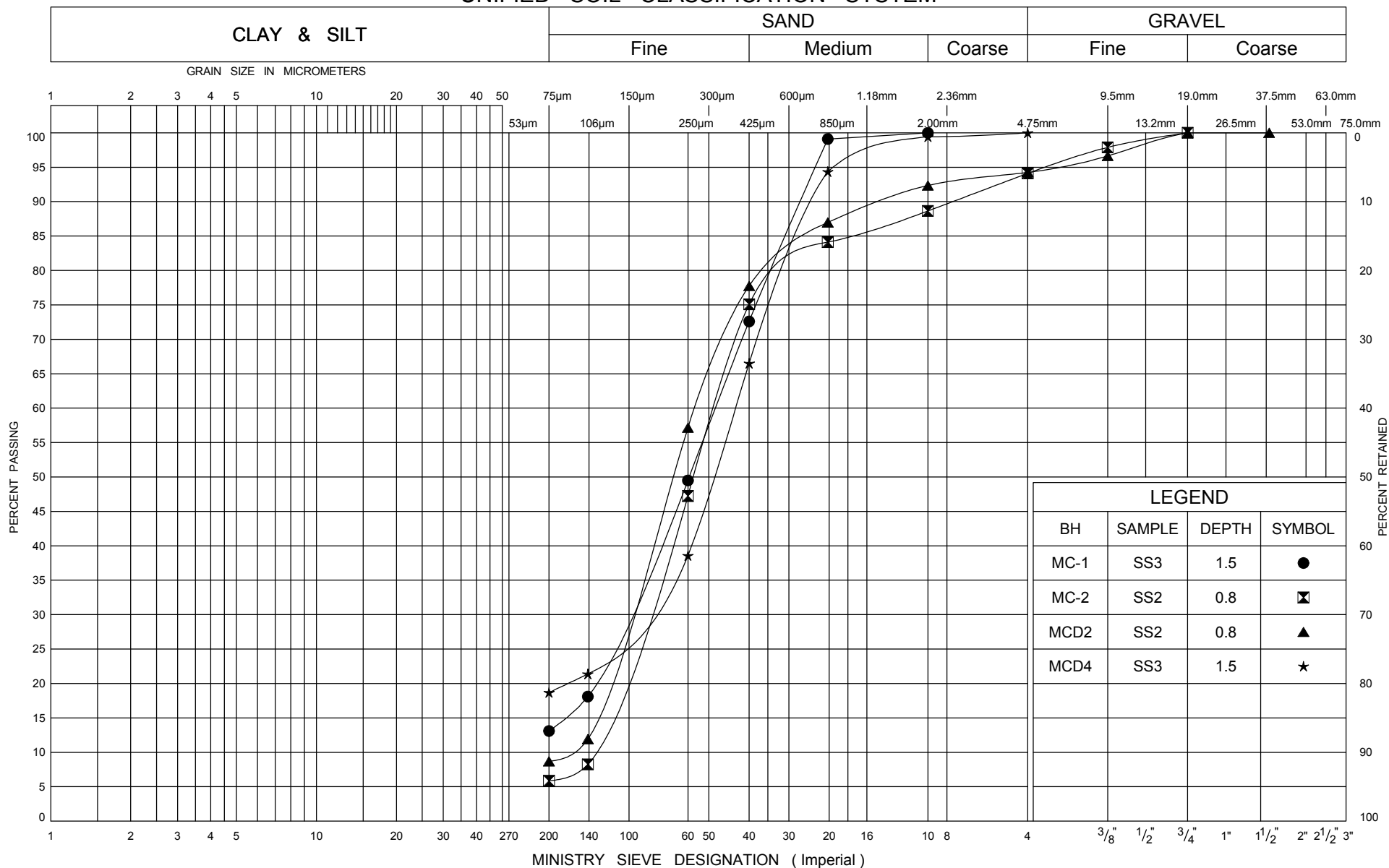


# UNIFIED SOIL CLASSIFICATION SYSTEM



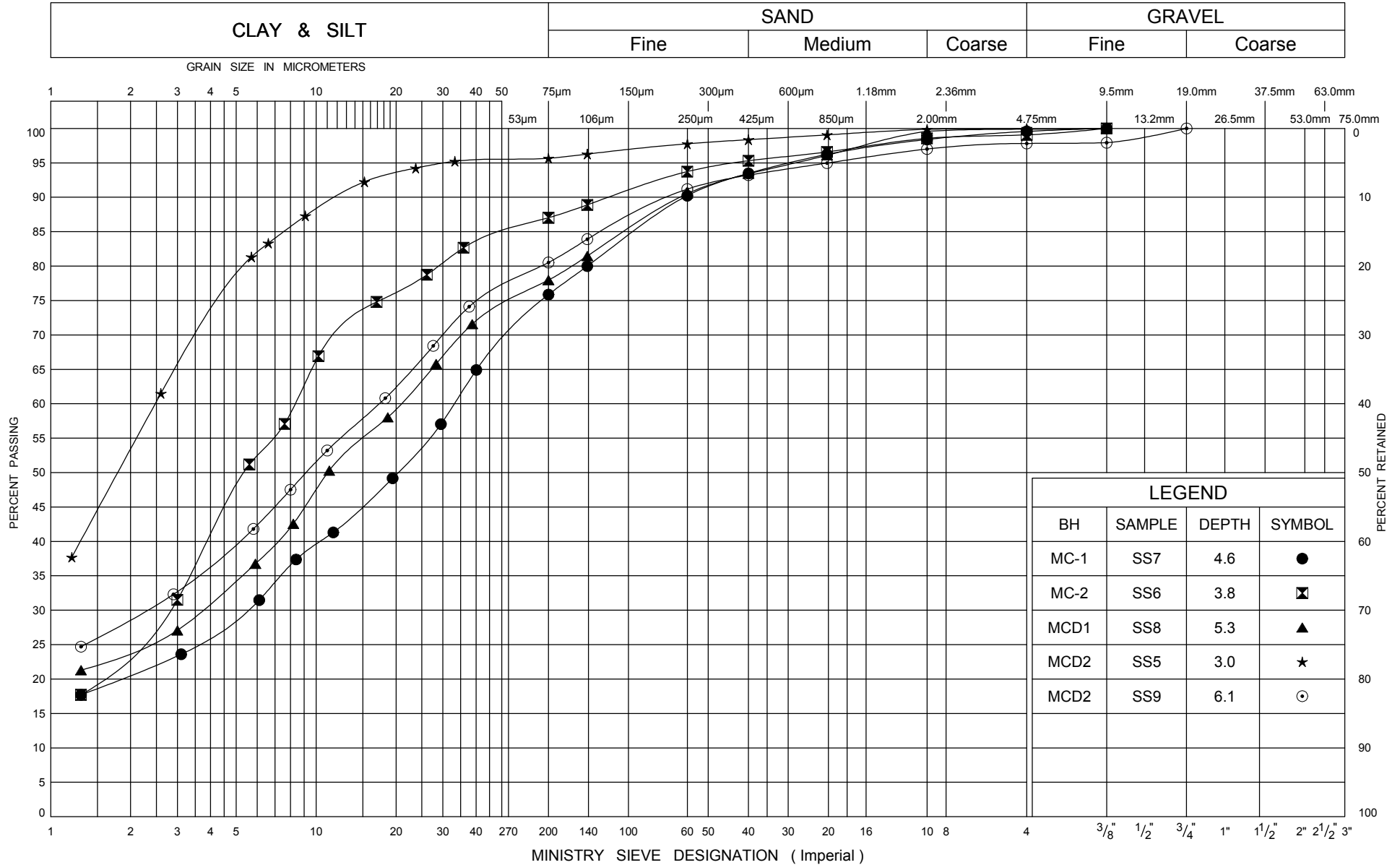


# UNIFIED SOIL CLASSIFICATION SYSTEM





# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SILTY CLAY TO CLAYEY SILT

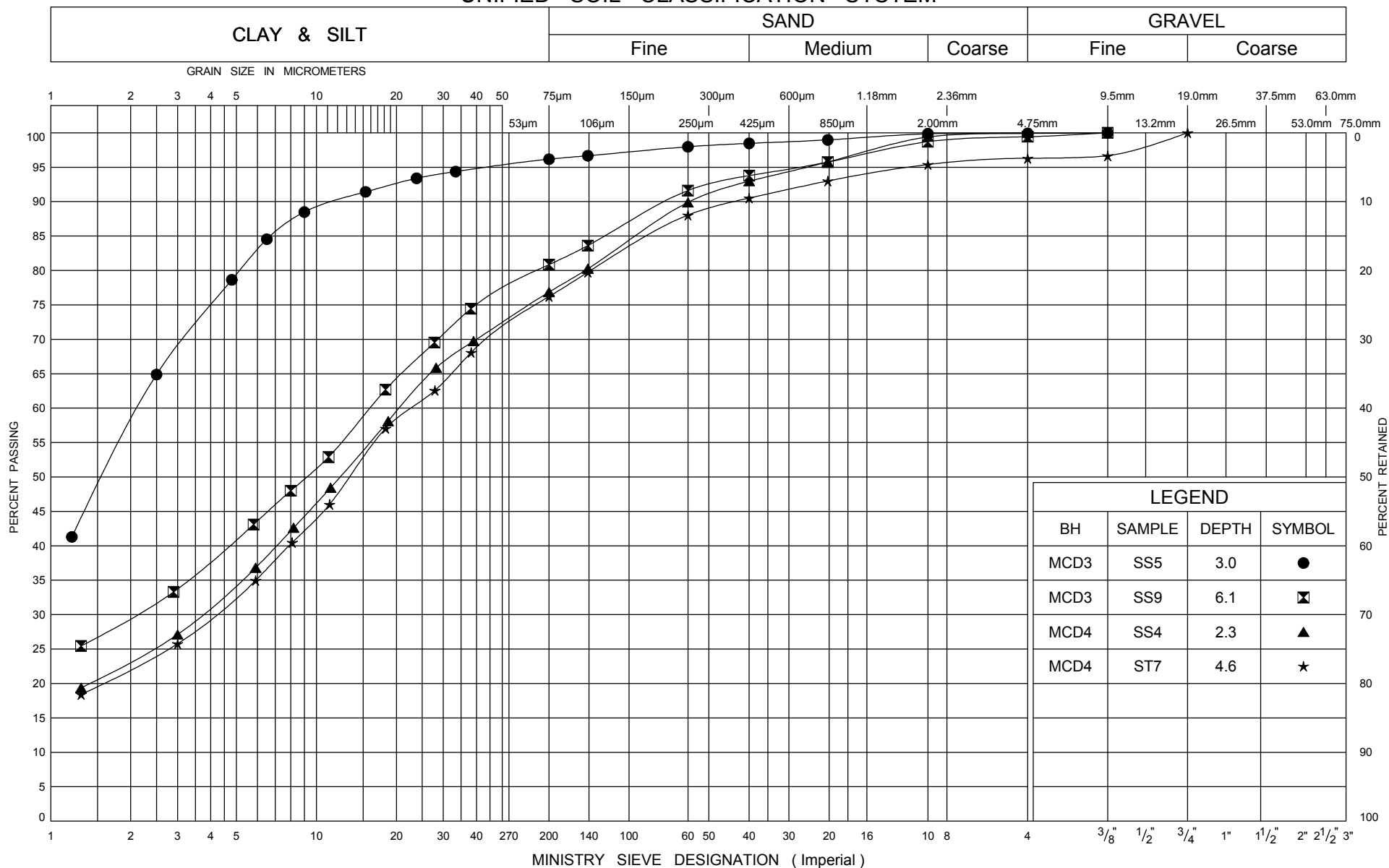
FIG No B1-3

G W P 5233-06-00

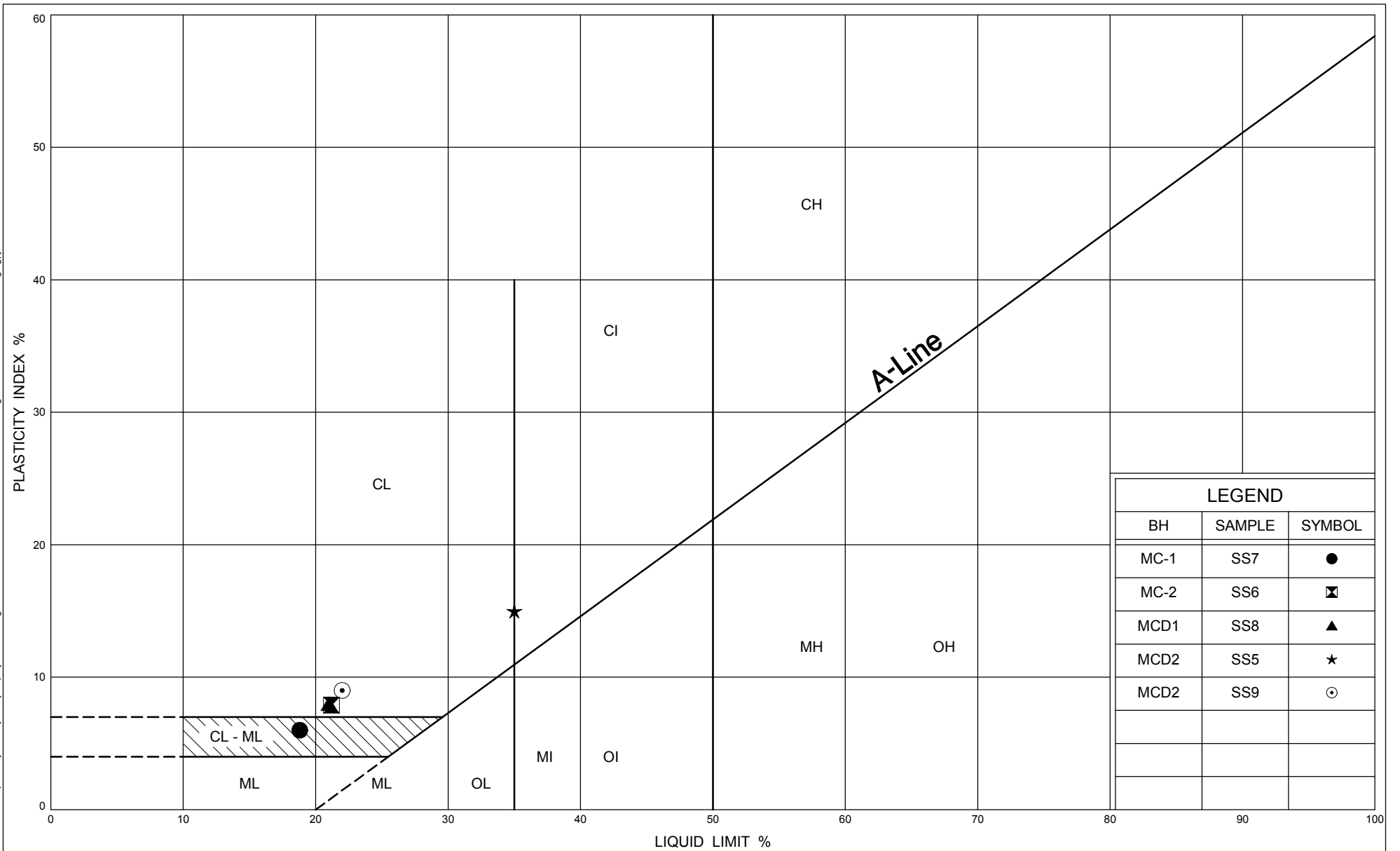
Montcalm Bridge Replacement



# UNIFIED SOIL CLASSIFICATION SYSTEM







Ministry of  
Transportation

## PLASTICITY CHART

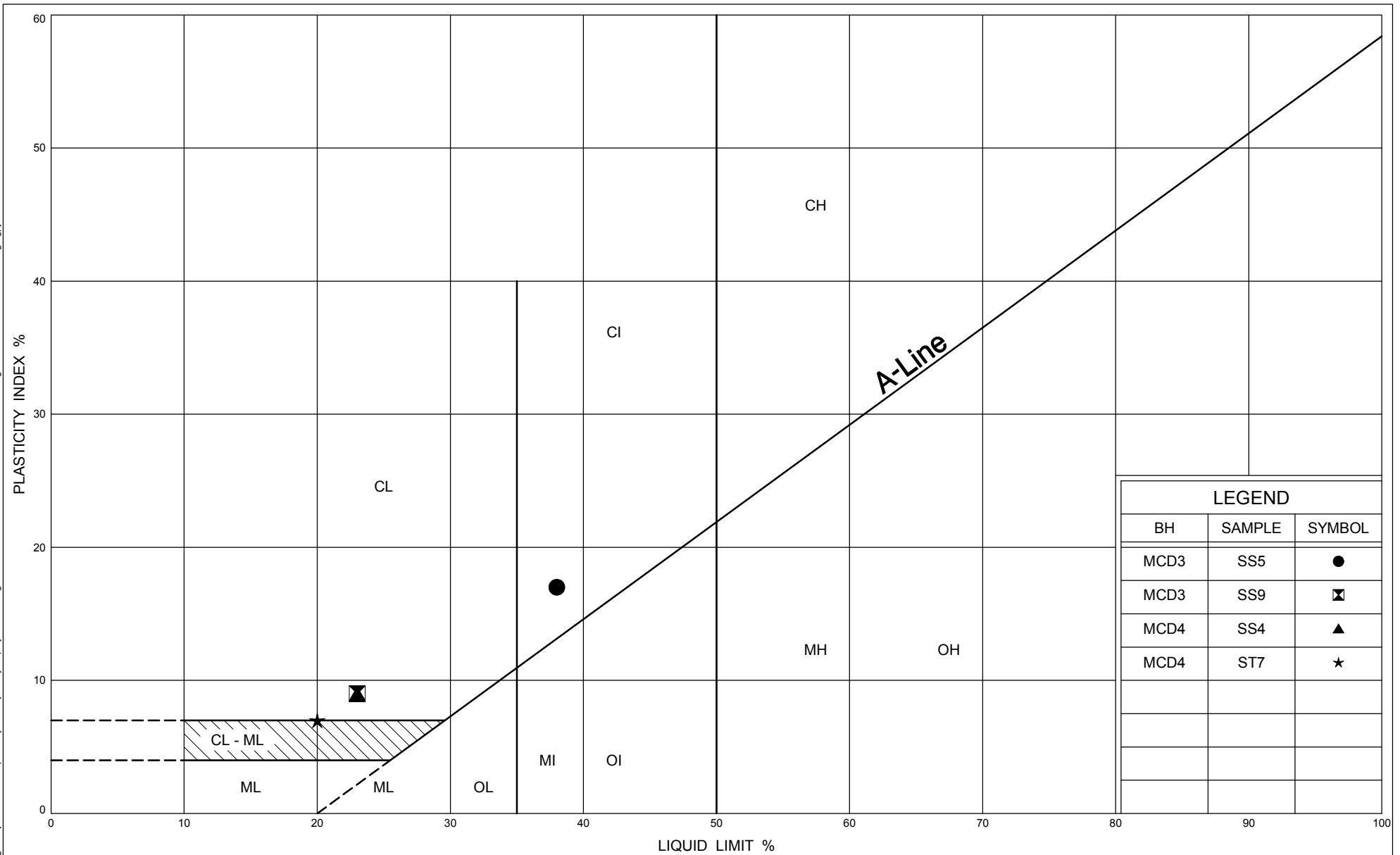
### SILTY CLAY TO CLAYEY SILT

FIG No B1-5

G W P 5233-06-00

Montcalm Bridge Replacement





Ministry of  
Transportation

# **PLASTICITY CHART** **SILTY CLAY TO CLAYEY SILT**

FIG No B1-6

G W P 5233-06-00

Montcalm Bridge Replacement

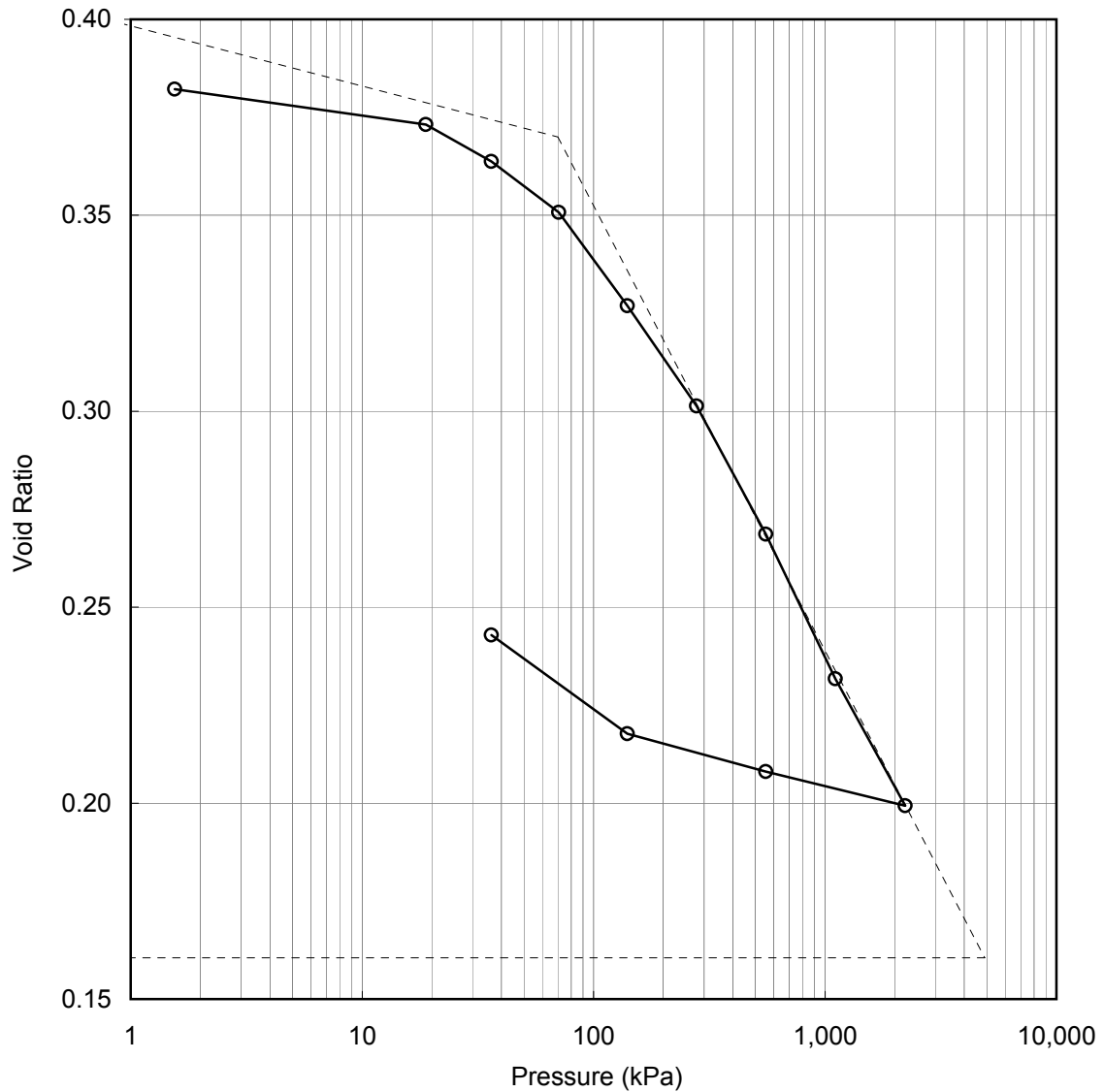


# CONSOLIDATION TEST

FIG No B1-7

MONTCLAM BRIDGE, BH MCD4, SAMPLE Sa7  
SILTY CLAY

e vs Pressure



Soil Type : Silty Clay

$e_o =$	0.40	$\omega_L =$	-	$Po' =$	0 kPa
$\omega =$	14%	$\omega_P =$	-	$Pc' =$	70 kPa
$\gamma =$	21.9 kN/m <sup>3</sup>	$PI =$	-	$Cc =$	0.114
$G_s =$	2.71			$Cr =$	0.015

