Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

FOUNDATION INVESTIGATION & DESIGN REPORT

CROW CREEK BRIDGE REPLACEMENT

HIGHWAY 11, 3.7 KM WEST OF LOWTHER

G.W.P. No. 5233-06-00, W.P. 5147-05-01, SITE 39W-055

GEOCRES No. 42G-35

MINISTRY OF TRANSPORTATION, ONTARIO

NORTHEASTERN REGION

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PART 1: FACTUAL INFORMATION

1 INTRODUCTION

This report presents the factual findings obtained from foundation investigations conducted at the Crow Creek Bridge site where a bridge replacement and a detour structure are proposed. The site is located on Highway 11, 3.7 km west of Lowther 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, Crow Creek Bridge Replacement, Highway 11, 3.7 km West of Lowther, G.W.P. No. 5233-06-00, W.P. 5147-05-01, Site 39W-055, Geocres No. 42G-33, 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 are reported under separate cover.

2 SITE DESCRIPTION & PHYSIOGRAPHY

Highway 11 crosses Crow Creek via an 11.7 m wide five span timber bridge measuring about 23 m in length. At this site Highway 11 is a two-lane highway with fully paved shoulders carrying east and west bound traffic. A CN Railway track runs parallel to Highway 11 and is located approximately 45 m south of Highway 11 centre line.

Crow 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 C1 to C4 inclusive were drilled at the preliminary design stage between July 27 and August 6, 2010. Boreholes C1 and C2 were drilled at the existing bridge site and Boreholes C3 and C4 were drilled at the site of the proposed detour structure. The second phase of the investigation was carried out between October 6 and November 8, 2011 and consisted of drilling and sampling six additional boreholes, designated CD1 to CD6 inclusive. Boreholes CD1 and CD2 were drilled at the existing bridge and boreholes CD3 and CD4 were drilled in the approaches to the existing bridge. Boreholes CD5 and CD6 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.

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 in-situ by means of field vane tests 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.

The 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 summarized 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
C1	27.4/214.2	Piezometer with 1.5 m slotted screen installed with filter sand to 25.6 m, bentonite seal from 25.6 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
C2	21.0/220.6	Piezometer with 1.5 m slotted screen installed with filter sand to 18.9 m, bentonite seal from 18.9 m to 6.1 m, drill cuttings from 6.1 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
C3	25.8/214.0	Piezometer with 1.5 m slotted screen installed with filter sand to 24.0 m and bentonite seal from 24.0 m to ground surface.
C4	22.9/217.1	Piezometer with 1.5 m slotted screen installed with filter sand to 21.1 m, bentonite seal from 21.1 m to 7.7 m and drill cuttings from 7.7 m to ground surface.
CD1	18.3/223.4	Piezometer with 3.0 m slotted screen installed with filter sand to 14.6 m, bentonite seal from 14.6 m to 14.0 m, drill cuttings from 14.0 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
CD2	20.7/220.9	Piezometer with 3.0 m slotted screen installed with filter sand to 17.1 m, bentonite seal from 17.1 m to 16.5 m, drill cuttings from 16.5 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
CD3	9.1/232.5	Piezometer with 3.0 m slotted screen installed with filter sand to 5.5 m, bentonite seal from 5.5 m to 4.9 m, drill cuttings from 4.9 m to 0.6 m and a concrete encased flush mount cover from 0.6 m to ground surface.
CD4	11.3/230.3	Piezometer with 3.0 m slotted screen installed with filter sand to 7.6 m, bentonite seal from 7.6 m to 0.3 m and a concrete encased flush mount cover from 0.3 m to ground surface.
CD5	7.9/230.7	Piezometer with 3.0 m slotted screen installed with filter sand to 4.3 m, bentonite seal from 4.3 m to 3.7 m, drill cuttings from 3.7 m to 0.6 m and bentonite seal from 0.6 m to ground surface.
CD6	9.4/229.3	Piezometer with 3.0 m slotted screen installed with filter sand to 5.8 m, bentonite seal from 5.8 m to 5.2 m, drill cuttings from 5.2 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 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 C1, C2, CD1, CD2, CD3 & CD4)

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

5.1.1 Flexible Pavement/Gravel Shoulder

A flexible pavement comprising of 150 mm to 200 mm thick of asphalt underlain by a layer of sand and gravel ranging in thickness from 130 mm to 250 mm was encountered in Boreholes C1, C2, CD1, CD2 and CD3. Borehole CD4 was drilled through the road shoulder and encountered a layer of sand and gravel fill approximately 450 mm thick. The granular fill extended to elevations ranging from 241.1 m to 241.3 m and was inferred to be in a compact state.

5.1.2 Fill – Sand

Fill consisting of sand, trace silt, trace gravel was encountered beneath the pavement and to depths of 1.4 m (Elev. 240.2 m) and 2.1 m (Elev. 239.5 m) below the existing ground surface.

The grain size distribution plots of samples of the sand fill recovered from the boreholes are presented in Figure B1-1. These results show a grain size distribution consisting about of 0-5% gravel, 87-95% sand and 5-8% silt and clay size particles.

'N' values in the range of 6 to 29 blows for 0.3 m were determined in the standard penetration testing carried out in the sand fill, inferring a loose to compact relative density. The water content of samples of the sand fill ranged from 2% to 14% by weight.

5.1.3 Sand and Silt to Silt

A near surface deposit ranging in composition from sand and silt to silt was encountered in all of the boreholes extending to depths ranging from 2.9 m (Elev. 238.8 m) to 3.7 m (Elev. 237.9 m) below ground surface.

The results of grain size distribution analysis of samples recovered from this deposit are shown in Figure B1-2. These results show a grain size distribution consisting of 11-44% sand, 44-71% silt and 10-18% clay size particles.

The N values determined in this deposit ranged from 4 to 13 blows per 0.3 m indicating a loose to compact relative density. The moisture content of samples from this stratum ranged from 14% to 19%.

5.1.4 Clayey Silt to Silty Clay

A clayey silt to silty clay deposit was encountered beneath the fill and surficial sands and silts and to depths ranging from 8.2 m (Elev. 233.4 m) to 9.8 m (Elev. 231.8 m) below ground surface.

The grain size distribution curves of samples of the clayey silt to silty clay are presented in Figures B1-3 and B1-4. These results show a grain size distribution consisting of 0-3% gravel, 1-17% sand, 44-75% silt and 21-53% 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:	21-30%
Plastic Limit:	12-21%
Plasticity Index:	4-12%
Natural Moisture Content:	12-38%

These values indicate low plasticity clayey silt to silty clay soils.

The N values determined in the clayey silt to silty clay ranged from 2 to 14 blows for 0.3 m penetration. Field vane shear tests indicated undrained shear strengths ranging from 20 kPa to 88 kPa. These values indicate that the consistency of the clayey silt to silty clay was generally firm to stiff with infrequent soft zones. The natural moisture content of samples of the clayey silt to silty clay ranged from 12% to 43%.

5.1.5 Sand and Silt Till

A deposit of sand and silt till was encountered beneath the clayey silt to silty clay. These till strata were fully penetrated in some of the boreholes at depths ranging from 14.6 m to 17.6 m below ground surface or at elevations ranging from 227.1 m to 224.0 m. The approach boreholes were terminated in this deposit at depths of 10.5 m (Elev. 231.1 m) and 12.4 m (Elev. 229.2 m).

The results of grain size distribution tests carried out on samples obtained from the sand and silt till are shown in Figure B1-7. These results show grain size distributions consisting of 0-17% gravel, 37-50% sand, 35-55% silt and 4-12% 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 sand and silt till ranged from 19 to more than 100 blows per 0.3 m, indicating a compact to very dense relative density. The natural water content of samples of the till ranged from 1% to 16% by weight.

5.1.6 Clayey Silt Till

A clayey silt till deposit was encountered beneath the sand and silt till and overlying the bedrock surface in boreholes C1, C2 and CD1 and CD2. The clayey silt till extended to depths ranging from 22.5 m (Elev. 219.1 m) to 28.0 m (Elev. 213.6 m) below ground surface.

The grain size distribution plots of samples of the clayey silt till deposit are presented in Figure B1-8. These results show a grain size distribution consisting of 1-9% gravel, 20-36% sand, 44-64% silt and 11-23% clay size particles. Cobbles and boulders were also thought to have been encountered in the clayey silt till.

The results of Atterberg Limits determinations on samples of the clayey silt till are presented in Figure B1-9 and summarized below:

Liquid Limit:	15-22%
Plastic Limit:	11-17%
Plasticity Index:	3-10%
Natural Moisture Content:	8-12%

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

The N values in the clayey silt till ranged from 60 to more than 100 blows for 0.3 m penetration indicating a hard consistency. The natural water content of samples of the clayey silt till was in the range of 7% to 12%.

5.1.7 Bedrock

The overburden described above was underlain by metamorphic phyllite and 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 5.1.

Table 5.1 – Depth to Bedrock

BH No.	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
C1	28.0	213.6
C2	22.5	219.1
CD1	23.5	218.2
CD2	26.4	215.2

In Borehole CD1 the phyllite bedrock has been described as moderately to highly weathered and unweathered in the Boreholes C1, C2 and CD2. The phyllite bedrock had sub-vertical foliations and was generally grey to dark grey in colour. Total core recovery in this bedrock ranged from 72% to 100% and the RQD values ranged from 0% to 100% however the RQD values were typically above 50%. Based on these results the rock quality is considered to be fair to good with occasional zones of very poor to poor quality rock.

5.2 Detour Structure (Boreholes C3, C4, CD5 & CD6)

In general, the site was underlain by topsoil, peat, silty clay fill and native deposits of clayey silt to silty clay, sandy silt till and clayey silt till. The overburden soils were underlain by bedrock consisting of phyllite.

5.2.1 Topsoil and Peat

A surface topsoil layer about 0.2 to 0.3m thick was encountered in Boreholes C3 and C4. Amorphous peat was encountered to depths of 0.7 to 1.1 m below ground surface in Boreholes CD5 and CD6. The samples or peat recovered from the penetration testing had natural water contents in the range of about 172 to 194%.

Peat was also encountered in several of the boreholes drilled along the detour alignment as part of the pavement design investigation. The natural water content of the peat recovered from these boreholes ranged from about 60 to 700 %. The consolidation characteristics inferred from the results of a one dimensional consolidation test carried out on a sample of the peat are summarized below. The results of the consolidation testing are shown on Figures B2-12 B2-13.

Table 5.2 – Consolidation Characteristics of Peat

Parameter	
Natural water content	586 %
Bulk Unit weight	9.8 kN/m³
Dry Unit weight	5.4 kN/m³
Compression index	1.67
Recompression index	0.426
Void ratio	2.8
Preconsolidation Pressure	20 kPa
Consolidation Coefficient	0.146 m2/yr

5.2.2 Fill – Silty Clay

Fill material consisting of silty clay mixed with peat was encountered in Boreholes C3 and C4 extending to depths ranging from 1.4 m (Elev. 238.6 m) to 2.1 m (Elev. 237.7 m) below ground surface. It is thought that the fill may have been surplus excavated soil from the construction of HWY 11 which could account for the mixture of peat and silty clay.

The grain size distribution curve of a sample of this fill is shown in Figure B2-1. These results show a grain size distribution consisting of 5% gravel, 8% sand, 41% silt and 46% clay size particles.

A sample of the silty clay fill 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:	64%
Plastic Limit:	33%
Plasticity Index:	31%
Moisture Content:	31%

N values in the range of 5 to 8 blows for 0.3 m were determined in the fill, indicating a firm consistency. The moisture content of samples of this fill ranged from about 28 to 76%.

5.2.3 Clayey Silt to Silty Clay

Native clayey silt to silty clay deposits were encountered in all of the boreholes. These deposits extended to depths ranging from 5.5 m to 7.1 m below ground surface or to elevations ranging from 233.1 m to 232.7 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 0-1% gravel, 1-14% sand, 27-76% silt and 23-71% 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:	21-43%
Plastic Limit:	14-22%
Plasticity Index:	5-21%
Natural Moisture Content:	17-36%

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

Standard Penetration tests in these strata gave ‘N’ values that ranged from 1 to 10 blows for 0.3 m penetration and field vane tests gave in-situ undrained shear strengths ranging from 8 kPa to greater than 100 kPa. A laboratory vane test on a relatively undisturbed Shelby tube sample gave undrained shear strength of 24 kPa. Based on these results the clayey silt to silty clay was generally firm to stiff with some soft to very soft zones. The moisture content of samples of the clayey silt to silty clay ranged from 16% to 40% and the unit weight of a tested sample was 17.4 kN/m^3.

A one dimensional consolidation test was carried out on a tube sample of the clayey silt to silty clay deposit from Borehole CD5 and the results are presented on Figures B2-7 and B2-8. The consolidation characteristics listed in Table 5.3 were determined from the results of the consolidation testing.

Table 5.3 - Summary of Consolidation Testing on Silty Clay

Parameter	
Natural water content	33 %
Bulk Unit weight	17.4 kN/m^3
Dry Unit weight	13.2 kN/m^3
Compression index	0.341
Recompression index	0.042
Void ratio	1.04
Preconsolidation Pressure	60 kPa
Consolidation Coefficient	0.041 m^2/yr

5.2.4 Sandy Silt Till

Sandy silt till was encountered across this site extending to depths ranging from 14.6 m to 14.7 m below ground surface or to elevations ranging from 225.2 m to 225.3 m. The approach boreholes were terminated in this deposit at depths of 8.1 m (Elev. 230.5 m) and 9.6 m (Elev. 229.1 m).

The results of grain size distribution tests conducted on samples of this till are illustrated in Figure B2-9. These results show grain size distributions of 5-16% gravel, 31-33% sand, 41-55% silt and 7-11% 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 31 to more than 100 blows per 0.3 m penetration indicating a dense to very dense relative density. The moisture content of samples from this stratum ranged from about 8 to 17%.

5.2.5 Clayey Silt Till

Clayey silt till was encountered beneath the sand and silt till and to depths ranging from 25.4 m (Elev. 214.6 m) to 28.2 m (Elev. 211.6 m) below ground surface.

The grain size distribution plots of samples of the clayey silt till deposit are presented in Figure B2-10. These results show a grain size distribution consisting of 2-19% gravel, 16-35% sand, 40-62% silt and 13-24% 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 till were also subjected to Atterberg Limits tests and the results are presented in Figure B2-11. The index values from these tests are summarized below:

Liquid Limit:	18-22%
Plastic Limit:	12-14%
Plasticity Index:	5-10%
Natural Moisture Content:	8-15%

These values indicate low plasticity clayey silt soils.

The N values in the clayey silt till were typically greater than 100 blows for 0.3 m penetration indicating a hard consistency. The natural water content of the clayey silt till ranged from about 7 to 15 per cent.

5.2.6 Bedrock

The overburden was underlain by metamorphic phyllite bedrock. Bedrock was proved by coring in both abutment boreholes and the bedrock depth and top of bedrock elevations are summarized in Table 5.4.

Table 5.4 – Depth to Bedrock

BH No.	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
C3	28.2	211.6
C4	25.4	214.6

The bedrock has been described as weathered at depths extending to between 28.9 m (Elev. 210.9 m) and 29.0 m (Elev. 211.0 m). Below these depths the bedrock was described as unweathered and was colour is grey. Total core recovery in the bedrock ranged from 33% to 98%. The RQD values ranged widely from 0% to 74% 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 quality 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 5.5.

Table 5.5 – Water Level Measurements

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
Existing Bridge Site			
C1	August 06, 2010 August 10, 2010 September 03, 2010 April 26, 2012	0.2 0.9 0.9 1.0	241.4 240.7 240.7 240.6
C2	August 06, 2010 August 10, 2010 September 03, 2010 April 26, 2012	0.7 0.7 0.7 0.8	240.9 240.9 240.9 240.8
CD1	December 06, 2011 December 12, 2011 April 26, 2012	1.2 1.3 1.1	240.5 240.4 240.6
CD2	December 06, 2011 December 12, 2011	0.8 0.8	240.8 240.8
CD3	November 07, 2011 November 08, 2011 December 12, 2011 April 26, 2012	1.4 1.1 1.0 1.2	240.2 240.5 240.6 240.4
CD4	November 07, 2011 November 08, 2011 December 12, 2011 April 26, 2012	1.1 1.2 0.7 1.0	240.5 240.4 240.9 240.6
Detour Structure			
C3	August 06, 2010 August 10, 2010 September 03, 2010 April 26, 2012	0.8(*ag) 1.0(*ag) 1.2(*ag) 1.2	240.6 240.8 241.0 238.6
C4	August 10, 2010 September 03, 2010	1.1(*ag) 1.6(*ag)	241.1 241.6
CD5	November 07, 2011 November 08, 2011 December 12, 2011 April 26, 2012	0.1 0.0 0.8(*ag) 0.0	238.5 238.6 239.4 238.6
CD6	November 07, 2011 November 08, 2011 December 12, 2011 April 26, 2012	0.1 0.2 0.0 0.0	238.6 238.5 238.7 238.7

*ag: recorded water level above the ground.

The free water level in the creek was recorded at Elev. 239.18 m in August, 2010 indicating that the ground water table exists just below the ground surface in the flood plain area.

The recorded water levels in the standpipe piezometers indicated the presence of excess hydrostatic pressure at depth in the underlying till since the piezometric water levels were at or slightly higher than the ground surface of the flood plain.

At the existing bridge the piezometric head was estimated to range between Elev. ±240.4 m and Elev. ±240.9 m. Along the Detour Structure the recorded water levels are at the surface to 1.6 m higher than ground surface and the piezometric head ranged between Elev. ±238.7 and Elev. ±241.6 m.

All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and with precipitation conditions. The ground water level may also 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 and the installation and decommissioning of piezometers was 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 Boreshnavand, 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
CROW CREEK BRIDGE REPLACEMENT
HIGHWAY 11
3.7 KM WEST OF LOWTHER
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 5233-06-00, W.P. 5147-05-01, SITE 39W-055
GEOCRES No. 42G-35
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 on Highway 11 and a temporary detour bridge at Crow Creek located 3.7 km west of Lowther in the Township of McCrea; District of Cochrane, Ontario.

The existing Highway 11 bridge consists of a five span timber bridge measuring approximately ±23 m in length and about 11.7 m wide that carries Highway 11 east bound and west bound traffic over Crow Creek. This bridge will be replaced with a new single span structure. A detour will be constructed south of the existing highway to maintain Highway 11 traffic during construction of the new bridge. A temporary single lane bridge will be constructed over Crow Creek on the detour alignment. Final construction will consist of removing the temporary structure and site restoration work.

The replacement structure will be a single span bridge approximately 14 m wide and measuring 19 m in length between abutments. The proposed finished grades at the structure will be about Elevation 242.275 at the east abutment and at Elevation 242.150 at the west abutment. At the east and west abutments the approach fills will be about 0.4 to 0.5m higher than the existing grades. Highway 11 will be widened and the alignment shifted 2.2m to the south of the present alignment.

The detour structure will be a single span modular bridge approximately 7 m wide and measuring 21 m in length. The proposed finished grades at the structure will be about Elevation 241.75 at both ends and approach fills of up to 2m in height are required.

The 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 investigations.

7 STRUCTURE FOUNDATIONS

Existing Bridge Site (Boreholes C1, C2, CD1 & CD2)

The stratigraphy encountered at the abutment locations consisted of flexible pavement, sand fill and native deposits of sand and silt to silt, clayey silt to silty clay, sand and silt till and clayey silt till. The overburden extended to depths of 22.5 m (Elevation 219.1 m) and 28 m (Elevation 213.6 m) and was underlain by bedrock consisting of metamorphic phyllite and igneous granitoid. The ground water level approximately coincided with the flood plain level i.e. Elevation 239.5 m for design purposes. Excess hydrostatic pressure was encountered at depth in the till strata with a piezometric head estimated to range between about Elevations 240.4 m and 240.9 m.

Detour Structure (Boreholes C3 & C4)

The stratigraphy encountered at the abutment locations was similar to the conditions encountered at the existing bridge as described above. Bedrock was encountered at depths of 25.4 m (Elevation. 214.6 m) and 28.2 m (Elevation 211.6 m). Excess hydrostatic pressure exists was indicated in the glacial till deposits with a piezometric head of about 1.2 m (Elevation 241.0 m) to 1.6 m (Elevation 241.6m) above the ground surface.

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

The geotechnical resistance of the near surface soils are low and foundation settlements will be high. Consequently, spread footings on native ground were not considered to be practical and have not been recommended.

It is noted that competent till soils capable of supporting spread footings exist at depths ranging from 7.1 m to 9 m below existing grade. However, designing a footing or an engineered fill pad to bear on these competent soils will require relatively deep and extensive excavations with potentially difficult ground water conditions. Therefore, this option was not considered feasible.

7.2 Augered Caissons (Drilled Shafts)

Augered caisson foundations were also considered for supporting the structures. However, the caissons must be founded on the very dense sand and silt to sandy silt till.

The base of the caissons would be about 10 to 12 m below the ground water level, resulting in high hydrostatic heads at the base in relatively permeable sand and silt to sandy silt till strata. 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 possibly 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 ended steel pipe piles were considered but excluded in favour of low displacement H piles. Open ended steel pipe piles with toes strengthened using cruciform plates and/or pile points could also be considered; however there would be a high risk of damage due to cobbles and boulders in the till deposits. H-pile sections are low displacement sections that have a higher probability of achieving the desired penetration and being installed successfully.

Steel H-piles are likely to be driven to practical refusal in till soils at all 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.

7.3.1 Axial Resistance

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 sand and silt till or sandy silt till 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 Pile Sections Driven to Bedrock

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	C1	229.0±	Sand and Silt Till	1600	1200
	CD2	227.0±	Sand and Silt Till		
East Abutment	C2	227.5±	Sand and Silt Till		
	CD1	229.0±	Sand and Silt Till		
Detour Structure					
West Abutment	C3	228.0±	Sandy Silt Till	1600	1200
East Abutment	C4	229.0±	Sandy Silt Till		

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	C1	228.0±	Sand and Silt Till	2100	1600
	CD2	226.0±			
East Abutment	C2	223.0±	Clayey Silt Till		
	CD1	228.0±			
Detour Alignment					
West Abutment	C3	226.0±	Sandy Silt Till	2100	1600
East Abutment	C4	228.0±	Sandy Silt Till		

The H-piles for the recommended foundation scheme will be driven to effective refusal in the overburden soils. 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 0.4 m. However, to accommodate the integral abutment construction a 3.0 m long CSP will surround the pile in the clayey silt to silty clay stratum. Consequently, downdrag forces on the piles due to embankment reconstruction and the grade raise will be minimal.

Embankment construction required for the detour will cause settlement of the underlying soils thereby imparting downdrag forces on piles. Downdrag forces on piles are estimated based on compressible silty clay soils that extend to Elev. 232.5 m. Unfactored downdrag loads of 175 kN/pile (HP 310 x 110 section) and 200 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:

$k_s = n_h \cdot z / D$ [cohesionless soils] (kN/m^3)
$k_s = 67\ S_u/D$ [cohesive soils] (kN/m^3)
$p_{ult} = 3 \cdot \gamma \cdot z \cdot K_p$ [cohesionless soils] (kPa)
$p_{ult} = 9\ S_u$ [cohesive soils] (kPa)

where z = depth of embedment of pile (m)
D = pile width (m)
S_u = undrained shear strength (Table 7.4) (kPa)
n_h = coefficient of horizontal subgrade reaction (Table 7.3) (kN/m^3)
γ = unit weight (Table 7.4) (kN/m^3)
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^3), 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/m³)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m³)*
Existing Bridge Site						
West Abutment C1	241.3 – 239.5	Fill – Sand	19	28	–	2200
	239.5 – 237.9	Sandy Silt	19	28	–	1300
	237.9 – 232.6	Silty Clay	19	0	40	–
	232.6 – 224.0	Sand and Silt Till	20	35	–	11000
	224.0 – 213.6	Clayey Silt Till	20	0	225	–
West Abutment CD2	241.2 – 239.5	Fill – Sand	19	28	–	2200
	239.5 – 237.9	Silt	19	28	–	1300
	237.9 – 233.1	Clayey Silt to Silty Clay	19	0	50	–
	233.1 – 229.5	Sand and Silt Till	20	35	–	4400
	229.5 – 225.4	Sand and Silt Till	20	35	–	11000
	225.4 – 215.2	Clayey Silt Till	20	0	225	–
East Abutment C2	241.3 – 239.5	Fill – Sand	19	28	–	2200
	239.5 – 238.7	Sand and Silt	19	28	–	1300
	238.7 – 236.5	Silty Clay	19	0	75	–
	236.5 – 232.9	Silty Clay	19	0	40	–
	232.9 – 231.5	Sand and Silt Till	20	35	–	4400
	231.5 – 226.9	Sand and Silt Till	20	35	–	11000
	226.9 – 219.1	Clayey Silt Till	20	0	225	–
East Abutment CD1	241.3 – 239.6	Fill – Sand	19	28	–	2200
	239.6 – 238.8	Sand and Silt	19	28	–	1300
	238.8 – 233.2	Clayey Silt to Silty Clay	19	0	40	–
	233.2 – 227.1	Sand and Silt Till	20	35	–	11000
	227.1 – 218.2	Clayey Silt Till	20	0	225	–
Detour Structure						
West Abutment C3	239.5 – 237.7	Fill – Silty Clay	18.5	0	30	–
	237.7 – 232.7	Silty Clay	19	0	40	–
	232.7 – 225.2	Sandy Silt Till	20	35	–	11000
	225.2 – 211.6	Clayey Silt Till	20	0	225	–
East Abutment C4	239.8 – 238.6	Fill – Silty Clay	18.5	0	30	–
	238.6 – 232.9	Silty Clay	19	0	50	–
	232.9 – 225.3	Sandy Silt Till	20	35	–	11000
	225.3 – 214.6	Clayey Silt Till	20	0	225	–

* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd 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_s quoted in this section may be used in conjunction with appropriate reduction factors.

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

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

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

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

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

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

Intermediate values 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.

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

7.3.8 Recommended Foundation

From a geotechnical point of view, it is recommended that all foundations for the new bridge and detour structure be supported on steel H-piles.

7.4 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).

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	ϕ (deg)	γ (kN/m^3)	K_a	K_o	K_p
Existing Bridge Site (Boreholes C1, C2, CD1 & CD2)					
Fill – Sand	28	19	0.36	0.53	2.77
Sand and Silt to Silt	28	19	0.36	0.53	2.77
Clayey Silt to Silty Clay	27	19	0.38	0.55	2.66
Sand and Silt Till	35	20	0.27	0.43	3.70
Clayey Silt Till	27	20	0.38	0.55	2.66
Detour Structure (Boreholes C3 & C4)					
Fill – Silty Clay	27	18.5	0.38	0.55	2.66
Silty Clay	27	19	0.38	0.55	2.66
Sandy Silt Till	35	20	0.27	0.43	3.70
Clayey Silt Till	27	20	0.38	0.55	2.66

It is envisaged that the shoring will consist of a system of interlocking steel sheet piling. The shoring can be designed as a cantilevered system or supported by a system of struts and wales in the case of closed cell cofferdam, or supported by employing a soil anchor system depending on the depth of soil to be retained and the performance criteria. It is expected that sheet piling would encounter refusal in the glacial till strata. .

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.

9 EXCAVATION AND BACKFILL

9.1 General

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 Clay) – Type 3 soils above the water table and Type 4 soils below the water table.
- Sand and Silt to Silt – Type 4 soils below the water table.
- Clayey Silt to Silty Clay – Type 4 soils below the water table.
- Sand and Silt to Sandy Silt Till – Type 4 soils below the water table.
- Clayey Silt 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, 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 239.18 m in August, 2010 indicating that the ground water table is generally just below the ground surface in the flood plain area. The recorded water levels in the standpipe piezometers indicate the presence of excess hydrostatic pressure in the underlying till strata. However, excess hydrostatic pressure will not be encountered in shallow excavations extending into the underlying silty clay soils.

Excavations at the bridge sites may penetrate surficial strata of sandy silt and sand and silt soils and terminate in the firm to stiff clayey silt to silty clay below the ground water level. These soils will be easily disturbed by construction activity. The overlying sandy silt sand and silt strata will yield water due to their relatively high permeability. To alleviate construction related problems we recommend that the ground water table be lowered and maintained at least 1 m below the base of the excavation. Alternatively, a system of interlocking steel sheet piling as outlined above and penetrating several metres into the underlying clayey silt to silty clay, will nearly cut off the ground water seepage from the surficial strata. Depending on the actual design of the sheeting and the workmanship in the installation, it may be feasible to achieve adequate control of the ground water and surface water by pumping from a series of properly filtered sumps located as required within the sheeted area. Similar conditions are expected at the Detour Structure.

The pile driving operations will cause significant remoulding of the clay soils around the pile shafts thereby forming a watertight barrier that will impede the upward movement of ground water at the soil/pile interface. Therefore, an inverted granular filter below the pile caps will not likely be required.

11 APPROACH EMBANKMENTS

The new bridge will result in the highway alignment being shifted 2.2m to the south of the existing alignment. The working point elevations of 242.275 m and 242.150 m on the east and west sides of the bridge will result in a grade rise of about 0.4 to 0.5m over the structure. The existing approach embankments will therefore need to be widened and raised.

11.1 Stability

11.1.1 Highway 11

The results of a series of shallow boreholes drilled during the pavement investigation suggest that discontinuous, near surface deposits of peat exist beyond the existing road embankments. In addition it is considered that peat and silty clay was excavated during the initial highway construction was probably deposited adjacent to the highway. This may account for the mixture of silty clay and peat fill encountered in some of the boreholes (i.e BH CD5 and CD6). The peat and fill that contains peat will need to be removed beneath any areas where embankment widening is required for Highway 11, to minimize differential settlement as well as to enhance the stability of the embankments. The sub-excavated areas will have to be restored using Granular B Type 1 or select subgrade material.

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.

Slope Stability analyses of the embankments were carried out as part of the preliminary investigation using the commercially available slope stability program Slide 5.0 by Rocscience Inc. The Janbu, Morgenstern-Price and Bishop's simplified method for stability analysis were employed. Both drained and undrained stability analyses were conducted for a range of embankment slope inclinations. It was assumed that the shallow peat deposits would be removed in preparation for embankment construction and that the new embankment fill would be benched into the existing embankment.

The results of the analyses indicated Factors of Safety of greater than 1.3 for embankment heights of up to 4.5m for earth fill embankments constructed with 2 horizontal to 1 vertical or flatter side slopes and for rock fill embankments at 1.25H:1V side slopes. The slope stability models are included in Appendix E.

Mid-height berms will not be required since the embankment heights will be less than 8 m (earth fill) or 10 m (rock fill).

11.1.2 Detour Embankments

The detour will be constructed on the south side of the existing bridge and will traverse areas underlain by fill containing peat as well as relatively shallow deposits of peat. Section 11.2.2 outlines an approach for constructing the embankment and roadway for the detour. This approach involves leaving the peat in place and a staged embankment construction. An embankment over-build (i.e. preloading) has been recommended in order that the settlements of the detour pavement would be within a tolerable range when the detour will be in service. The results of the initial slope stability analysis indicated that a geotextile as well as a reinforcing grid would be required to achieve a Factor of Safety of greater than 1.3 for embankments constructed with 3 horizontal to 1 vertical side slopes.

It is understood that side slope inclinations of 3 horizontal to 1 vertical are not feasible along the entire length of the temporary embankment due to property constraints and that inclinations approaching 2 horizontal to 1 vertical will consequently be required in some sections. A factor of safety of less than 1.3 has been indicated for these steeper inclinations. The stability analyses indicate that a second layer of geogrid nominally about 0.6m above the bottom layer will improve the factor of safety for 2 horizontal to 1 side slopes to an acceptable range and is required in areas where the side slope inclinations will be steeper than 3 horizontal to 1 vertical.

The slope stability analyses for the 2 horizontal to 1 vertical side slopes with reinforcement are presented in Appendix F. The recommended cross section for the temporary embankments is shown on Figure G1 in Appendix C.