Project No. : 11-10-5076  
Date : February 2012



**Terraprobe Inc.**

Prepared By : MD  
Checked By : JC

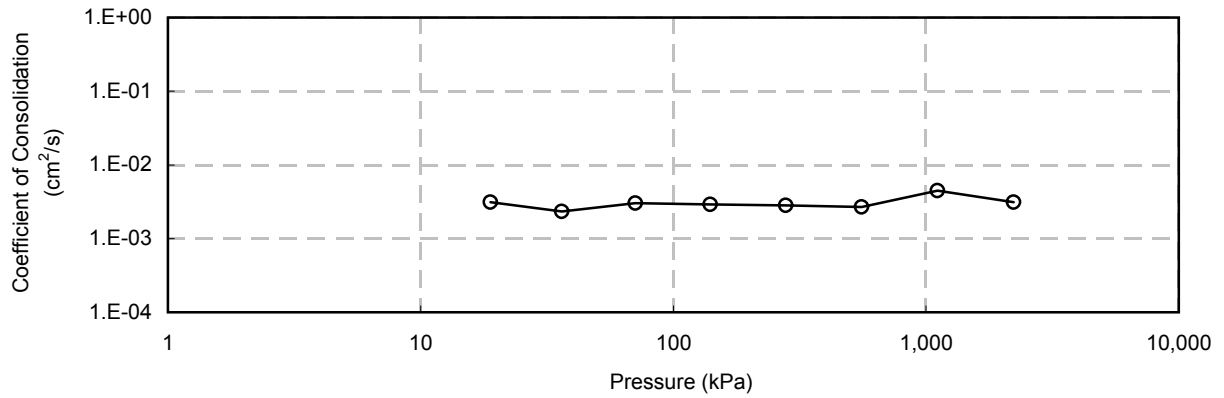


# CONSOLIDATION TEST

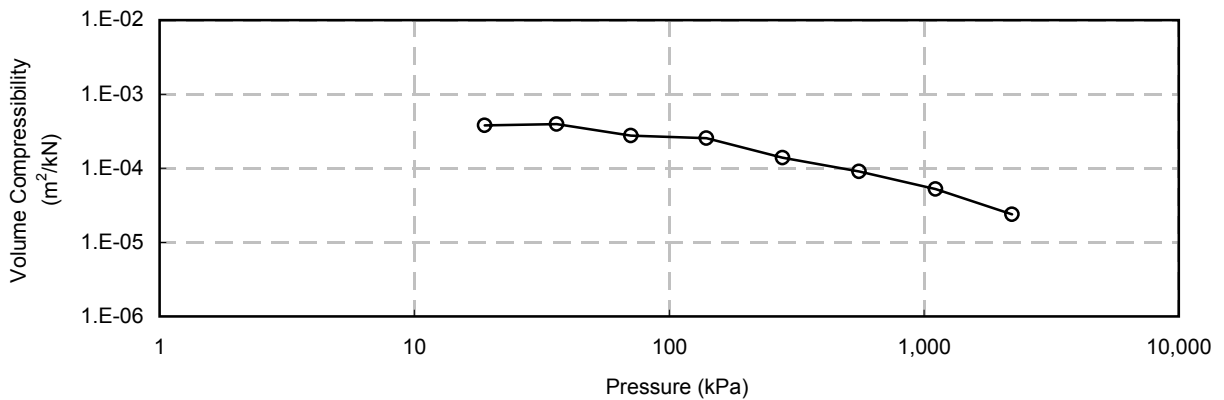
FIG No B1-8

MONTCLAM BRIDGE, BH MCD4, SAMPLE Sa7  
SILTY CLAY

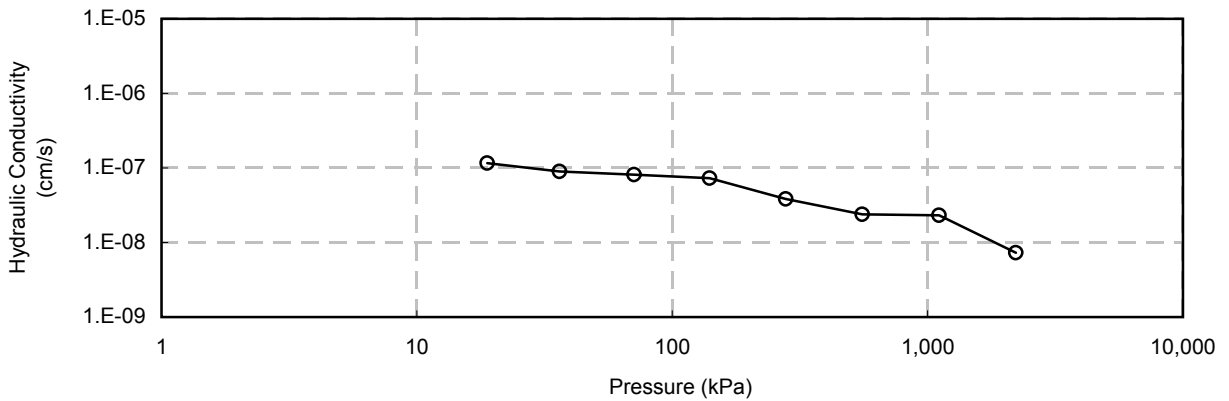
Cv vs Pressure



mv vs Pressure



k vs Pressure



Project No. : 11-10-5076  
Date : February 2012



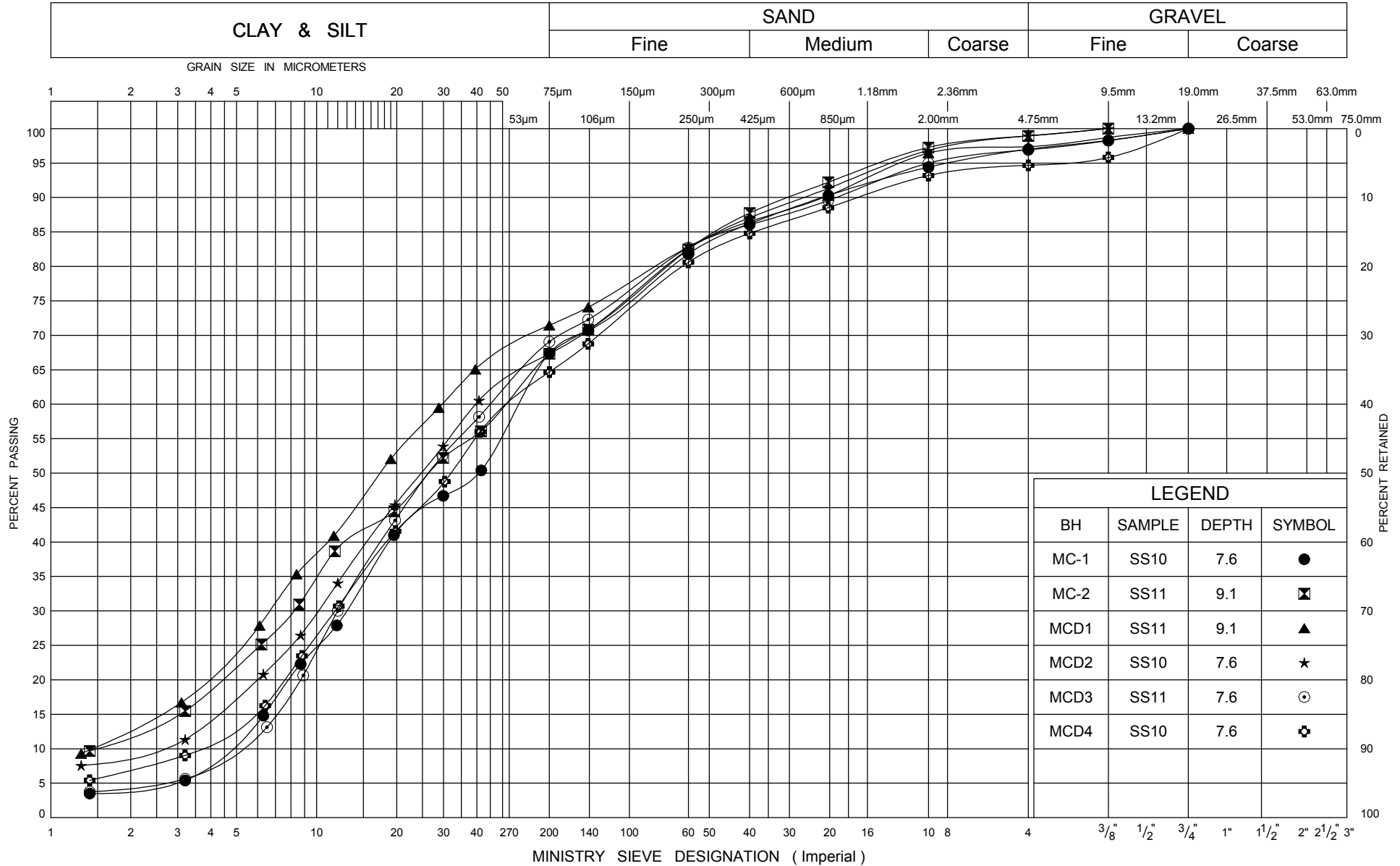
**Terraprobe Inc.**

Prepared By : MD  
Checked By : JC

\\Pdo\server1-Project Files\11-Geotechnical\2010\11-10-5001 to 5099\11-10-5076\11-10-5076.D\_Calcs\_Analysis\_Info\Settlement (MD)\11-10-5076 Consolidation Results montclam silty clay.xls

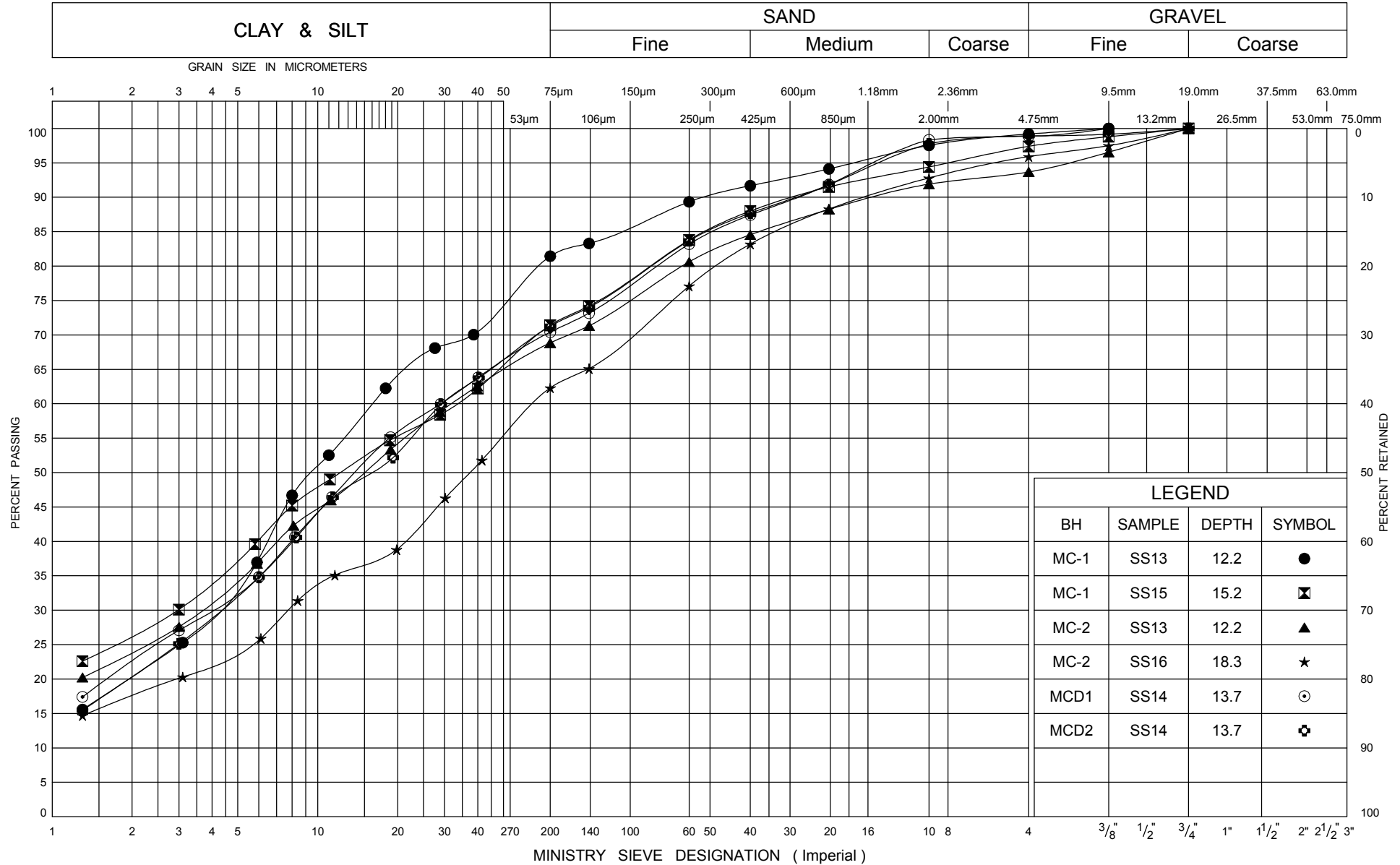


# UNIFIED SOIL CLASSIFICATION SYSTEM





# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

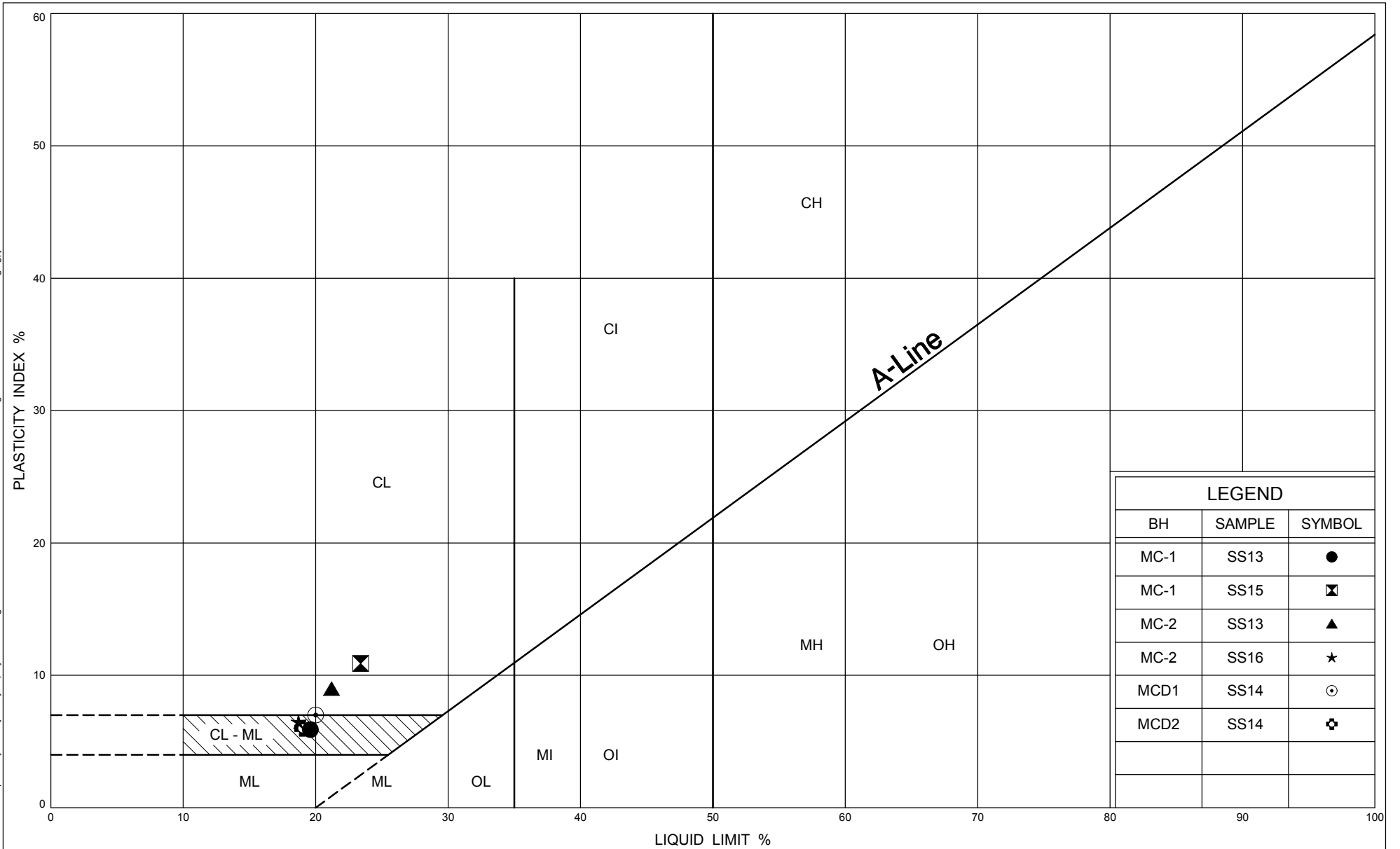
## GRAIN SIZE DISTRIBUTION CLAYEY SILT TO SILTY CLAY TILL

FIG No B1-10

G W P 5233-06-00

Montcalm Bridge Replacement





Ministry of  
Transportation

## PLASTICITY CHART

### CLAYEY SILT TO SILTY CLAY TILL

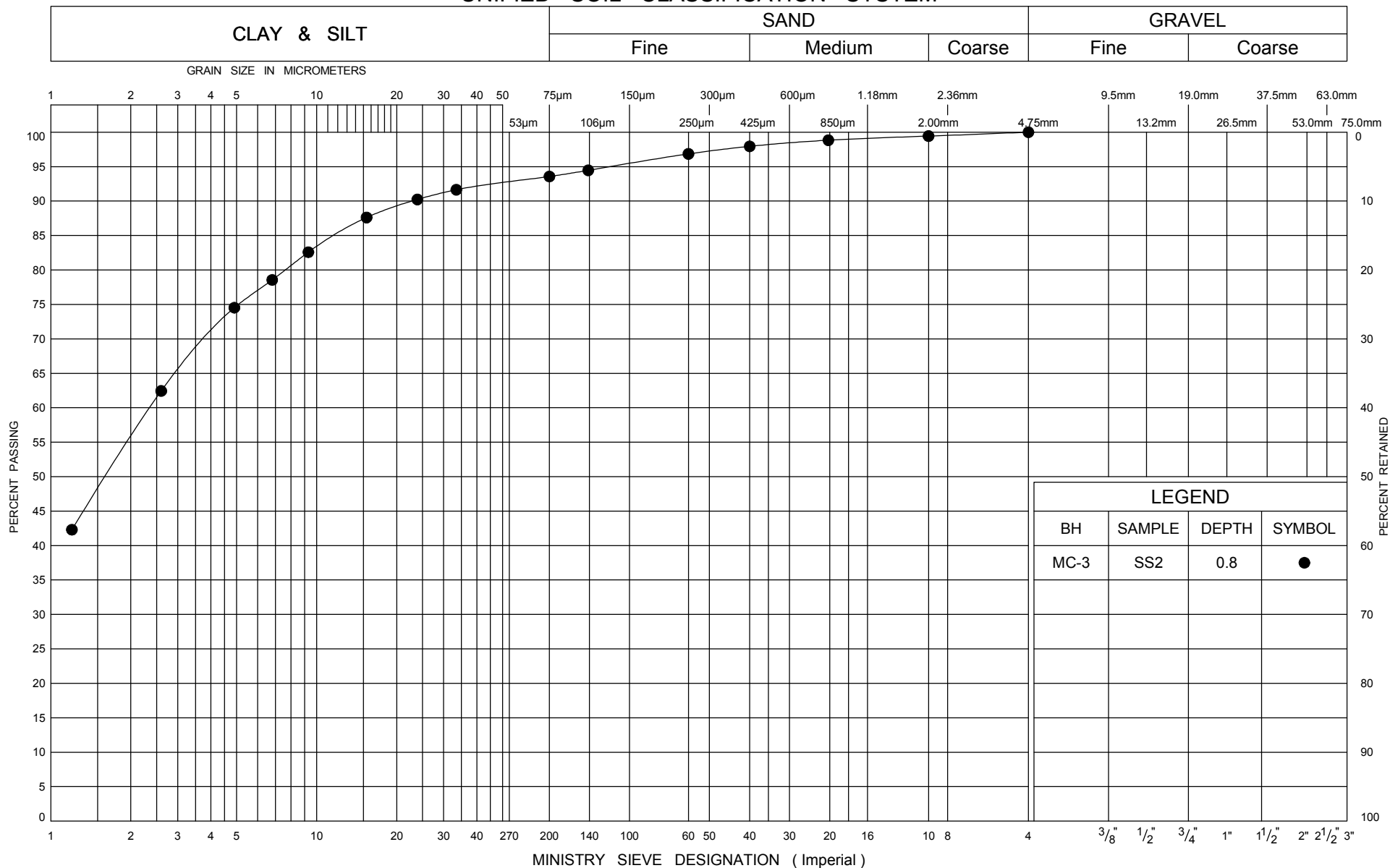
FIG No B1-11

G W P 5233-06-00

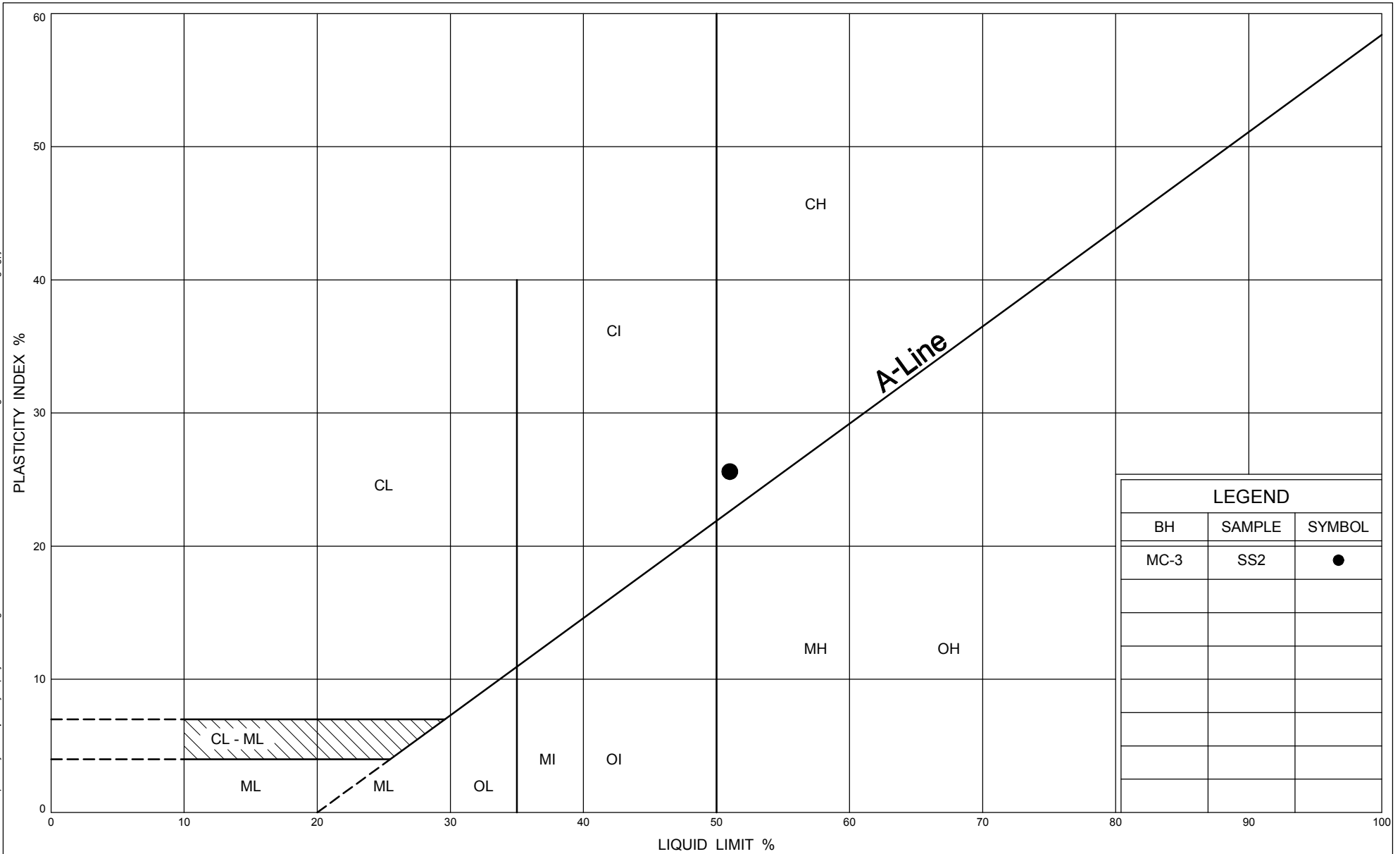
Montcalm Bridge Replacement



# UNIFIED SOIL CLASSIFICATION SYSTEM







Ministry of  
Transportation

## PLASTICITY CHART

### FILL - SILTY CLAY

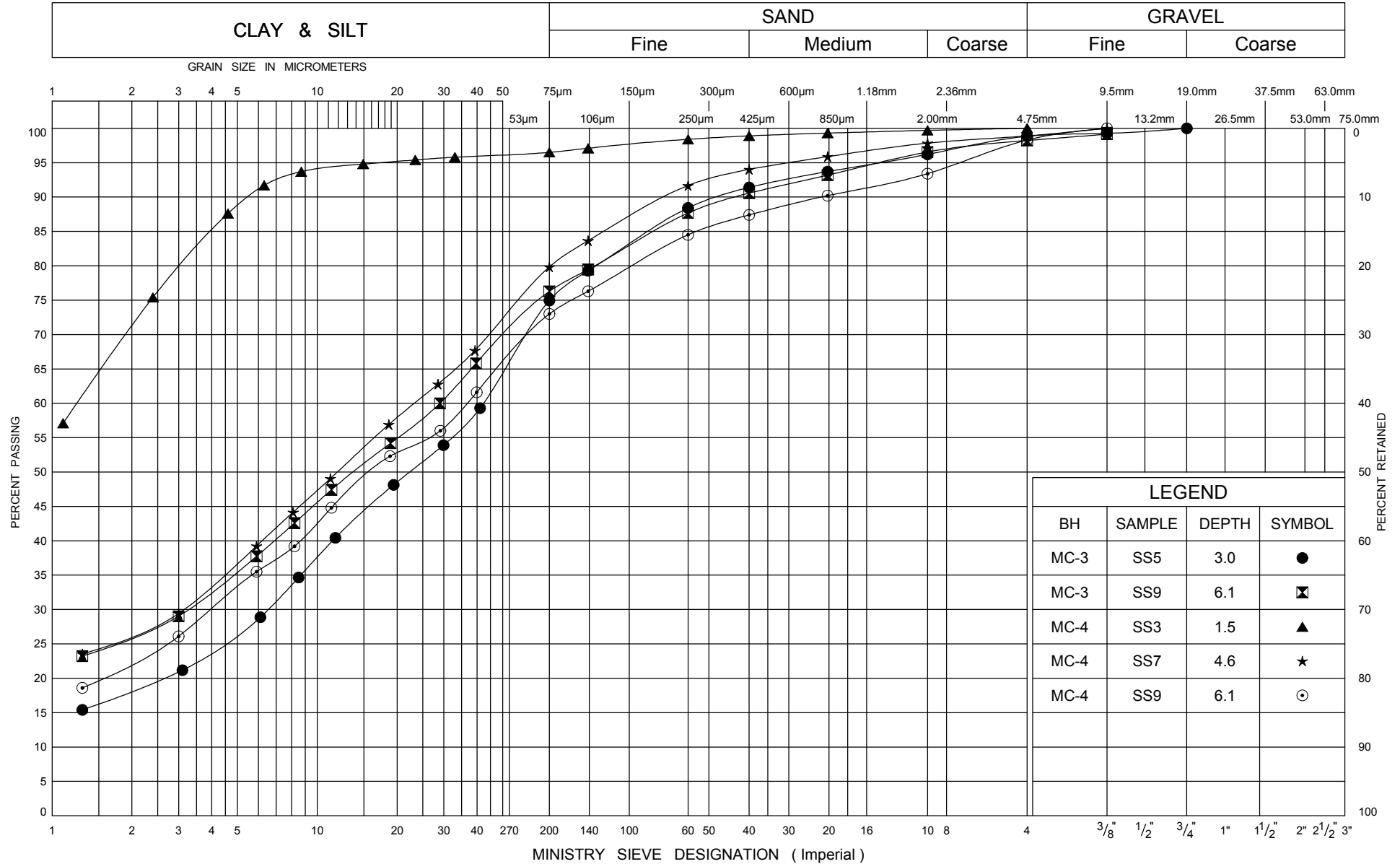
FIG No B2-2

G W P 5233-06-00

Montcalm Bridge Replacement



# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
CLAYEY SILT TO SILTY CLAY

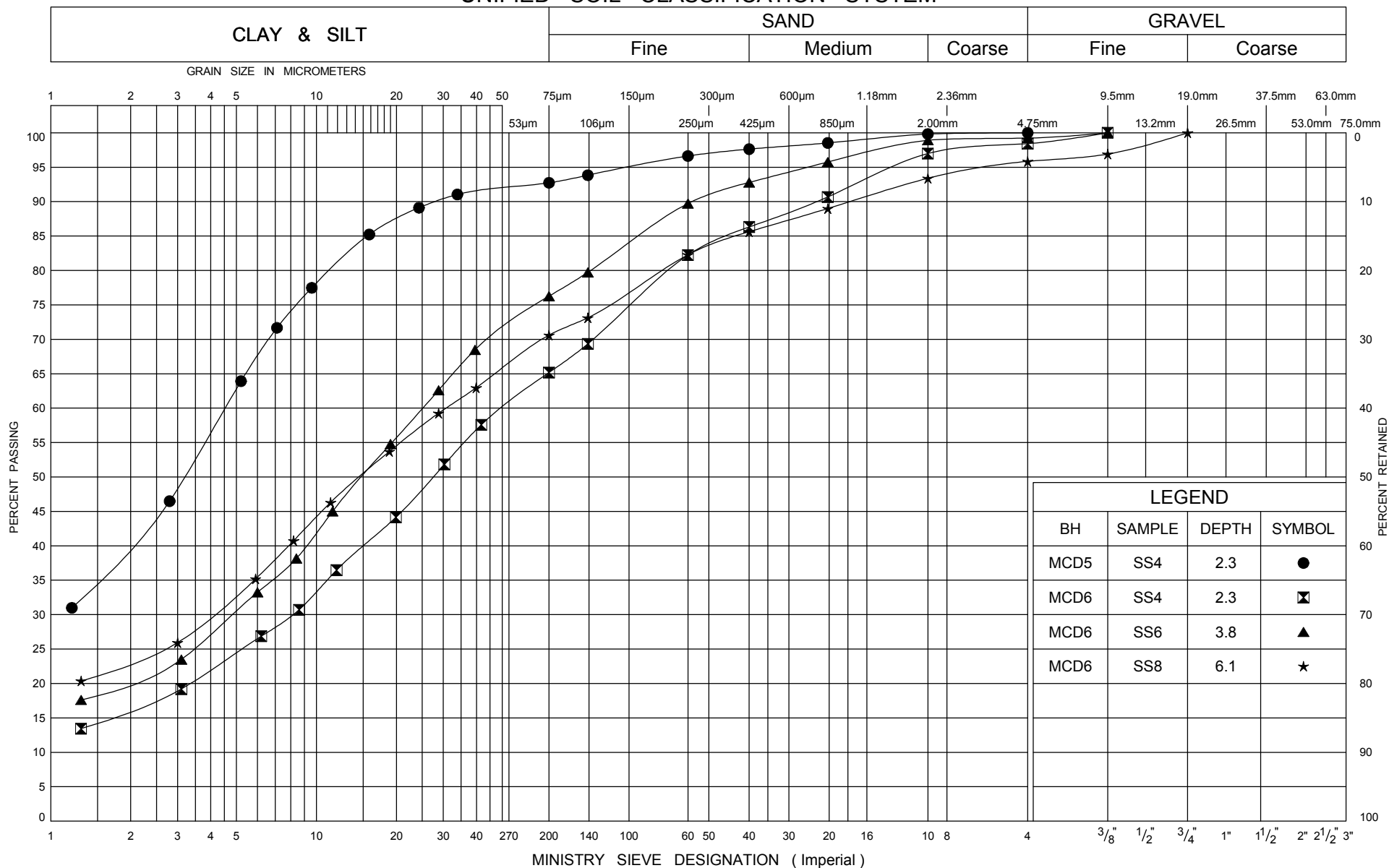
FIG No B2-3

G W P 5233-06-00

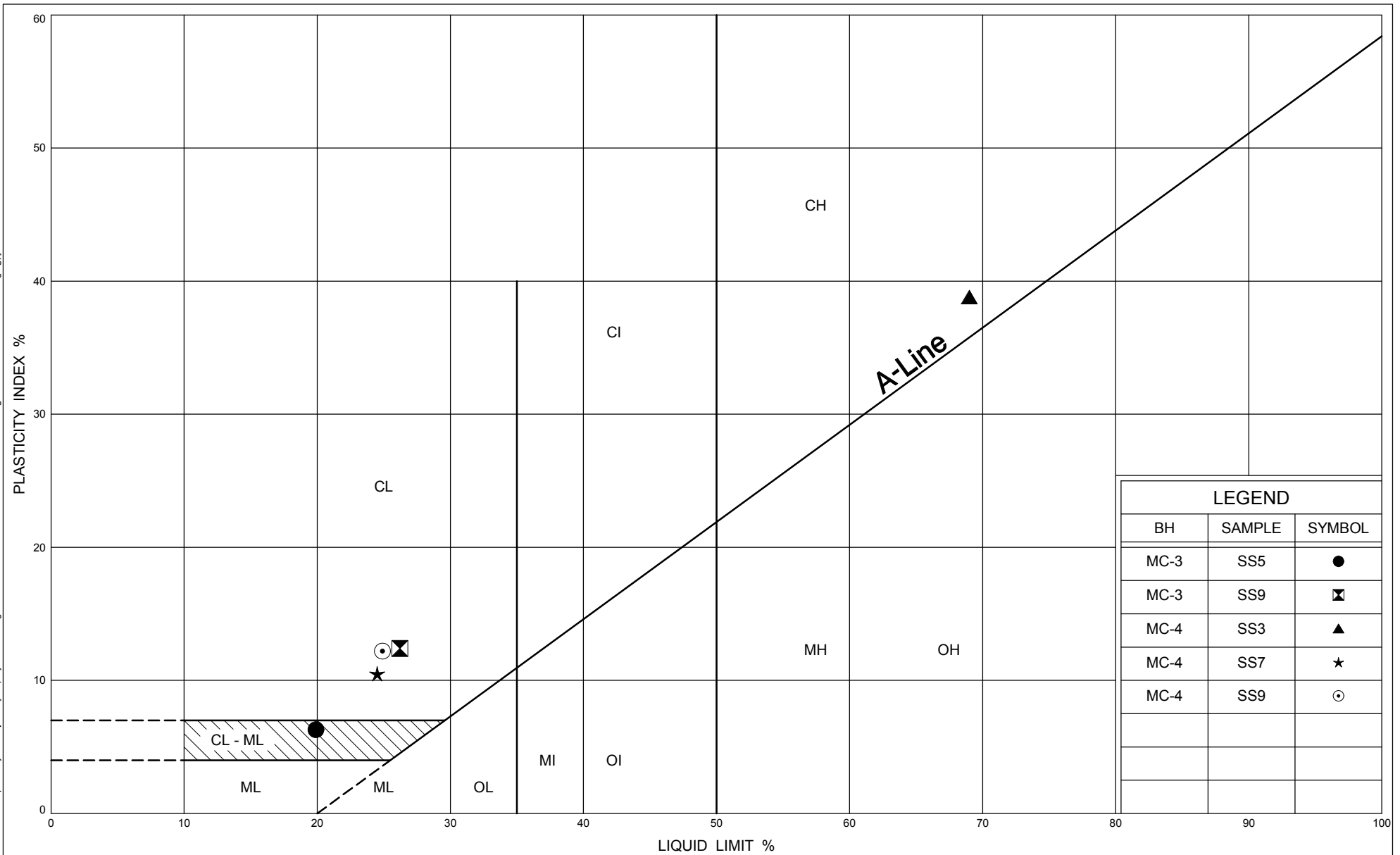
Montcalm Bridge Replacement



# UNIFIED SOIL CLASSIFICATION SYSTEM







Ministry of  
Transportation

## PLASTICITY CHART

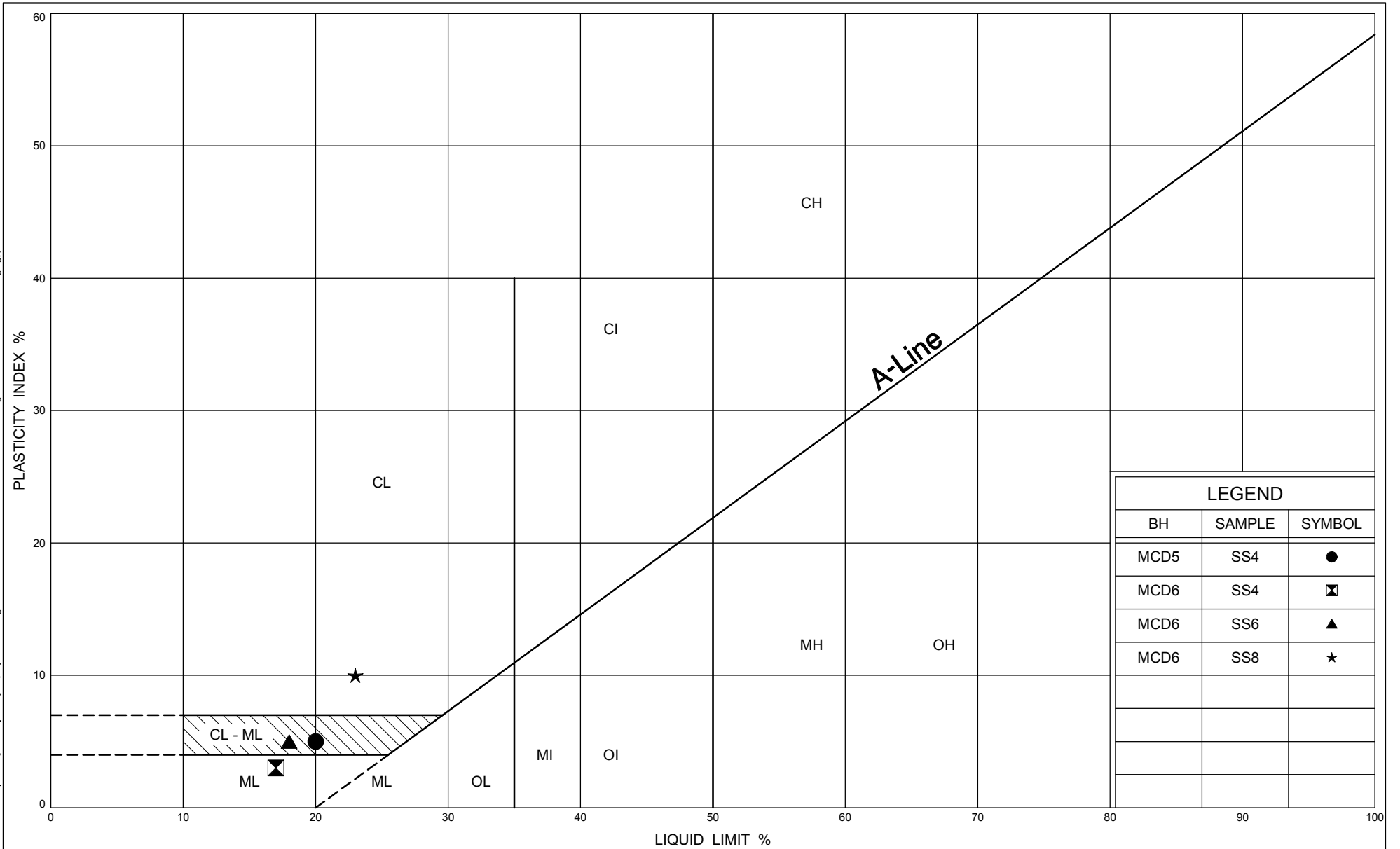
### CLAYEY SILT TO SILTY CLAY

FIG No B2-5

G W P 5233-06-00

Montcalm Bridge Replacement





Ministry of  
Transportation

## PLASTICITY CHART

### CLAYEY SILT TO SILTY CLAY

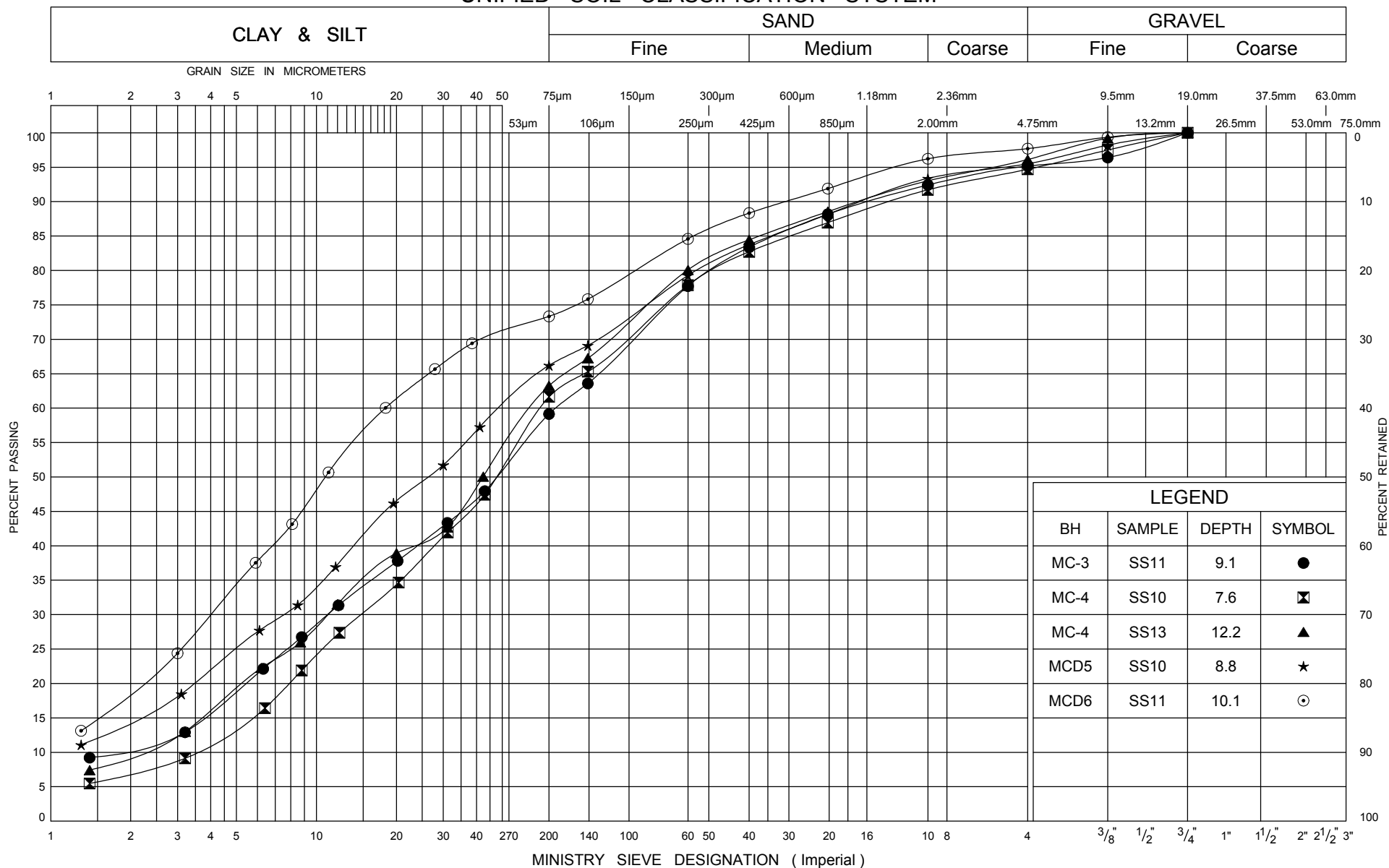
FIG No B2-6

G W P 5233-06-00

Montcalm Bridge Replacement