It is understood that granular pads are required on both sides of the creek to support the approach ramps for the temporary modular bridge. Based on the proximity of the temporary bridge to the new bridge, the zone of influence of the granular pads will encroach into the embankment widening required at the Highway 11 Bridge. To minimize the potential for differential settlement between sections of the pad underlain by peat and sections encroaching into the new highway embankment (i.e. where the peat will be removed and replaced with engineered fill), and to ensure uniform support for the approach ramps, the peat must be subexcavated beneath the pads and replaced with a suitable engineered fill. Based on the results of the pavement investigation it is expected that subexcavation to depths of about 1.8m below the existing ground will be required to remove the peat. Due to the difficult ground water conditions at the site, the removal of the peat and the backfilling will have to be carried out as a continuous operation and using wet construction techniques (i.e. swamp excavation). The excavation can be restored to the original grade using 50mm clear crushed stone. This material should be placed using forward spreading techniques to at least 300mm above the flood plain. Final construction will consist of thoroughly compacting the surface using a large diameter smooth drum roller and placing and compacting the Granular "A" to the usual standards.

The side slope inclinations of the 50 mm clear stone must be at 1.25horizontal to 1 vertical or flatter for stability. The side slopes on the Granular A placed on the 50mm clear stone should be 1 horizontal to 1 vertical of flatter.

11.2 Settlement

11.2.1 Highway 11

At the existing bridge site the grade raise will be approximately 0.4 to 0.5 m above the existing grade. The grade raise in combination with the alignment shift will necessitate widening the existing embankments, primarily on the south side. Embankment fill heights approaching about 2m will be required in some locations to achieve these intents. Settlements due to the consolidation of the underlying silty clay are expected. A high proportion of the settlement will be differential in nature since the grade raise over the existing embankment will generally be less than 0.5m. The greatest settlement will occur along the south edge of the roadway platform and would be progressively less toward the centre of the road as the fill height reduces. The effects of the differential settlement could impact the performance of the approach slab and therefore warrants further consideration.

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 B2-7 and summarized in Table 5.2. A range of settlement was calculated due to raising the existing embankment and for the embankment widening (ie maximum fill height) at various stages. This data is summarized below.

Location	Settlement at Various Times (mm)				
Time Period	6 months	1 year	2 years	5 years	Total settlement
Existing Embankment	5 -15	10 – 20	15 – 30	20 – 35	20 -40
Embankment widening	30-50	50-70	70-100	100 – 135	110 -150

The construction staging presently under consideration involves constructing the embankment widening to match the existing road grade in the first phase, when the detour embankment is also being constructed. The embankment will be constructed to the final grade in the second phase of the work (which will commence about six months after the initial phase) which will also include the Highway 11 bridge construction. It is estimated that the approach slabs would probably be completed within about 1 year of completing the initial grading. In this scenario the post construction differential settlement across the width of the approach embankment could be in the range of about 50 to 60 mm.

The effects of post construction differential settlement could be addressed by one of the following approaches:

- Allow the settlement to take place and return to the site later (ie 1 to 2 years) to pad the surface as required and to restore the approach slabs to the design grade by mud jacking;
- Allow the settlement to take place and modify the design of the slab to resist the effects of the settlement to the extent practicable;
- Reduce the embankment weight in the widened sections (and the settlement) by using light weight backfill;
- Carry out ground improvement work possibly with the use of rammed aggregate piers.

The first two approaches may address the issues with the performance of the slabs to some degree however padding may eventually be required to address the settlement in the shoulders beyond the slabs. The widening and grade raise extend well beyond the structure and fill heights will approach about 1m some 70 m east of the bridge. The settlement will be primarily in the south shoulder.

Use of certain types of slag have been used as light weight fill however in this instance shipping such materials to the site will result in a relatively high cost with only marginal benefit since the fill heights are relatively low. In addition there may be some adverse environmental impacts associated with the use of slag.

Alternatively use of a geofoam product would have proportionately the largest impact because of its extremely low density. For the 2m high fill, substitution of 1m of the embankment fill with geofoam could decrease the expected settlement by about 50% which may be within a tolerable range, however the potential for post construction differential settlement will still exist.

Other alternatives could consist of use of ground improvement techniques such as the use of mini piles, or supporting the approach slab on a series of driven piles terminating in the silt and sand till at depths of about 10m. Post construction maintenance would still be required to address settlement in the adjacent pavement. Use of the rammed aggregate piers is considered feasible and may be effective; however the cost of this proprietary technique is extremely high relative to the overall cost of the project.

Based on the above it is recommended that one of or a combination of the first two approaches be used. The performance of the slabs should be assessed every six months for up to two years following construction to assess the need for and scope of any remedial work. It is understood that this approach is being used in similar conditions on several other sites in Northeastern Region.

11.2.2 Detour

A substantial length of the detour alignment is underlain by highly compressible peat and soft silty 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:

- Excavate and replace the peat with engineered fill;
- Select an alternate alignment;
- Float the embankment on the existing ground and stage the construction 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.

Substantial deformations are expected due to the consolidation of the peat under the embankment loading. A large proportion of the consolidation will however take place during the months immediately following embankment construction and the rate of settlement would be expected to decrease after the primary consolidation phase. Staged construction of the detour would involve construction of the embankment (and possibly a surcharge) in the fall with resumption of construction of the detour pavement and the temporary modular bridge the following spring (i.e. after the primary settlement has occurred). Settlement would be expected to continue during the service 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. The conditions at Station 24+450 were selected for analysis as these conditions were typical of the thicker layers of peat under the higher fills. At this location, the profile grade will be about 2 m above the existing grade and the thickness of the underlying peat was about 2 m. Three cases were considered using embankments initially over built by 0.5 m, 0.8 m and 1.4 m. Table 11.1 below shows for each case, the anticipated settlement in the first 6 months and the consolidation that can be expected after the detour is completed and is in service (i.e. between 6 and 10 months after the initial embankment construction). The consolidation characteristics of the peat summarized in Table 5.2 of this report were used in the analyses.

Table 11.1 – Potential Settlement

	Case 1 0.5 m Overbuild	Case 2 0.8 m Overbuild	Case 3 1.4 m Overbuild
Consolidation 0 - 6 months (mm)	450-550	500-600	600-700
Consolidation 6 -10 months (mm)	50 – 100	60 - 100	80 - 160

The results of the analyses indicate that for Case 1, where the embankment is overbuilt by 0.5 m the resulting grade after a nominal six month period may approach the existing grade, more or less. An additional 50 mm to 100 mm of settlement could be expected during the serviceable life of the detour (i.e. between six and 10 months following embankment construction). If construction of the detour would be delayed beyond ten months the anticipated additional settlement would be less than about 50mm.

It has been concluded that surcharging the embankment in excess of 0.5 m will not be effective. This is due in part to the consolidation of the underlying silty clay.

Based on the above consideration it has been recommended that the detour be overbuilt by 0.5 m to account for the anticipated primary settlement.

11.2.3 Approach Ramps

The granular pads that will support the approach ramps for the temporary modular bridge as outlined in Section 11.1.2 will also experience some settlement due to consolidation of the underlying silty clay (it has been recommended that the peat be removed from the footprint of the pads). Consideration could also be given to constructing the granular pad during the first phase of the construction to minimize the magnitude of the settlement and the effects on the temporary approach slab. The expected settlement for the approach described above is estimated to be in the range of about 20 to 40mm.

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.

12 EMBANKMENT CONSTRUCTION

Embankment construction should be in accordance with OPSS 206, November 2009. As outlined above, consideration should be given to constructing the embankment widening during the first phase of the construction and the approach fills early in the second phase, in order to reduce post construction settlement. Oversize materials (e.g. greater than 75 mm nominal diameter) should not be used in the embankment fills through which piles will be driven.

Earth fill embankment slopes and cut slopes must be provided with erosion protection in accordance with OPSS 571 and OPSS 572. Bonding between the embankment fill and the existing soils should be established by benching as per OPSD 208.010.

13 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. Alternatively suitable filter protection must be provided between the rock fill and the backfill.

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.

14 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), active horizontal earth pressure may be used in the geotechnical design of the structure. If the support system does not allow yielding (restrained system), 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)
γ = 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 14.1.

Table 14.1 – Earth Pressure Coefficients

Wall Condition	Earth Pressure Coefficient (K)					
	OPSS Granular A or OPSS Granular B Type II $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°$; $\gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.40*	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.

15 EROSION PROTECTION

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

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

It is recommended that the new highway embankment slopes and detour fill slopes 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 scheduled to correspond with optimal germination conditions in the spring of 2014.

16 SEISMIC CONSIDERATIONS

16.1 Seismic Design Parameters

The site is treated as lying in Seismic Zone 0. The following seismic parameters (Hearst) 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 0.059

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.

16.2 Liquefaction Potential

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 with 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 value of 30 blows per 0.3m has been considered.

Although the grain size and plasticity characteristics, and degree of saturation of the sandy 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.

16.3 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 ϕ. For the design of retaining walls, the coefficients of horizontal earth pressure in Table 16.1 may be used:

Table 16.1 – Earth Pressure Coefficient for Earthquake Loading

Earth Pressure Coefficient (K) for Earthquake Loading						
Wall Condition	Granular A or Granular B Type II $\phi = 35°$; $\delta = 17.5°$ $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\delta = 16°$ $\gamma = 21.2$ kN/m^3		Rock Fill $\phi = 42°$; $\delta = 21°$ $\gamma = 19.0$ kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (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

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23-5-2012
PROVINCE OF ONTARIO

TABLE

TERRAPROBE INC.

TABLE 1

DOCUMENT	TITLE
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

TERRAPROBE INC.

APPENDIX A

TERRAPROBE INC.

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.

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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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ε	%	LINEAR STRAIN
$\varepsilon_1, \varepsilon_2, \varepsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

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

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

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:

		Approximate φ
T	= Tight, hard, non-softened	
O	= Oxidation surface staining only	25 - 35
SA	= Slightly altered; clay-free	25 - 30
S	= Sandy particles; clay-free	25 - 30
Si	= Sandy and silty, minor clay	20 - 25
NC	= Non-softening Clays; 5mm	16 - 24
SC	= Swelling Clay fillings; 5mm	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:

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

Ministry of Transportation Ontario

Terraprobe

Foundation Design

RECORD OF BOREHOLE No CD1

METRIC

G.W.P. 5233-06-00 | LOCATION Coords: E:372347.7 N:5492810.1 | ORIGINATED BY PB

DIST - HWY Hwy 11 | BOREHOLE TYPE HOLLOW STEM AUGERS / WASH BORING | COMPILED BY DB

DATUM GEODETIC | DATE 11.7.11 | CHECKED BY HA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.7	GROUND SURFACE					
241.5 0.2	200mm **ASPHALTIC CONCRETE**	1A	AS			
241.3 0.4	180mm FILL, sand and gravel, trace silt, brown, damp	1B			241	
	FILL, sand, trace silt, trace gravel; loose to compact, brown, damp	2	SS	10		
		3	SS	6	240	0 95 5 0
239.6 2.1	**SAND and SILT**, some clay, loose, brown, wet	4	SS	4	239	
238.8 2.9	**CLAYEY SILT to SILTY CLAY**, trace sand, trace gravel, firm to stiff, grey, damp to moist	5	SS	6	238	
		6	SS	12		0 8 71 21
		7	SS	4	237	
		8	ST		236	
		9	SS	2	235	
		10	AS			
		11	AS		234	0 10 66 24
233.2 8.5	**SAND and SILT**, some clay, trace gravel, occasional cobbles and boulders, dense to very dense, grey, damp to moist (GLACIAL TILL)	12A 12B	AS		233	
		13	SS	50	232	commence casing and washboring
		14	SS	116	231	
		15	SS	194 / 275mm	230 / 229	1 39 48 12
		16	SS	100 / 100mm	228	
227.1 14.6	**CLAYEY SILT**, sandy, trace gravel, hard, grey, moist (GLACIAL TILL)				227	
		17	SS	99	226	

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No CD1

METRIC

G.W.P. 5233-06-00 LOCATION Coords: E:372347.7 N:5492810.1 ORIGINATED BY PB

DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS / WASH BORING COMPILED BY DB

DATUM GEODETIC DATE 11.7.11 CHECKED BY HA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued) **CLAYEY SILT**, sandy, trace gravel, hard, grey, moist (GLACIAL TILL) *(continued)*				
		18	SS	60	3 28 47 22
		1	RUN		
		2	RUN		
		3	RUN		
		4	RUN		
218.2 / 23.5	For details see rock core log cd1 (BEDROCK)	5	RUN		
		6	RUN		
		7	RUN		
		8	RUN		
		9	RUN		
		10	RUN		
		11	RUN		
212.3 / 29.4					

Remarks (groundwater): Nov.7 / Nov.8

Elevation scale: 225, 224, 223, 222, 221, 220, 219, 218, 217, 216, 215, 214, 213

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m^3

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)
December 6, 2011	1.2	240.5
December 12, 2011	1.3	240.4
April 26, 2012	1.1	240.6

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

CORE LOG

Terraprobe

Project: Crow Creek Bridge Replacement	Orientation: Vertical	Ground Elevation: 241.7m	Datum: Geodetic	Borehole No.: CD1
Location: Hwy 11, Township of McCrea, Ontario	Date Started: November 8, 2011	Completed: November 8, 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

ELEVATION (m)	DEPTH (m)
222.7	19.0
222.5	19.2
221.7	20.0
220.7	21.0
219.7	22.0
218.7	23.0
217.7	24.0
216.7	25.0
215.7	26.0
214.7	27.0
213.7	28.0
212.7	29.0
212.3	29.4
211.7	30.0
210.7	31.0

SYMBOL (3)	GENERAL DESCRIPTION (4)	No. OF SETS (5)	JOINT TYPE (6)	ORIENTATION (7)	SPACING (8)	ROUGHNESS (9)	FILLING (10)	APERTURE (11)	WEATHERING (12)	STRENGTH (13)	FRACTURE FREQUENCY (14)	RUN NO. CORE RECOVERY % (15)	R Q D % (16)	CORE SIZE/CASING (17)	UNCONFINED COMPRESSIVE STRENGTH MPa (18)	UNIT WEIGHT (KN/m³) (19)
												#1		NQ		
												#2		NQ		
												#3		NQ		
	Overburden, see Borehole Log CD1											#4		NQ		
Bedrock	BEDROCK – PHILLITE (Metasediments) moderately to highly weathered, subvertica foliations, dark grey, low strength.	1	CC	F	VC	SP	O	0 to 1				#5 TCR 89 SCR 72	0	NQ		
Bedrock												#6 TCR 90 SCR 60		NQ		
Bedrock	Rubblized zones at: 24.9m, 25.1m, 25.5m, 26.4m, 28.5m.	2	CC	DV	VC	SP	O	0 to 1				#7 TCR 98 SCR 78	37	NQ		
Bedrock		2	CC	DV	M	SP	O	0 to 1				#8 TCR 84 SCR 70	19	NQ		
Bedrock		2	CC	FV	VC	SP	O	0 to 1				#9 TCR 100 SCR 100	90	NQ		
Bedrock		2	CC	FV	M	SP	O	0 to 1				#10 TCR 100 SCR 83	58	NQ		
Bedrock		2	CC	FV	M	SP	O	0 to 1				#11 TCR 100 SCR 100	100	NQ		
	End of Core Log															

Remarks:

LEGEND:

Bedrock

Y:\1-Project Files\11-Geotechnical\2010\1-10-5001 to 5099\1-10-5076A, Dwgs, Logs\AutoCAD\1-10-5076 CORE.dwg, DB

Ministry of Transportation Ontario

Terraprobe

Foundation Design

RECORD OF BOREHOLE No CD2

METRIC

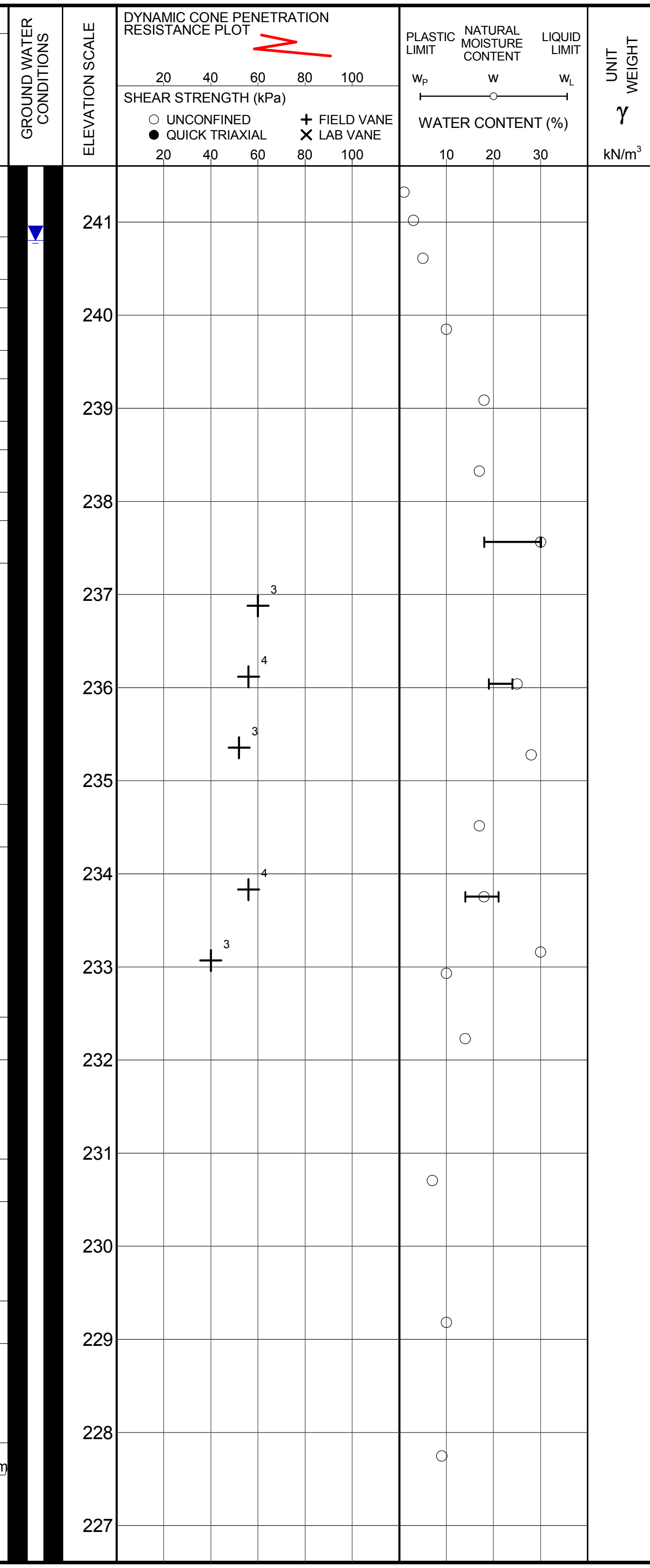
G.W.P. 5233-06-00	LOCATION Coords: E:372322.4 N:5492831.1		ORIGINATED BY PB
DIST -	HWY Hwy 11	BOREHOLE TYPE HOLLOW STEM AUGERS / WASH BORING	COMPILED BY DB
DATUM GEODETIC		DATE 11.5.11	CHECKED BY HA

SOIL PROFILE

ELEV / DEPTH (m)	DESCRIPTION
241.6	**GROUND SURFACE**
241.5 / 0.2	150mm **ASPHALTIC CONCRETE**
241.2 / 0.4	250mm FILL, sand and gravel, trace silt, brown, damp
	FILL, sand, trace silt, trace gravel; loose to compact, brown, damp to moist
239.5 / 2.1	**SILT**, some clay, some sand, loose, brown, wet
237.9 / 3.7	**CLAYEY SILT to SILTY CLAY**, trace to some sand, trace gravel, firm to stiff, grey, moist
233.1 / 8.5	**SAND and SILT**, trace clay, trace gravel, occasional cobbles, dense to very dense, grey, damp to moist (GLACIAL TILL)

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1A	AS		
1B			
2	SS	18	
3	SS	8	
4	SS	6	
5	SS	6	0 19 69 12
6	SS	6	0 3 44 53
7	ST		
8	AS		0 6 64 30
9	AS		
10	SS	3	
11	AS		0 15 61 24
12A	AS		
12B			
13	SS	30	8 39 45 8
14	SS	33	
15	SS	91	Nov.5 / Nov.6
16	SS	100 / 125mm	3 44 47 6 commence casing and washboring



Continued Next Page

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

RECORD OF BOREHOLE No CD2

METRIC

G.W.P. 5233-06-00	LOCATION Coords: E:372322.4 N:5492831.1		ORIGINATED BY PB
DIST -	HWY Hwy 11	BOREHOLE TYPE HOLLOW STEM AUGERS / WASH BORING	COMPILED BY DB
DATUM GEODETIC	DATE 11.5.11		CHECKED BY HA

ELEV DEPTH (m)	DESCRIPTION *(continued)*	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SAND and SILT, trace clay, trace gravel, occasional cobbles, dense to very dense, grey, damp to moist (GLACIAL TILL) *(continued)*	17	SS	198 / 200mm	226		
225.4 16.2	**CLAYEY SILT**, sandy, trace gravel, occasional cobbles, hard, grey, moist (GLACIAL TILL)				225		
		18	SS	81			6 31 47 16
					224		
		19	SS	196	223		
					222		
		20	SS	124	221		
		21	SS	115	220		1 28 54 17
					219		
		22	SS	100 / 125mm			
		1	RUN		218		
		2	RUN		217		
		3	RUN		216		
215.2 26.4	For details see rock core log cd2 (BEDROCK)	4	RUN		215		
					214		
		5	RUN		213		
		6	RUN		212		

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; GROUND WATER CONDITIONS; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa) ○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m^3

Continued Next Page

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

RECORD OF BOREHOLE No CD2

METRIC

G.W.P. 5233-06-00 LOCATION Coords: E:372322.4 N:5492831.1 ORIGINATED BY PB

DIST - HWY Hwy 11 BOREHOLE TYPE HOLLOW STEM AUGERS / WASH BORING COMPILED BY DB

DATUM GEODETIC DATE 11.5.11 CHECKED BY HA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE						
	(continued)										
210.7 30.9	For details see rock core log cd2 (BEDROCK) *(continued)*		7	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)
December 6, 2011	0.8	240.8
December 12, 2011	0.8	240.8
April 26, 2012	0.8	240.8

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

CORE LOG

Terraprobe

Project	Orientation	Ground Elevation	Datum	Borehole No.
Crow Creek Bridge Replacement	Vertical	241.6m	Geodetic	CD2
Location	**Date Started**	**Completed**	**Logged By**	**Sheet**
Hwy 11, Township of McCrea, Ontario	November 6, 2011	November 6, 2011	B.R.	1 of 1
Client	**Drilling Agency**	**Drill Type**	**Core Barrel & Bit Design**	**Project No.**
MTO	Landcore Drilling	CME55	NQ	11-10-5076

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics: No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	UNCONFINED COMPRESSIVE STRENGTH MPa	UNIT WEIGHT (KN/m³)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
219.6	22.0																	
218.6	23.0																	
218.4	23.2													#1				
217.6	24.0													#2				
216.6	25.0													#3				
215.6	26.0		Overburden, see Borehole Log CD2															
214.6	27.0	Bedrock	<u>BEDROCK – PHILLITE (Metasediments)</u> unweathered, subvertical foliations, dark grey, high strength.	1	C	F	VC C	SP	O	0 to 1				#4 TCR 100 SCR 100	15	NQ		
213.6	28.0	Bedrock	Moderately to highly fractured: from 26.4m to 27.6m	2	CC	DV	M	SP	O	0 to 1				#5 TCR 100 SCR 92	74	NQ		
212.6	29.0	Bedrock		2	CC	FV	VC	SP	O	0 to 1				#6 TCR 100 SCR 28	33	NQ		
211.6	30.0	Bedrock		2	CC	FV	M	SP	O	0 to 1				#7 TCR 100 SCR 47	13	NQ		
210.7	30.9		End of Core Log															
210.6	31.0																	
209.6	32.0																	
208.6	33.0																	
207.6	34.0																	

Remarks:

<u>LEGEND:</u>

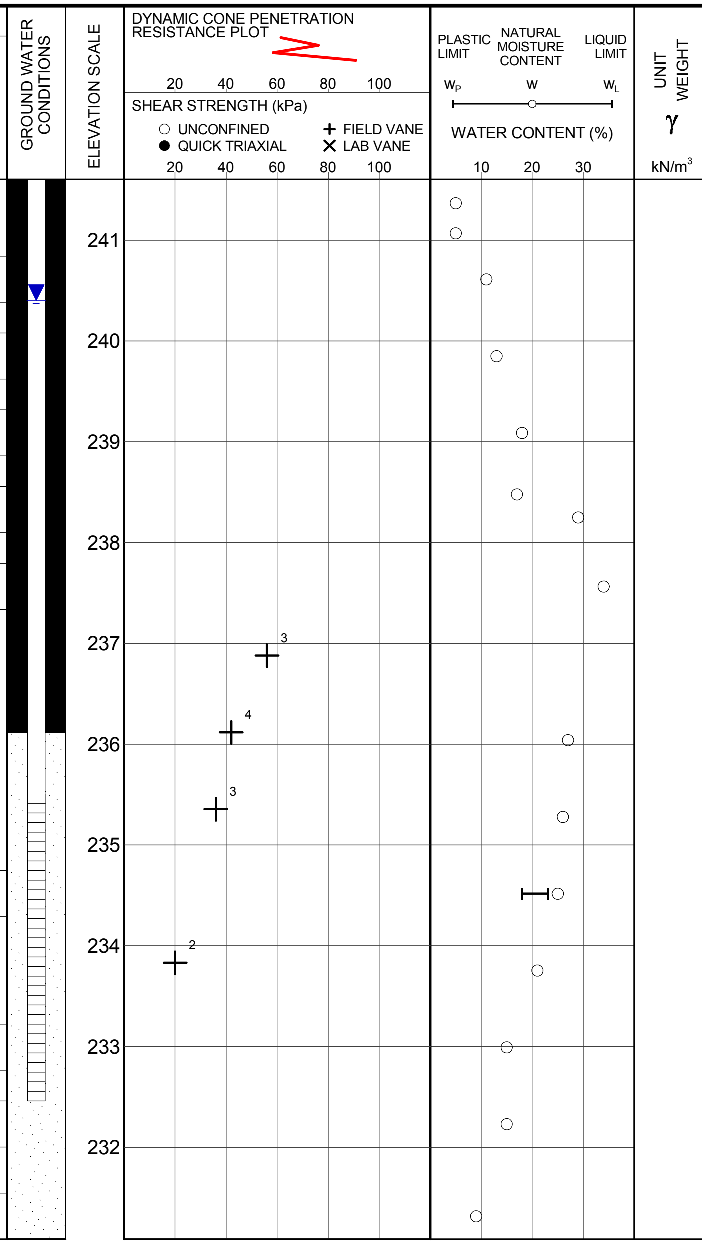
Bedrock

RECORD OF BOREHOLE No CD3

METRIC

G.W.P. 5233-06-00	LOCATION Coords: E:372293.4 N:5492832.4		ORIGINATED BY PB
DIST -	HWY Hwy 11	BOREHOLE TYPE HOLLOW STEM AUGERS	COMPILED BY DB
DATUM GEODETIC	DATE 10.19.11		CHECKED BY HA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.6	**GROUND SURFACE**				
241.4 / 0.2	170mm **ASPHALTIC CONCRETE**	1A			
241.3 / 0.3	130mm FILL, sand and gravel, trace silt, brown, damp	1B	AS		
	FILL, sand, trace silt, trace gravel; compact, brown, moist to wet	2	SS	19	
	...at 1.5m, silty	3	SS	18	
239.5 / 2.1	**SILT**, some clay, some sand, trace organics, compact, brown, wet	4	SS	13	0 11 71 18
238.4 / 3.2	Trace organics	5A / 5B	SS	8	
		6	SS	5	
237.2 / 4.4	**CLAYEY SILT to SILTY CLAY**, trace sand, trace gravel, soft to stiff, grey, moist	7	ST		
		8	AS		
		9	AS		
		10	SS	3	0 4 72 24
		11	AS		
233.4 / 8.2	**SAND and SILT**, trace clay, trace gravel, occasional cobbles and boulders, dense to very dense, grey, moist (GLACIAL TILL)	12	SS	37	0 37 55 8
		13	SS	69	
231.1 / 10.5		14	SS	61	



END OF BOREHOLE
Auger refusal

Unstabilized water level measured at 8.5m and borehole caved to 9.1m below grade upon completion of drilling

25mm piezometer installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
November 7, 2011	1.4	240.2
November 8, 2011	1.1	240.5
December 12, 2011	1.0	240.6
April 26, 2012	1.2	240.4

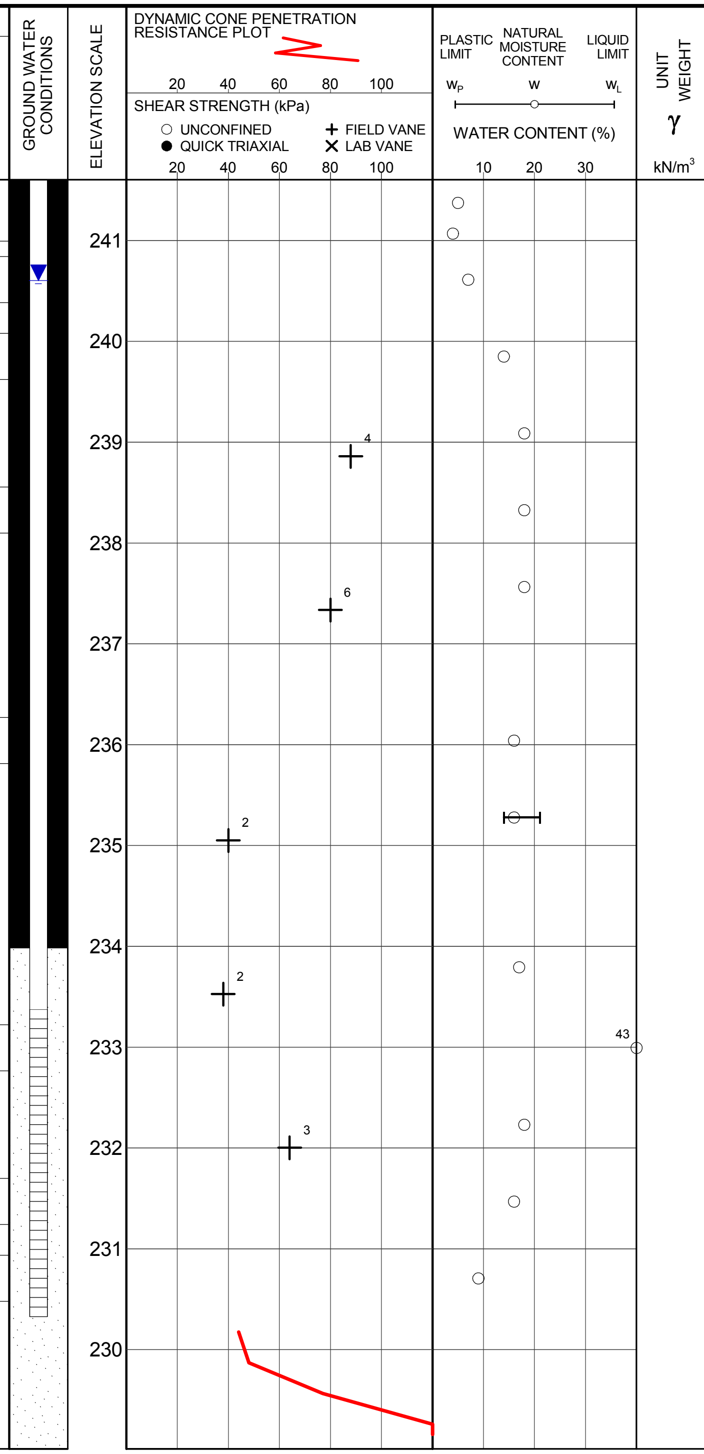
+3, ×3: Numbers refer to Sensitivity   ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CD4

METRIC

G.W.P. 5233-06-00	LOCATION Coords: E:372373.2 N:5492800		ORIGINATED BY PB
DIST -	HWY Hwy 11	BOREHOLE TYPE HOLLOW STEM AUGERS	COMPILED BY DB
DATUM GEODETIC	DATE 10.6.11		CHECKED BY HA

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.6	**GROUND SURFACE**				
	450mm FILL, sand and gravel, trace silt, compact, brown, damp	1A	SS	19	
241.1 / 0.5	**FILL**, sand, trace silt, trace gravel; compact, brown, damp	1B			
		2	SS	12	
240.2 / 1.4	**SAND AND SILT**, trace to some clay, trace gravel, loose, brown, wet	3	SS	7	
		4	AS		
238.7 / 2.9	Occasional peat inclusions	5	SS	10	
237.9 / 3.7	**CLAYEY SILT to SILTY CLAY**, trace to some sand, trace gravel, firm to stiff, grey, moist to moist	6	AS		
		7	ST		
		8	SS	2	
		9	AS		1 17 59 23
		10	ST		
		11	AS		
		12	SS	4	
	...at 8.8m, sandy	13	AS		
231.8 / 9.8	**SAND and SILT**, trace clay, trace gravel, compact, grey, moist (GLACIAL TILL)	14	SS	19	
		15	SS	27	
229.0 / 12.6					



END OF BOREHOLE

Borehole was dry upon completion of drilling.