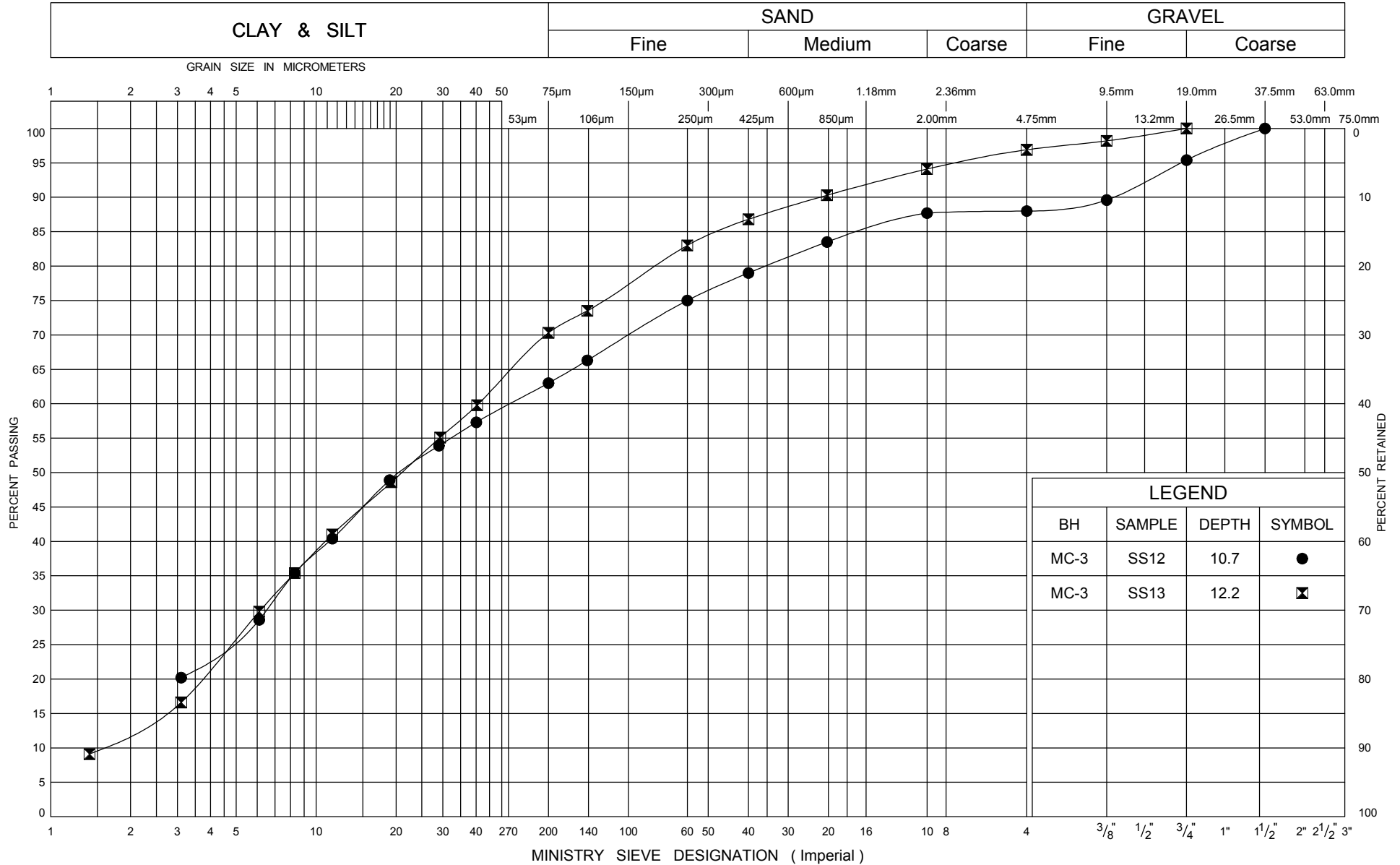


# UNIFIED SOIL CLASSIFICATION SYSTEM



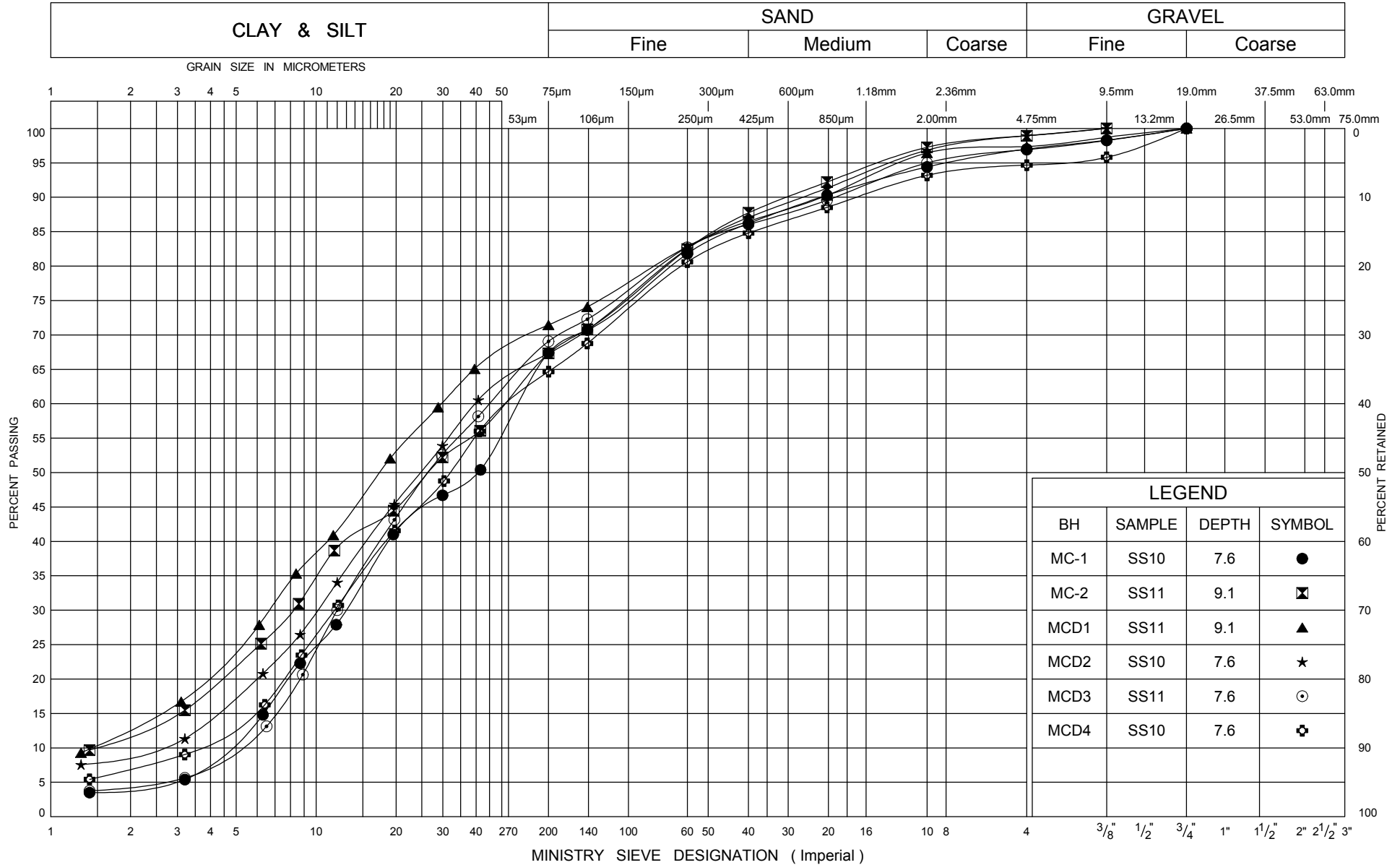


# UNIFIED SOIL CLASSIFICATION SYSTEM





# UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of  
Transportation

## GRAIN SIZE DISTRIBUTION SANDY SILT TILL

FIG No B2-9

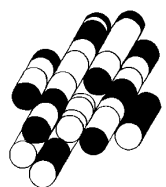
G W P 5233-06-00

Montcalm Bridge Replacement



# APPENDIX C

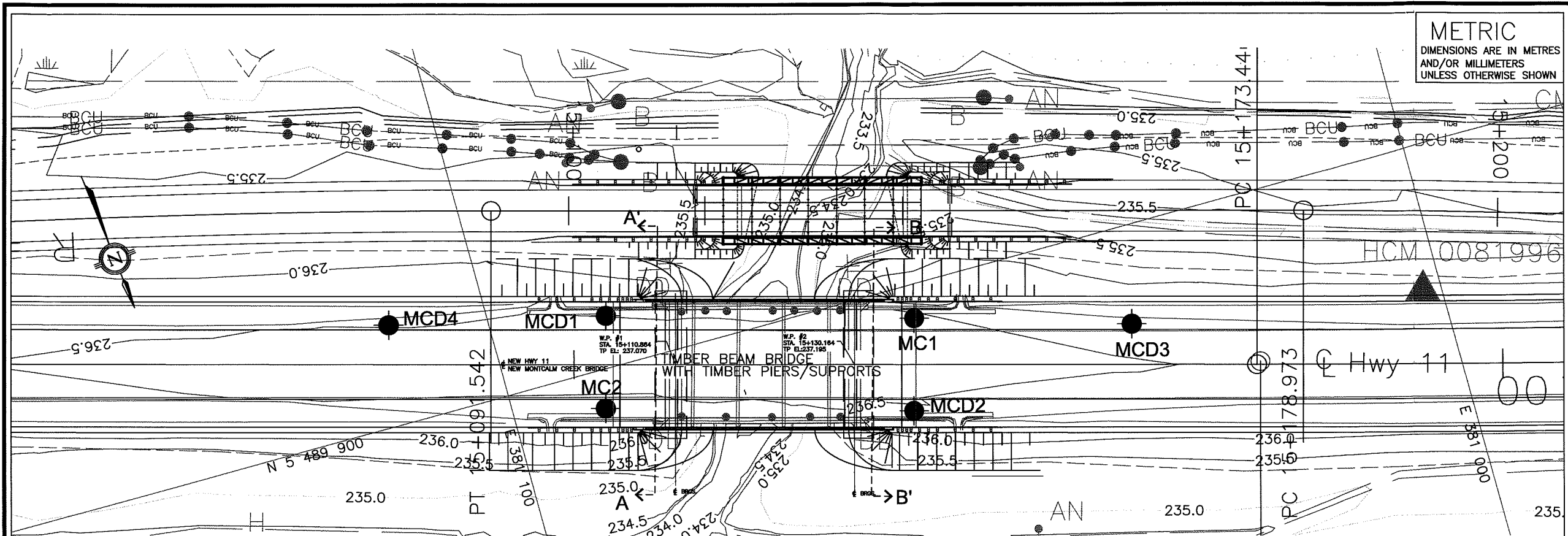
**TERRAPROBE INC.**











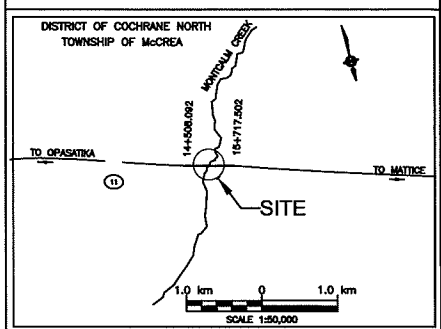
METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETERS  
UNLESS OTHERWISE SHOWN

CONT No  
GWP No 5233-06-00

MONTCALM CREEK BRIDGE  
BOREHOLE LOCATION AND SOIL STRATA

MCCORMICK RANKIN CORPORATION  
MRC

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



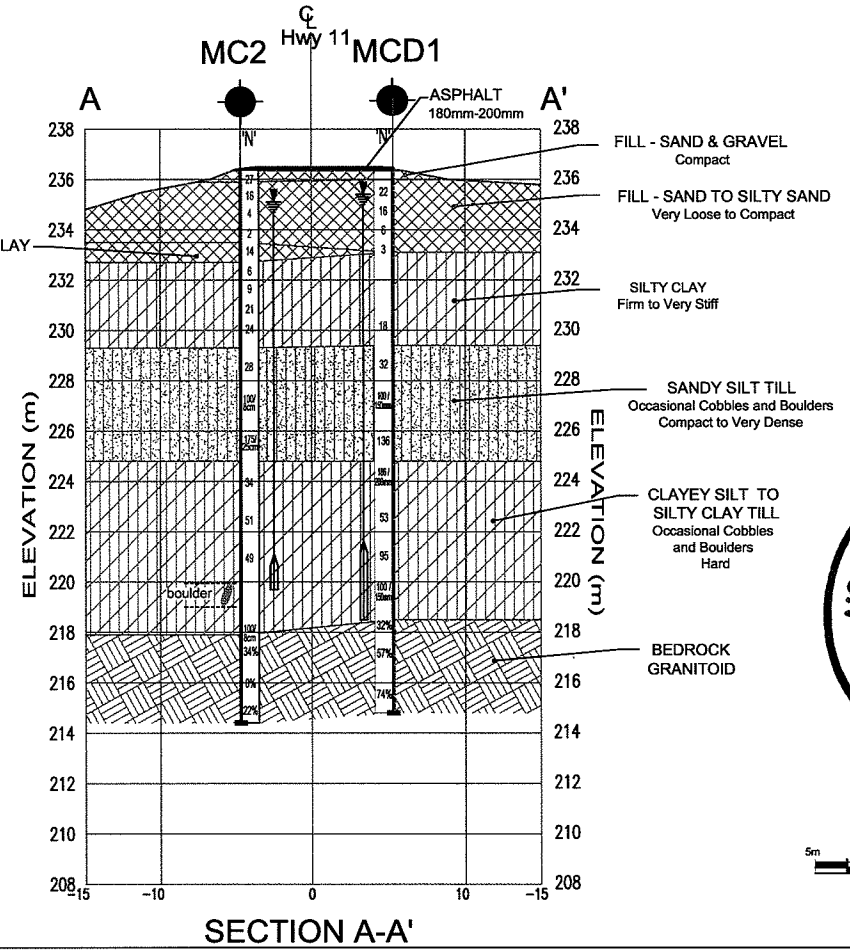
KEY PLAN

- 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
  - 90% Rock Quality Designation
  - A/R Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
MC1	236.5	5 489 903.6	381 055.6
MC2	236.4	5 489 904.1	381 090.2
MCD1	236.5	5 489 894.5	381 087.5
MCD2	236.5	5 489 913.2	381 058.3
MCD3	236.4	5 489 910.4	381 033.2
MCD4	236.5	5 489 889.2	381 110.4

NOTE  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.  
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS	DATE	BY	DESCRIPTION	
			DESIGN	DATE
			G.M.	MAY 2012
			K.C.	
			CHK G.M.	
			STRUCT 39W-058	
			GEORES 42G-36	