25mm piezometer installed.

Dynamic cone penetration test conducted.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
November 7, 2011	1.1	240.5
November 8, 2011	1.2	240.4
December 12, 2011	0.7	240.9
April 26, 2012	1.0	240.6

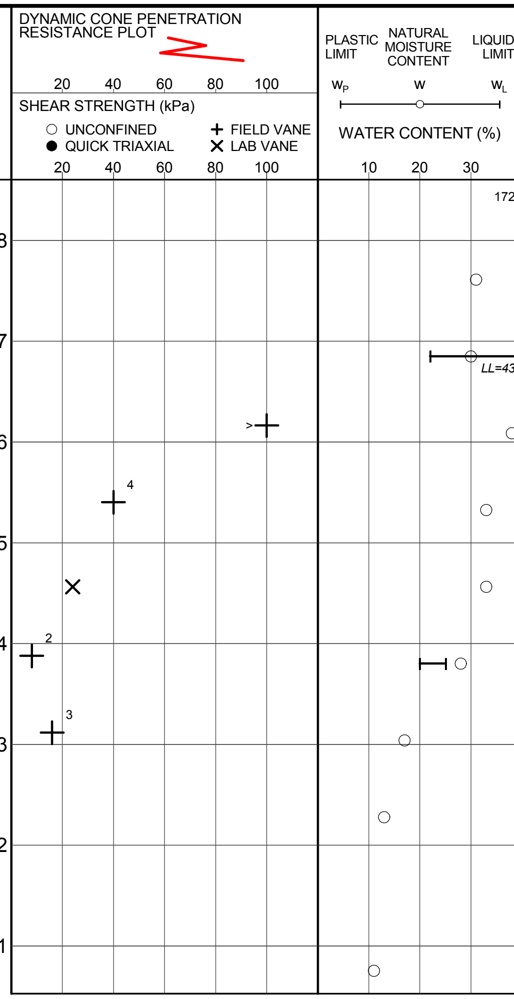
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No CD5

METRIC

G.W.P. 5233-06-00	LOCATION Coords: E:372276.6 N:5492820.9		ORIGINATED BY PB
DIST -	HWY Hwy 11	BOREHOLE TYPE HOLLOW STEM AUGERS	COMPILED BY DB
DATUM GEODETIC	DATE 10.24.11		CHECKED BY HA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
238.6	**GROUND SURFACE**					
	PEAT, black, wet	1	SS	3		
237.9 / 0.7	**CLAYEY SILT to SILT AND CLAY**, trace sand, very soft to firm, grey, moist	2	SS	3		
		3	SS	7		0 2 27 71
		4	AS			
		5	AS			
		6	ST		17.4	0 1 76 23
		7	AS			0 4 68 28
233.1 / 5.5	**SANDY SILT**, trace clay, trace gravel, occasional cobbles and boulders, very dense, grey, moist (GLACIAL TILL)	8	AS			
		9	SS	65		
230.5 / 8.1		10	SS	102		



END OF BOREHOLE
Auger refusal

Wet cave to 5.5m below grade upon completion of drilling.

25mm piezometer installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
November 7, 2011	0.1	238.5
November 8, 2011	0.0	238.6
December 12, 2011	-0.8	239.4
April 26, 2012	0.0	238.6

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

Ministry of Transportation Ontario

Terraprobe

Foundation Design

RECORD OF BOREHOLE No CD6

METRIC

G.W.P. 5233-06-00	LOCATION Coords: E:372367.8 N:5492786.3		ORIGINATED BY PB
DIST -	HWY Hwy 11	BOREHOLE TYPE HOLLOW STEM AUGERS	COMPILED BY DB
DATUM GEODETIC	DATE 10.24.11		CHECKED BY HA

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

ELEV DEPTH (m)	DESCRIPTION
238.7	**GROUND SURFACE**
	PEAT, black, wet
237.6 / 1.1	Trace rootlets
237.3 / 1.4	**CLAYEY SILT to SILTY CLAY**, trace to some sand, trace gravel, firm, grey, moist
232.8 / 5.9	**SANDY SILT**, trace to some clay, trace to some gravel, dense to very dense, grey, moist to wet (GLACIAL TILL)
	...at 9.1m, frequent sand interlayers
229.1 / 9.6	**END OF BOREHOLE**

NUMBER	TYPE	SPT 'N' VALUE	GR SA SI CL
1	SS	1	
2	SS	2	
3	SS	3	
4	AS		0 2 30 68
5	SS	5	
6	AS		
7	SS	4	
8	SS	0	1 12 60 27
9	SS	40	
10	SS	31	6 31 53 10
11	SS	74	

Elevation scale: 238, 237, 236, 235, 234, 233, 232, 231, 230

194

4

END OF BOREHOLE
Auger refusal

Unstabilized water level measured at 2.7m and borehole caved to 6.7m below grade upon completion of drilling

25mm piezometer installed.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
November 7, 2011	0.1	238.6
November 8, 2011	0.2	238.5
December 12, 2011	0.0	238.7
April 26, 2012	0.0	238.7

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

RECORD OF BOREHOLE No C-1

METRIC

W.P. 5233-06-00	LOCATION Coords: N:5492821.8 E:372318.7		ORIGINATED BY PK
DIST	HWY Hwy 11	BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring	COMPILED BY DB
DATUM Geodetic	DATE 7.28.10 - 7.29.10		CHECKED BY RA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
241.6	Ground Surface
241.5	150mm ASPHALT
241.3 / 0.3	170mm FILL - Sand and Gravel, trace silt, inferred compact, brown, damp
	FILL - Sand, trace silt, trace gravel, loose to compact, brown, damp to moist
239.5 / 2.1	SANDY SILT some clay, loose, brown, wet
237.9 / 3.7	SILTY CLAY trace to some sand, trace gravel, occasional silt seams, soft to firm, grey, moist
232.6 / 9.0	SAND AND SILT trace clay, trace gravel, occasional cobbles and boulders, very dense, grey, damp to moist (GLACIAL TILL)

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	16	5 87 (8)
2	SS	8	
3	SS	7	
4	SS	6	
5	SS	4	0 23 67 10
6	SS	7	
7	SS	3	0 4 56 40
8	TW	PH	
9	SS	2	1 15 58 26
10	SS	70	commence casing and washboring
11	SS	100/ 13cm	
12	SS	100/ 8cm	
13	SS	100/ 10cm	6 42 45 7

ELEVATION SCALE: 241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 3.7, 3.4, 2.0

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

WATER CONTENT (%): 10 20 30

UNIT WEIGHT γ kN/m^3

GROUND WATER CONDITIONS

STRAT PLOT

Continued Next Page

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-1

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492821.8 E:372318.7 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 7.28.10 - 7.29.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
						Jul.28 ---------- Jul.29
		14	SS	100/ 15cm	226	
		15	SS	100/ 15cm	225	
224.0 17.6	CLAYEY SILT some sand to sandy, trace gravel, frequent cobbles and boulders below 21.8m, hard, grey, damp (GLACIAL TILL)				224	
		16	SS	100/ 13cm	223	1 20 64 15
					222	
					221	
		17	SS	100	220	7 36 46 11
	---- brown				219	
					218	
					217	
		18	SS	158/ 28cm	216	Commence NQ Coring
		19	WS	-	215	6 29 44 21
		20	WS	-	214	
213.6 28.0	BEDROCK - PHYLLITE unweathered, sub-vertical foliations, grey, high strength.	1	RUN	NQ	213	RUN#1 TCR=96% SCR=77% RQD=61%
		2	RUN	NQ	212	RUN#2 TCR=91% SCR=91% RQD=88%

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m^3

Continued Next Page

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No C-1

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492821.8 E:372318.7 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 7.28.10 - 7.29.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
			3	RUN	NQ		211				RUN#3 TCR=91% SCR=80% RQD=80%
210.3 31.3	End of Borehole										

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	0.2	241.4
Aug.10.10	0.9	240.7
Sep.03.10	0.9	240.7
Apr. 26, 12	1.0	240.6

Borehole was open to 30.2m and filled with drill water on completion of drilling.

Continous soil core sample collected from 25.4m to 28.0m.

$+^3, \times^3$: 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

CORE LOG

Terraprobe

Project: Crow Creek Bridge Replacement	Orientation: Vertical	Ground Elevation: 241.6m	Datum: Geodetic	Borehole No. C-1
Location: Hwy 11, Township of McCrea, Ontario	Date Started: July 29, 2010	Completed: July 29, 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: No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	UNCONFINED COMPRESSIVE STRENGTH MPa	UNIT WEIGHT (KN/m³)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
214.6	27.0		Overburden, see Borehole Log C-1															
213.6	28.0		<u>BEDROCK - PHYLLITE</u> Unweathered, sub-vertical foliations, grey, high strength.	1	C C C	F F F	VC C M	SP SP SP	O O O	0 to 1				#1 TCR 96 SCR 77	61	NQ		
212.6	29.0		Rubbilized zones from 28.0m to 28.2m and 30.7m to 30.8m. Highly fractured from 28.2m to 28.7m.	2	CC	DV	M	SP	O	0 to 1				#2 TCR 91 SCR 91	88	NQ		
211.6	30.0																	
210.6	31.0			2	CC CC CC	FV FV FV	M VC M	SP SP SP	O O O	0 to 1				#3 TCR 91 SCR 80	80	NQ		
210.3	31.3		End of Core Log															

Remarks:

LEGEND:
Bedrock

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RECORD OF BOREHOLE No C-2

METRIC

W.P. 5233-06-00	LOCATION Coords: N:5492819.4 E:372351.3		ORIGINATED BY PK
DIST	HWY Hwy 11	BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring	COMPILED BY DB
DATUM Geodetic	DATE 7.27.10		CHECKED BY RA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
241.6	Ground Surface
241.5	150mm ASPHALT
241.3 / 0.3	150mm FILL - Sand and Gravel, inferred compact, brown, damp
	FILL - Sand, trace silt, loose to compact, brown, dry ---- wet
239.5 / 2.1	SAND AND SILT some clay, loose, brown, wet
238.7 / 2.9	SILTY CLAY trace sand, occasional gravel inclusions, firm to stiff, grey, damp to moist
232.9 / 8.7	SAND AND SILT trace to some gravel, trace clay, occasional cobbles, compact to very dense, grey, damp to moist (GLACIAL TILL)
226.9 / 14.7	

Continued Next Page

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	29	
2	SS	9	0 94 (6)
3	SS	8	
4	SS	4	0 44 44 12
5	SS	14	0 3 61 36
6	SS	11	
7	SS	12	0 1 75 24
8	SS	5	3 7 55 35
9	TW	PH	
10	SS	6	
11	SS	24	17 44 35 4
12	SS	117	
13	SS	44	
14	SS	156	9 50 37 4

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (241, 240, 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³.

Field vane sensitivity values: 2.3, 1.6, 1.8

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

RECORD OF BOREHOLE No C-2

 METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492819.4 E:372351.3 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 7.27.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT sandy, trace gravel, some cobbles, hard, grey, damp (GLACIAL TILL) *(continued)*	15	SS	85	226	9 32 44 15; commence casing and washboring
					225	
					224	
		16	SS	100/ 13cm	223	
					222	
					221	
		17	SS	125	220	2 30 45 23
219.1 / 22.5	BEDROCK - GRANITOID unweathered, massive, bluish white, high strength.	1	RUN	NQ	219	RUN#1 TCR=72% SCR=61% RQD=25%
		2	RUN	NQ	218	RUN#2 TCR=100% SCR=100% RQD=90%
		3	RUN	NQ	217, 216	RUN#3 TCR=91% SCR=89% RQD=62%
215.7 / 25.9	End of Borehole					

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	0.7	240.9
Aug.10.10	0.7	240.9
Sep.03.10	0.7	240.9
Apr. 26, 12	0.8	240.8

Borehole was open to full depth and filled with drill water on completion of drilling.

Unable to push vane beyond 9.0m.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30; UNIT WEIGHT γ kN/m³; STRAT PLOT; GROUND WATER CONDITIONS

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

CORE LOG

Terraprobe

Project	Crow Creek Bridge Replacement	Orientation: Vertical	Ground Elevation: 241.6m	Datum: Geodetic	Borehole No.: C–2
Location	Hwy 11, Township of McCrea, Ontario	Date Started: July 27, 2010	Completed: July 27, 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: No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	UNCONFINED COMPRESSIVE STRENGTH MPa	UNIT WEIGHT (KN/m³)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
219.6	22.0		Overburden, see Borehole Log C-2															
219.1	22.5			2	CC	FV	C	SP	O	0 to 1				#1 TCR 72 SCR 61	25	NQ		
218.6	23.0		BEDROCK – GRANITOID Unweathered, massive, bluish white, high strength.	2	CC	FV	C	SP	T	0 to 1				#2 TCR 100 SCR 100	90	NQ		
217.6	24.0																	
					CC	FV	C	SP	NC	10								
216.6	25.0			2	CC	FV	C	SP	O	0 to 1				#3 TCR 91 SCR 89	62	NQ		
					CC	FV	C	SP	NC	5								
215.7	25.9		End of Core Log															

Remarks:

LEGEND:
Bedrock

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Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-3

METRIC

W.P.	5233-06-00	LOCATION	Coords: N:5492806.5 E:372316.7
DIST		HWY	Hwy 11
BOREHOLE TYPE	Hollow Stem Augers / Casing and Washboring / NQ Coring		
DATUM	Geodetic	DATE	8.4.10 - 8.5.10
ORIGINATED BY	PK	COMPILED BY	DB
CHECKED BY	RA		

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.8	Ground Surface				
0.0 / 239.5	300mm TOPSOIL	1	SS	8	5 8 41 46
0.3	FILL - Silty Clay and Peat, trace sand, trace gravel, firm, dark brown / black, moist	2	SS	6	
		3	SS	5	
237.7 / 2.1	SILTY CLAY trace to some sand, trace gravel, firm to stiff, brown, moist	4	SS	10	
		5	SS	5	1 7 64 28
		6	SS	5	
		7	TW	PH	
		8	SS	1	1 14 62 23
232.7 / 7.1	SANDY SILT trace clay, trace gravel, occasional cobbles and boulders, very dense, brown, damp to moist (GLACIAL TILL)	9	SS	51	5 33 55 7
		10	SS	78	commence casing and washboring
		11	SS	100/ 13cm	
		12	SS	100/ 13cm	
		13	SS	100/ 10cm	
225.2 / 14.6					

Elevation scale: 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226, 225

Plot annotations: 64; 76; 3.1; 2.5

Continued Next Page

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No C-3

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492806.5 E:372316.7 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 8.4.10 - 8.5.10 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL	REMARKS
	CLAYEY SILT sandy, trace to some gravel, frequent cobbles and boulders below 19.2m, hard, brown, damp (GLACIAL TILL)	14	SS	151/ 23cm		**	19	27	40	14	
					224						
					223						
					222						
		15	SS	180/ 23cm			2	29	56	13	
					221						Aug.04 / Aug.05
					220						
		16	WS	-							
					219						
					218						
		17	WS	-			12	25	43	20	
					217						
		18	WS	-							
					216						
					215						
		19	WS	-							
					214						
		20	SS	149			4	29	43	24	
					213						
					212						RUN#1 TCR=46% SCR=23% RQD=0%
211.6 28.2	BEDROCK - PHYLLITE unweathered below 28.9m, sub-vertical foliations, grey, medium to high strength.	1	RUN	NQ							
					211						RUN#2 TCR=90% SCR=79% RQD=29%
		2	RUN	NQ							
					210						

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE — 20 40 60 80 100
PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 10 20 30

Continued Next Page

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

RECORD OF BOREHOLE No C-3

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492806.5 E:372316.7 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 8.4.10 - 8.5.10 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT w / LIQUID LIMIT W_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
			3	RUN	NQ		209				RUN#3 TCR=97% SCR=84% RQD=74%
208.0 / 31.8	End of Borehole						208				

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	0.8(ag)*	240.6
Aug.10.10	1.0(ag)*	240.8
Sep.03.10	1.2(ag)*	241.0
Apr. 26, 12	1.2	238.6

*(ag) - above ground

Borehole filled with drill water on completion of drilling.

**Enough sample not available to perform Atterberg Limits Test.

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

CORE LOG

Terraprobe

Project	Orientation	Ground Elevation	Datum	Borehole No.
Crow Creek Bridge Replacement	Vertical	239.8m	Geodetic	C–3
Location	**Date Started**	**Completed**	**Logged By**	**Sheet**
Hwy 11, Township of McCrea, Ontario	August 5, 2010	August 5, 2010	A.W.	1 of 1
Client	**Drilling Agency**	**Drill Type**	**Core Barrel & Bit Design**	**Project No.**
MTO	Landcore Drilling	CME55	NQ	1–10–5076

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	Joint Characteristics: No. OF SETS	JOINT TYPE	ORIENTATION	SPACING	ROUGHNESS	FILLING	APERTURE	WEATHERING	STRENGTH	FRACTURE FREQUENCY	RUN NO. CORE RECOVERY %	R Q D %	CORE SIZE/CASING	MPa UNCONFINED COMPRESSIVE STRENGTH	UNIT WEIGHT (KN/m³)
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19
212.1	27.7		Overburden, see Borehole Log C–3															
211.8	28.0													#1 TCR 46 SCR 23	0	NQ		
211.6	28.2		BEDROCK – PHYLLITE	1	C	F	C	SP	SA	0 to 1								
			Unweathered below 28.9m, sub–vertical foliations, grey, medium to high strength.		CCC	FDV	VC	SP	SA									
210.8	29.0		Slightly to moderately weathered from 28.2m to 28.9m. Highly fractured from 28.7m to 28.9m.	3	CCC	FDV	C	SP	T	0 to 1				#2 TCR 90 SCR 79	29	NQ		
209.8	30.0																	
208.8	31.0			2	CC	DV	C	SP	T	0 to 1				#3 TCR 97 SCR 84	74	NQ		
208.0	31.8		End of Core Log															

Remarks:

LEGEND:
Bedrock

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RECORD OF BOREHOLE No C-4

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492800.2 E:372339.6 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 8.6.10 CHECKED BY RA

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
240.0	Ground Surface
239.8 / 0.0	200mm TOPSOIL
0.2	FILL - Silty Clay and Peat, trace sand, trace gravel, firm, dark brown, moist
238.6 / 1.4	SILTY CLAY trace to some sand, trace gravel, firm to stiff, brown, moist
232.9 / 7.1	SANDY SILT trace to some clay, trace to some gravel, occasional cobbles and boulders, dense to very dense, brown, damp to moist (GLACIAL TILL)
225.3 / 14.7	

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	7	
2	SS	5	
3	SS	8	
4	SS	9	1 2 66 31
5	TW	PH	
6	SS	5	
7	SS	4	
8	SS	4	0 10 64 26
9	SS	42	5 32 55 8 commence casing and washboring
10	SS	100/ 15cm	
11	SS	100/ 15cm	
12	SS	100/ 15cm	
13	SS	99	16 32 41 11

GROUND WATER CONDITIONS

ELEVATION SCALE: 239, 238, 237, 236, 235, 234, 233, 232, 231, 230, 229, 228, 227, 226

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values: 4.3, 3.6, 3.3

PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L

WATER CONTENT (%): 10 20 30 (42)

UNIT WEIGHT γ kN/m³

Continued Next Page

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

RECORD OF BOREHOLE No C-4

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492800.2 E:372339.6 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 8.6.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT some sand to sandy, trace gravel, occasional cobbles, hard, brown, damp to moist (GLACIAL TILL)	14	SS	100/ 13cm			5 16 62 17
					224		
					223		
					222		
		15	SS	100/ 13cm			
					221		
					220		
					219		
		16	SS	100/ 15cm			2 35 44 19
					218		
					217		
					216		
		17	WS	-			
		2	RUN	NQ	215		RUN#2 TCR=33% SCR=7% RQD=7%
214.6 / 25.4	BEDROCK - PHYLLITE unweathered below 29.0m, sub-vertical foliations, grey, very low to high strength.						
		3	RUN	NQ	214		RUN#3 TCR=59% SCR=28% RQD=9%
					213		
		4	RUN	NQ	212		RUN#4 TCR=63% SCR=34% RQD=23%
					211		
		5	RUN	NQ			RUN#5 TCR=98% SCR=92% RQD=65%

Continued Next Page

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

Ministry of Transportation Ontario

RECORD OF BOREHOLE No C-4

METRIC

W.P. 5233-06-00 LOCATION Coords: N:5492800.2 E:372339.6 ORIGINATED BY PK

DIST HWY Hwy 11 BOREHOLE TYPE Hollow Stem Augers / Casing and Washboring / NQ Coring COMPILED BY DB

DATUM Geodetic DATE 8.6.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 / SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT w_P NATURAL MOISTURE CONTENT w LIQUID LIMIT w_L / WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
209.5 30.5	End of Borehole 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 1.1(ag)* 241.1 Sep.03.10 1.6(ag)* 241.6 *(ag) - above ground Borehole filled with drill water on completion of drilling. Unable to push vane beyond 7.5m.										

ON_MOT OLD 1-10-5076 CROW MONTCALM BRIDGE RPL - ORIGINAL.GPJ ON_MOT.GDT 5/22/12

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

CORE LOG

Terraprobe

Project: Crow Creek Bridge Replacement	Orientation: Vertical	Ground Elevation: 240.0m	Datum: Geodetic	Borehole No. C-4
Location: Hwy 11, Township of McCrea, Ontario	Date Started: August 6, 2010	Completed: August 6, 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

1 ELEVATION (m)	2 DEPTH (m)	3 SYMBOL	4 GENERAL DESCRIPTION	5 No. OF SETS	6 JOINT TYPE	7 ORIENTATION	8 SPACING	9 ROUGHNESS	10 FILLING	11 APERTURE	12 WEATHERING	13 STRENGTH	14 FRACTURE FREQUENCY	15 RUN NO. CORE RECOVERY %	16 R Q D %	17 CORE SIZE/CASING	18 UNCONFINED COMPRESSIVE STRENGTH MPa	19 UNIT WEIGHT (KN/m³)
216.0	24.0																	
215.6	24.4																	
215.0	25.0		Overburden, see Borehole Log C-4											#2 TCR 33 SCR 7	7	NQ		
214.6	25.4	Bedrock	BEDROCK – PHYLLITE	2	CC	FV	VC	SP	SA	0 to 1								
214.0	26.0	Bedrock	Unweathered below 29.0m, sub-vertical foliations, grey, very low to high strength.		CC	FV	VC	SU	SI					#3 TCR 59 SCR 28	9	NQ		
213.0	27.0	Bedrock	Completely weathered from 26.2m to 27.4m. Highly weathered from 25.9m to 26.2m and 27.6m to 27.9m.	2	CC	FV	C	SU	SI	0 to 3								
212.0	28.0	Bedrock	Slightly to moderately weathered from 25.4m to 25.9m, 27.4m to 27.6m and 27.9m to 29.0m. Highly fractured / rubbilized from 25.4m to 26.2m.	2	CC	FV	C	SU	SI	0 to 1				#4 TCR 63 SCR 34	23	NQ		
		Bedrock			CC	FV	M	SU	SI									
211.0	29.0	Bedrock																
210.0	30.0	Bedrock		2	CC	FV	M	SP	T	0 to 1				#5 TCR 98 SCR 92	65	NQ		
209.5	30.5		End of Core Log															

Remarks:

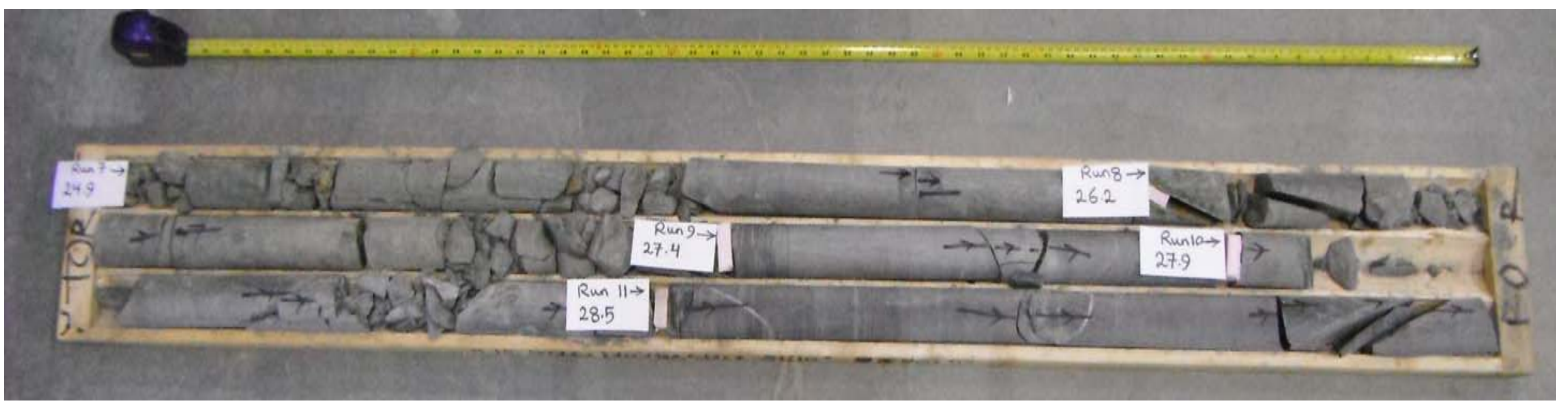
LEGEND:

Bedrock

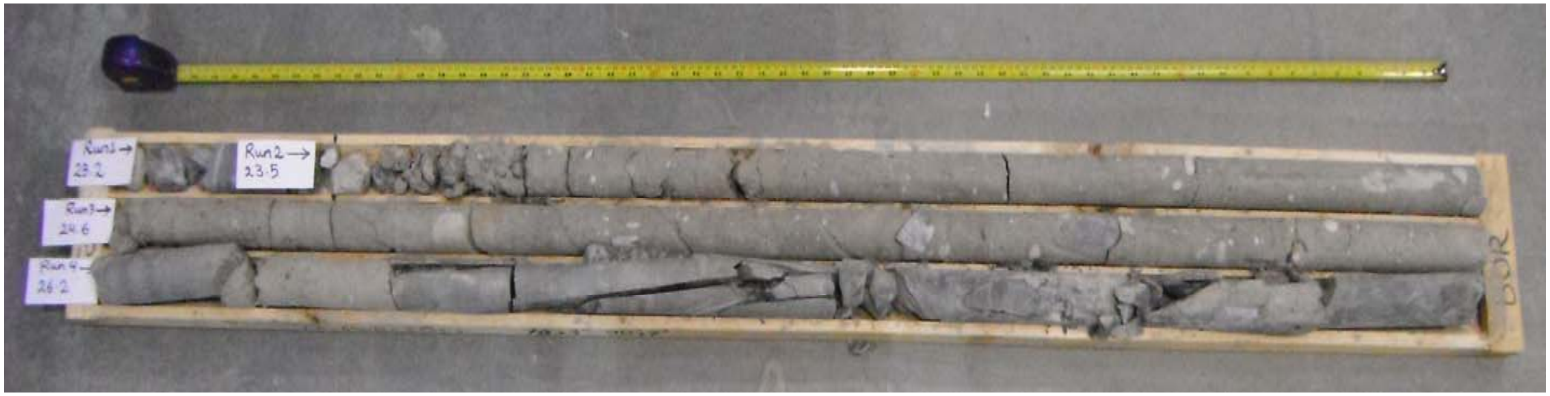
Bedrock Core Sample
Borehole: C1
Runs: 1, 2 & 3
Depth: 28.0m – 31.3m

Bedrock Core Sample
Borehole: C2
Runs: 1, 2 & 3
Depth: 22.2m – 25.9m

Soil/Bedrock Core Sample
Borehole: CD1; Runs: 1 to 6; Depth: 19.2m – 24.9m



Bedrock Core Sample
Borehole: CD1; Runs: 7 to 11; Depth: 24.9m – 29.4m



Soil/Bedrock Core Sample
Borehole: CD2; Runs: 1 to 4; Depth: 23.2m – 27.6m

Bedrock Core Sample
Borehole: CD2; Runs: 5 to 7; Depth: 27.6m – 30.9m

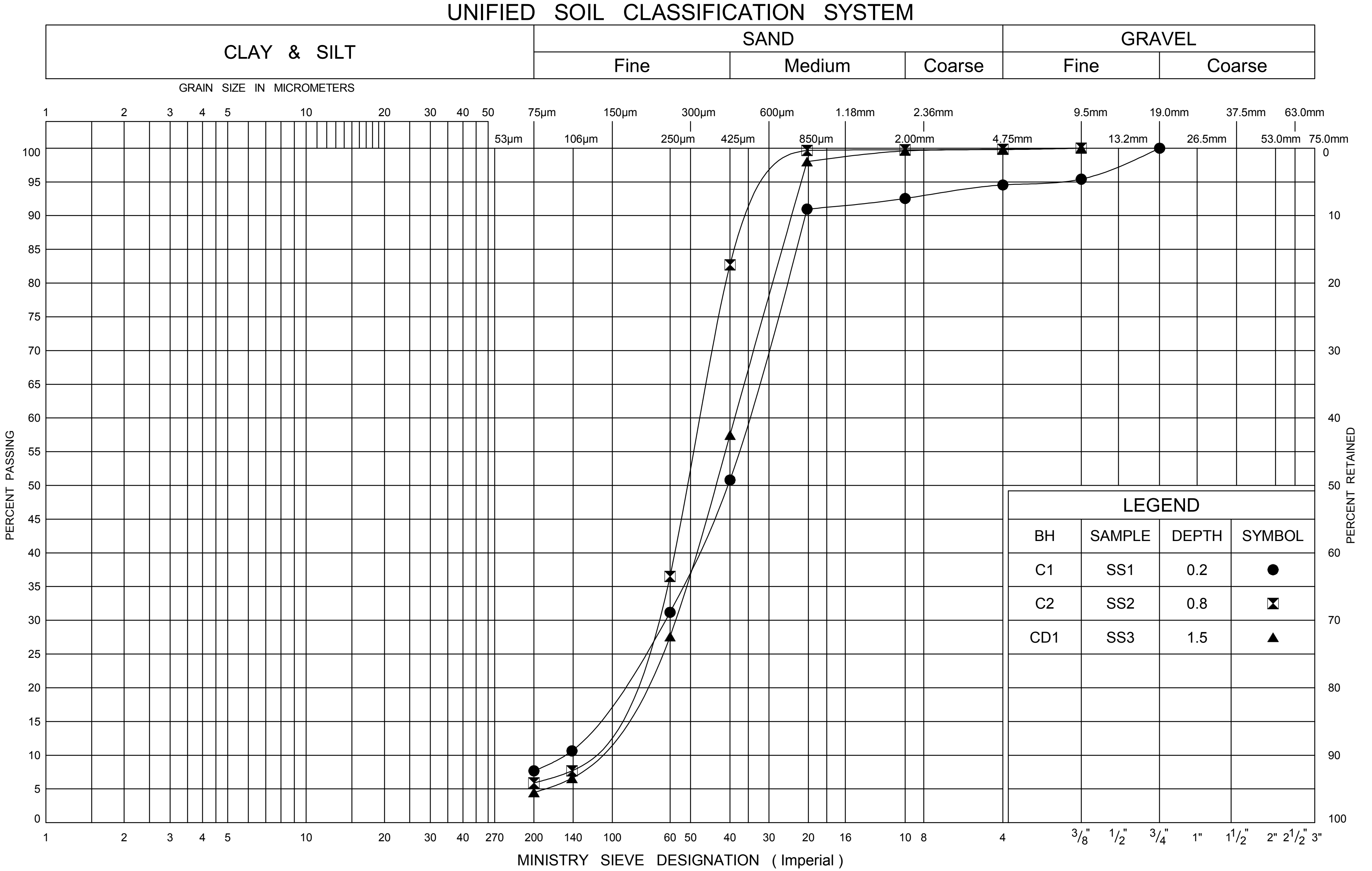
Bedrock Core Sample
Borehole: C3
Runs: 1, 2 & 3
Depth: 27.7m – 31.8m

Bedrock Core Sample
Borehole: C4
Runs: 1, 2, 3, 4 & 5
Depth: 23.9m – 30.5m

APPENDIX B

TERRAPROBE INC.