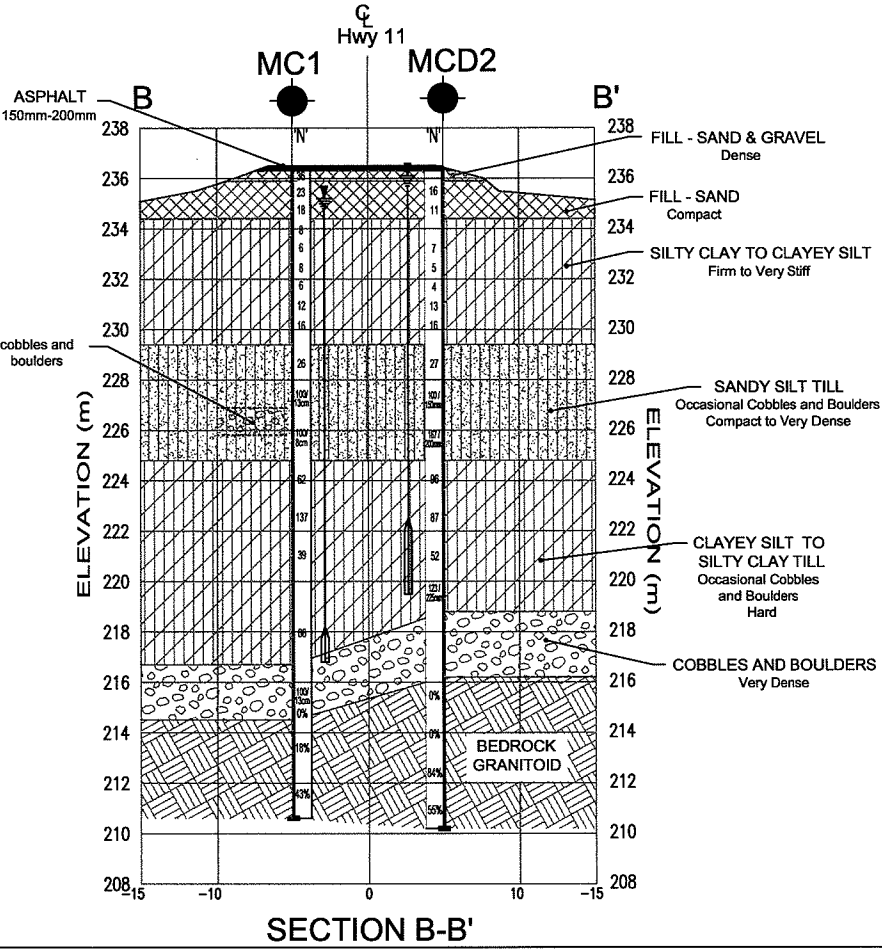


PLAN

REGISTERED PROFESSIONAL ENGINEER  
M. TANOS  
23-5-2012  
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER  
J. G. MUCKLE  
32997017  
MAY 23, 2012  
PROVINCE OF ONTARIO

HORIZONTAL SCALE  
5m 2.5 0 2.5 5m  
VERTICAL SCALE  
5m 2.5 0 2.5 5m

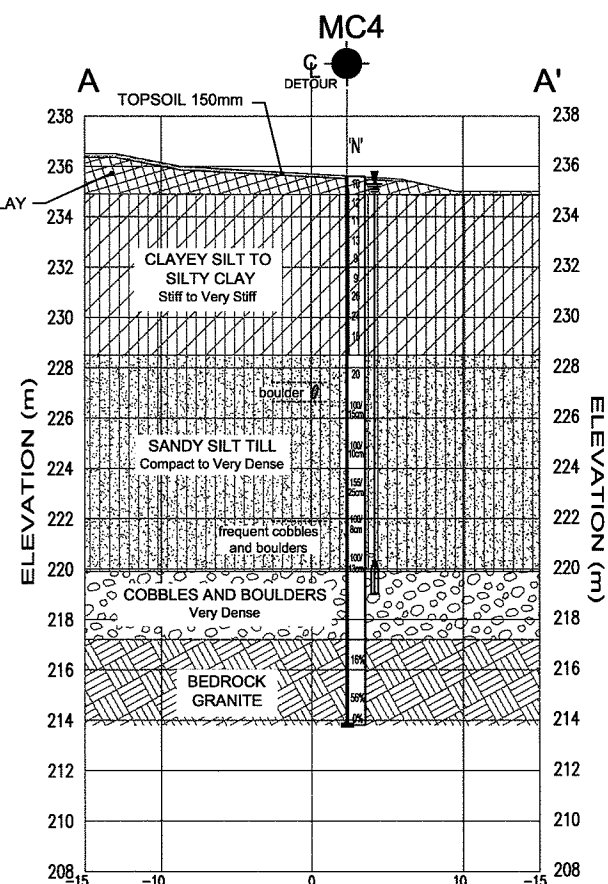
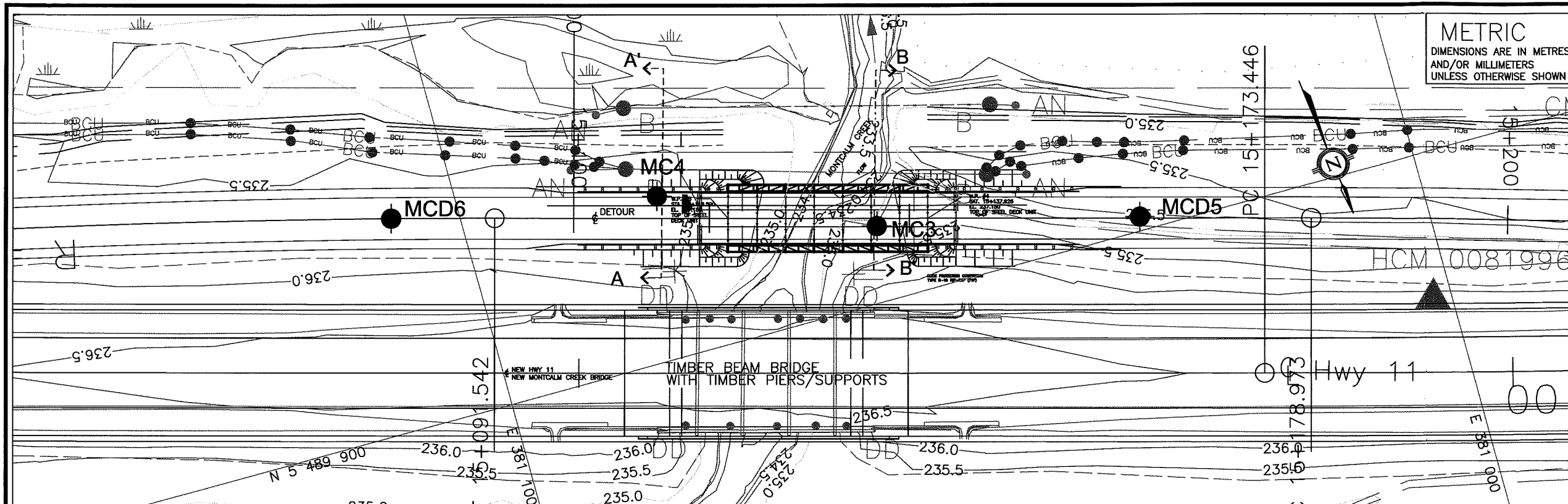


SECTION B-B'

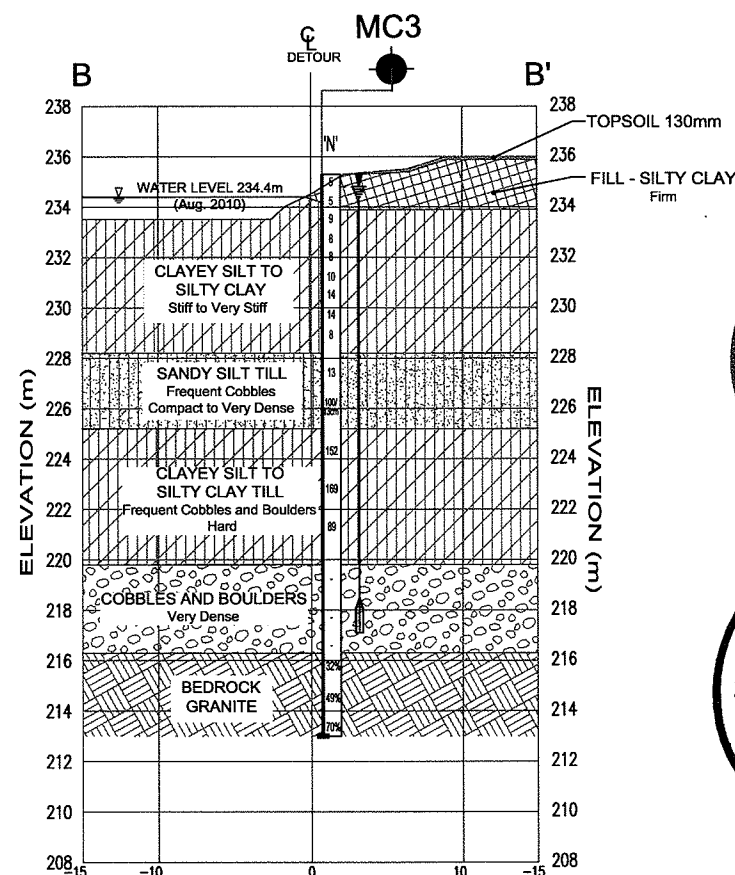
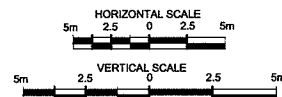








PROFILE C DETOUR ALIGNMENT



SECTION B-B'



CONT No  
GWP No 5233-06-00

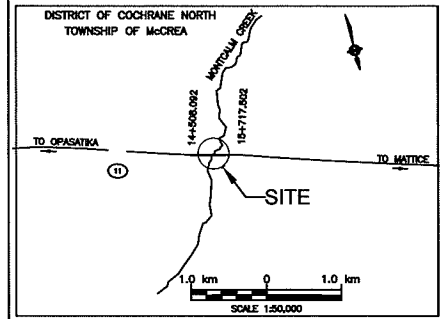
MONTCALM CREEK DETOUR  
BOREHOLE LOCATION AND SOIL STRATA

SHEET  
4 OF 4

MCCORMICK RANKIN CORPORATION

**MRC**

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



**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
MC3	235.3	5 489 892.0	381 057.3
MC4	235.6	5 489 882.6	381 079.1
MCD5	235.7	5 489 898.5	381 029.9
MCD6	235.7	5 489 877.4	381 107.1

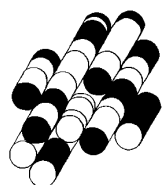
**NOTE**  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.  
This drawing is for subsurface information only. Surface details and features are for conceptual illustration.

REVISIONS			
DATE	BY	DESCRIPTION	
DESIGN G.M.	CODE CHBDC2006	LOAD	DATE MAY 2012
DRAWN K.C.	CHK G.M.	STRUCT 39W-058	GEORES 42G-38



# APPENDIX D

**TERRAPROBE INC.**





**COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT**

Foundation Element	Driven Piles	Augered Caissons	Footings on Native Soil	Footings on Engineered Fill
<b>MONTCALM CREEK EXISTING BRIDGE SITE</b>				
<b>East and West Abutments</b>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by driving piles to effective refusal.</li> <li>ii. Readily installed.</li> <li>iii. Reliable performance and low risk.</li> <li>iv. Allows for the design of an integral or semi-integral abutment.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by founding caissons on till soils.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Relatively high construction effort required to install caissons compared to driven piles.</li> <li>ii. Higher risk of encountering potential construction problems compared to driven piles.</li> <li>iii. Precludes consideration of an integral abutment structure.</li> </ul>	<p><b>Advantages:</b></p> <p>None</p> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Uneconomically large footings due to low geotechnical resistance of soils.</li> <li>ii. Unreliable performance and high risk due to settlement sensitive soils. Potential for unacceptable settlements and differential settlements.</li> <li>iii. Relatively long abutment stems required.</li> <li>iv. Precludes consideration of an integral abutment structure.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Possibility of shortening the abutment height.</li> <li>ii. Allows for the design of a semi-integral abutment.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. High risk due to settlement sensitive soils. Potential for unacceptable settlements and differential settlements</li> <li>ii. Requires relatively large and deep excavations in order to found the engineered fill pad on competent soils.</li> <li>iii. Precludes consideration of an integral abutment structure.</li> </ul>





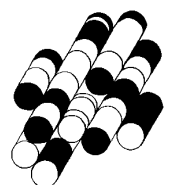
Foundation Element	Driven Piles	Augered Caissons	Footing on Native Soil	Footing on Engineered Fill
<b>MONTCALM CREEK DETOUR ALIGNMENT</b>				
<b>East and West Abutments</b>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by driving piles to effective refusal.</li> <li>ii. Readily installed.</li> <li>iii. Reliable performance and low risk.</li> <li>iv. Allows for the design of an integral or semi-integral abutment.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Construction concerns related to the possibility of piles being obstructed by a boulder during driving.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. High geotechnical resistances available by founding caissons on till soils.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Relatively high construction effort required to install caissons compared to driven piles.</li> <li>ii. Higher risk of encountering potential construction problems compared to driven piles.</li> <li>iii. Precludes consideration of an integral abutment structure.</li> </ul>	<p><b>Advantages:</b></p> <p>None</p> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. Uneconomically large footings due to low geotechnical resistance of soils.</li> <li>ii. Unreliable performance and high risk due to settlement sensitive soils. Potential for unacceptable settlements and differential settlements.</li> <li>iii. Relatively long abutment stems required.</li> <li>iv. Precludes consideration of an integral abutment structure.</li> </ul>	<p><b>Advantages:</b></p> <ul style="list-style-type: none"> <li>i. Possibility of shortening the abutment height.</li> <li>ii. Allows for the design of a semi-integral abutment.</li> </ul> <p><b>Disadvantages:</b></p> <ul style="list-style-type: none"> <li>i. High risk due to settlement sensitive soils. Potential for unacceptable settlements and differential settlements</li> <li>ii. Requires relatively large and deep excavations in order to found the engineered fill pad on competent soils.</li> <li>iii. Precludes consideration of an integral abutment structure.</li> </ul>





# APPENDIX E

**TERRAPROBE INC.**





# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-1)  
Method: Bishop Simplified  
Slope: 2H:1V  
Condition: Undrained

## MATERIAL PROPERTIES

1 Material: Embankment Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 31 deg

2 Material: Silty Clay to Clayey Silt

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 40 kPa

Friction Angle: 0 deg

3 Material: Sandy Silt Till

Unit Weight: 20 kN/m<sup>3</sup>

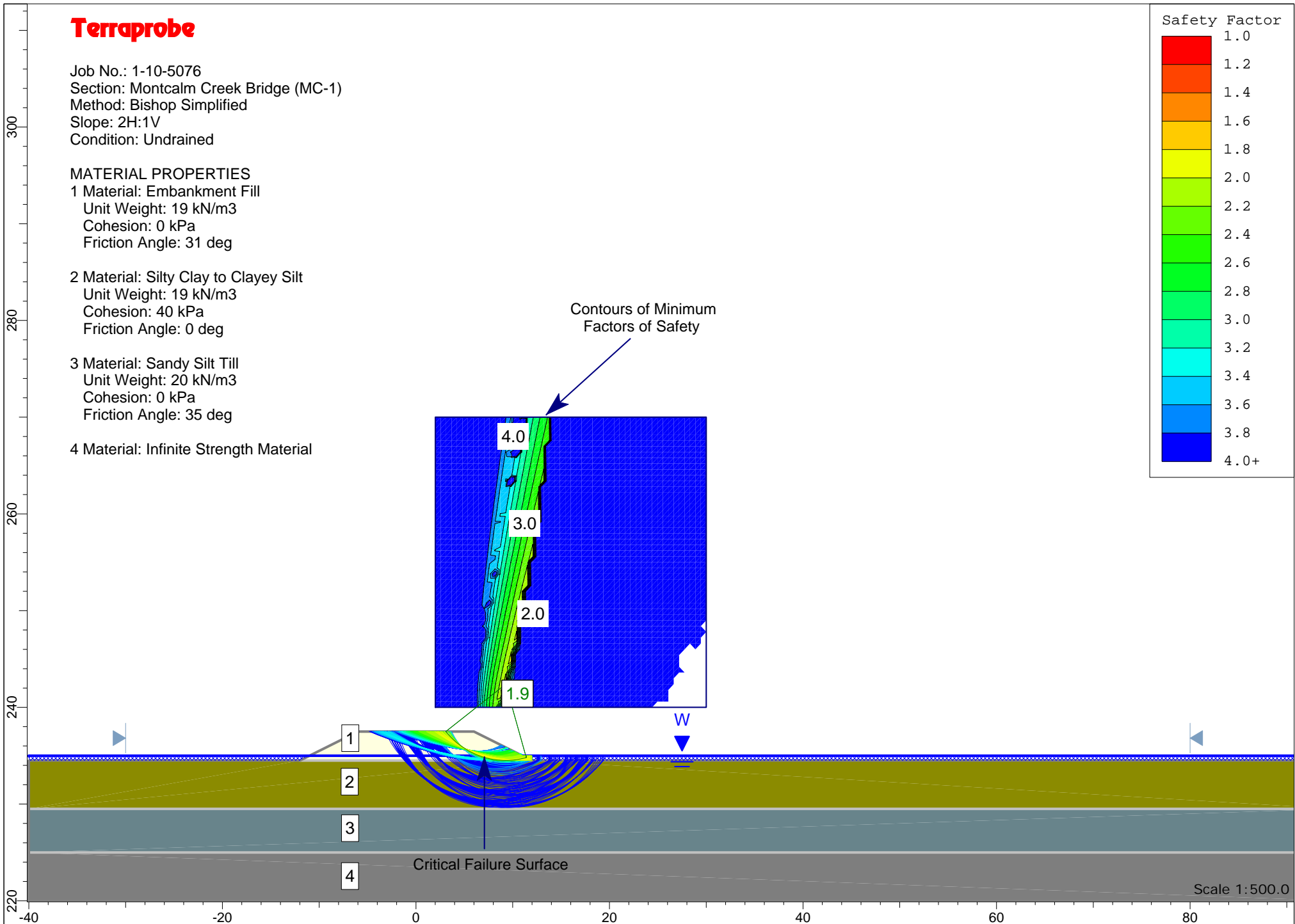
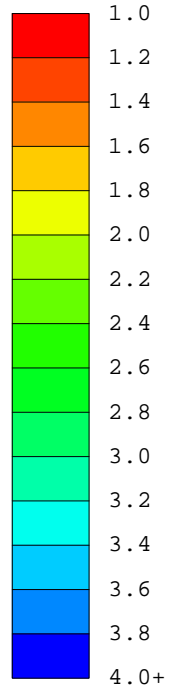
Cohesion: 0 kPa

Friction Angle: 35 deg

4 Material: Infinite Strength Material

Contours of Minimum  
Factors of Safety

Safety Factor





# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-1)  
Method: Bishop Simplified  
Slope: 2H:1V  
Condition: Drained

## MATERIAL PROPERTIES

1 Material: Embankment Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 31 deg

2 Material: Silty Clay to Clayey Silt

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 5 kPa

Friction Angle: 28 deg

3 Material: Sandy Silt Till

Unit Weight: 20 kN/m<sup>3</sup>

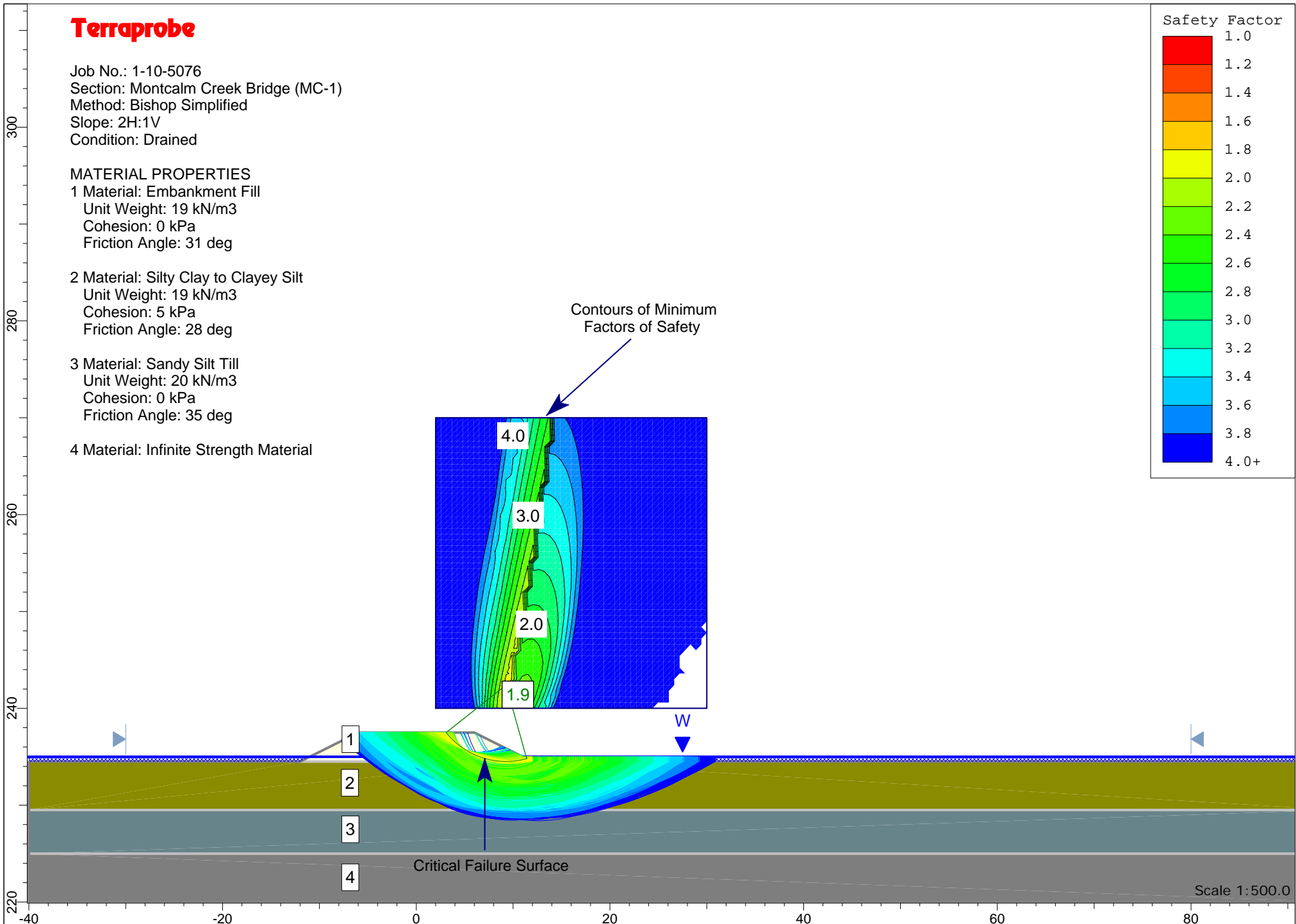
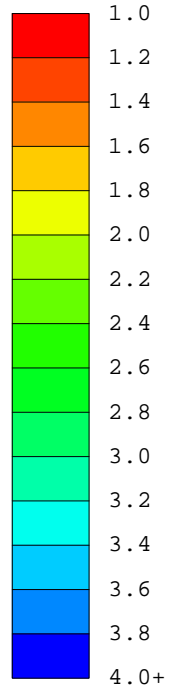
Cohesion: 0 kPa