78 12 M



BH	SAMPLE	DEPTH	SYMBOL
C1	SS1	0.2	●
C2	SS2	0.8	⧗
CD1	SS3	1.5	▲

Ministry of Transportation

Ontario

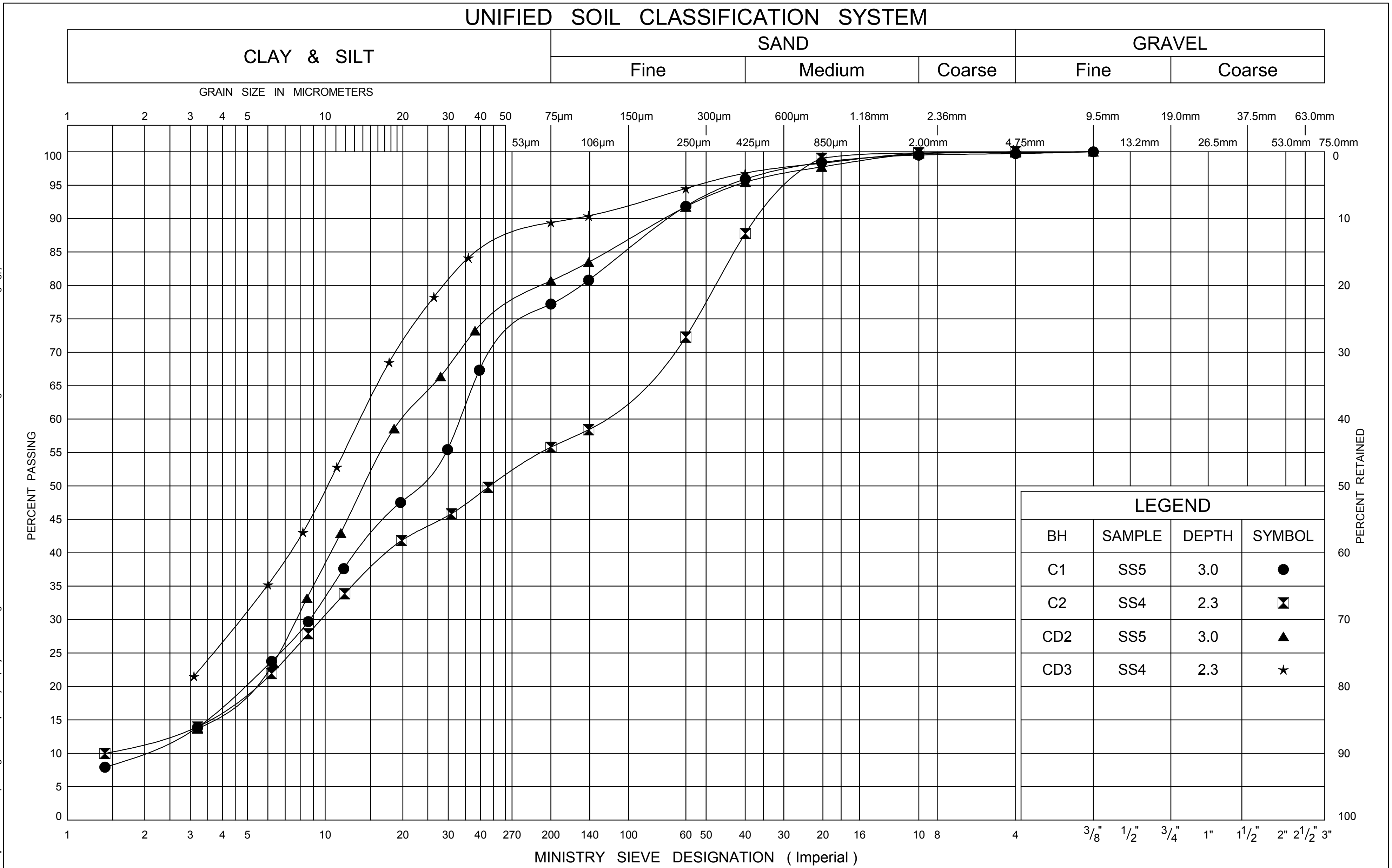
GRAIN SIZE DISTRIBUTION

FILL - SAND

FIG No B1 -1

G W P 5233-06-00

Crow Bridge Replacement



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
C1	SS5	3.0	●
C2	SS4	2.3	⧗
CD2	SS5	3.0	▲
CD3	SS4	2.3	★

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SAND AND SILT TO SILT

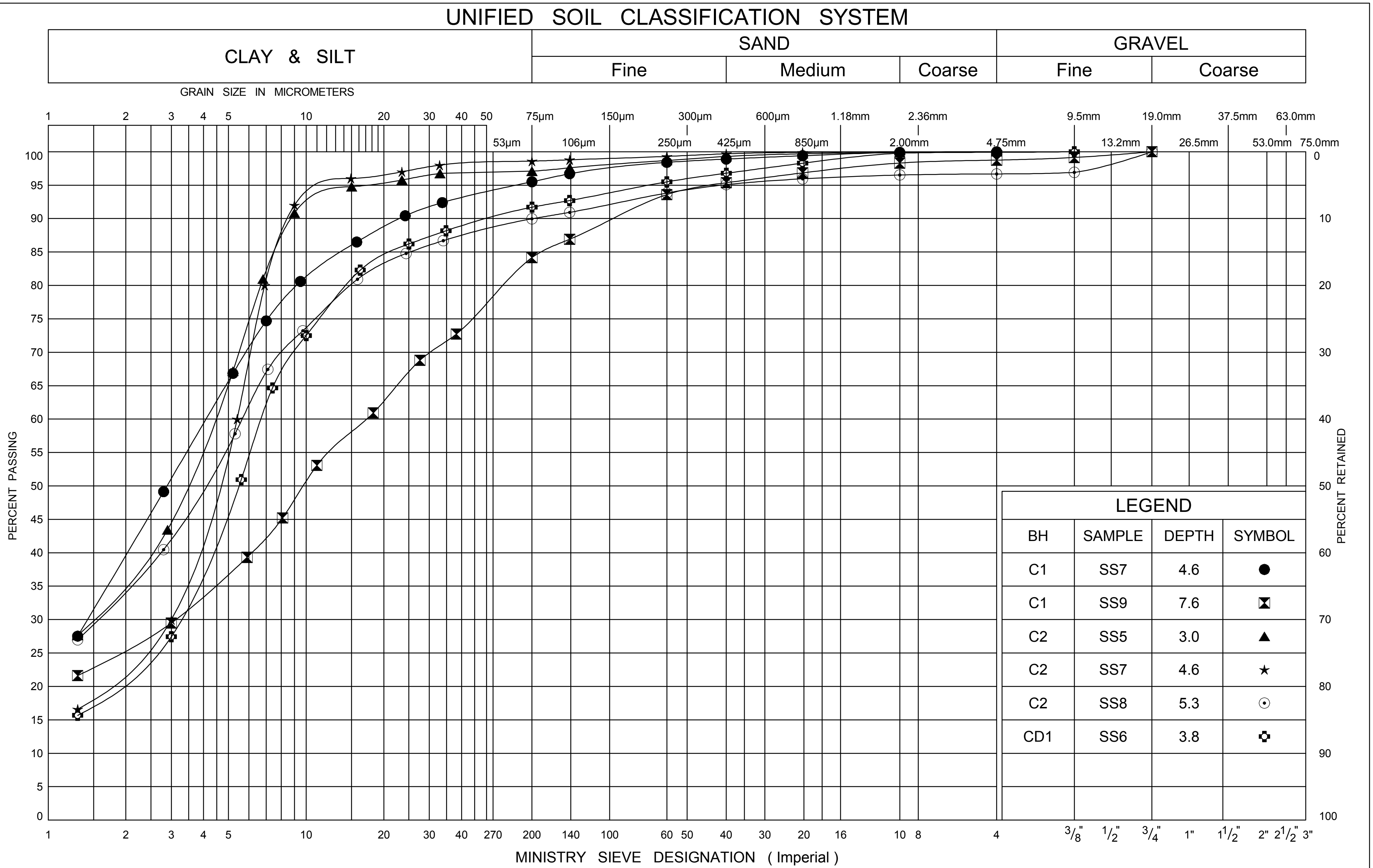
FIG No B1-2

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
C1	SS7	4.6	●
C1	SS9	7.6	⧗
C2	SS5	3.0	▲
C2	SS7	4.6	★
C2	SS8	5.3	⊙
CD1	SS6	3.8	✪

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY

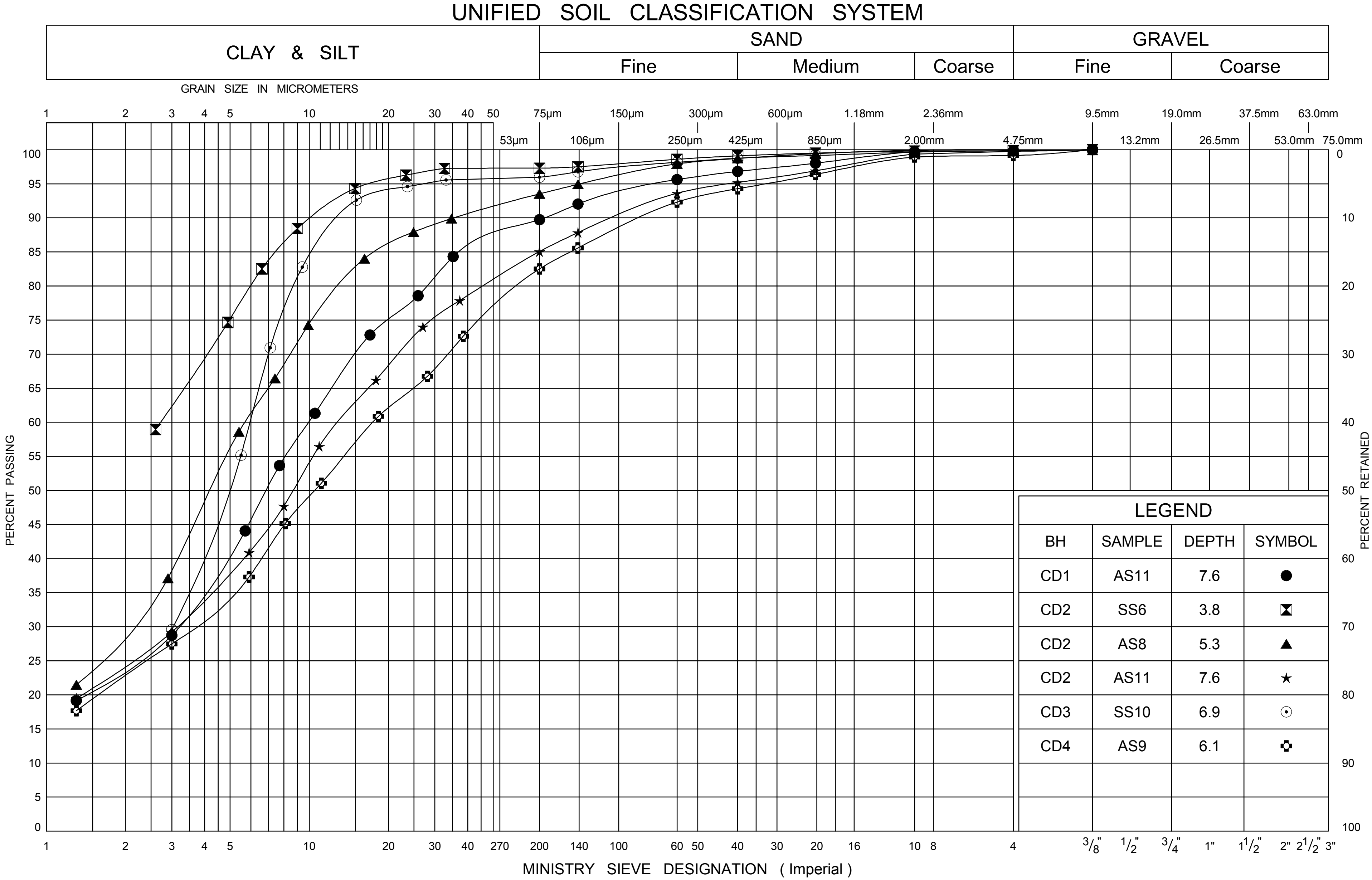
FIG No B1-3

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
CD1	AS11	7.6	●
CD2	SS6	3.8	⧗
CD2	AS8	5.3	▲
CD2	AS11	7.6	★
CD3	SS10	6.9	⊙
CD4	AS9	6.1	✪

Ministry of Transportation
Ontario

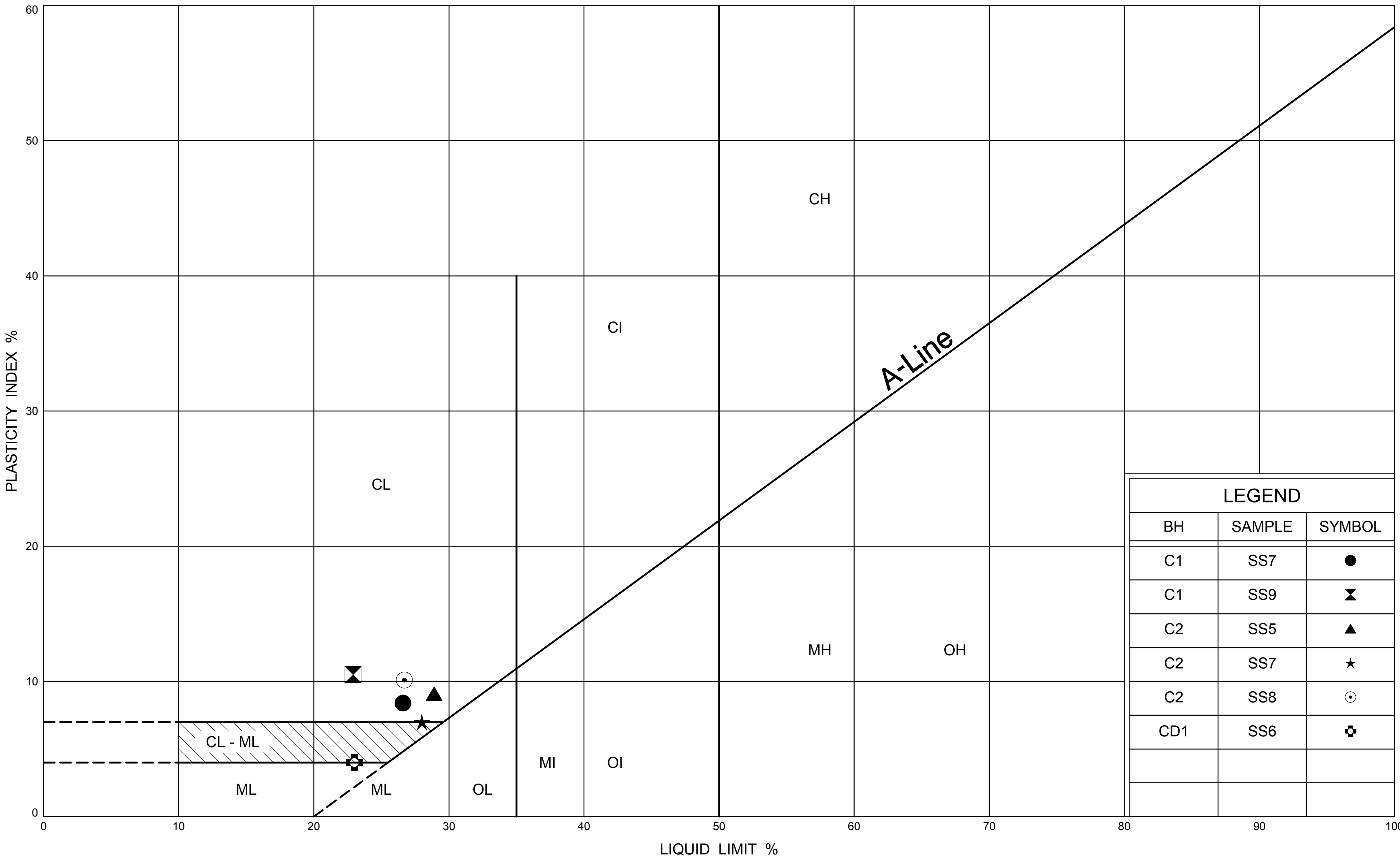
GRAIN SIZE DISTRIBUTION
CLAYEY SILT TO SILTY CLAY

FIG No B1-4

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe



LEGEND

BH	SAMPLE	SYMBOL
C1	SS7	●
C1	SS9	⧗
C2	SS5	▲
C2	SS7	★
C2	SS8	☉
CD1	SS6	✣

Ministry of Transportation
Ontario

PLASTICITY CHART

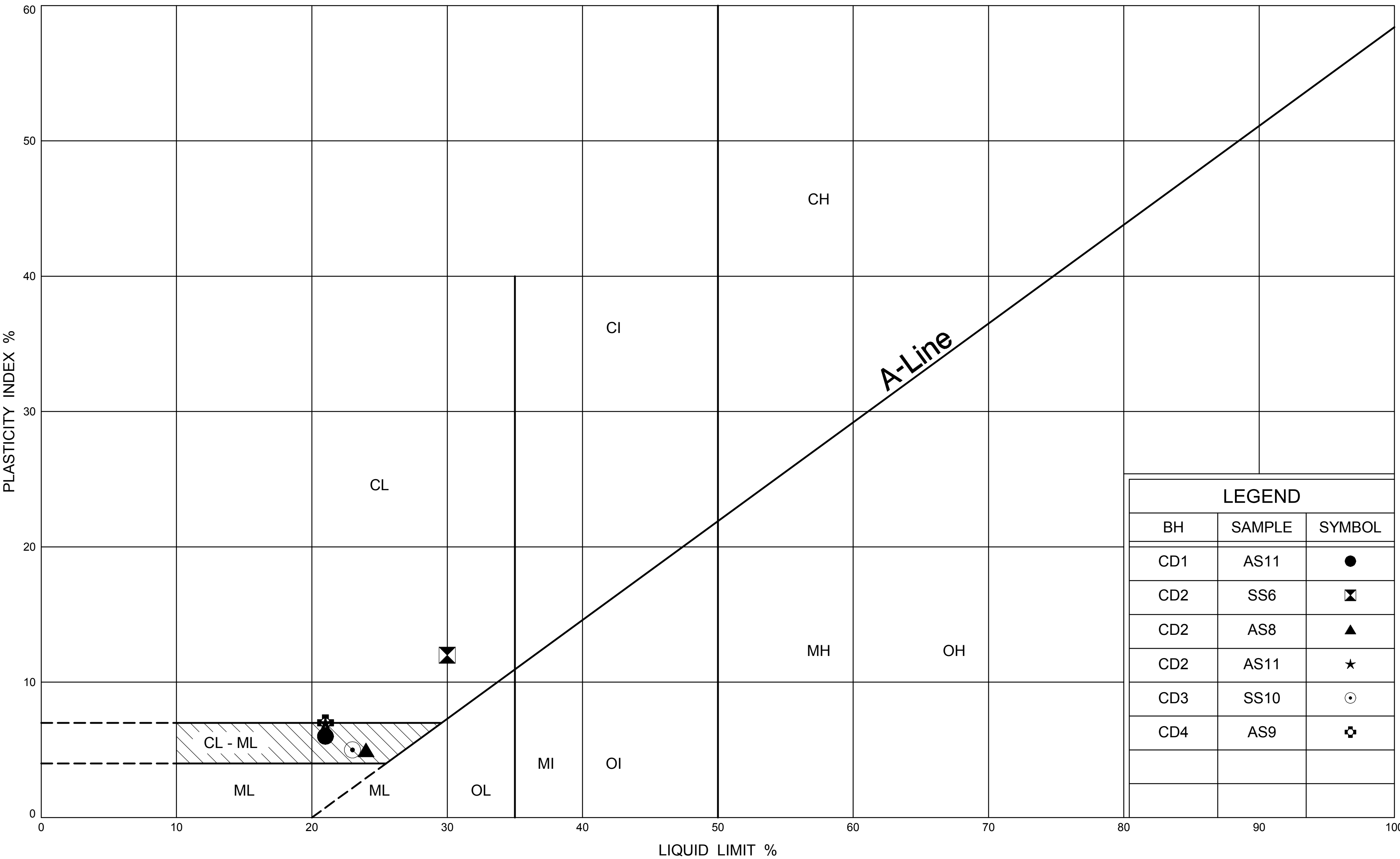
CLAYEY SILT TO SILTY CLAY

FIG No B1-5

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe

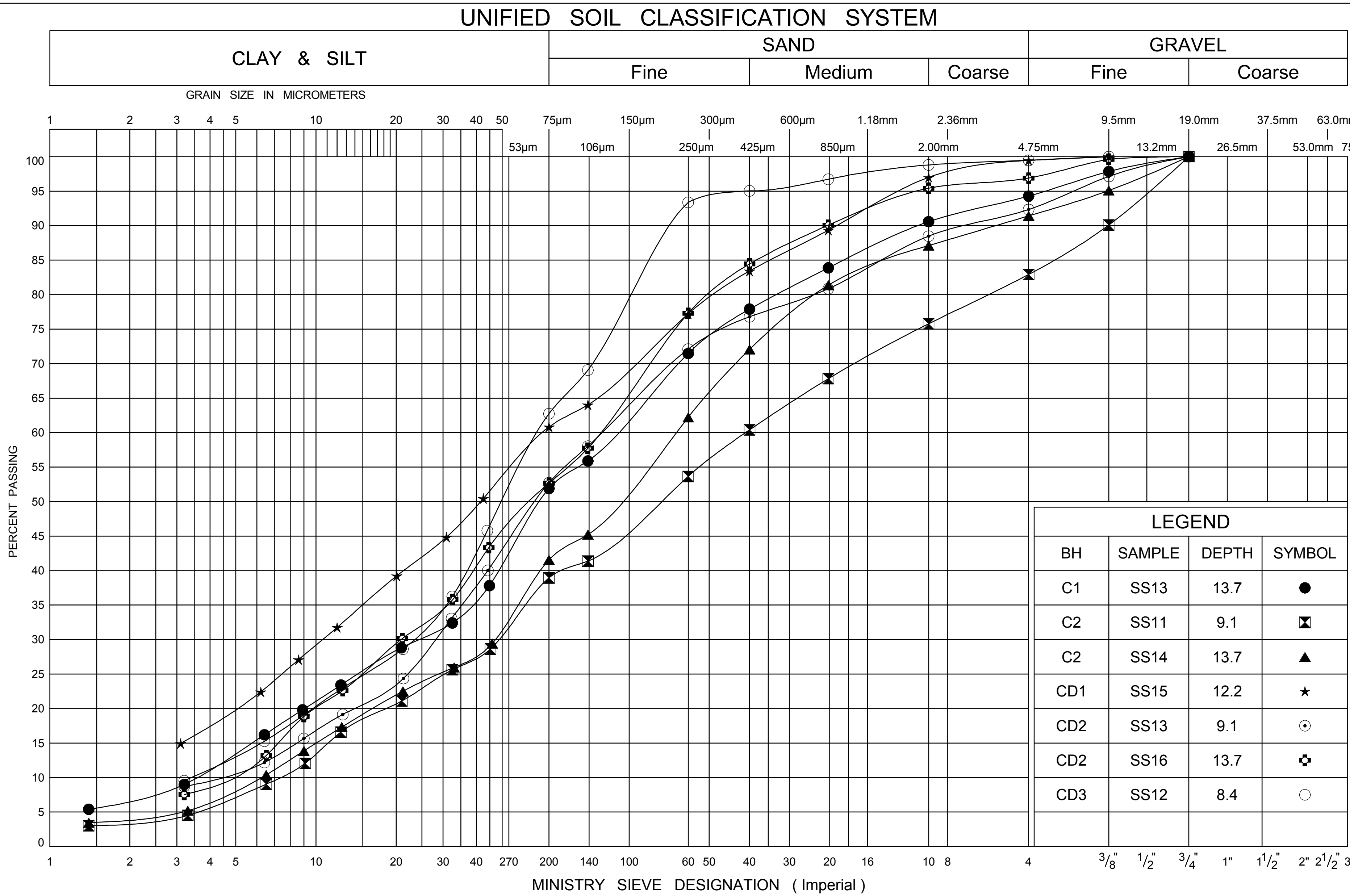


LEGEND		
BH	SAMPLE	SYMBOL
CD1	AS11	●
CD2	SS6	⧗
CD2	AS8	▲
CD2	AS11	★
CD3	SS10	⊙
CD4	AS9	✿

Ministry of Transportation
Ontario

PLASTICITY CHART
CLAYEY SILT TO SILTY CLAY

FIG No B1-6
G W P 5233-06-00
Crow Bridge Replacement



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
C1	SS13	13.7	●
C2	SS11	9.1	⧗
C2	SS14	13.7	▲
CD1	SS15	12.2	★
CD2	SS13	9.1	⊙
CD2	SS16	13.7	✪
CD3	SS12	8.4	○

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SAND AND SILT TILL

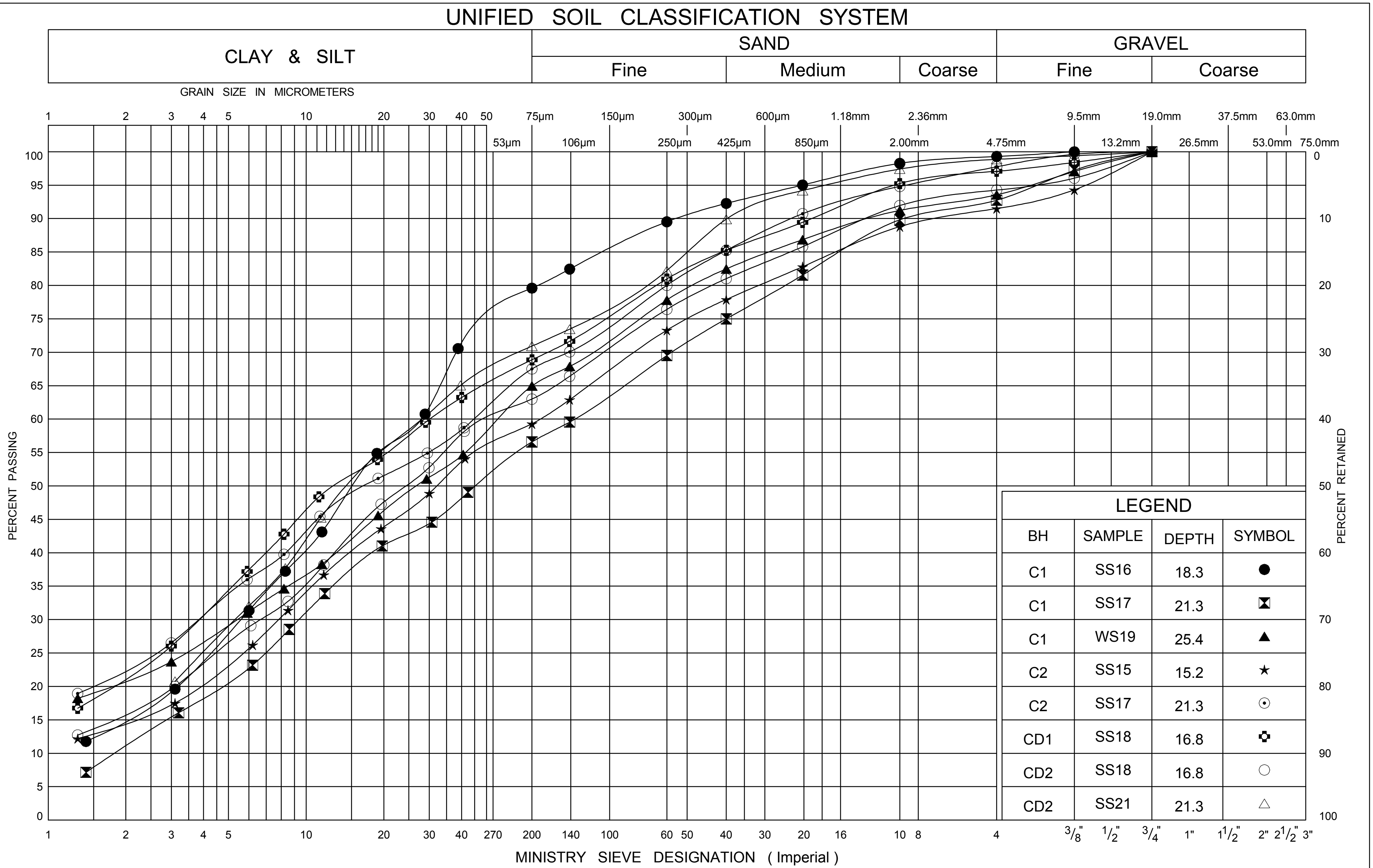
FIG No B1-7

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
C1	SS16	18.3	●
C1	SS17	21.3	⧗
C1	WS19	25.4	▲
C2	SS15	15.2	★
C2	SS17	21.3	⊙
CD1	SS18	16.8	✪
CD2	SS18	16.8	○
CD2	SS21	21.3	△

Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

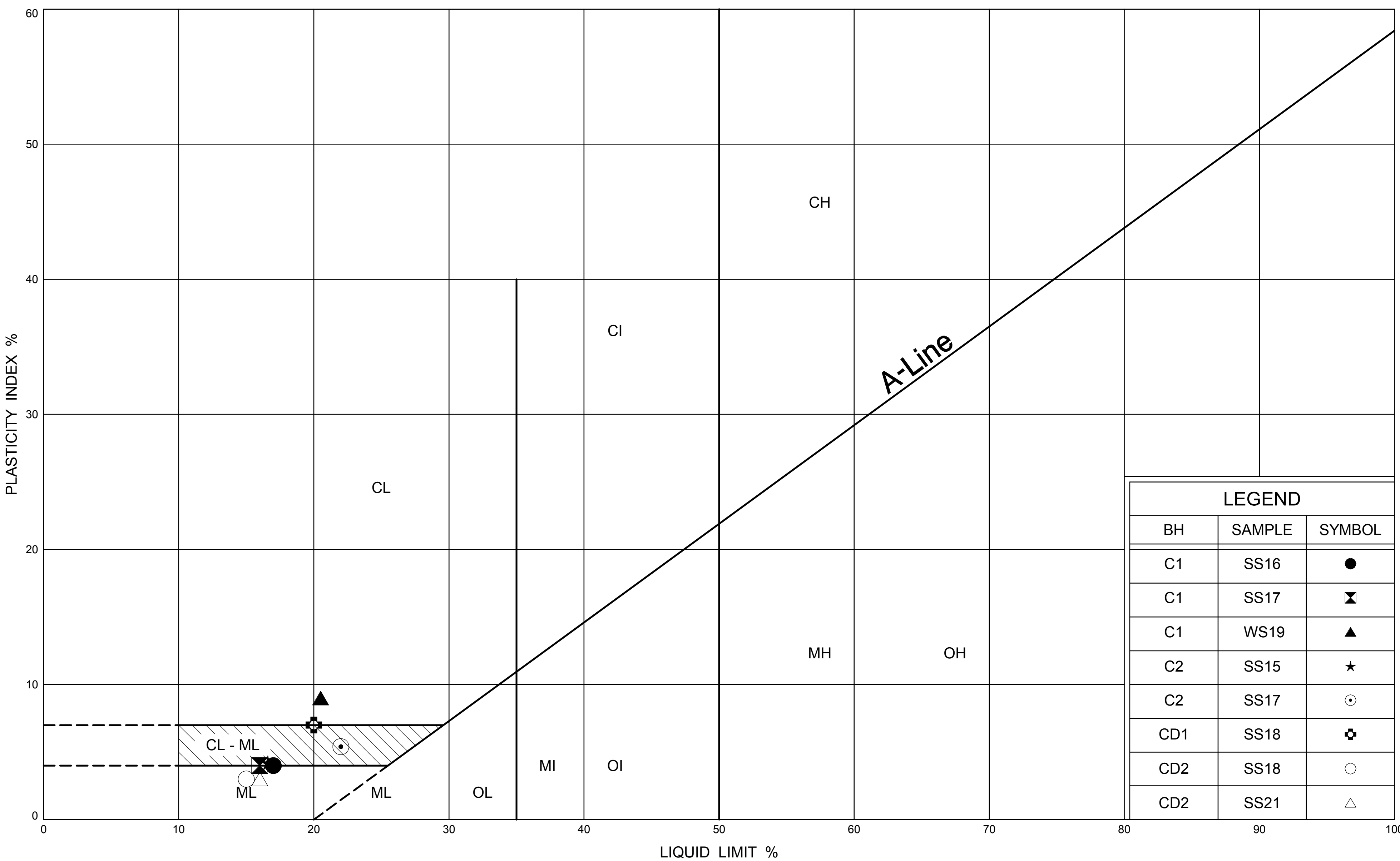
CLAYEY SILT TILL

FIG No B1-8

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe



Ministry of Transportation
Ontario

PLASTICITY CHART

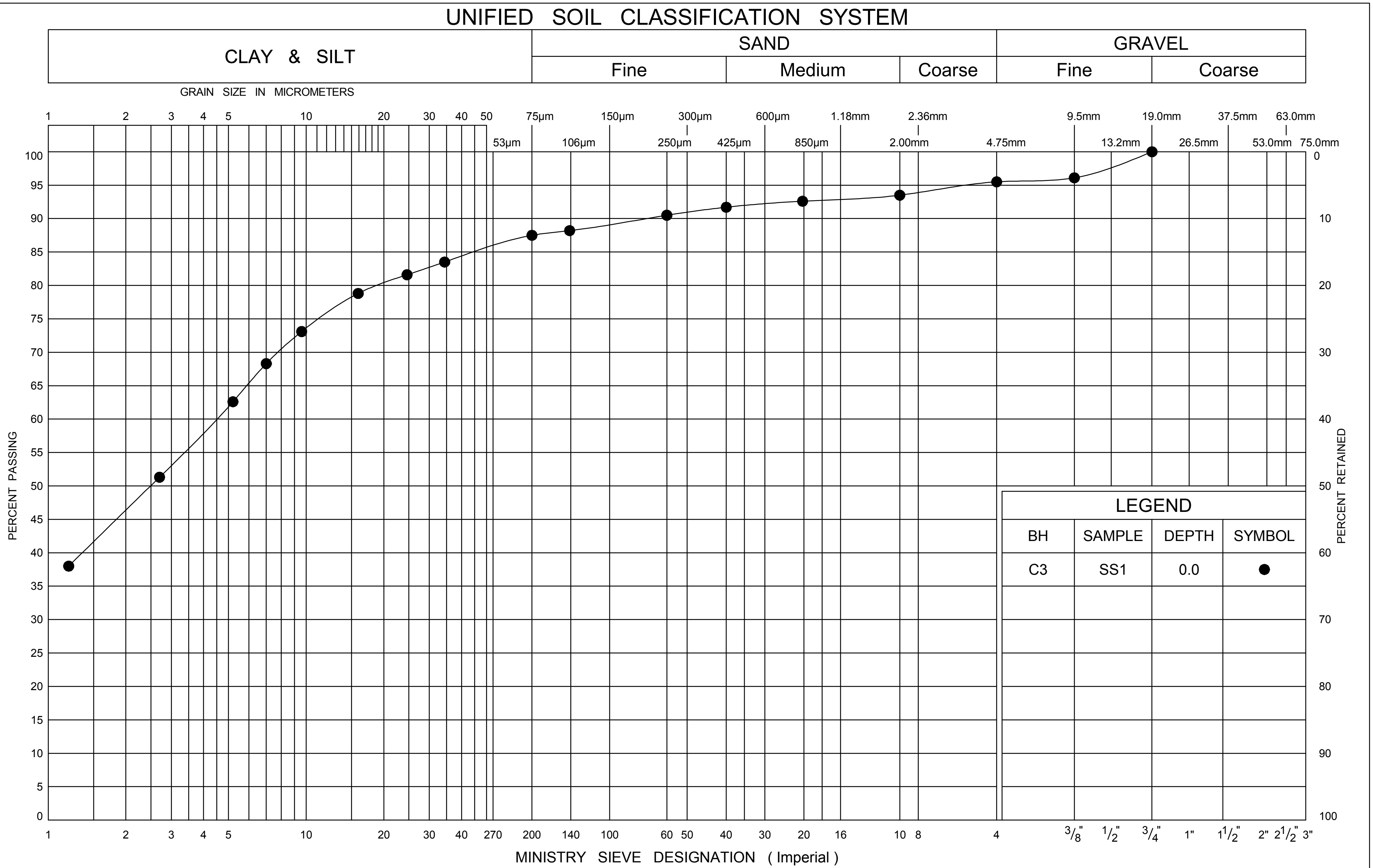
CLAYEY SILT TILL

FIG No B1-9

G W P 5233-06-00

Crow Bridge Replacement

78 12 M



LEGEND			
BH	SAMPLE	DEPTH	SYMBOL
C3	SS1	0.0	●

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

FILL - SILTY CLAY

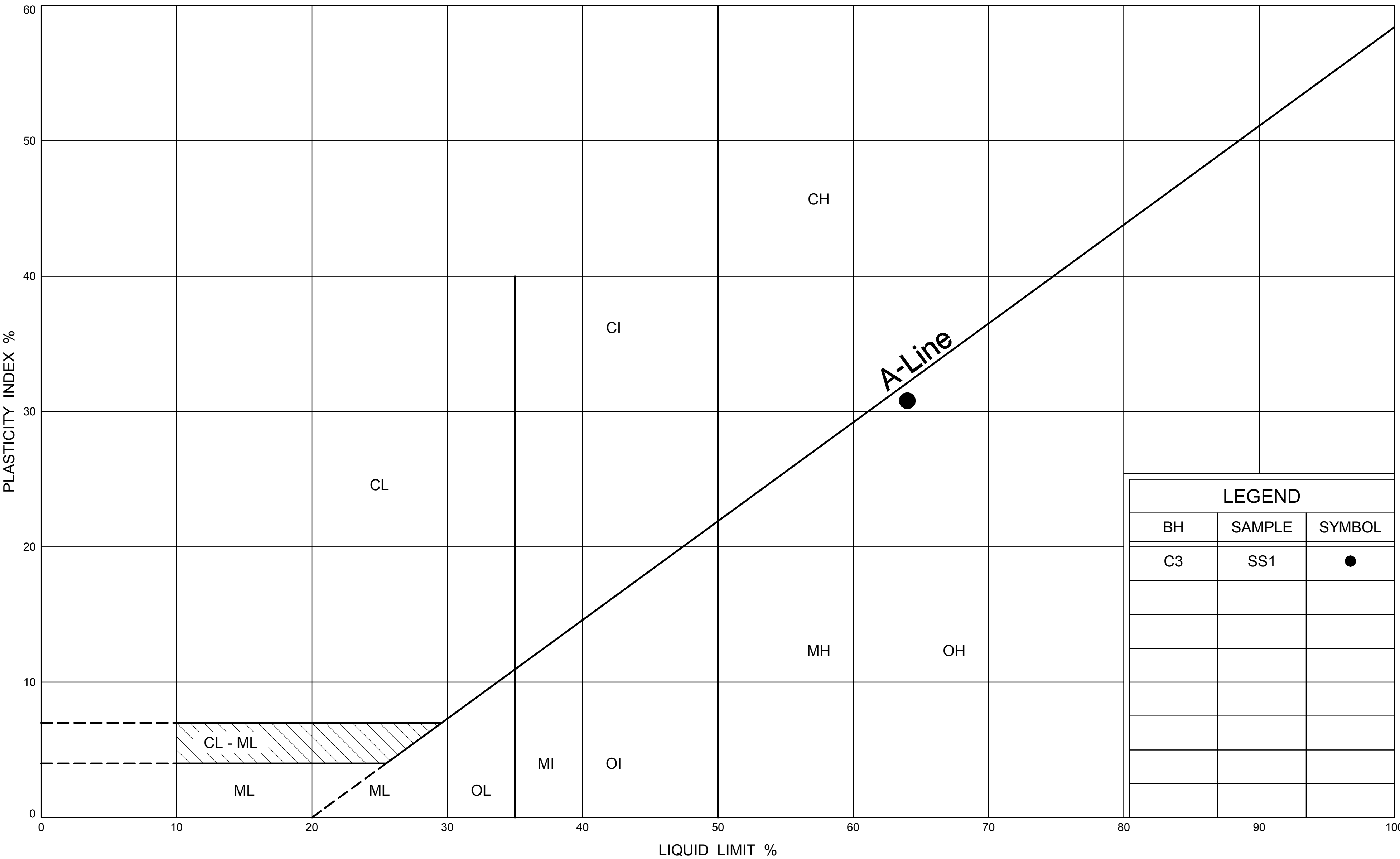
FIG No B2-1

G W P 5233-06-00

Crow Bridge Replacement

library: library - mto gint.glb report: mot-terraprobe grain size path: y:\1-project files\11-geotechnical\2010\11-10-5001 to 5099\11-10-5076\n. gint\11-10-5076 crow bridge.gpj

Terraprobe



Ministry of Transportation
Ontario

PLASTICITY CHART

FILL - SILTY CLAY

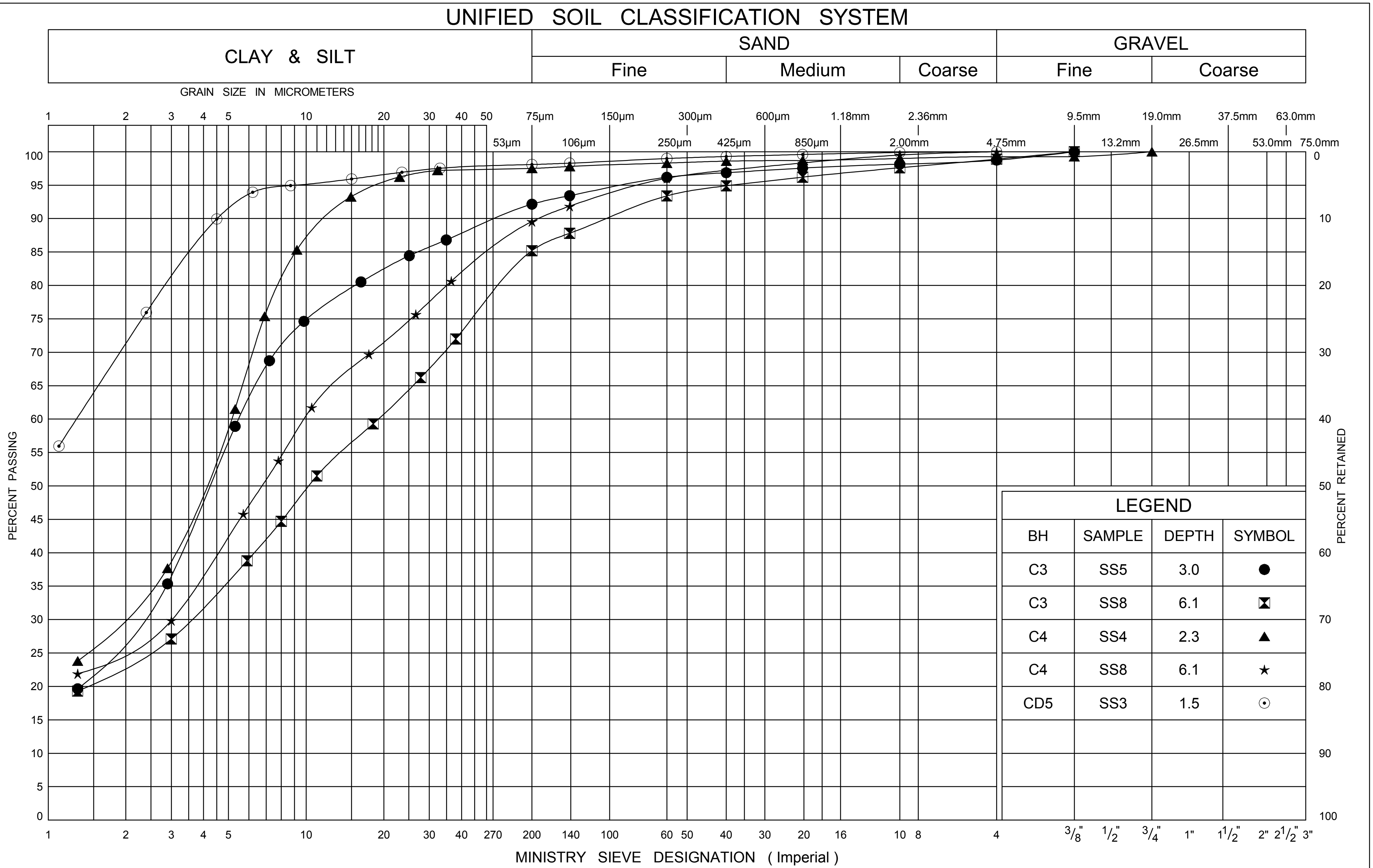
FIG No B2-2

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
C3	SS5	3.0	●
C3	SS8	6.1	⧗
C4	SS4	2.3	▲
C4	SS8	6.1	★
CD5	SS3	1.5	⊙

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILTY CLAY

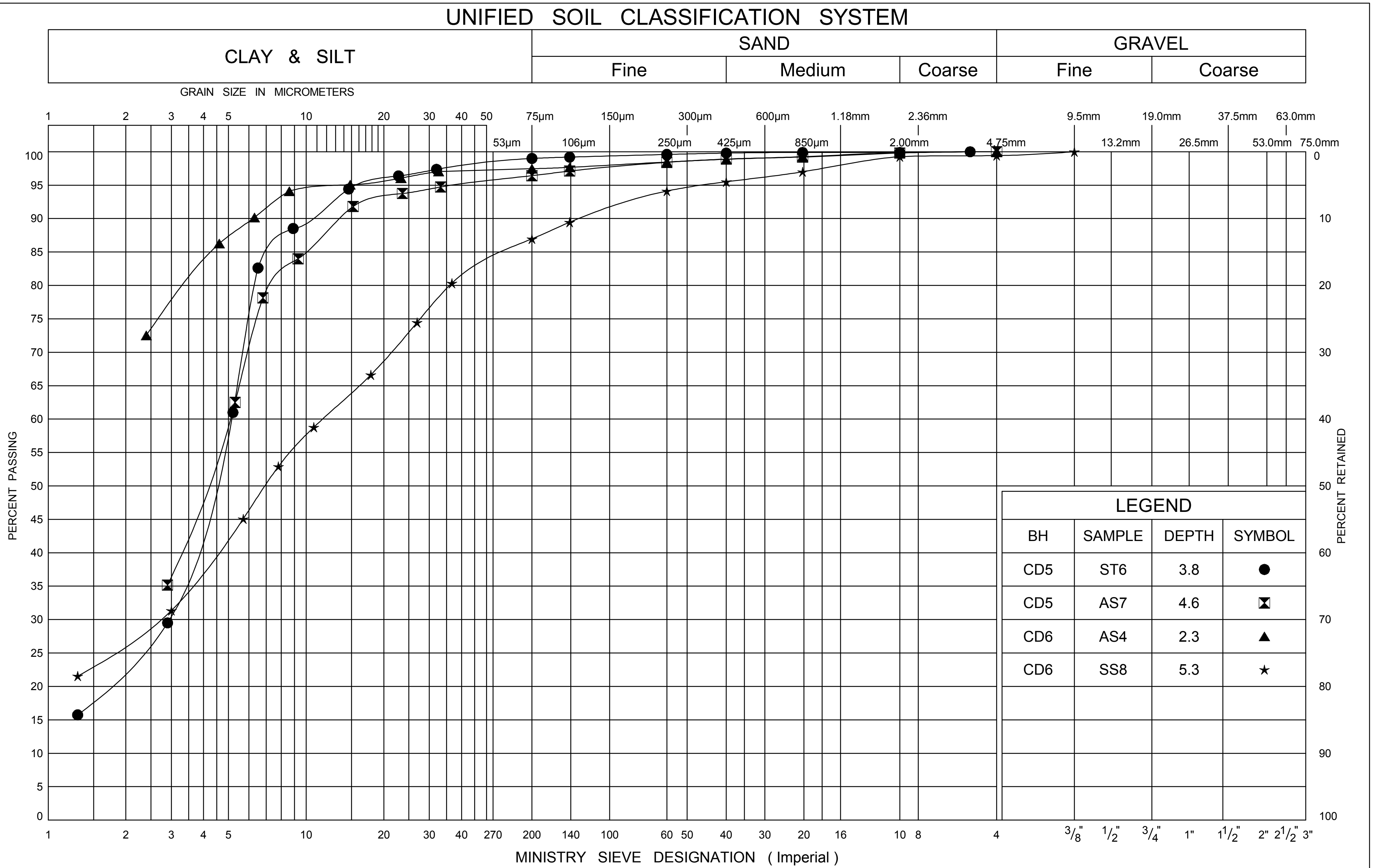
FIG No B2-3

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
CD5	ST6	3.8	●
CD5	AS7	4.6	⧗
CD6	AS4	2.3	▲
CD6	SS8	5.3	★

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

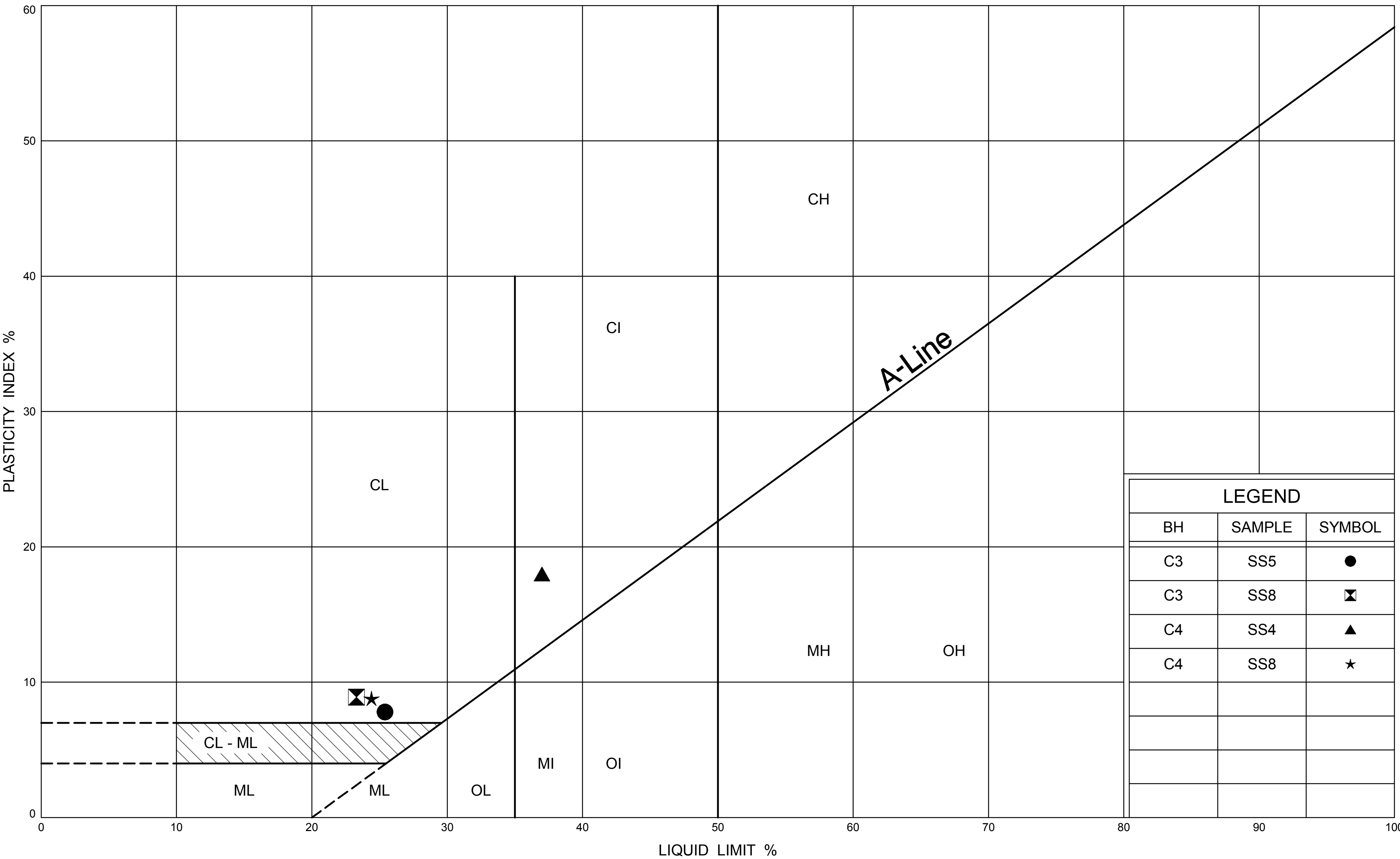
CLAYEY SILT TO SILTY CLAY

FIG No B2-4

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe



LEGEND		
BH	SAMPLE	SYMBOL
C3	SS5	●
C3	SS8	⧗
C4	SS4	▲
C4	SS8	★

Ministry of Transportation
Ontario

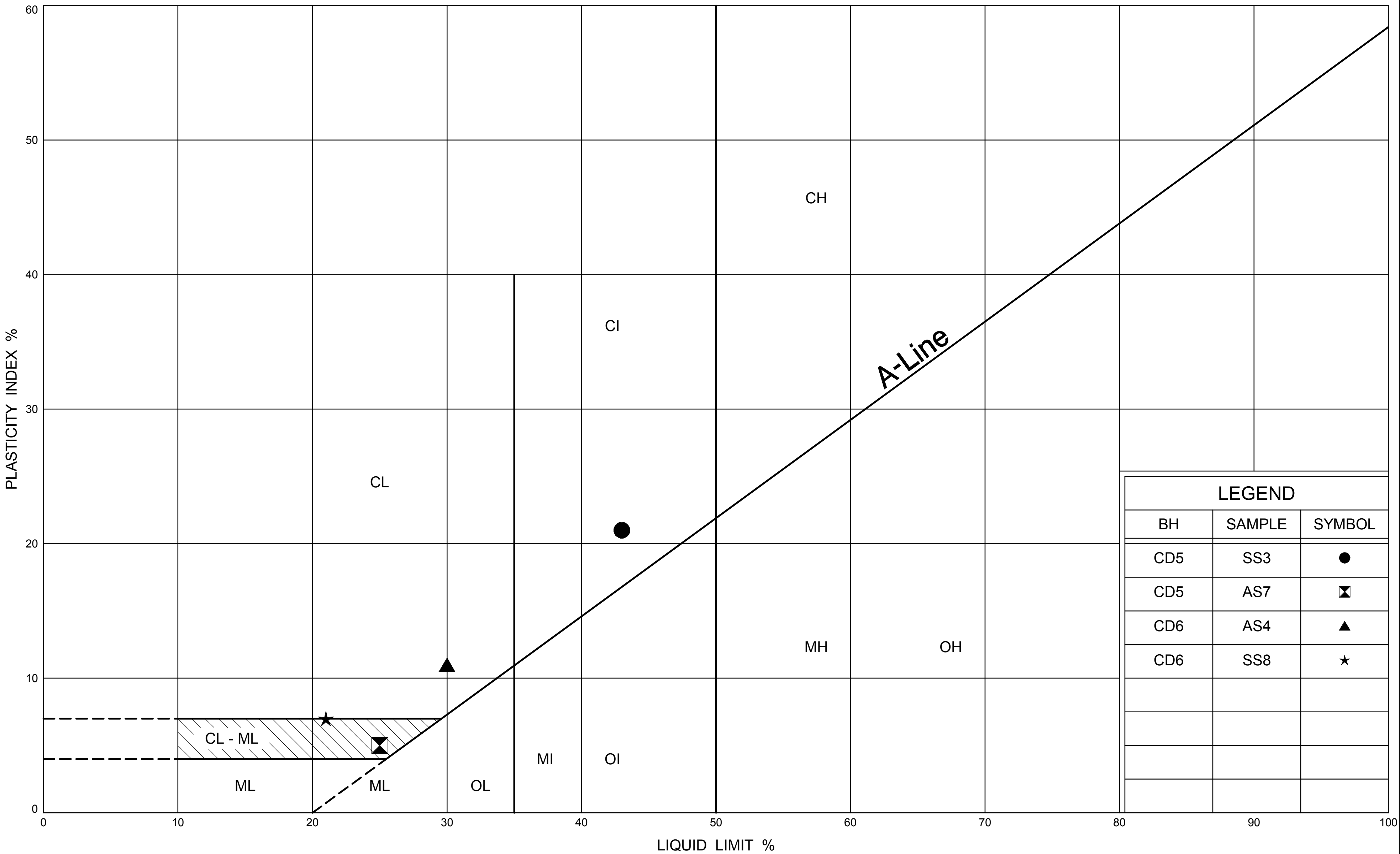
PLASTICITY CHART
SILTY CLAY

FIG No B2-5

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe



Ministry of Transportation
Ontario

PLASTICITY CHART

CLAYEY SILT TO SILTY CLAY

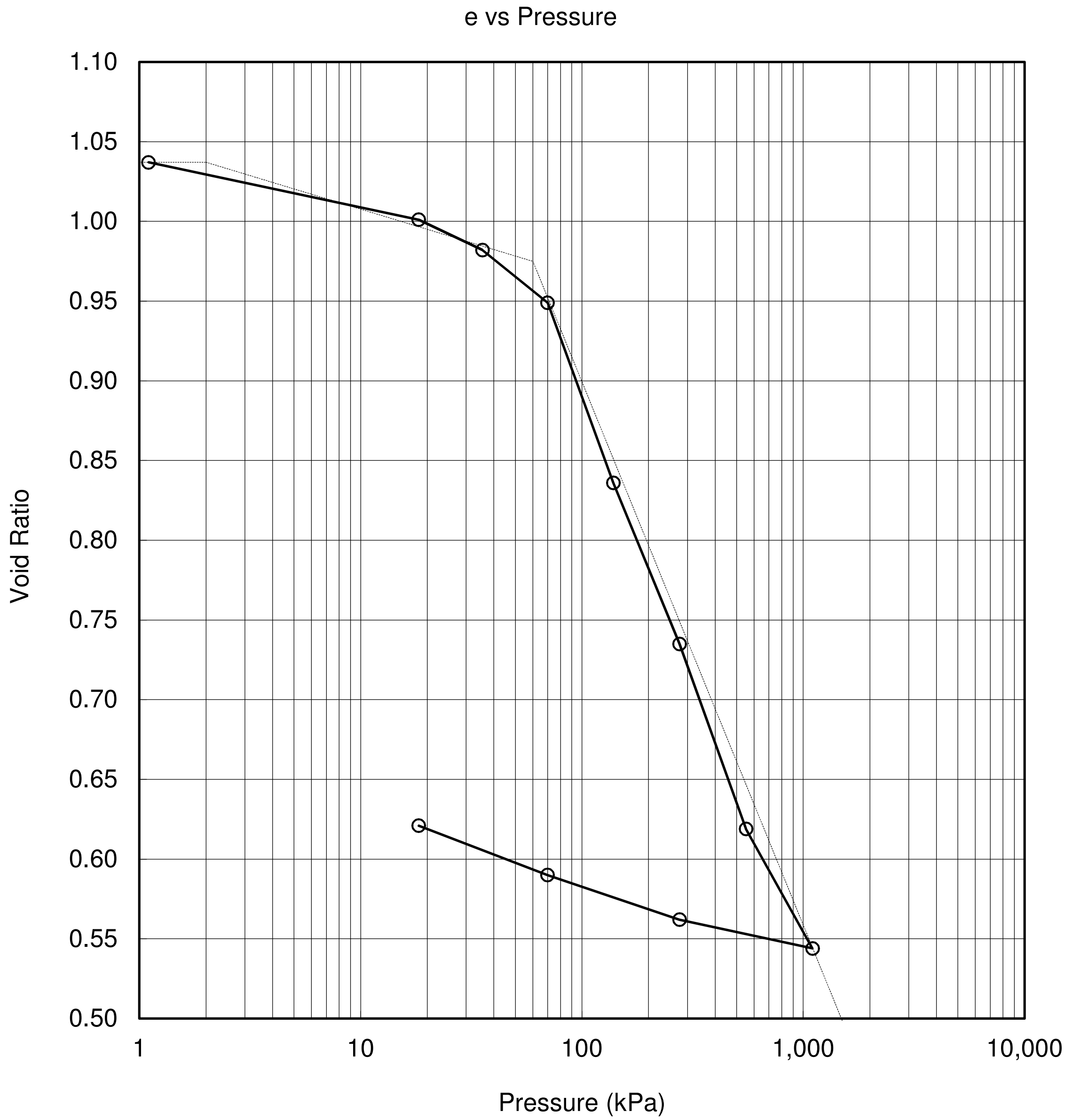
FIG No B2-6

G W P 5233-06-00

Crow Bridge Replacement

CONSOLIDATION TEST	FIG No B2-7

CROW BRIDGE, BH CD5, SAMPLE Sa6
SILTY CLAY



Soil Type : Silty Clay

e_o =	1.04	ω_L =	-	Po' =	2 kPa
ω =	33%	ω_P =	-	Pc' =	60 kPa
γ =	17.4 kN/m^3	PI =	-	Cc =	0.341
Gs =	2.73			Cr =	0.042

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

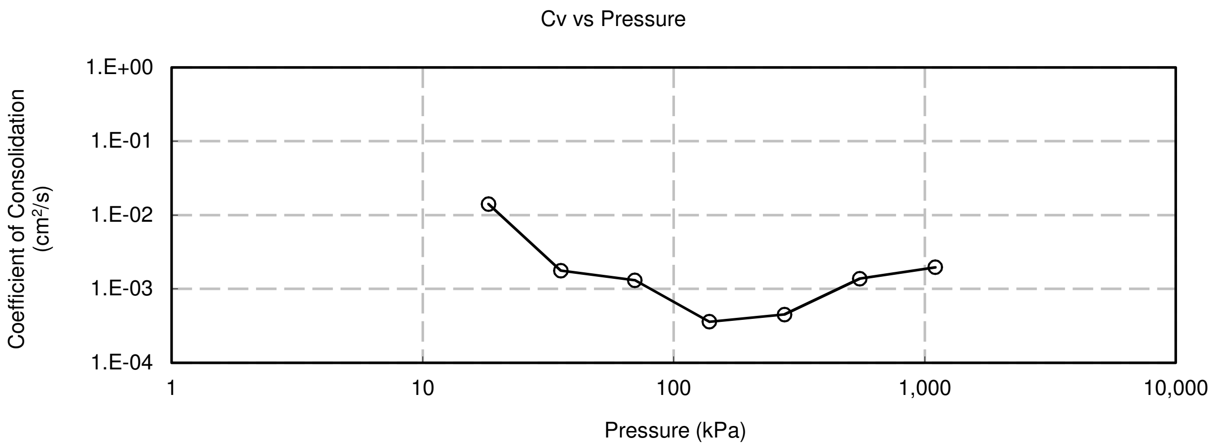
Terraprobe Inc.

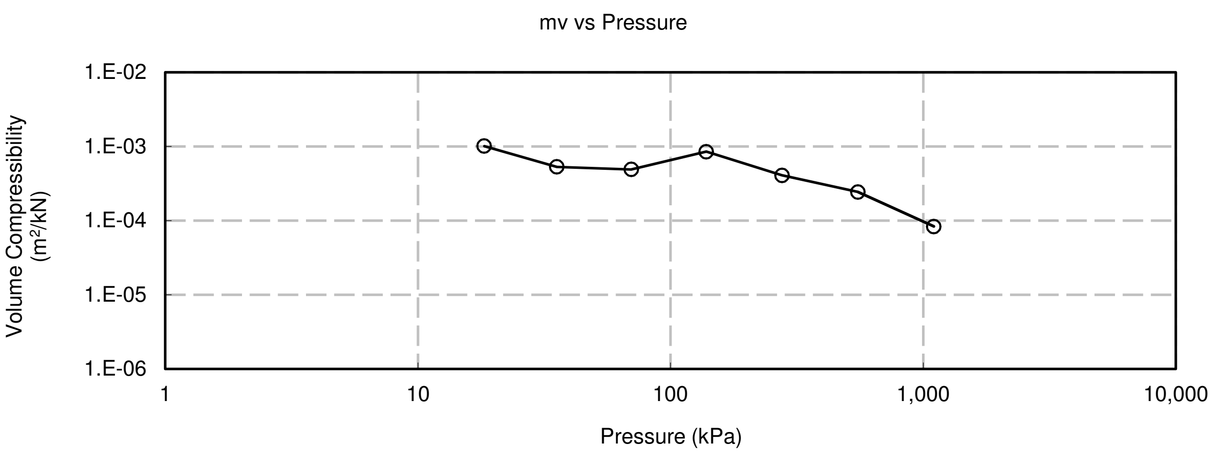
Prepared By : MD
Checked By : JC

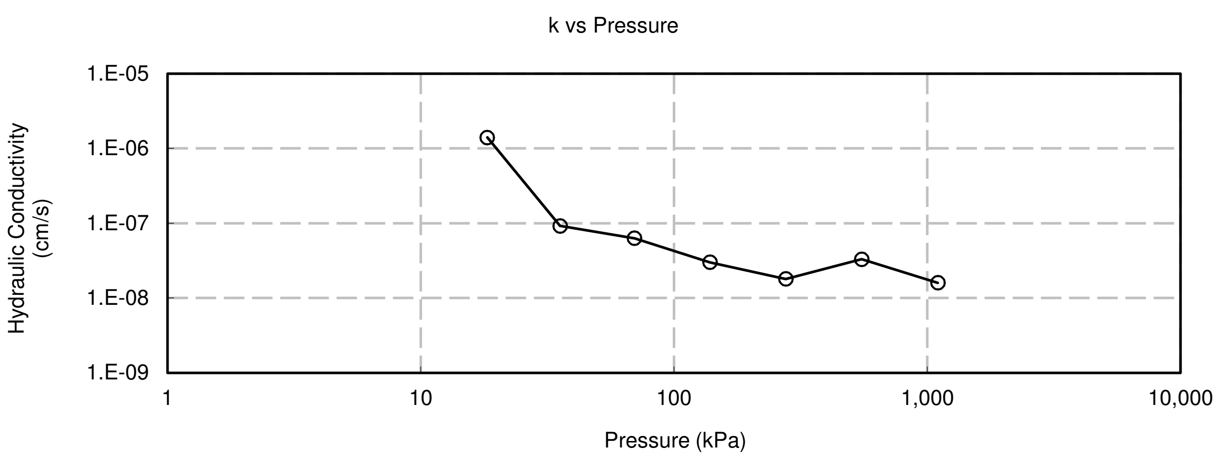
C:\Users\Toshiba\Documents\1 Projects\11-10-5076, Hwy 11, Crow and Montcalm Creeks\Reports\Crow Creek\Latest Draft\Appendix C3-3.xls

CONSOLIDATION TEST	FIG No B2-8

CROW BRIDGE, BH CD5, SAMPLE Sa6
SILTY CLAY



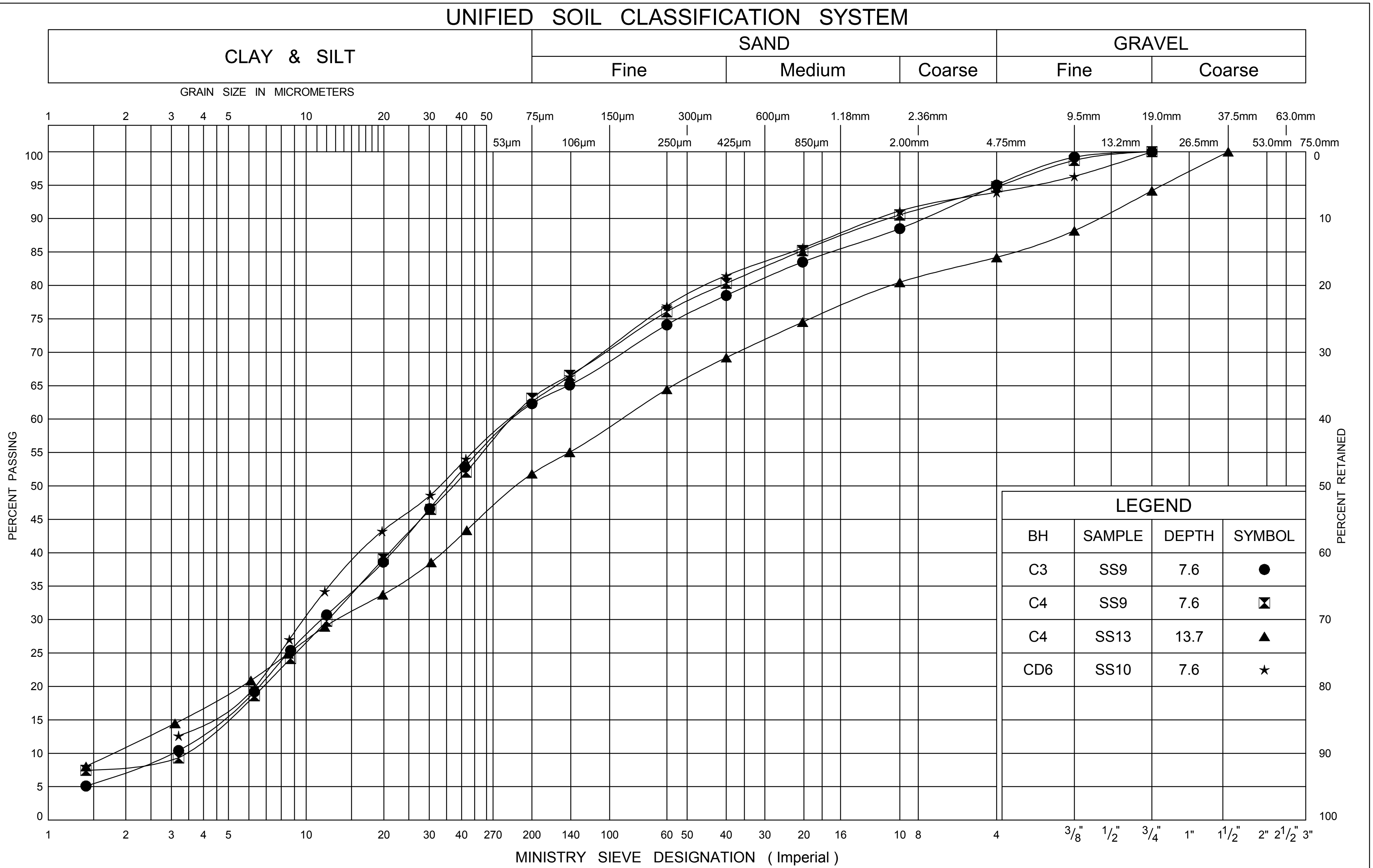




Project No. :	11-10-5076	Terraprobe Inc.	Prepared By :	MD
Date :	January 2012		Checked By :	JC

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
C3	SS9	7.6	●
C4	SS9	7.6	⧗
C4	SS13	13.7	▲
CD6	SS10	7.6	★

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SANDY SILT TILL

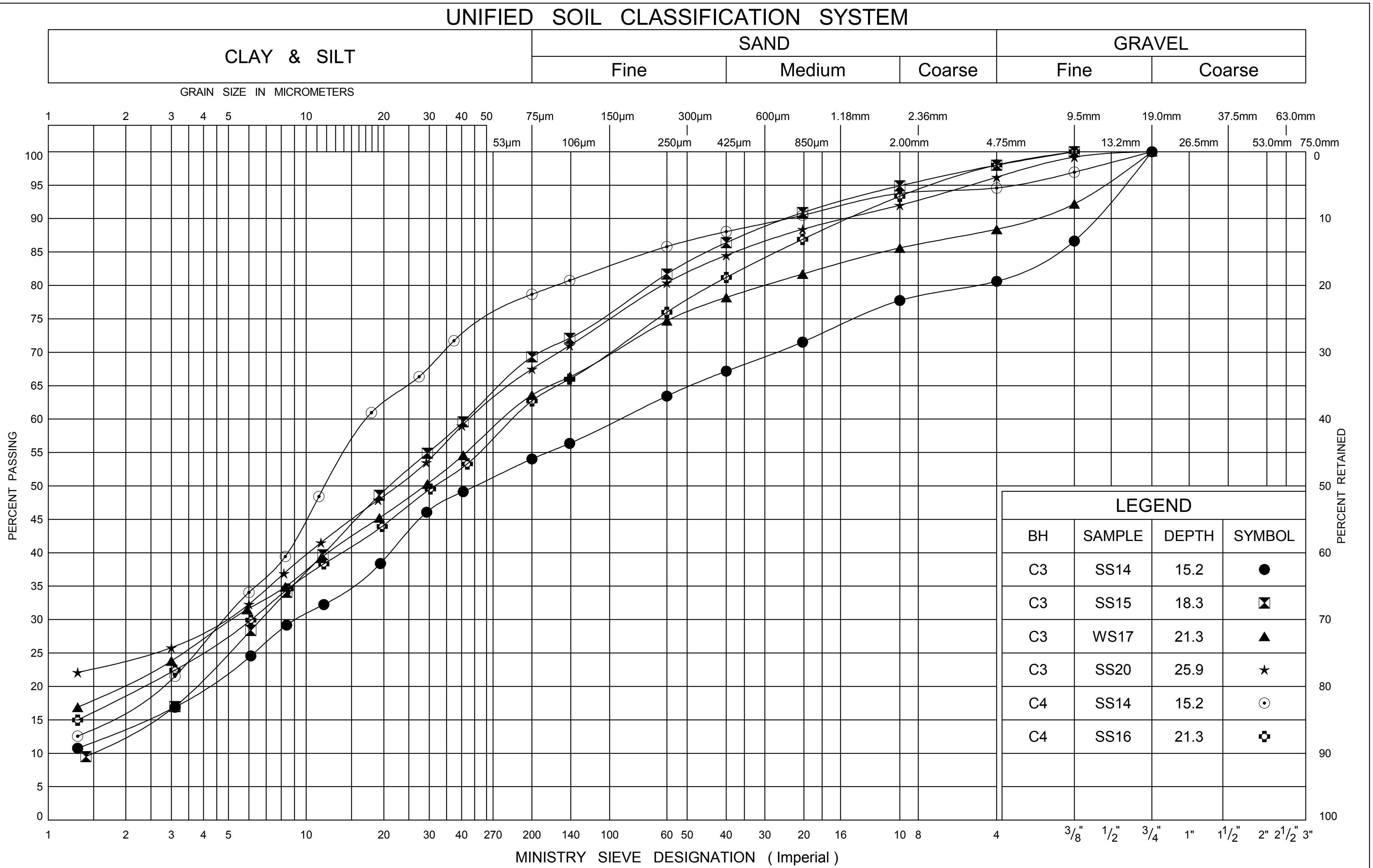
FIG No B2-9

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe

78 12 M



Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION

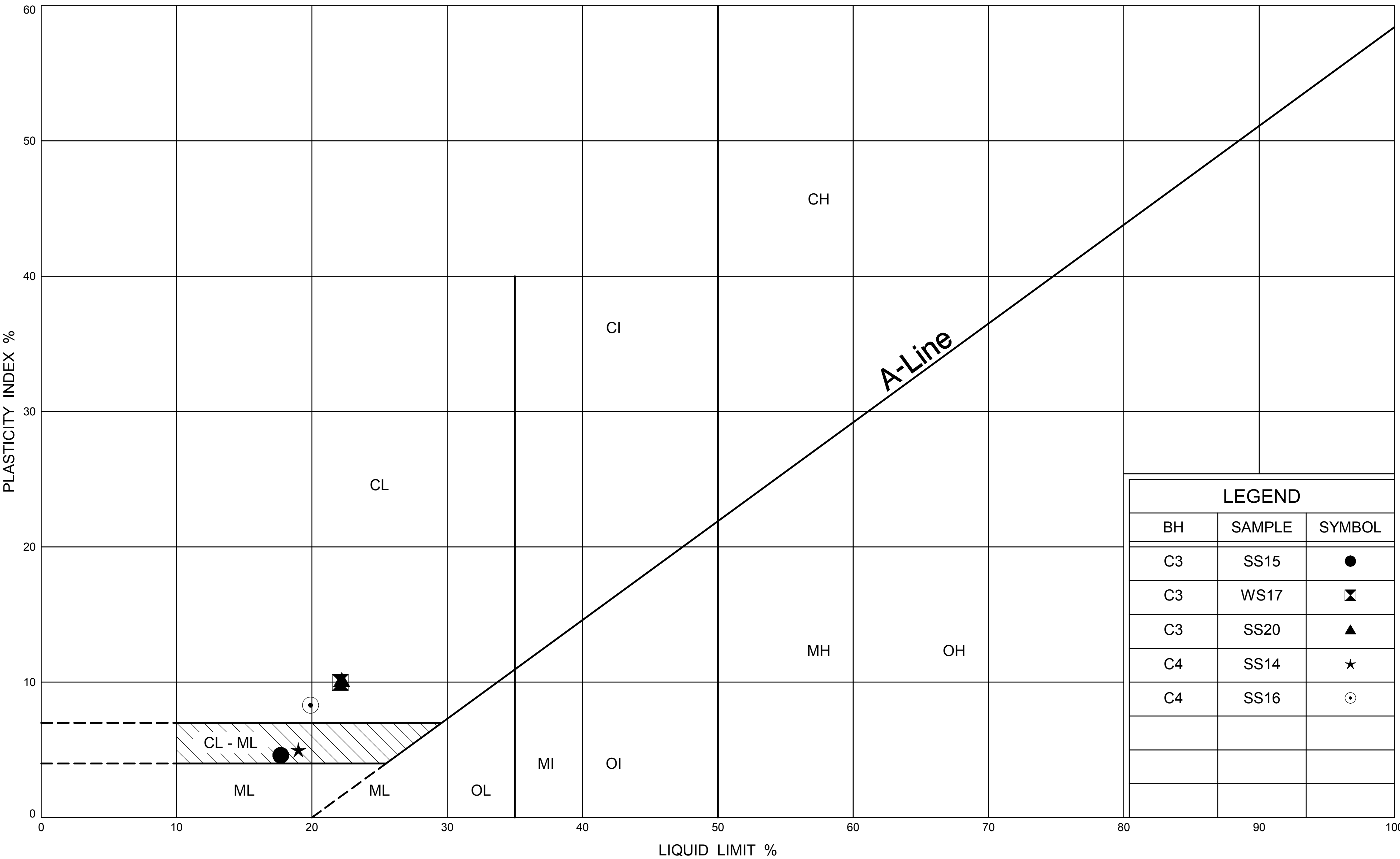
CLAYEY SILT TILL

FIG No B2-10

G W P 5233-06-00

Crow Bridge Replacement

Terraprobe



LEGEND

BH	SAMPLE	SYMBOL
C3	SS15	●
C3	WS17	⧗
C3	SS20	▲
C4	SS14	★
C4	SS16	⊙

Ministry of Transportation
Ontario

PLASTICITY CHART

CLAYEY SILT TILL

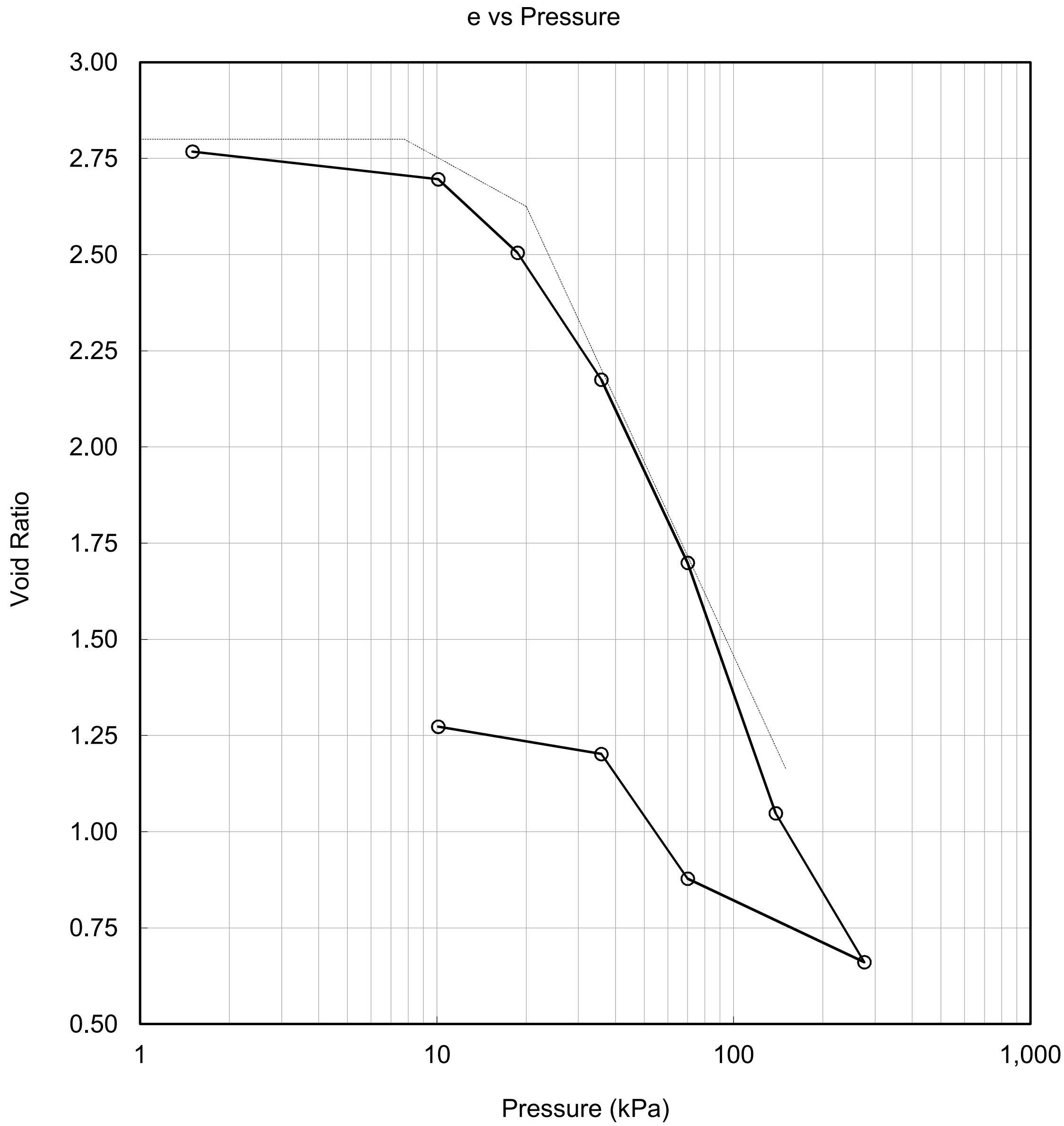
FIG No B2-11

G W P 5233-06-00

Crow Bridge Replacement

CONSOLIDATION TEST	APPENDIX B2-12

CROW CREEK DETOUR, BH 24+450
PEAT



Soil Type : Peat

e_o =	2.80	ω_L =	-	Po' =	7 kPa
ω =	586%	ω_P =	-	Pc' =	20 kPa
γ =	9.8 kN/m^3	PI =	-	Cc =	1.671
Gs =	0.55			Cr =	0.426

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

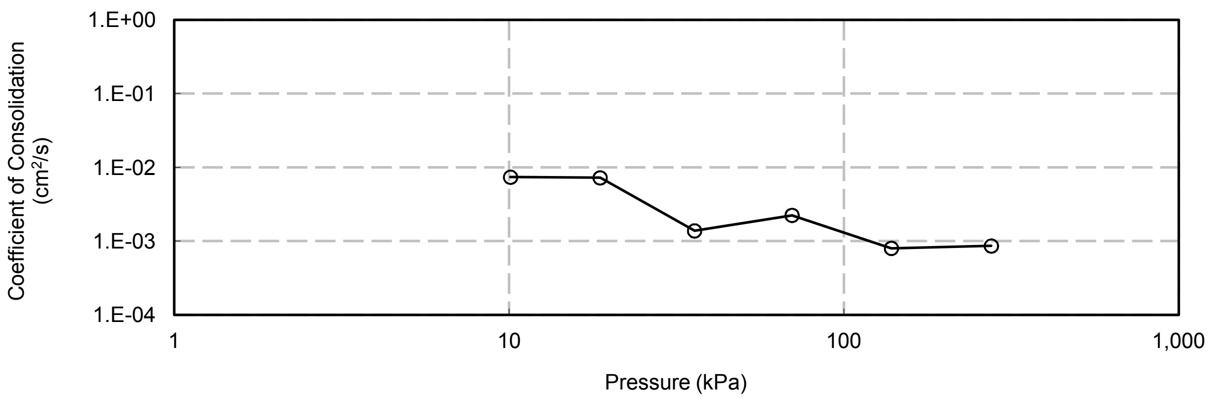
Terraprobe Inc.

Prepared By : MD
Checked By : JC

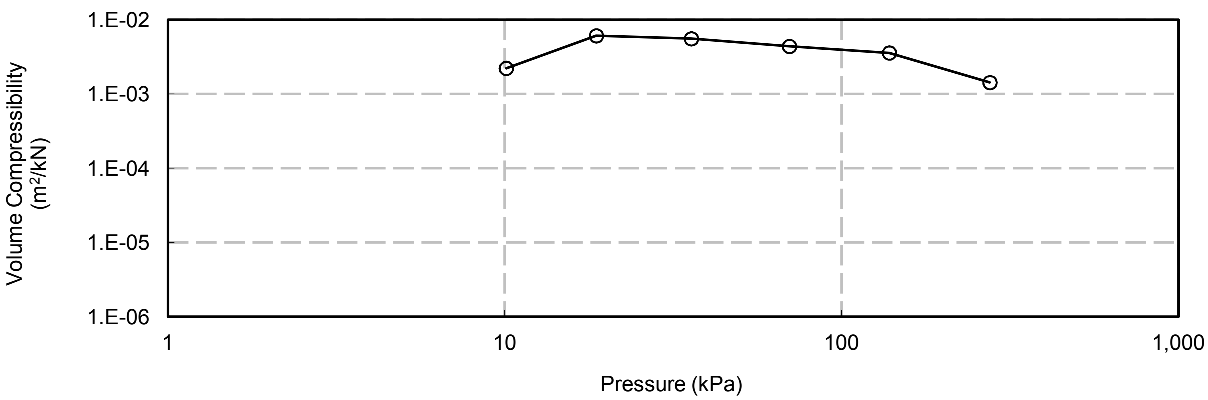
CONSOLIDATION TEST	APPENDIX B2-13

CROW CREEK DETOUR, BH 24+450
PEAT

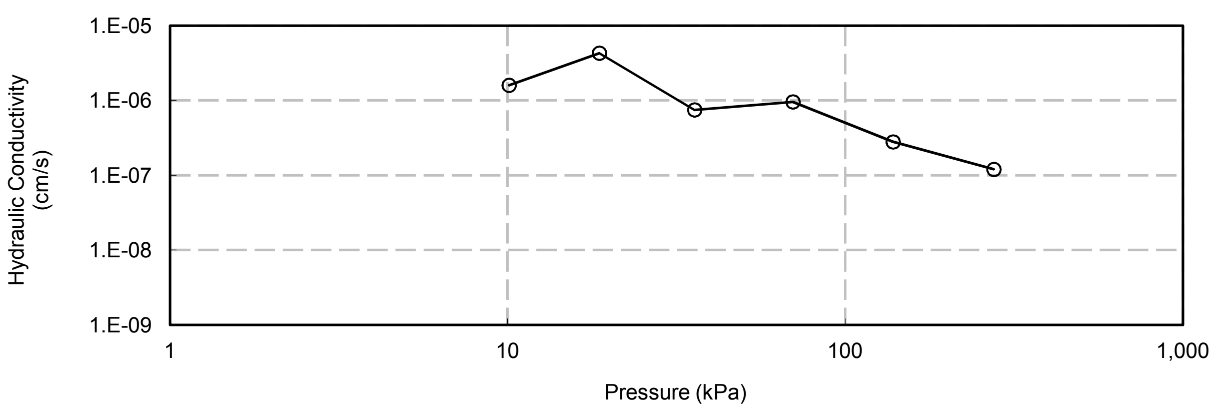
Cv vs Pressure



mv vs Pressure



k vs Pressure



Y:\1-Project Files\11-Geotechnical\2010\1-10-5001 to 5099\1-10-5076\D. Calcs, Analysis, Info\Settlement (MD)\11-10-5076 Consolidation Results 24+450 peat.xls

Project No. :	11-10-5076	Terraprobe Inc.	Prepared By :	MD
Date :	January 2012		Checked By :	JC

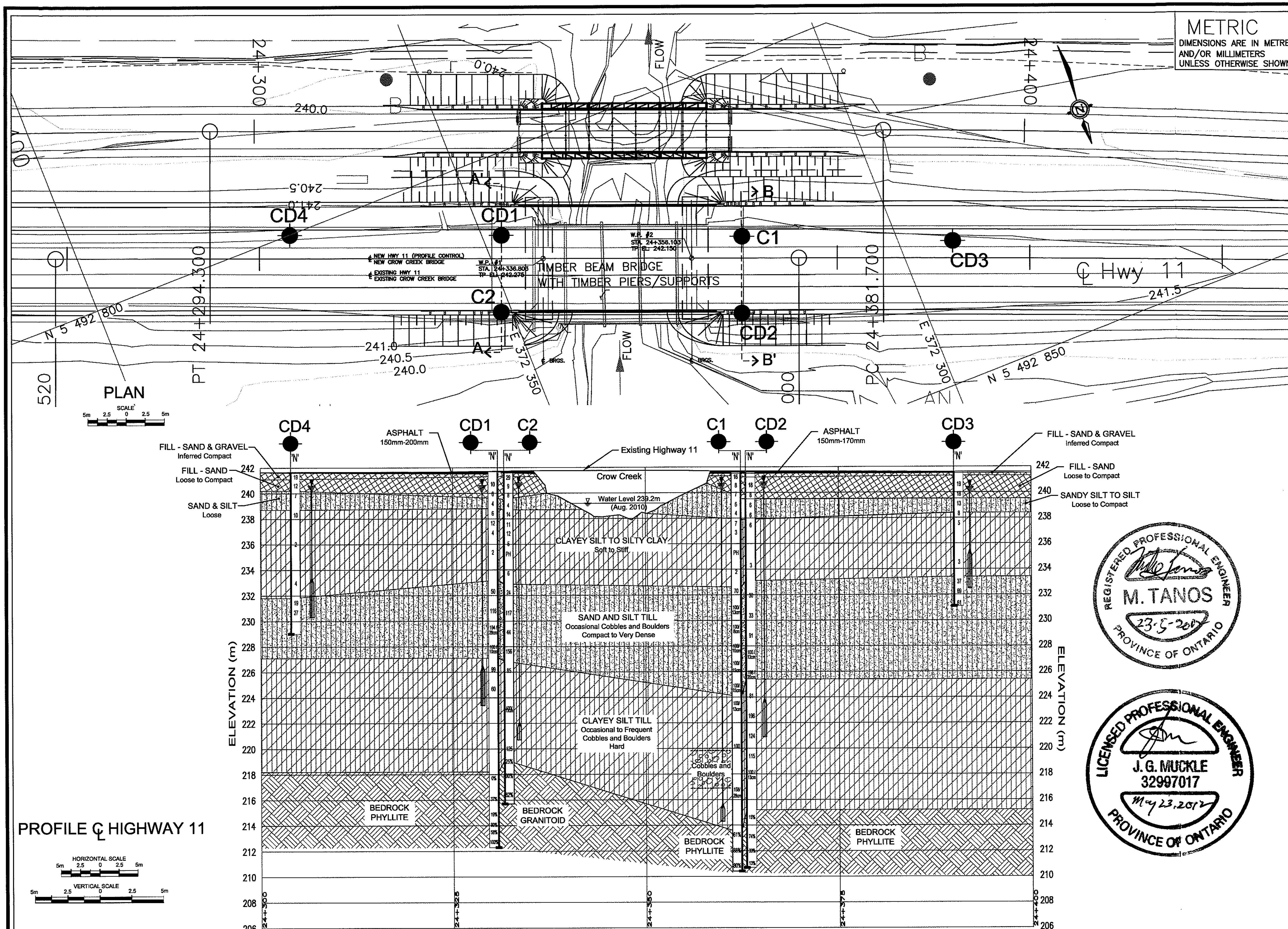
APPENDIX C

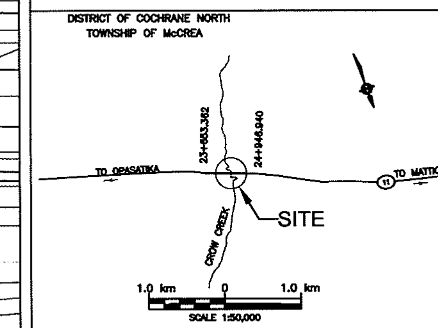
TERRAPROBE INC.