Friction Angle: 35 deg

4 Material: Infinite Strength Material

Contours of Minimum  
Factors of Safety

Safety Factor





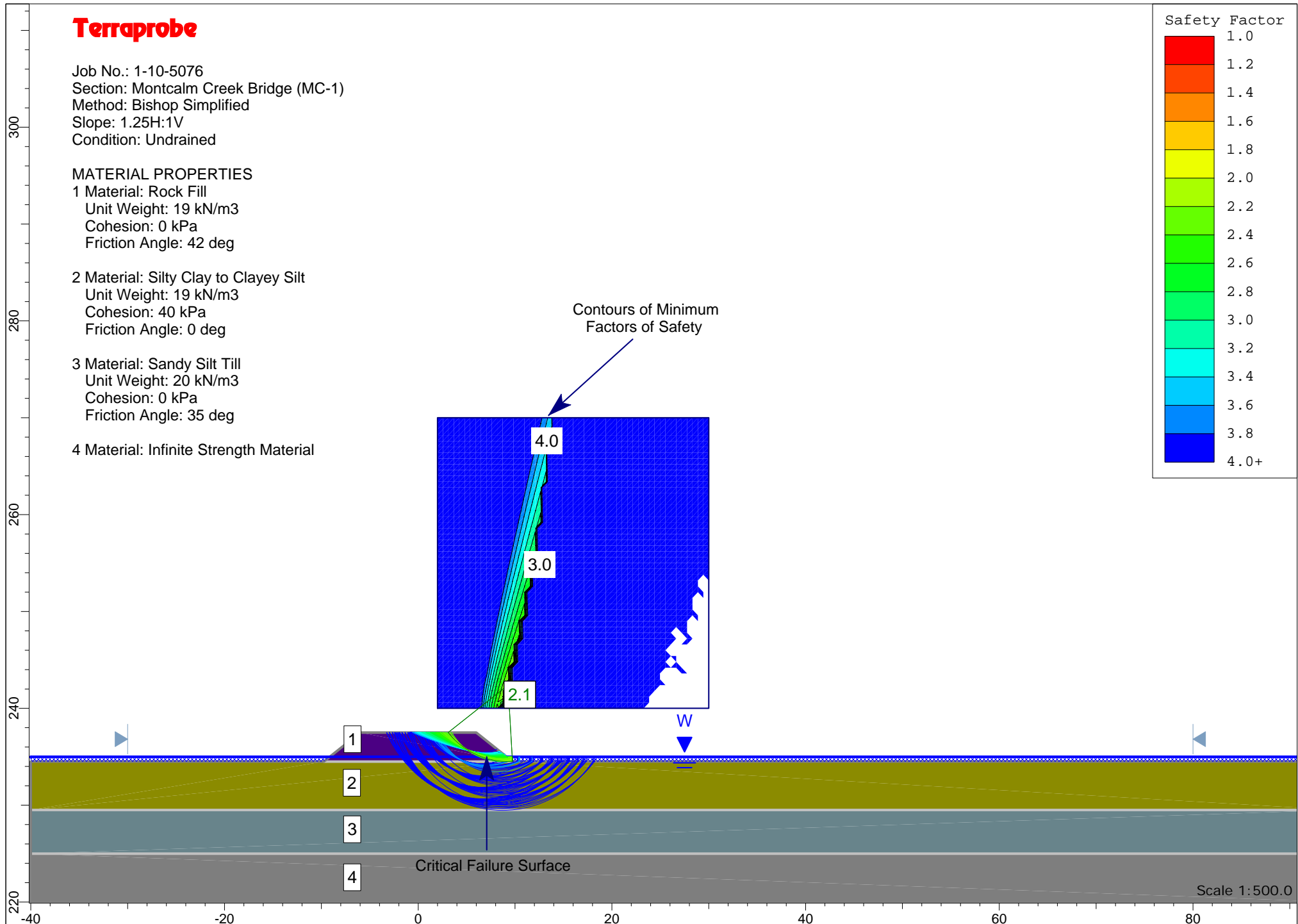
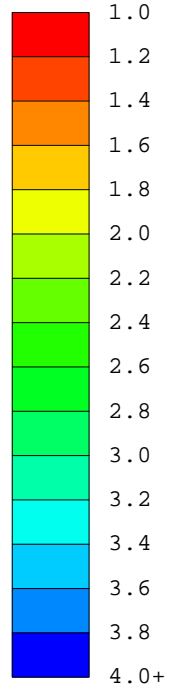
# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-1)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Undrained

## MATERIAL PROPERTIES

- 1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg
- 2 Material: Silty Clay to Clayey Silt  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 40 kPa  
Friction Angle: 0 deg
- 3 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 35 deg
- 4 Material: Infinite Strength Material

## Safety Factor





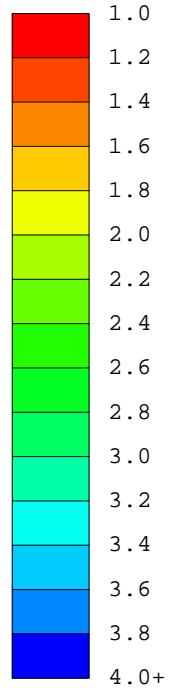
# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-1)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Drained

## MATERIAL PROPERTIES

- 1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg
- 2 Material: Silty Clay to Clayey Silt  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg
- 3 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 35 deg
- 4 Material: Infinite Strength Material

## Safety Factor



300

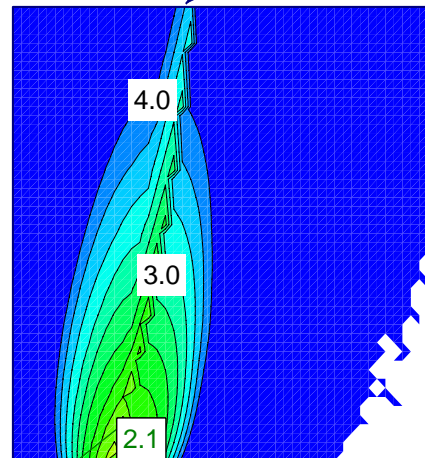
280

260

240

220

Contours of Minimum  
Factors of Safety



Critical Failure Surface

Scale 1:500.0



# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-2)  
Method: Bishop Simplified  
Slope: 2H:1V  
Condition: Undrained

## MATERIAL PROPERTIES

1 Material: Embankment Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 31 deg

2 Material: Silty Sand Fill

Unit Weight: 18.5 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 27 deg

3 Material: Silty Clay Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 40 kPa

Friction Angle: 0 deg

4 Material: Silty Clay

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 40 kPa

Friction Angle: 0 deg

5 Material: Sandy Silt Till

Unit Weight: 20 kN/m<sup>3</sup>

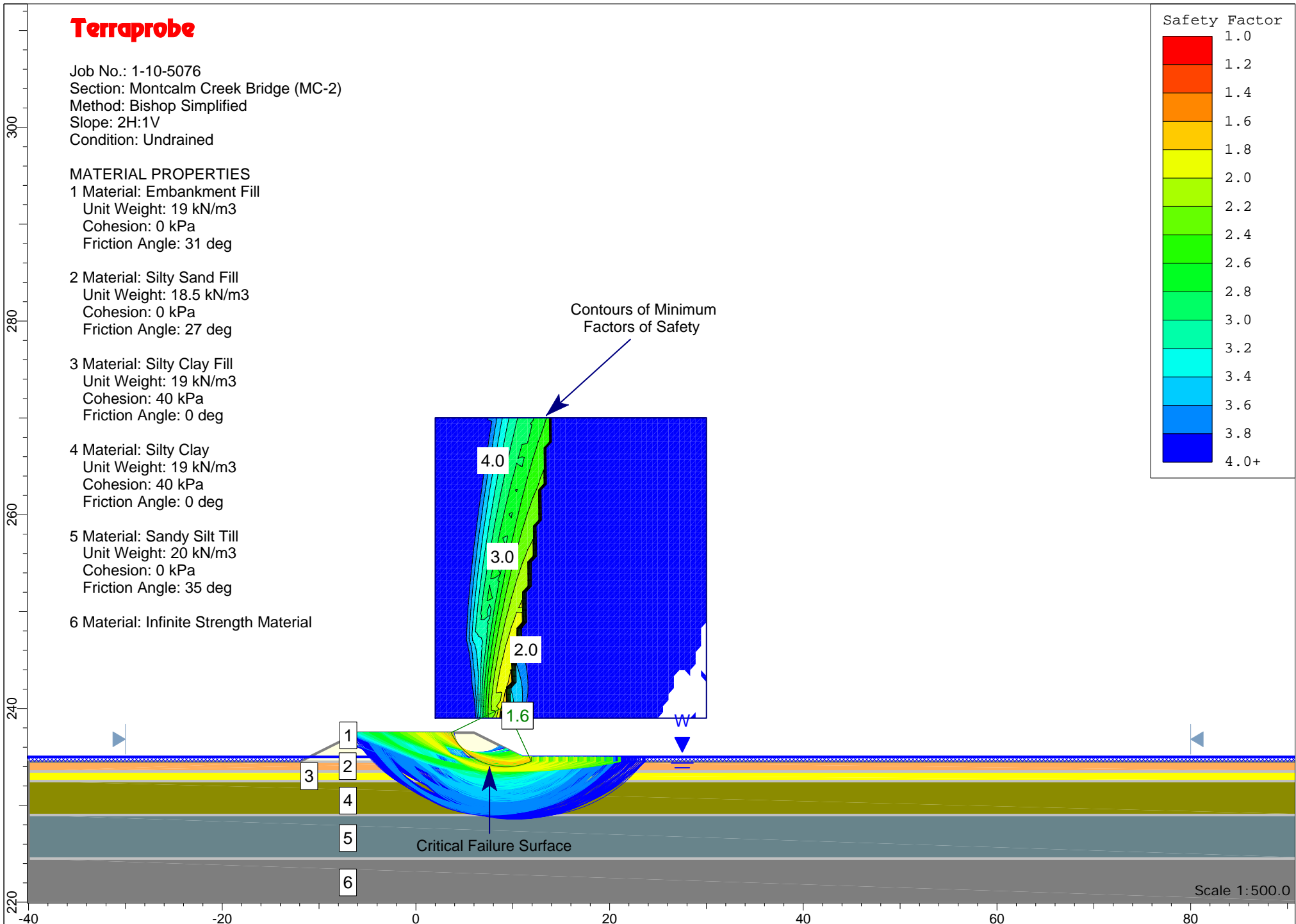
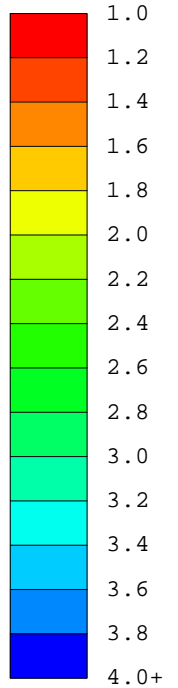
Cohesion: 0 kPa

Friction Angle: 35 deg

6 Material: Infinite Strength Material

Contours of Minimum  
Factors of Safety

Safety Factor





**Terraprobe**

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-2)  
Method: Bishop Simplified  
Slope: 2H:1V  
Condition: Drained

**MATERIAL PROPERTIES**

1 Material: Embankment Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 31 deg

2 Material: Silty Sand Fill  
Unit Weight: 18.5 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 27 deg

3 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

4 Material: Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

5 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 35 deg

6 Material: Infinite Strength Material

Contours of Minimum Factors of Safety

Critical Failure Surface

Scale 1:500.0

The figure is a slope stability analysis plot for a bridge embankment. It features a main cross-sectional view and an inset providing a magnified view of the failure surface. The main view shows a cross-section of the embankment with a critical failure surface (a curved line) and various material layers labeled 1 through 6. The inset shows a detailed view of the failure surface with contours of minimum factors of safety ranging from 1.6 to 4.0. A color scale on the right indicates the factor of safety values from 1.0 (red) to 4.0+ (blue). The plot also includes a title block with project information and material properties.

Material	Unit Weight (kN/m <sup>3</sup> )	Cohesion (kPa)	Friction Angle (deg)
1 Material: Embankment Fill	19	0	31
2 Material: Silty Sand Fill	18.5	0	27
3 Material: Silty Clay Fill	19	5	28
4 Material: Silty Clay	19	5	28
5 Material: Sandy Silt Till	20	0	35
6 Material: Infinite Strength Material	-	-	-

## 6 Material: Infinite Strength Material

Scale 1:500.0

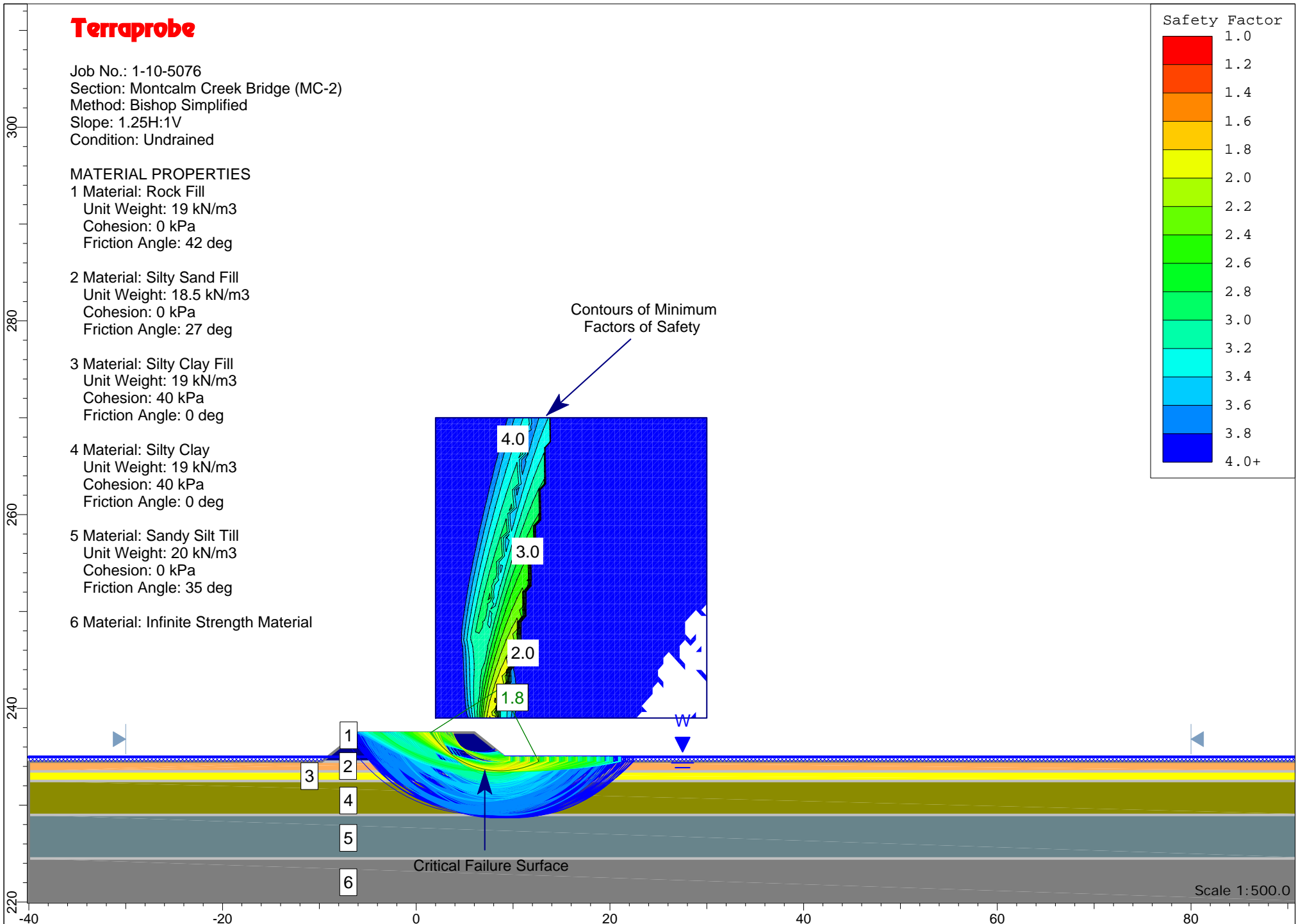
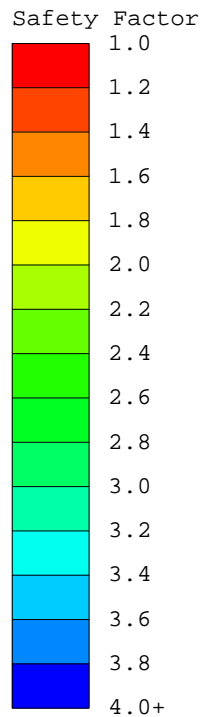


# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-2)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Undrained

## MATERIAL PROPERTIES

- 1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg
- 2 Material: Silty Sand Fill  
Unit Weight: 18.5 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 27 deg
- 3 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 40 kPa  
Friction Angle: 0 deg
- 4 Material: Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 40 kPa  
Friction Angle: 0 deg
- 5 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 35 deg
- 6 Material: Infinite Strength Material





**Terraprobe**

Job No.: 1-10-5076  
Section: Montcalm Creek Bridge (MC-2)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Drained

**MATERIAL PROPERTIES**

1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg

2 Material: Silty Sand Fill  
Unit Weight: 18.5 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 27 deg

3 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

4 Material: Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

5 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 35 deg

6 Material: Infinite Strength Material

Contours of Minimum Factors of Safety

Critical Failure Surface

Scale 1:500.0

Condition: Drained

Friction Angle: 42 deg

Friction Angle: 27 deg

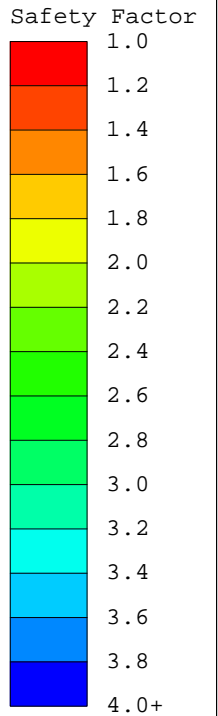
Friction Angle: 28 deg

Friction Angle: 28 deg

Friction Angle: 35 deg

## 6 Material: Infinite Strength Material

Scale 1:500.0





**TerraProbe**

Job No.: 1-10-5076  
 Section: Montcalm Creek Detour (MC-3)  
 Method: Bishop Simplified  
 Slope: 2H:1V  
 Condition: Undrained

**MATERIAL PROPERTIES**

1 Material: Embankment Fill  
 Unit Weight: 19 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Friction Angle: 31 deg

2 Material: Silty Clay Fill  
 Unit Weight: 19 kN/m<sup>3</sup>  
 Cohesion: 30 kPa  
 Friction Angle: 0 deg

3 Material: Clayey Silt to Silty Clay  
 Unit Weight: 19 kN/m<sup>3</sup>  
 Cohesion: 50 kPa  
 Friction Angle: 0 deg

4 Material: Sandy Silt Till  
 Unit Weight: 20 kN/m<sup>3</sup>  
 Cohesion: 0 kPa  
 Friction Angle: 30 deg

5 Material: Infinite Strength Material

Contours of Minimum Factors of Safety

4.0

Critical Failure Surface

Scale 1:500.0

Scale 1:500.0



**Terraprobe**

Job No.: 1-10-5076  
Section: Montcalm Creek Detour (MC-3)  
Method: Bishop Simplified  
Slope: 2H:1V  
Condition: Drained

**MATERIAL PROPERTIES**

1 Material: Embankment Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 31 deg

2 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

3 Material: Clayey Silt to Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

4 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 30 deg


5 Material: Infinite Strength Material

Contours of Minimum Factors of Safety

Critical Failure Surface

Scale 1:500.0

## 5 Material: Infinite Strength Material



## 2.7

Scale 1:500.0



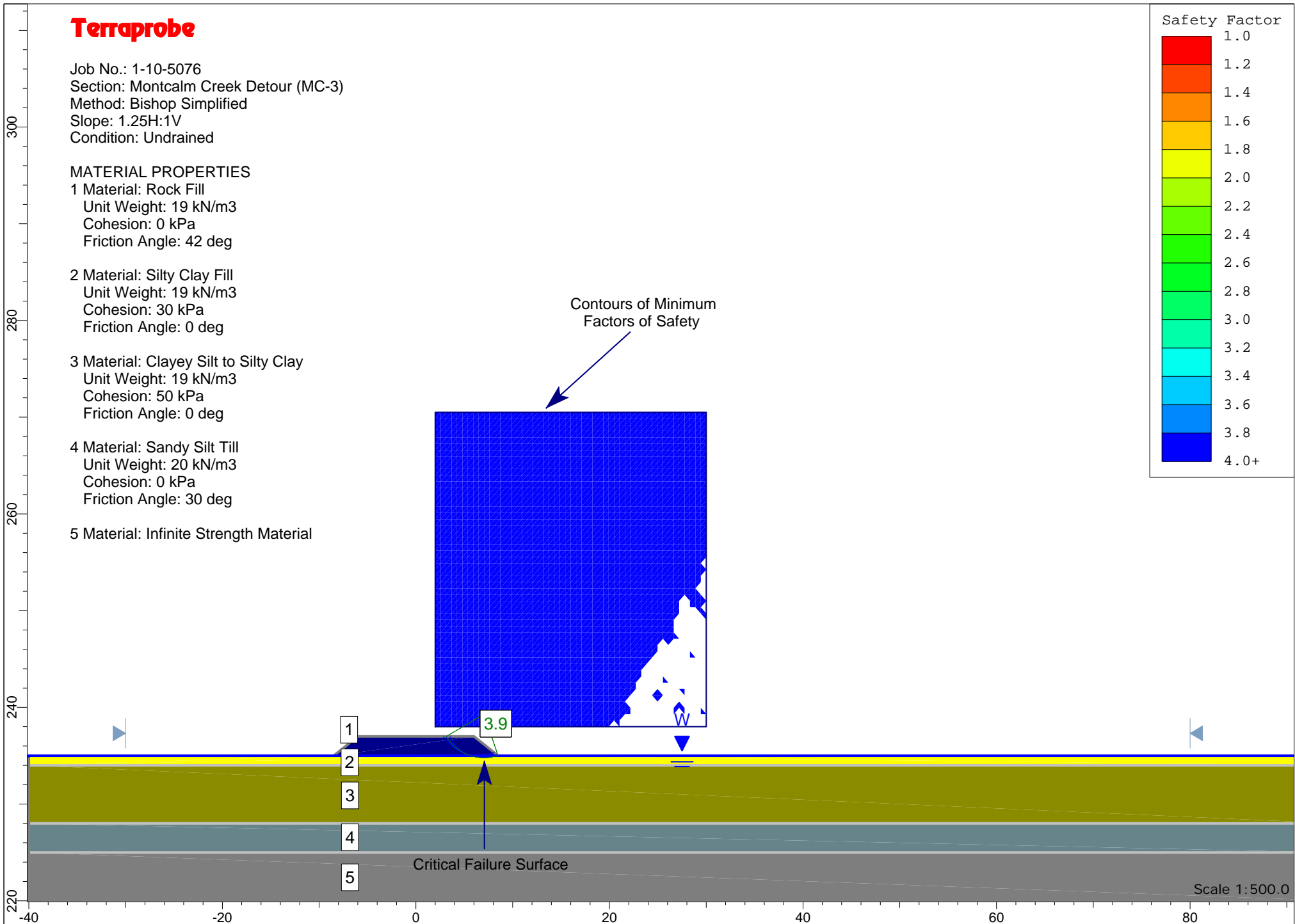
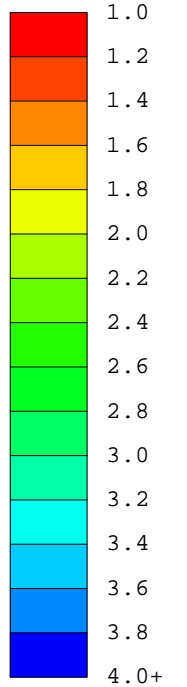
# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Detour (MC-3)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Undrained

## MATERIAL PROPERTIES

- 1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg
- 2 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 30 kPa  
Friction Angle: 0 deg
- 3 Material: Clayey Silt to Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 50 kPa  
Friction Angle: 0 deg
- 4 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 30 deg
- 5 Material: Infinite Strength Material

## Safety Factor





Job No.: 1-10-5076  
Section: Montcalm Creek Detour (MC-3)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Drained

1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg

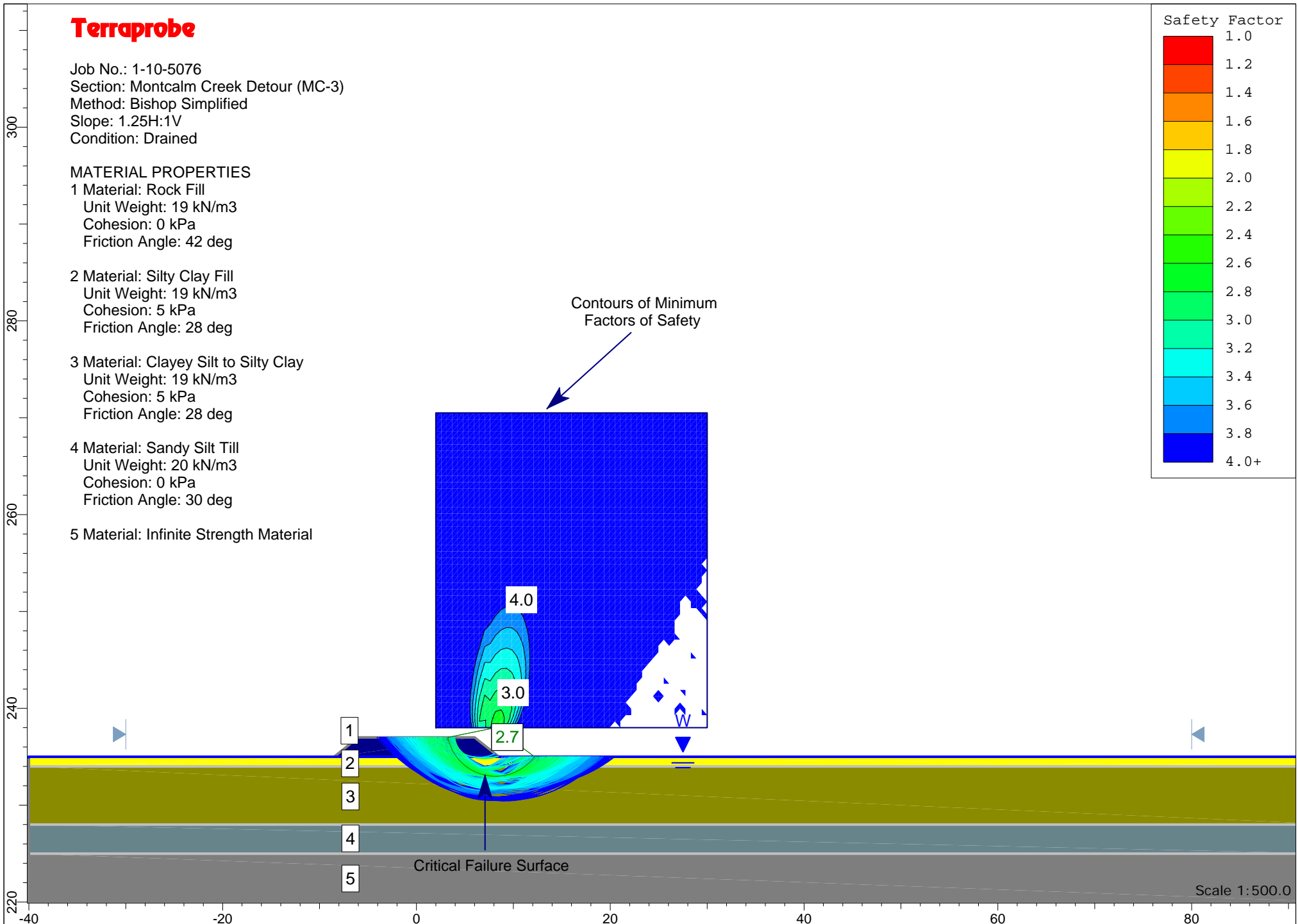
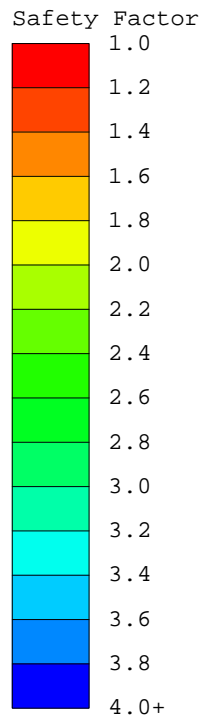
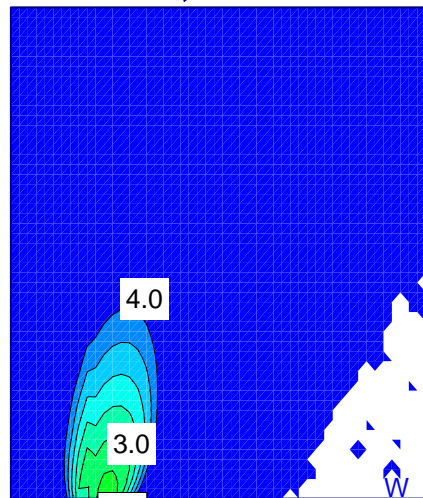
2 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

3 Material: Clayey Silt to Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

4 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 30 deg

## 5 Material: Infinite Strength Material

### Contours of Minimum Factors of Safety





# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Detour (MC-4)  
Method: Bishop Simplified  
Slope: 2H:1V  
Condition: Undrained

## MATERIAL PROPERTIES

1 Material: Embankment Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 31 deg

2 Material: Silty Clay Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 50 kPa

Friction Angle: 0 deg

3 Material: Silty Clay

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 75 kPa

Friction Angle: 0 deg

4 Material: Sandy Silt Till

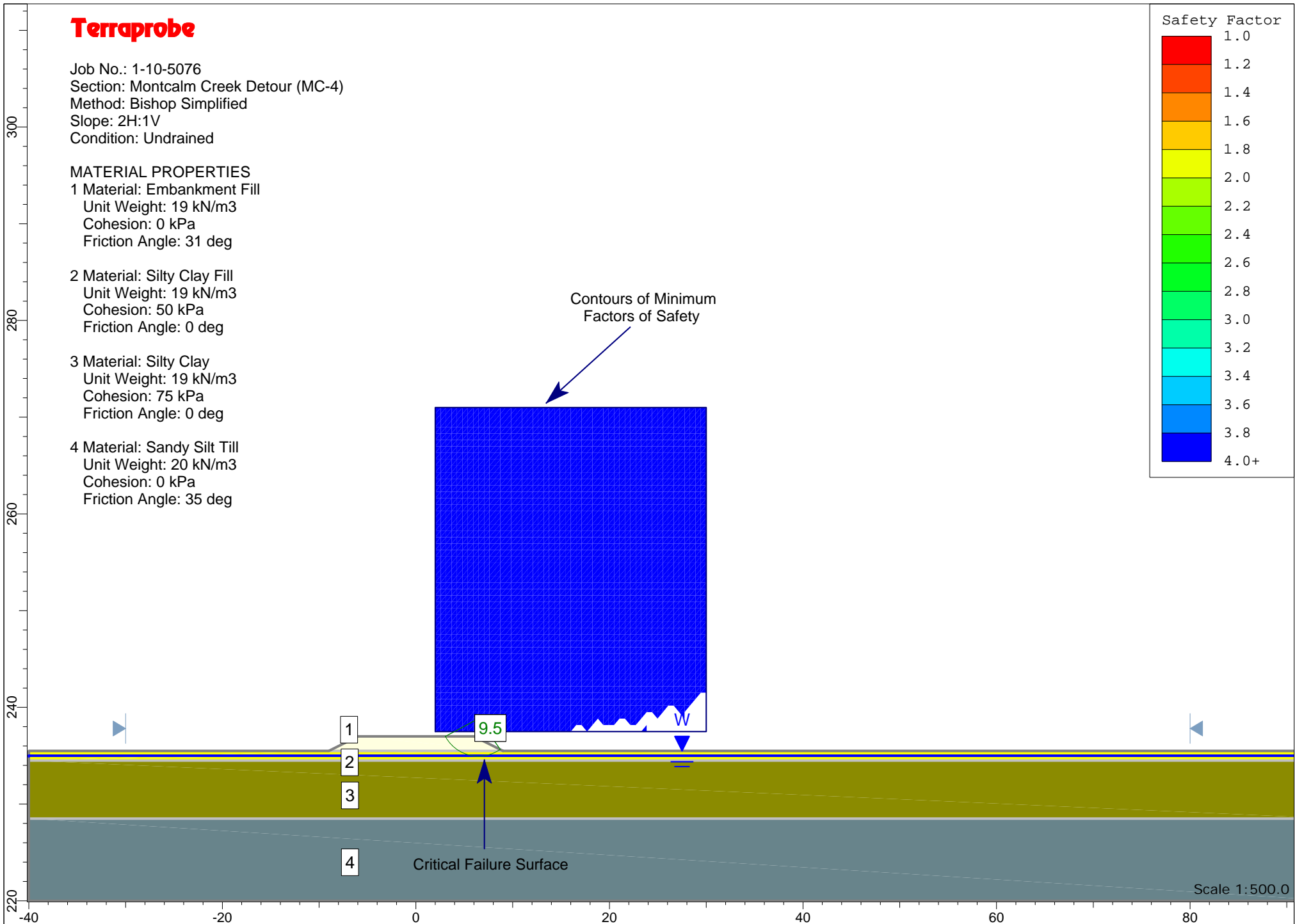
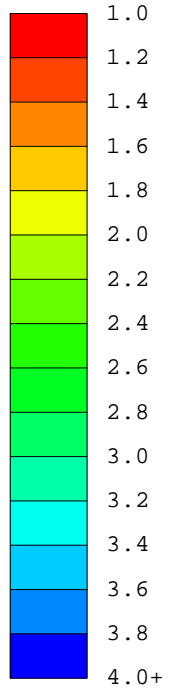
Unit Weight: 20 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 35 deg

Contours of Minimum  
Factors of Safety

Safety Factor





# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Detour (MC-4)  
Method: Bishop Simplified  
Slope: 2H:1V  
Condition: Drained

## MATERIAL PROPERTIES

1 Material: Embankment Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 31 deg

2 Material: Silty Clay Fill

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 5 kPa

Friction Angle: 28 deg

3 Material: Silty Clay

Unit Weight: 19 kN/m<sup>3</sup>

Cohesion: 5 kPa

Friction Angle: 28 deg

4 Material: Sandy Silt Till

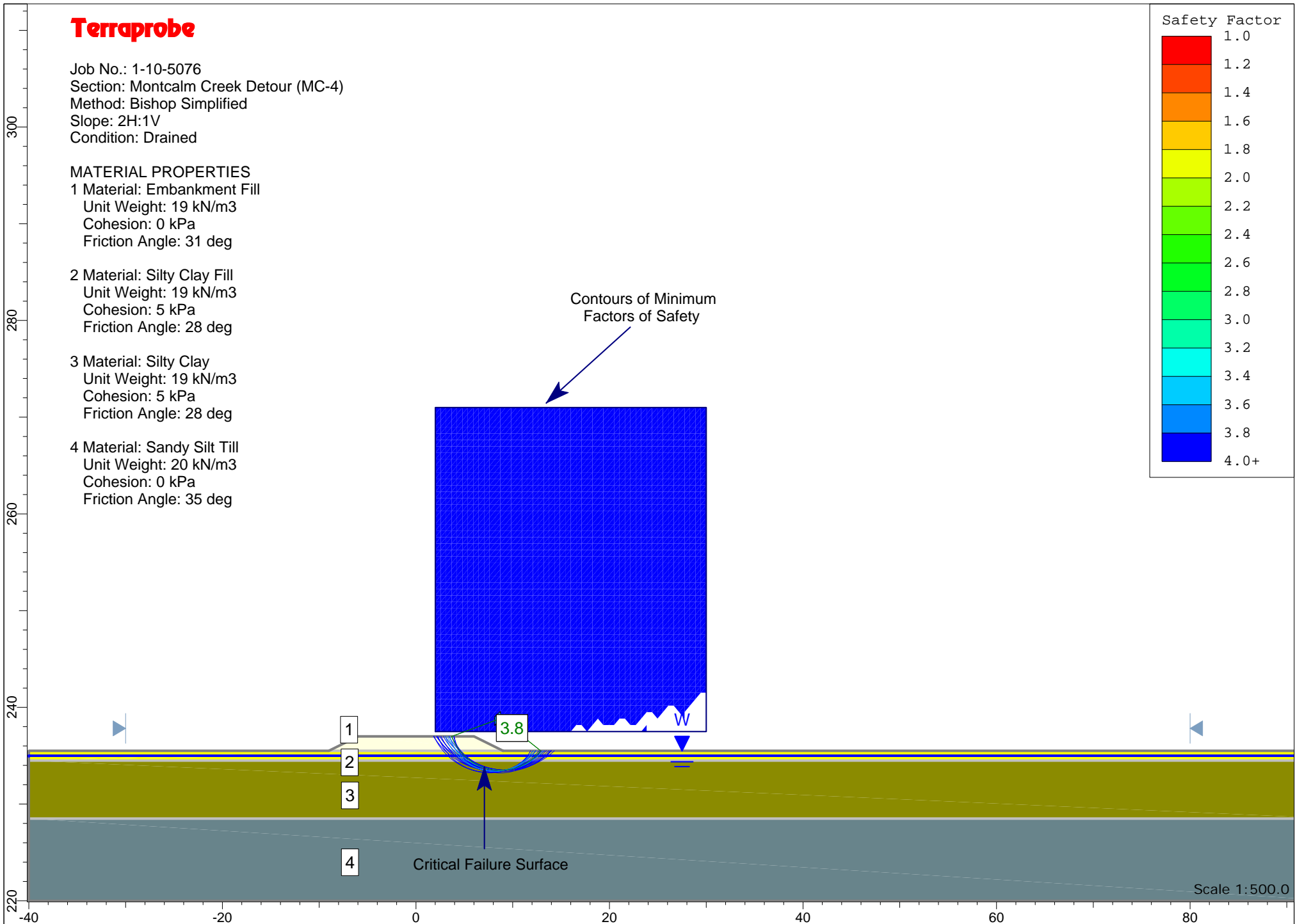
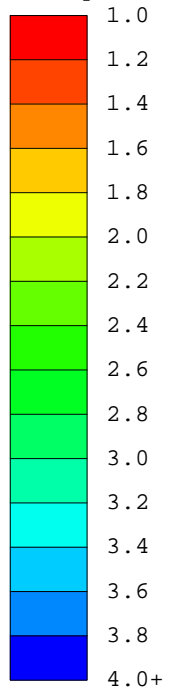
Unit Weight: 20 kN/m<sup>3</sup>

Cohesion: 0 kPa

Friction Angle: 35 deg

Contours of Minimum  
Factors of Safety

Safety Factor





# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Detour (MC-4)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Undrained

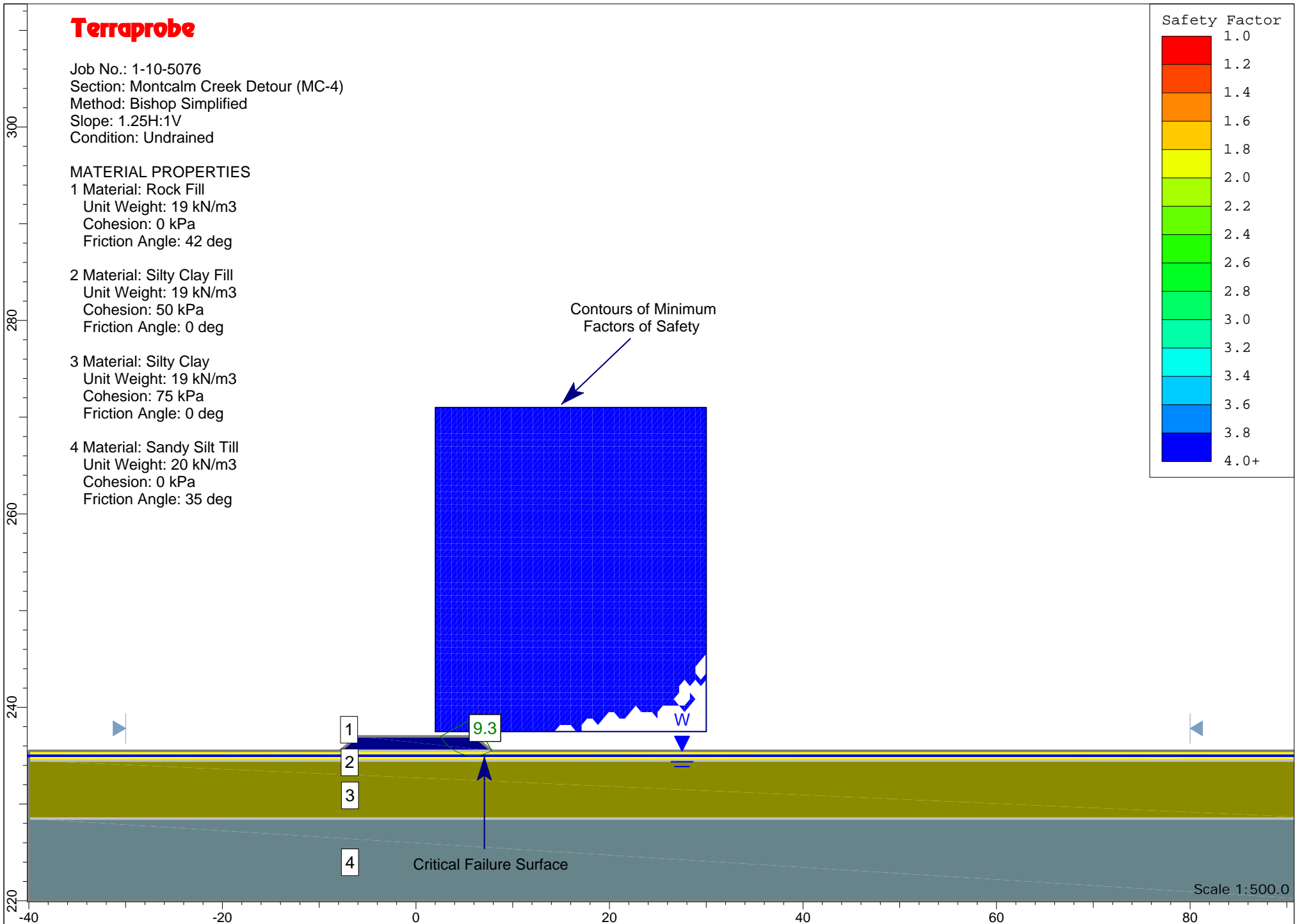
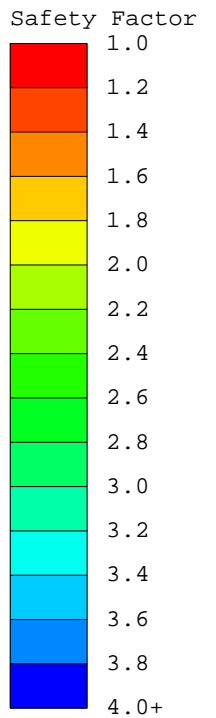
## MATERIAL PROPERTIES

1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg

2 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 50 kPa  
Friction Angle: 0 deg

3 Material: Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 75 kPa  
Friction Angle: 0 deg

4 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 35 deg





# Terraprobe

Job No.: 1-10-5076  
Section: Montcalm Creek Detour (MC-4)  
Method: Bishop Simplified  
Slope: 1.25H:1V  
Condition: Drained

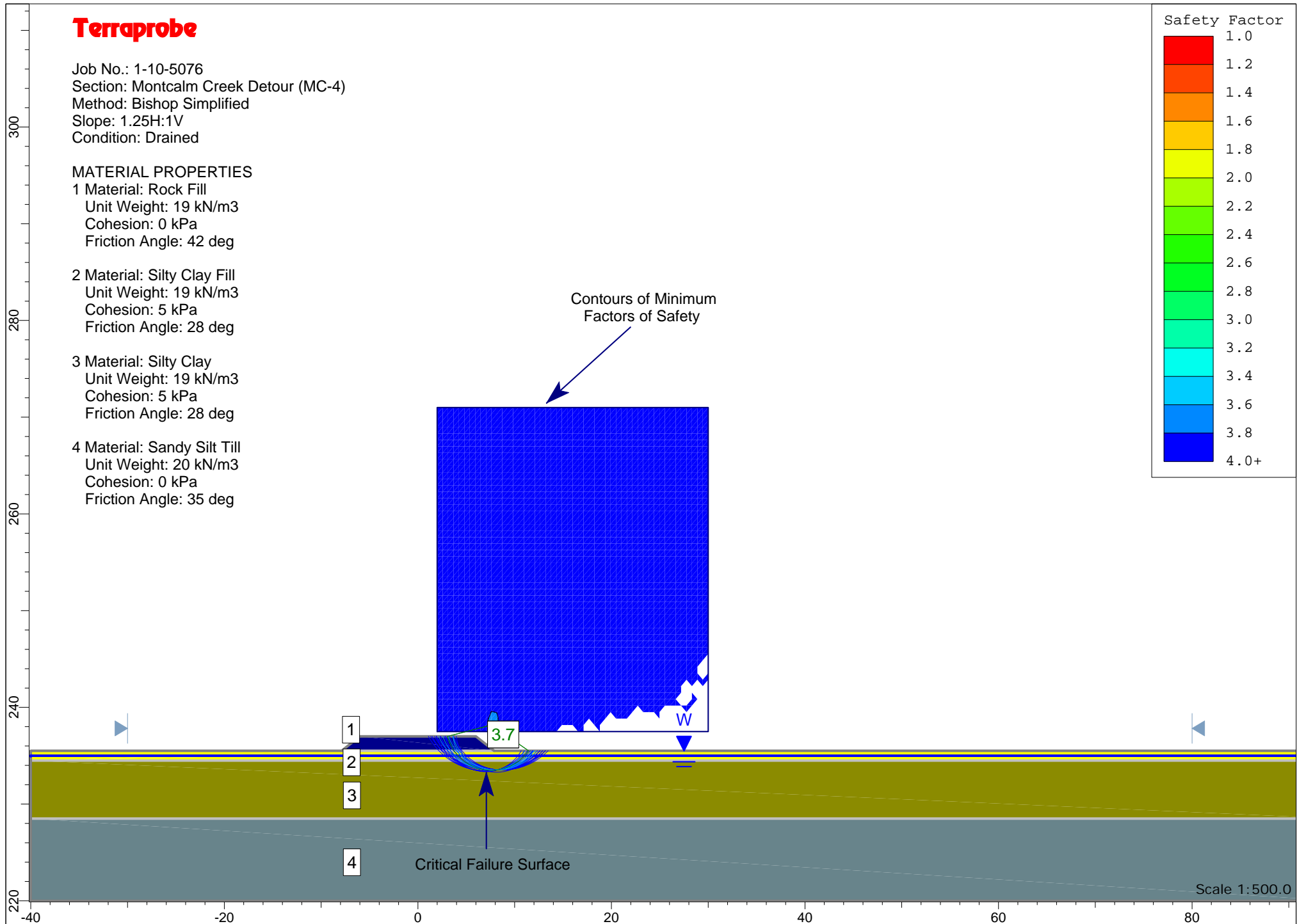
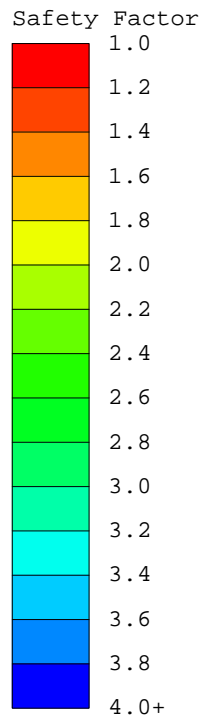
## MATERIAL PROPERTIES

1 Material: Rock Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 42 deg

2 Material: Silty Clay Fill  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

3 Material: Silty Clay  
Unit Weight: 19 kN/m<sup>3</sup>  
Cohesion: 5 kPa  
Friction Angle: 28 deg

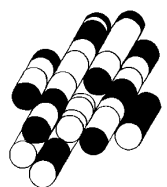
4 Material: Sandy Silt Till  
Unit Weight: 20 kN/m<sup>3</sup>  
Cohesion: 0 kPa  
Friction Angle: 35 deg



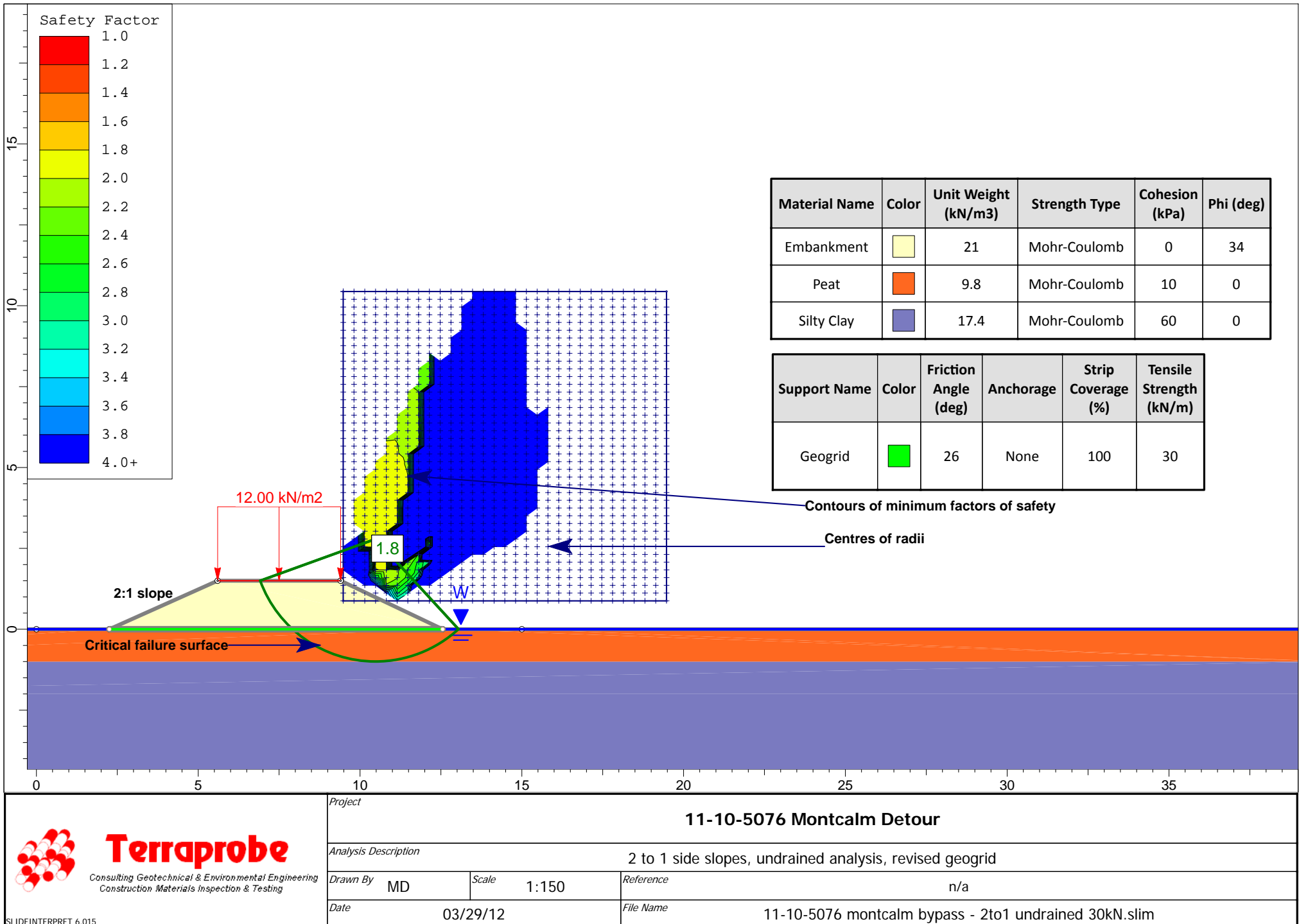


# APPENDIX F

**TERRAPROBE INC.**



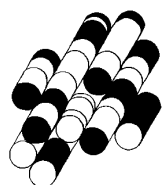




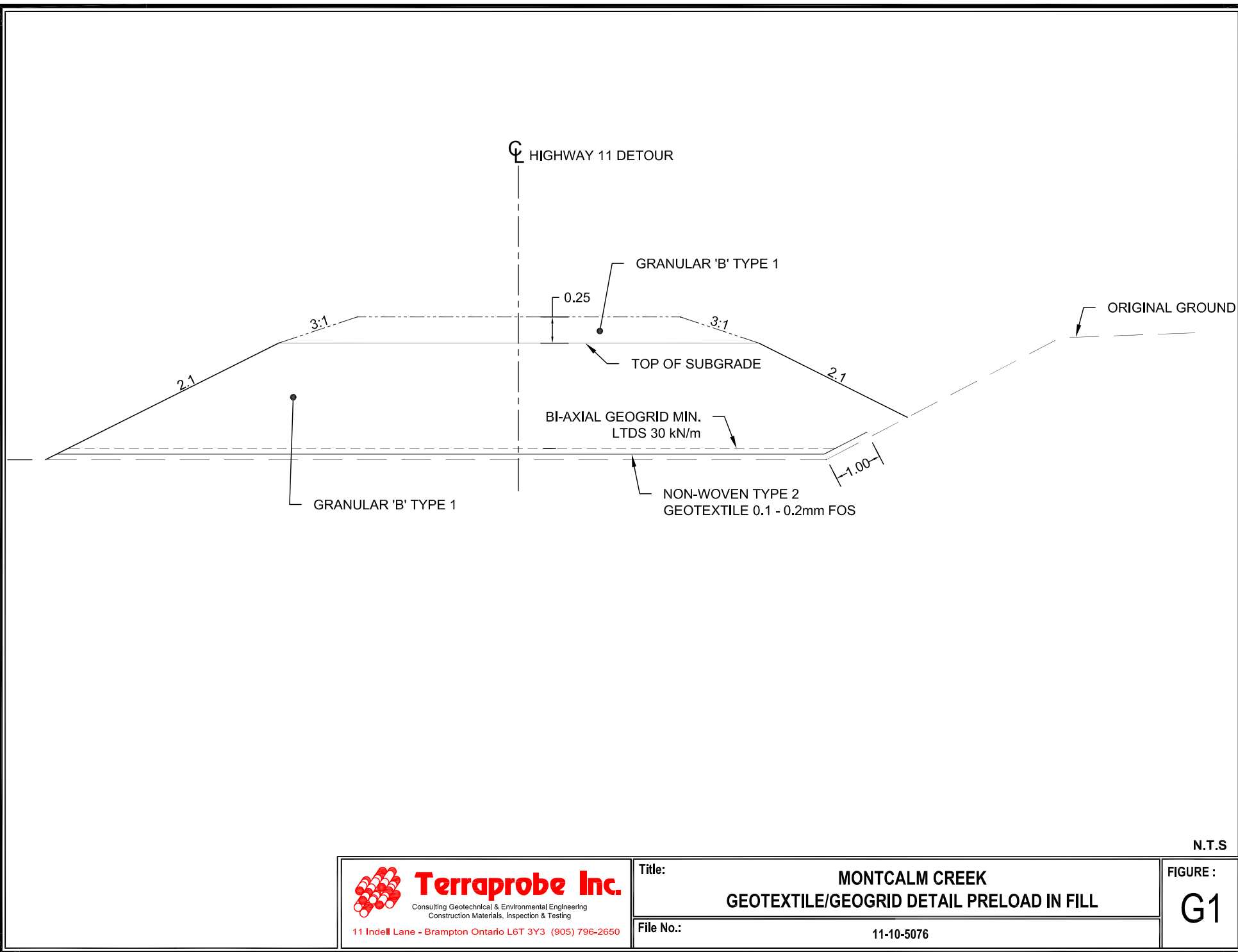


# APPENDIX G

**TERRAPROBE INC.**







N.T.S



**Terraprobe Inc.**  
Consulting Geotechnical & Environmental Engineering  
Construction Materials, Inspection & Testing

11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650

Title:

**MONTCALM CREEK  
GEOTEXTILE/GEOGRID DETAIL PRELOAD IN FILL**

File No.:

11-10-5076

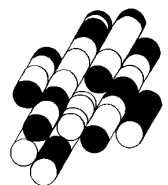
FIGURE :

**G1**



# APPENDIX H

**TERRAPROBE INC.**





In this report, reference is made to the following Provincial Standard:

- OPSS 903

The contract documents should contain a NSSP with the following wording:

**Cobbles and Boulders**

“The Contractor is informed that the soils at this site may contain cobbles and boulders that could impede the progress of pile driving operations. The soil conditions are described in the Foundation Investigation Report prepared for this site”.

If a pile encounters refusal on cobbles and boulders, the pile driving inspector should terminate driving before the pile is damaged by overdriving. If the required resistance according to the Hiley Formula is not achieved and further driving is likely to cause damage to the pile, pile driving should cease and the contract administrator should be notified.





**GEOTEXTILE** - Item No.  
**GEOGRID**, Item No.

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## Special Provision

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### **1.0 Scope**

This non-standard special provision specifies the material requirements and the work required for the supply and construction of the geotextile/geogrid system for the reinforced embankment of the temporary detour.

### **2.0 References**

This special provision refers to the following standards and specifications where applicable:

- OPSS 201 - Clearing, close cut clearing, grubbing and removal of surface boulders
- OPSS 206 - Grading
- OPSS 501 - Compacting
- OPSS 510 - Removal

The Contractor shall refer to the following reports for a description of subsurface conditions at this site:

- Foundations Investigation Report, Replacement of the Crow Creek Bridge,  
WP 5147-05-01, May 2012.  
Site 39W-055  
Geocres No. 42G-35

- Foundations Investigation Report, Replacement of the Montcalm Creek Bridge,  
WP 5146-05-01, May 2012.  
Site 39W-058  
Geocres No. 42G-36

### **3.0 Definitions**

Quality Verification Engineer (QVE): an Engineer with a minimum of five (5) years' experience related to the design and/or construction of Reinforced Embankment of similar scope to that in the Contract, or alternatively has demonstrated expertise by providing satisfactory quality verification services for the work at a minimum of two (2) projects of similar scope to the Contract. The Quality Verification Engineer shall be retained by the Contractor to ensure general conformance with the contract documents and issue certificate(s) of conformance.

### **4.0 Submission Requirements**

#### Construction Methods

The Contractor shall submit details of the sequence and methods of construction to the Quality Verification Engineer for review. The submittals shall satisfy the specifications and at a minimum contain the following specific information:



Proposed equipment.  
Detailed description of proposed installation procedures.  
Proposed methods for overcoming obstructions.  
Proposed methods for laying of geotextile and geogrid.  
Proposed methods for placing of backfill materials.  
Proposed methods for maintaining access road.

At least 21 calendar days prior to the construction of the detour embankment, the Contractor shall submit to the Contract Administrator for (review) details of the sequence and method of installation. The submittals shall satisfy the specifications and at a minimum contain the above information as provided to the Contractors Quality Verification Engineer.

The Contractor shall submit to the Contract Administrator a Certificate of Conformance sealed and signed by the Quality Verification Engineer a minimum of 7 calendar days prior to commencement of work under this item. The Certificate shall state that the installation procedures are in conformance with the requirements and specifications of the contract documents.

#### Final Certificate of Conformance

Prior to the acceptance of the work by the Owner, the Contractor shall obtain a certificate of conformance sealed and signed by Contractor's designated QVE and submit the certificate to the Contract Administrator. The certificate shall state that all work has been completed in general accordance with the contract drawings and specifications.

### **5.0 Materials**

Non-woven Geotextile Fabric Type 2 with Filtration Opening Size (FOS) 0.1- 0.2 mm.

Bi-axial geogrid reinforcing with a minimum Long Term Design Strength (LTDS) of 30 kN/m.

Granular B Type 1.

### **6.0 Construction**

Prior to construction of the reinforced embankment, the contractor shall close cut clear all trees and shrubs and clear all objects without disturbing the root mat.

The geotextile followed by the geogrid shall be placed on the prepared ground surface at the locations, elevations, orientations and lengths shown on the contract drawings. Prior to placing fill, the geogrid materials shall be placed flat and pulled taut to remove any slack.

Where a second layer of biaxial geogrid is required, it shall to be placed 0.6 m above the first layer.

Granular B Type 1 fill materials shall be placed from the middle of the reinforced zone towards the ends of the geogrid to ensure further tensioning.

Low ground pressure construction equipment should be used especially during the initial stages of construction.



Geogrid reinforcement shall be continuous throughout the embedment length(s). If splicing of the geogrid is required, it is to be spliced according to the manufacturer's recommendations. No splices shall be allowed for geogrid less than 2.0 m in length (each).

The Contractor shall be responsible for any damage caused to the geogrid during construction of the access road. If the geogrid is damaged, it shall be replaced at the Contractor's cost. Tracked construction equipment shall not be operated directly on the geogrid.

No changes to the geotxtile/geogrid layout, length, geogrid type, or elevation, shall be made without the prior written consent of the Contract Administrator

## **7.0 Operational Constraints**

All geogrid materials supplied shall be free of defects, rips, holes or flaws. During shipment the geogrid shall be protected from damage. During on-site storage the storage area shall be such that the geogrid is protected from sunlight, dirt, dust, mud, debris and any other detrimental substances.

The location of the geogrids shall not vary by more than 150 mm from the locations shown on the contract drawings.

The maximum contact pressure permitted by the Contractor's construction equipment on the access roadway is 70 kPa.

The Contractor is cautioned that the pad shall always be maintained to a minimum distance of 2.0 m beyond the limit of the maneuvering space of the equipment.

The Contractor is advised that the site is considered as an environmentally sensitive area and therefore the work area shall be limited to the area required to construct the temporary detour. Under no circumstance shall the area outside of the footprint of the temporary detour be used for any construction activities/purpose.

## **Measurement for Payment**

Measurement shall be by area in square metres with no allowance for overlap.

## **Basis for Payment**

Payment at the contract price for the above tender item shall be full compensation for all labour, equipment and material necessary to do the work.