CONT No GWP No 5233-06-00		
CROW CREEK BRIDGE BOREHOLE LOCATION AND SOIL STRATA	SHEET 1 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

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





KEY PLAN

	LEGEND
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/0.3m (60° Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
C1	241.6	5 492 821.8	372 318.7
C2	241.6	5 492 819.4	372 351.3
CD1	241.7	5 492 810.1	372 347.7
CD2	241.6	5 492 831.1	372 322.4
CD3	241.6	5 492 832.4	372 293.4
CD4	241.6	5 492 800.0	372 373.2

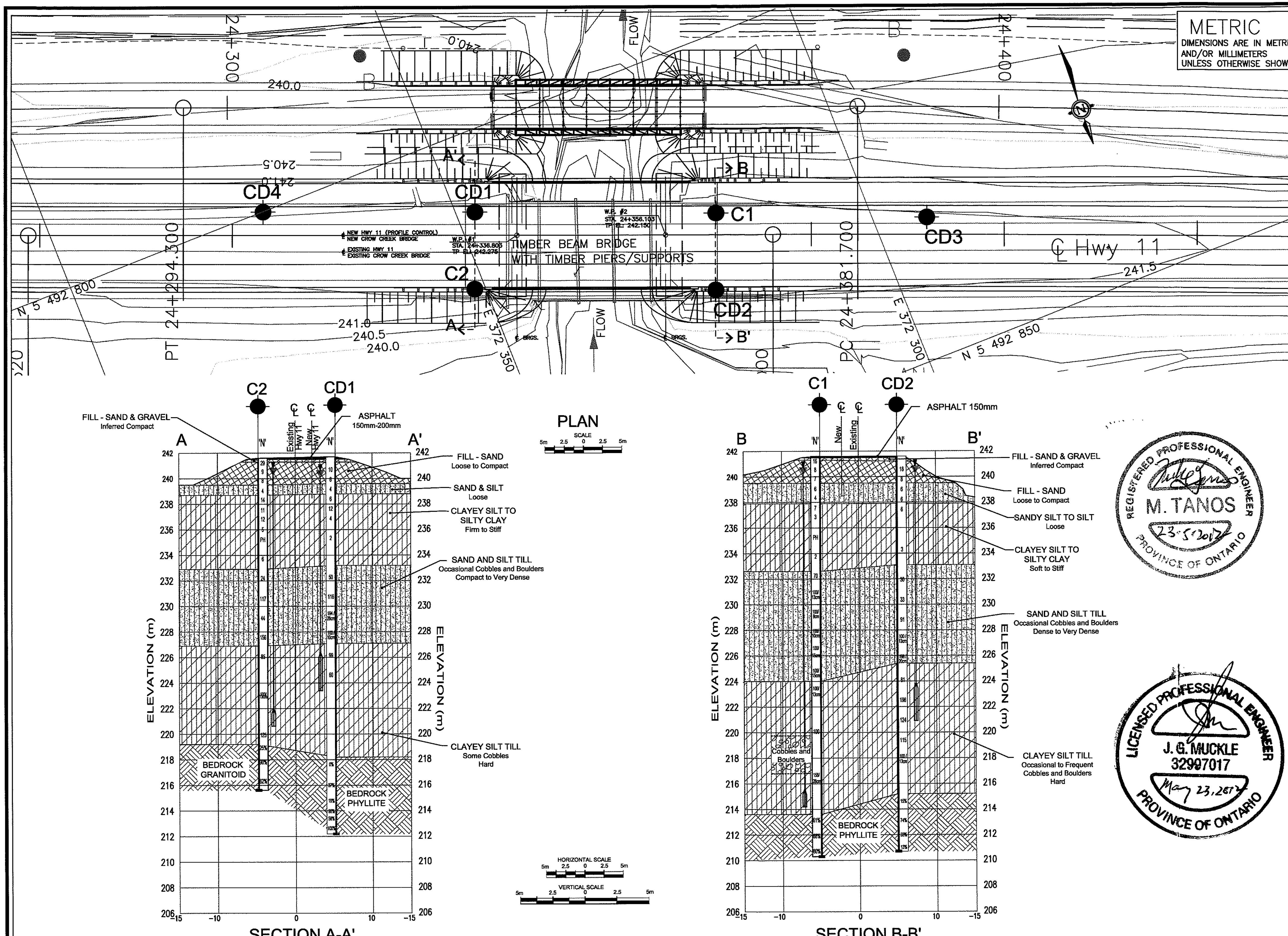
NOTE

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-055	GEOCRES 42G-33



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

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

CONT No

GWP No 5233-06-00

CROW CREEK BRIDGE

BOREHOLE LOCATION AND SOIL STRATA

SHEET 2 OF 4

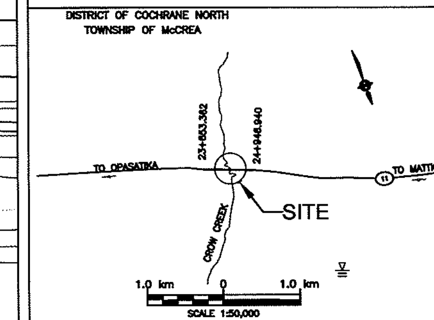
McCORMICK RANKIN CORPORATION

MRC

Terraprobe Inc.

Consulting Geotechnical & Environmental Engineering
Construction Materials, Inspection & Testing

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



KEY PLAN

LEGEND

●	Bore Hole
⌖	Dynamic Cone Penetration Test
⌖	Bore Hole And Cone
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/0.3m (60° Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
C1	241.6	5 492 821.8	372 318.7
C2	241.6	5 492 819.4	372 351.3
CD1	241.7	5 492 810.1	372 347.7
CD2	241.6	5 492 831.1	372 322.4
CD3	241.6	5 492 832.4	372 293.4
CD4	241.6	5 492 800.0	372 373.2

NOTE

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

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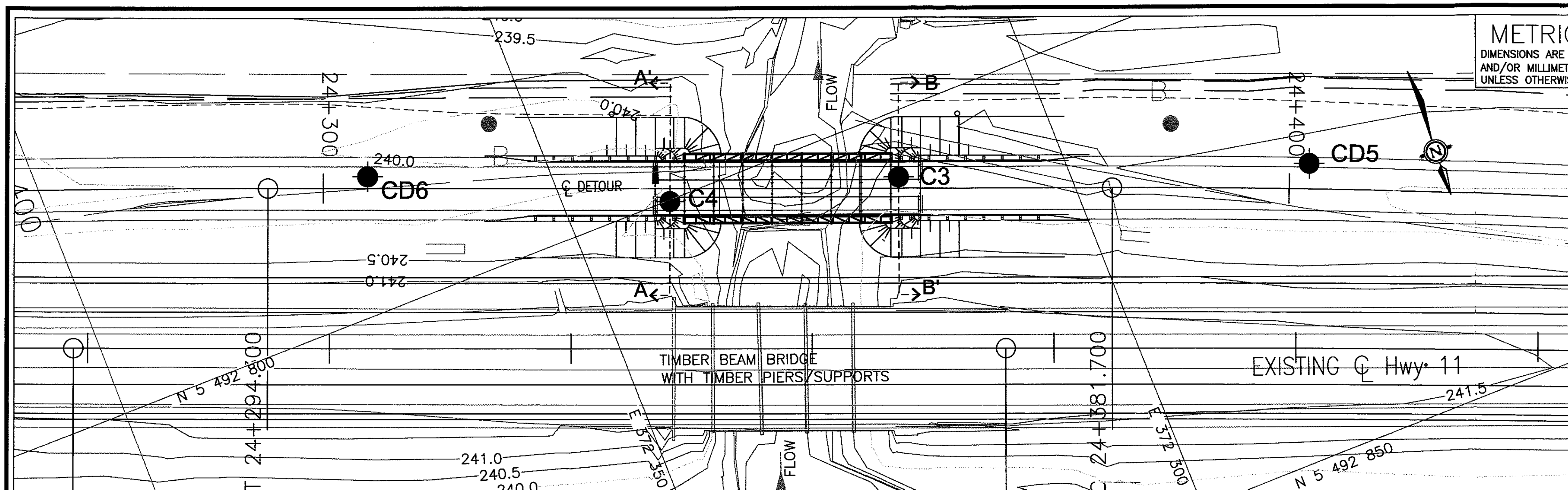
REVISIONS		
DATE	BY	DESCRIPTION

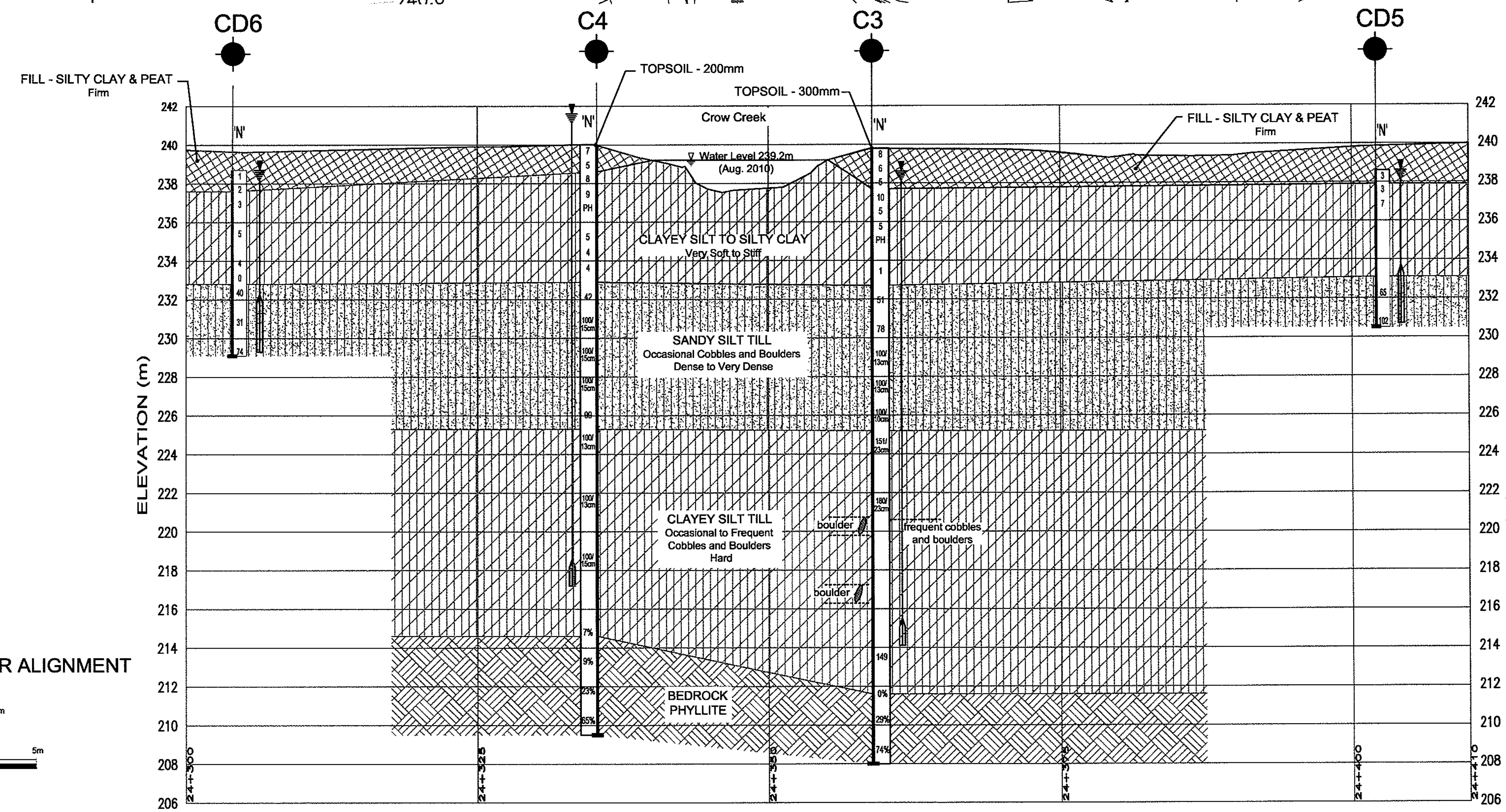
DESIGN G.M	CODE CHBDC2006	LOAD	DATE MAY 2012
DRAWN K.C	CHK G.M	STRUCT 39W-055	GEOCRES 42G-33

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



PLAN

SCALE 5m 2.5 0 2.5 5m



PROFILE ℄ DETOUR ALIGNMENT

HORIZONTAL SCALE 5m 2.5 0 2.5 5m

VERTICAL SCALE 5m 2.5 0 2.5 5m

REGISTERED PROFESSIONAL ENGINEER M. TANOS 23.5.2012 PROVINCE OF ONTARIO

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

CONT No

GWP No 5233-06-00

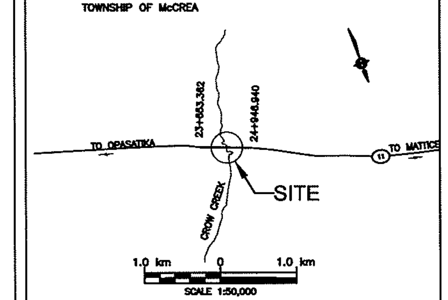
CROW CREEK DETOUR
BOREHOLE LOCATION AND SOIL STRATA

SHEET 3 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

DISTRICT OF COCHRANE NORTH
TOWNSHIP OF McCREA



KEY PLAN

LEGEND

⬤	Bore Hole
⊕	Dynamic Cone Penetration Test
⊕	Bore Hole And Cone
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/0.3m (60° Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
C3	239.8	5 492 806.5	372 316.7
C4	240.0	5 492 800.2	372 339.6
CD5	238.6	5 492 820.9	372 276.6
CD6	238.7	5 492 786.3	372 367.8

NOTE

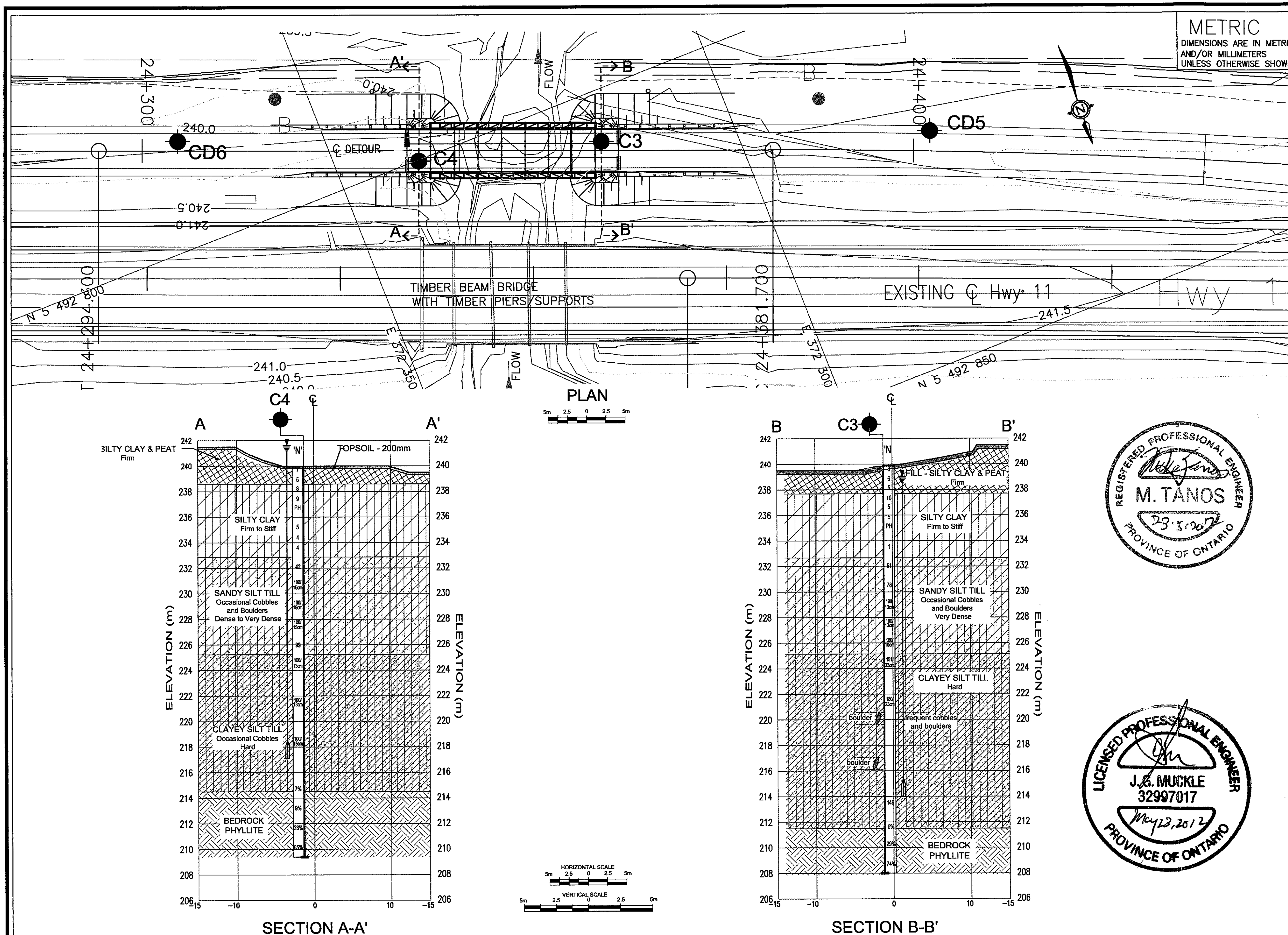
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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-055	GEOCRES 42G-33

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



CONT No

GWP No 5233-06-00

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BOREHOLE LOCATION AND SOIL STRATA

SHEET 4 OF 4

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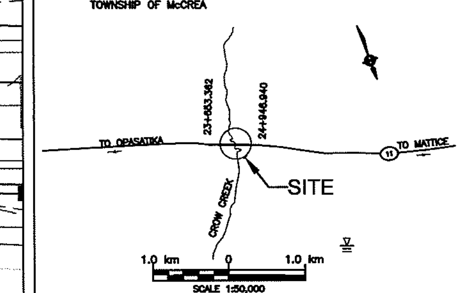
DISTRICT OF COCHRANE NORTH
TOWNSHIP OF McCREA



KEY PLAN

LEGEND

Symbol	Description
●	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
C3	241.6	5 492 806.5	372 316.7
C4	241.6	5 492 800.2	372 339.6
CD5	238.6	5 492 820.9	372 276.6
CD6	238.7	5 492 786.3	372 367.8

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-055	GEOCRES 42G-33

APPENDIX D

TERRAPROBE INC.

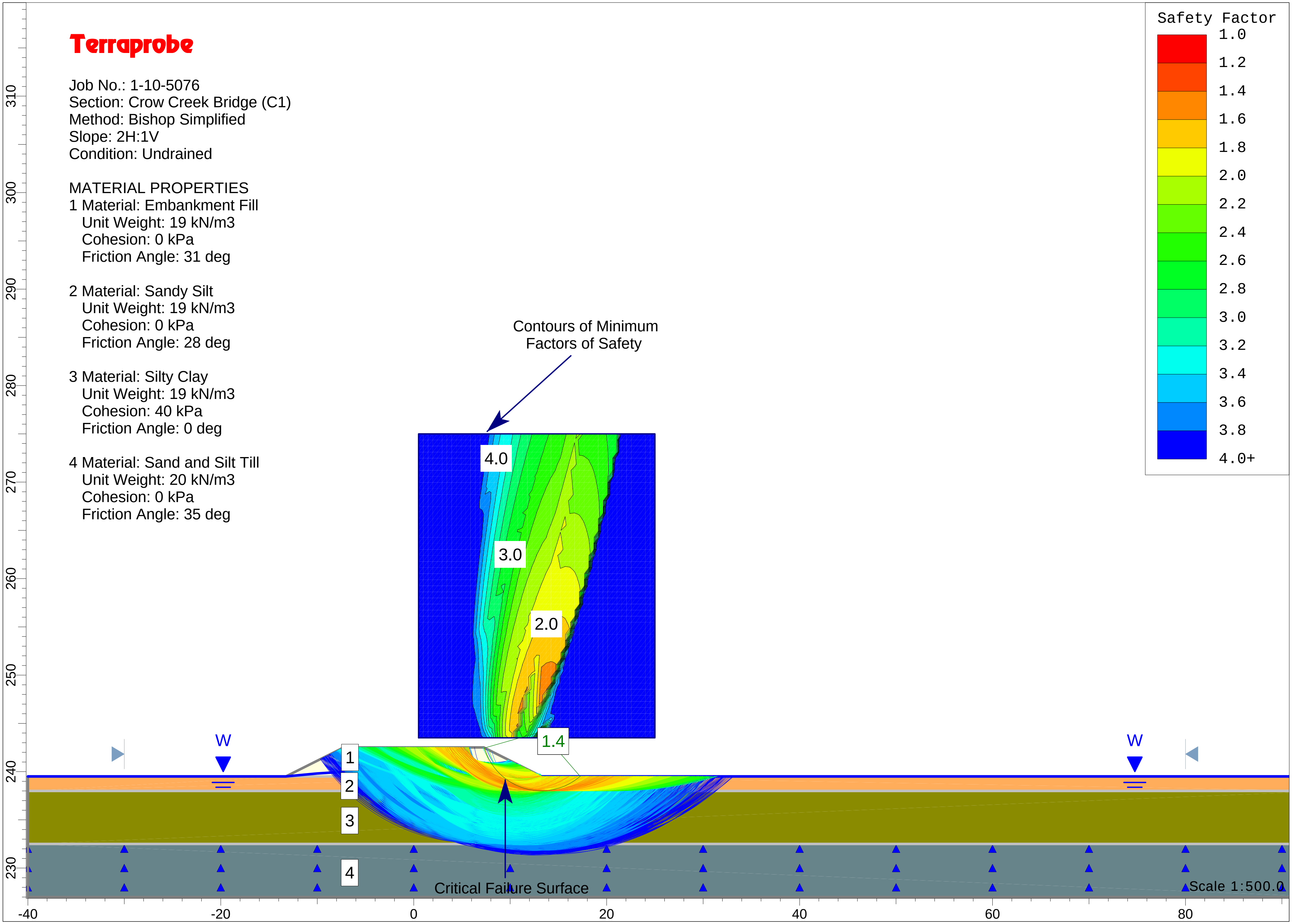
COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

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

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

APPENDIX E

TERRAPROBE INC.


Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C1)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sand and Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum
Factors of Safety
4.0
3.0
2.0
1.4
W
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
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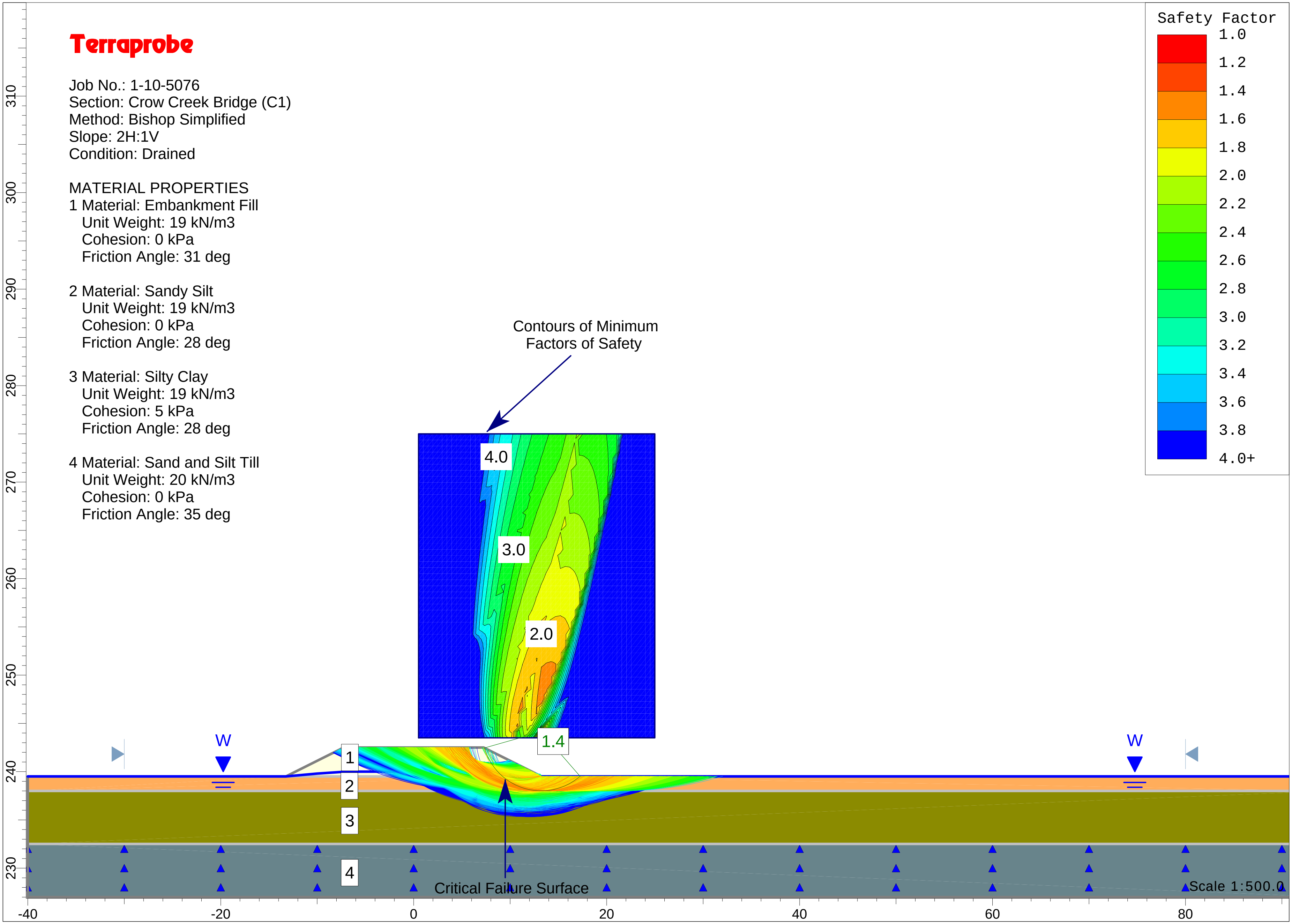
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C1)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
4 Material: Sand and Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum
Factors of Safety
4.0
3.0
2.0
1.4
W
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
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3.6
3.8
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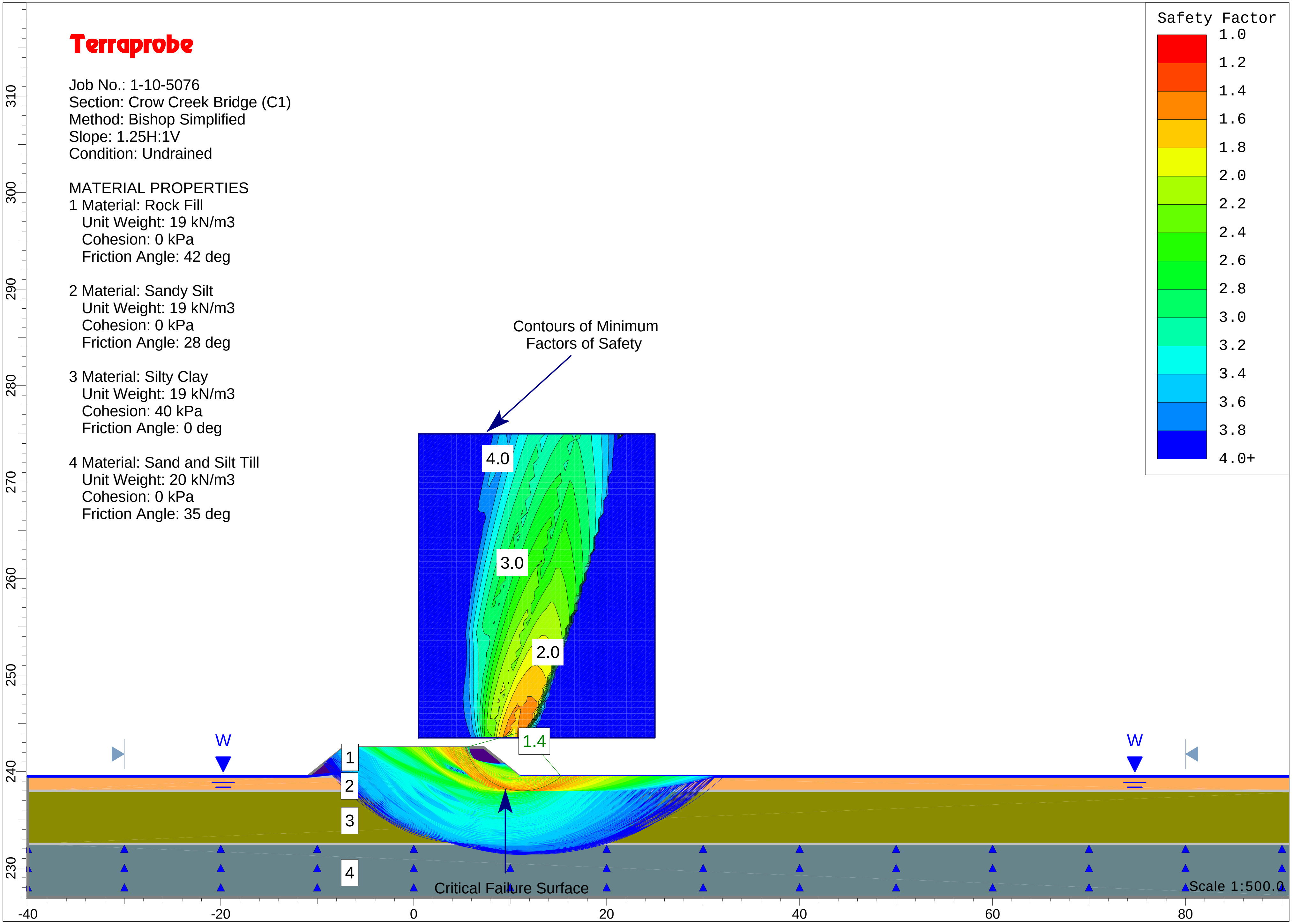
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C1)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sand and Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.0
1.4
W
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
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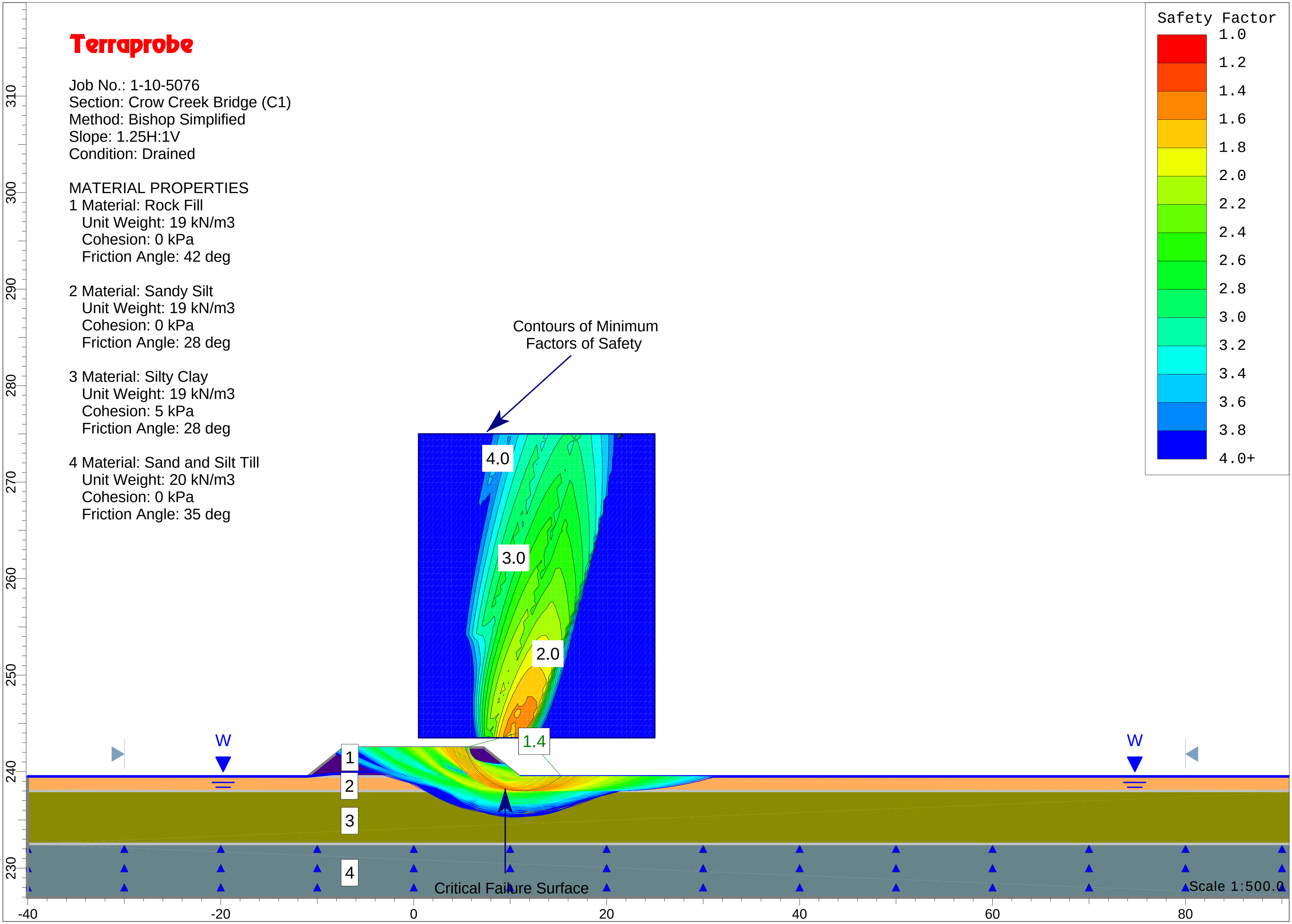
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C1)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
4 Material: Sand and Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.0
1.4
Critical Failure Surface
W
W
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3
4
Safety Factor
1.0
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2.2
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3.2
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3.8
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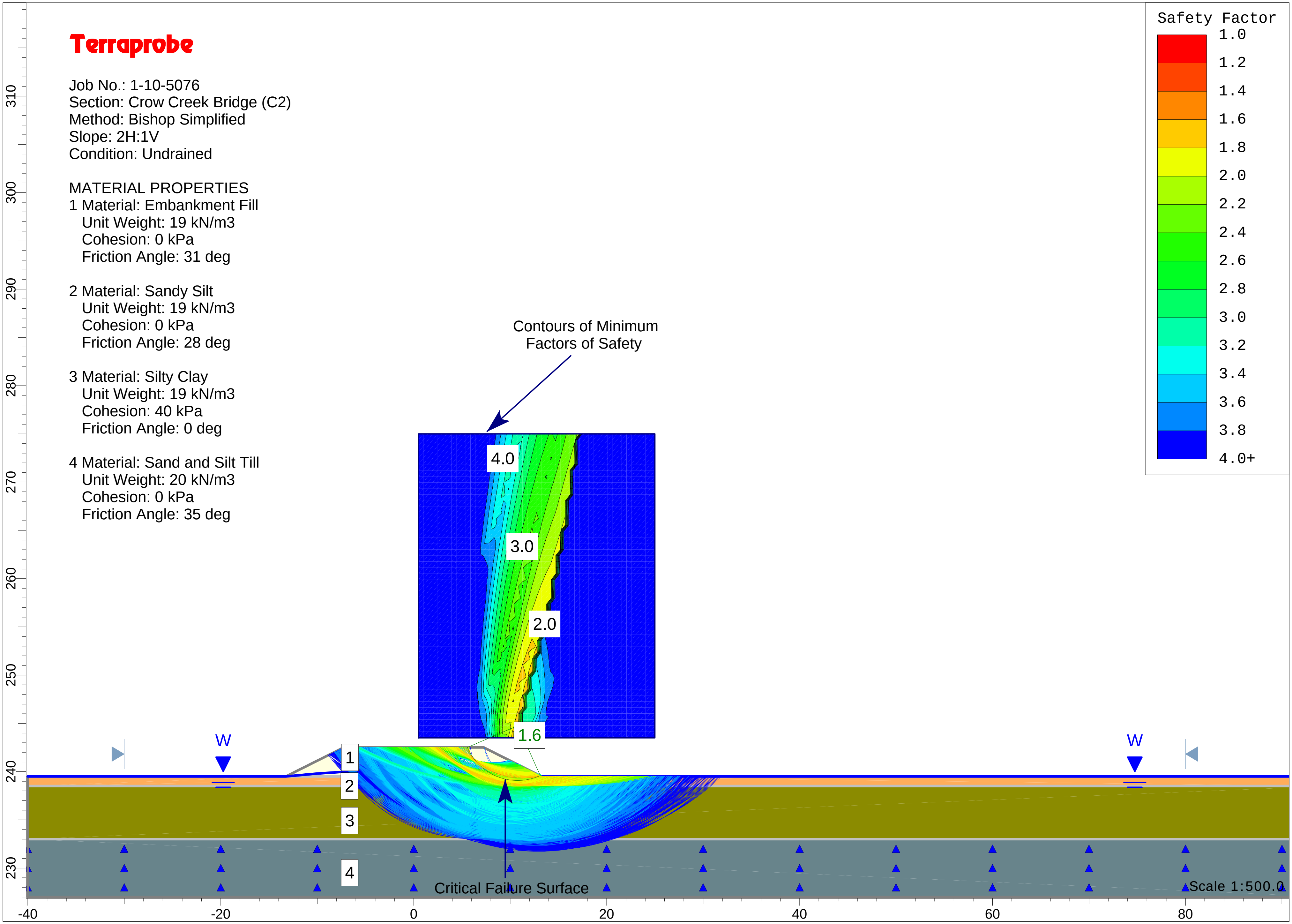
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Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sand and Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.0
1.6
Critical Failure Surface
W
W
1
2
3
4
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0+
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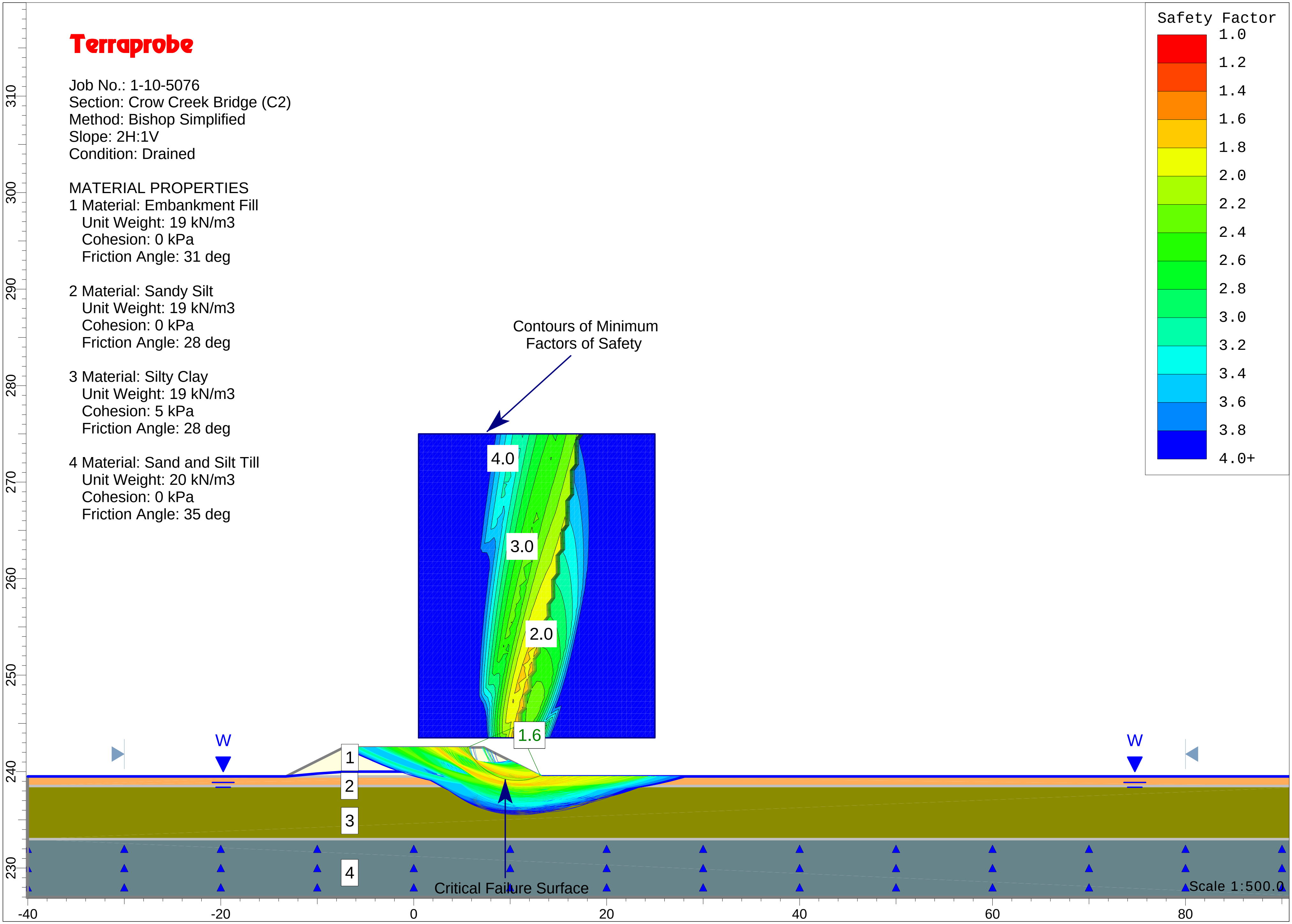

Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
4 Material: Sand and Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.0
1.6
Critical Failure Surface
W
W
1
2
3
4
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0+
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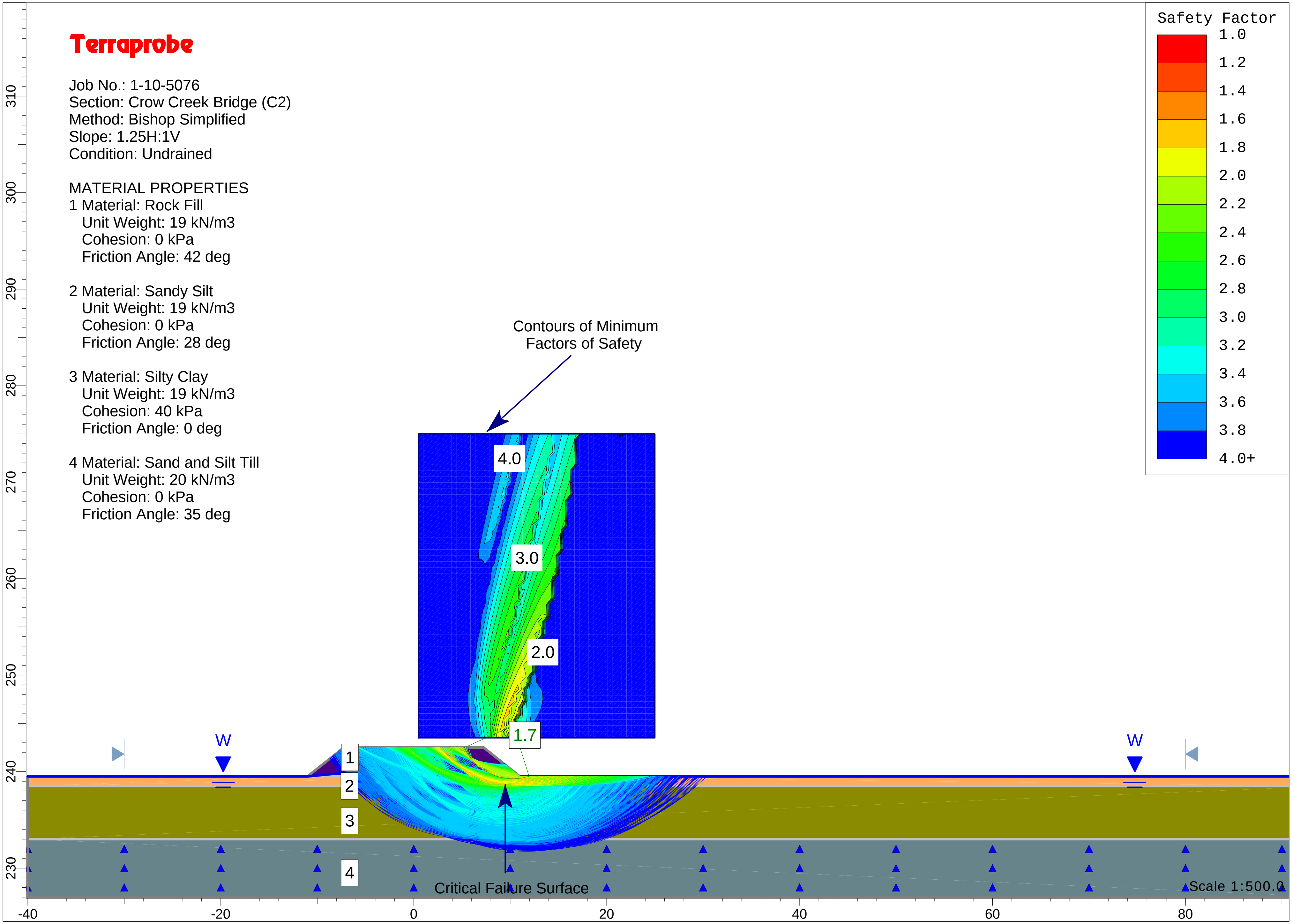

Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sand and Silt Till
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Cohesion: 0 kPa
Friction Angle: 35 deg
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0+
Contours of Minimum Factors of Safety
4.0
3.0
2.0
1.7
W
W
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Critical Failure Surface
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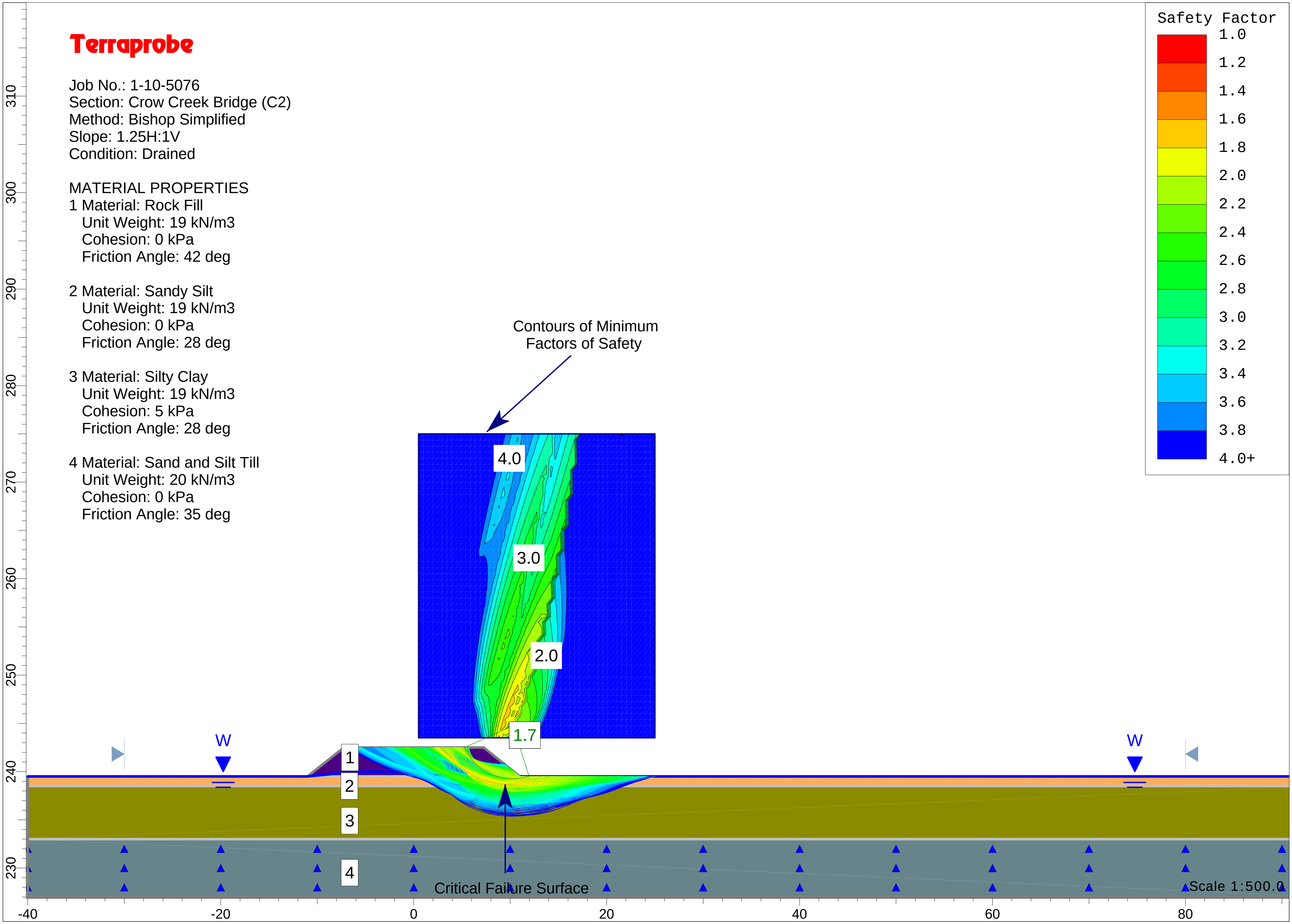
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
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Unit Weight: 19 kN/m3
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Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.0
1.7
Critical Failure Surface
Safety Factor
1.0
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1.4
1.6
1.8
2.0
2.2
2.4
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2.8
3.0
3.2
3.4
3.6
3.8
4.0+
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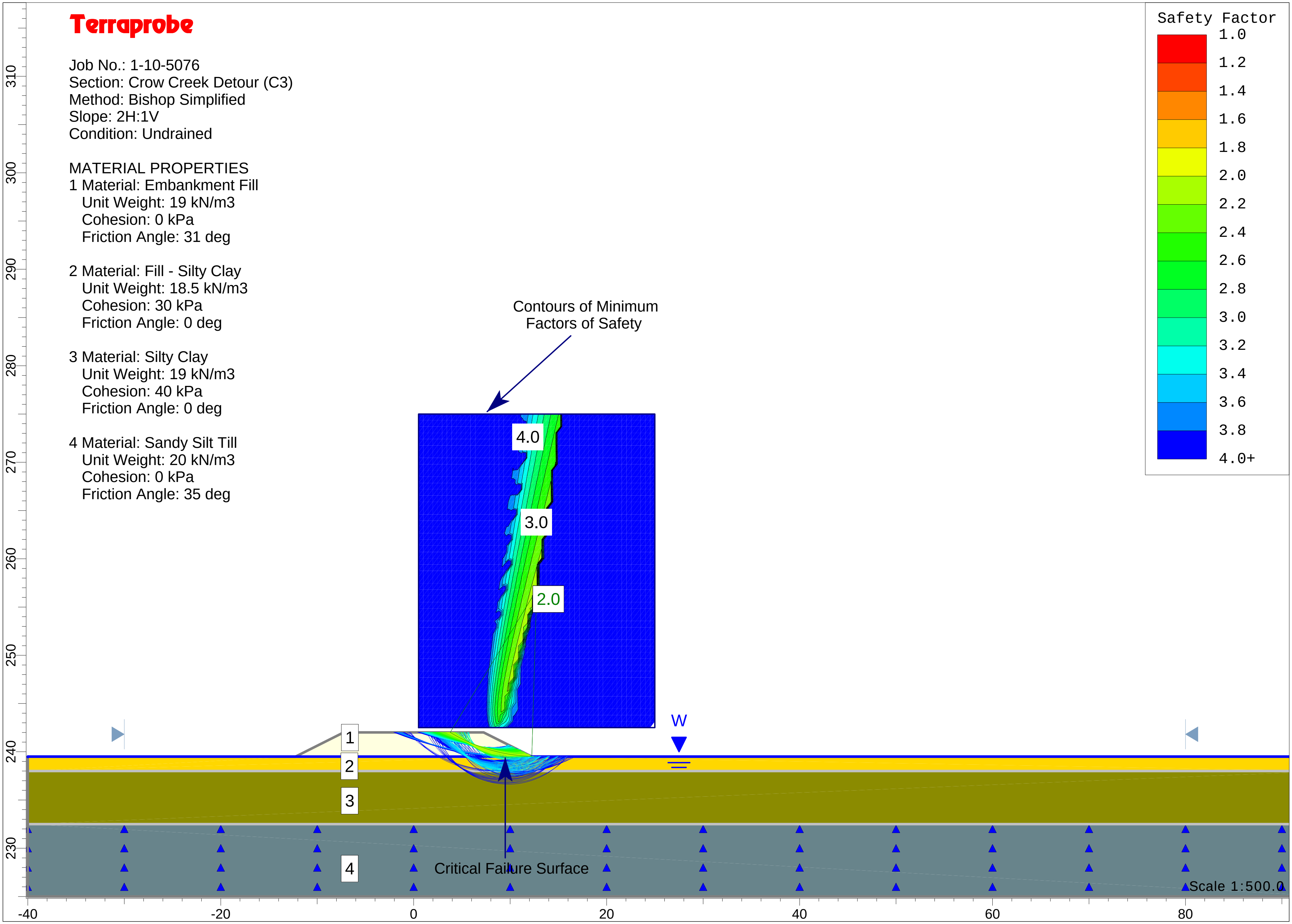
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 30 kPa
Friction Angle: 0 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.0
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
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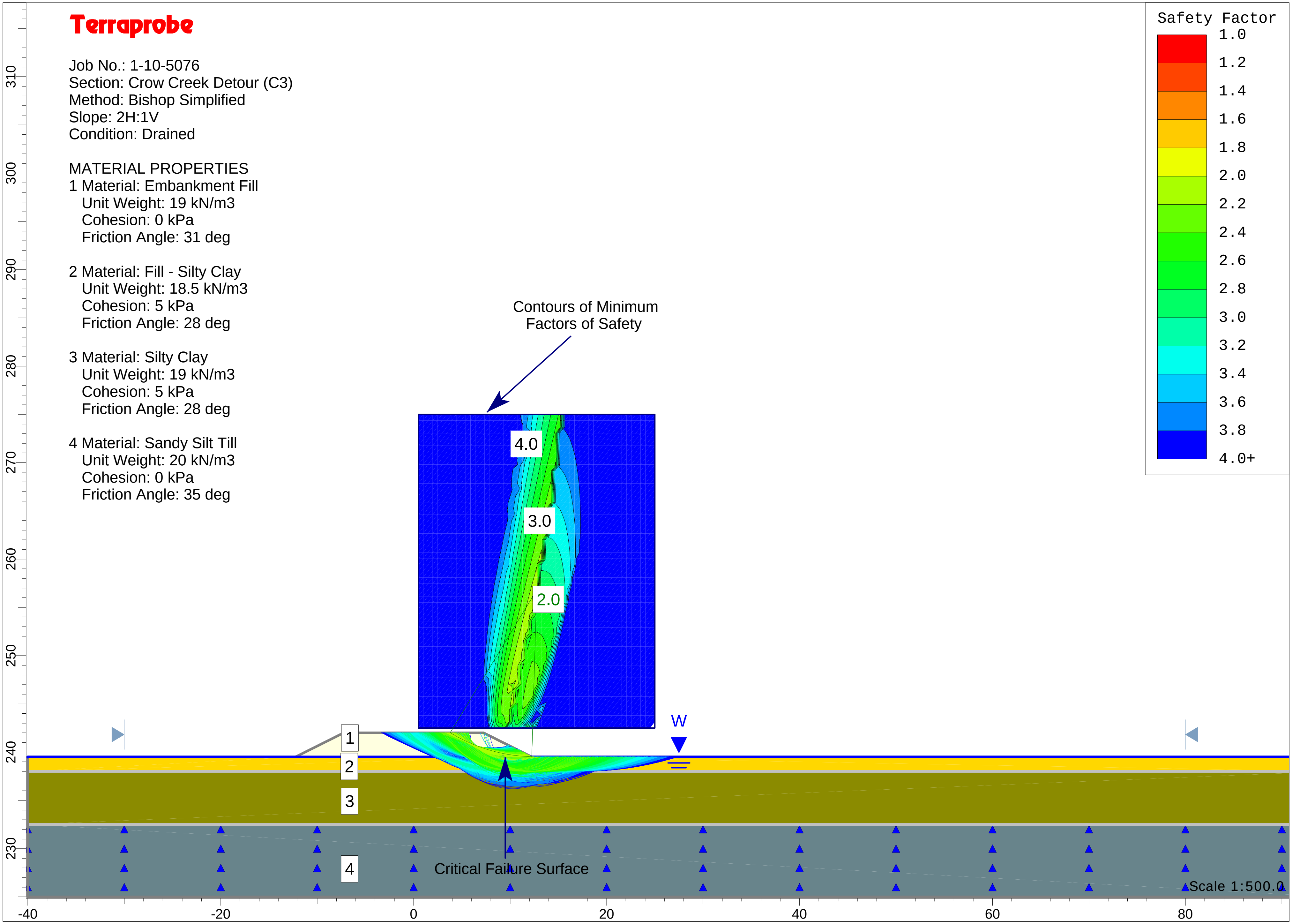
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum
Factors of Safety
4.0
3.0
2.0
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
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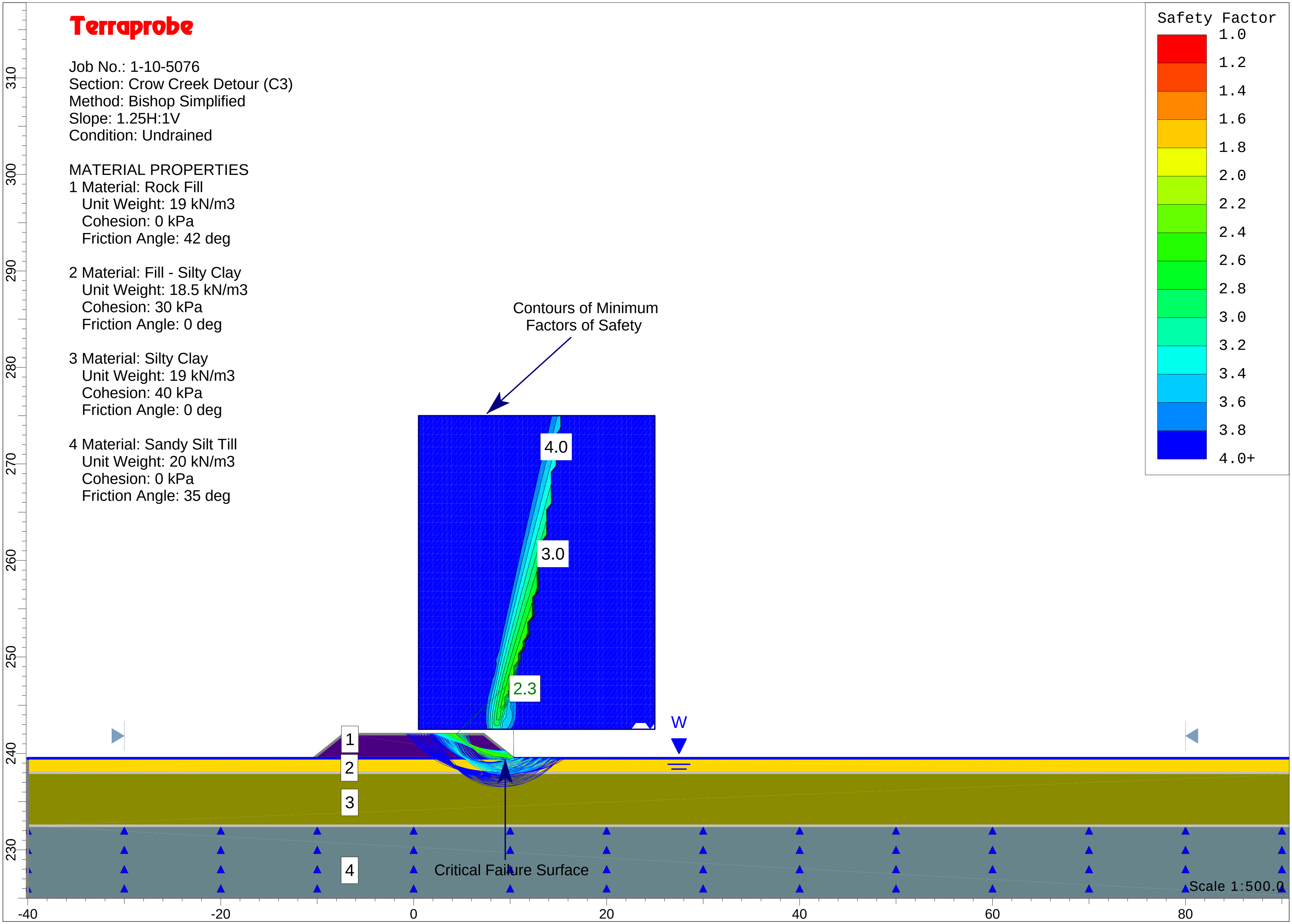
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Undrained
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1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 30 kPa
Friction Angle: 0 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.3
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0+
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290
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-40
-20
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60
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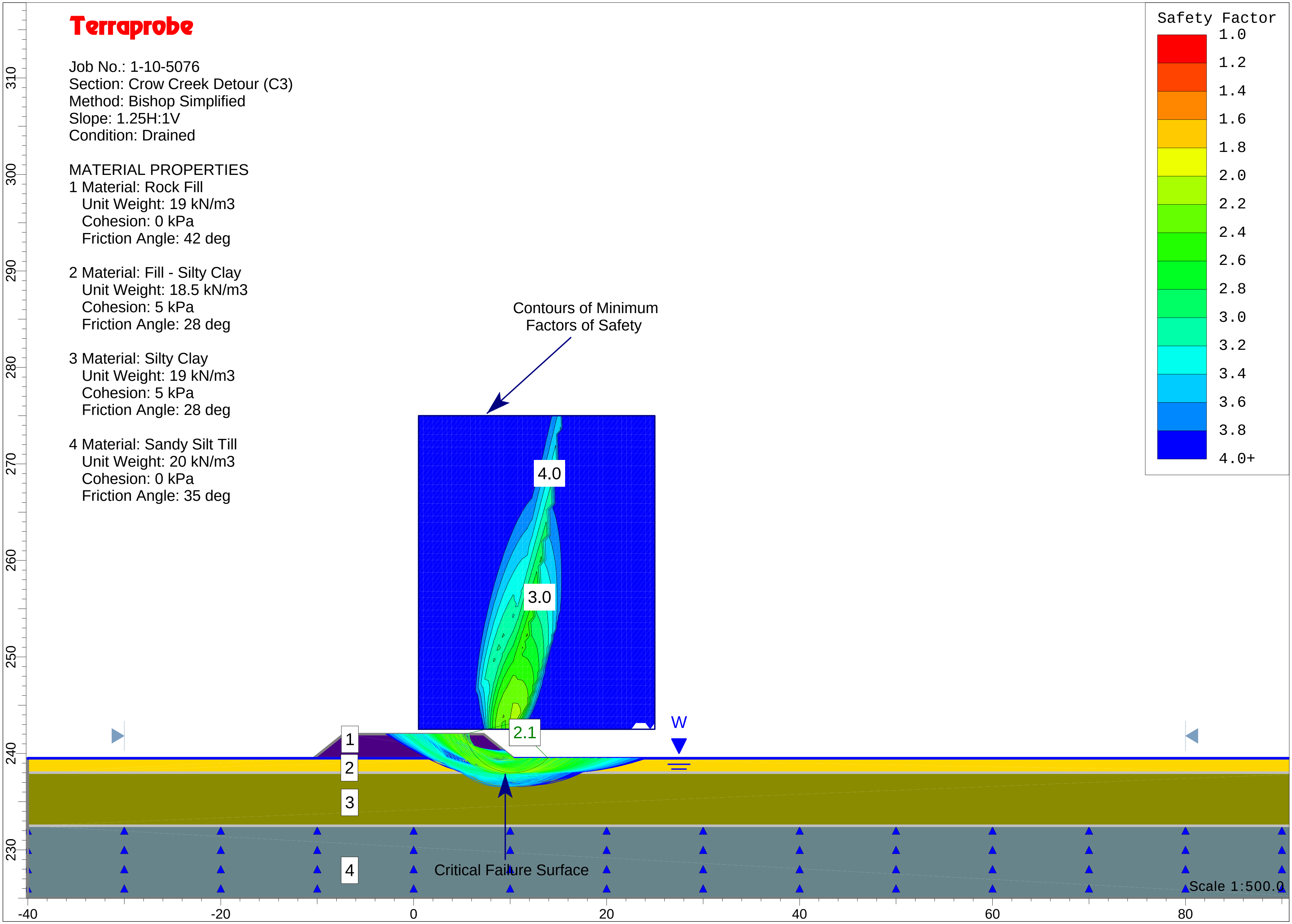
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.1
W
1
2
3
4
Critical Failure Surface
Safety Factor
1.0
1.2
1.4
1.6
1.8
2.0
2.2
2.4
2.6
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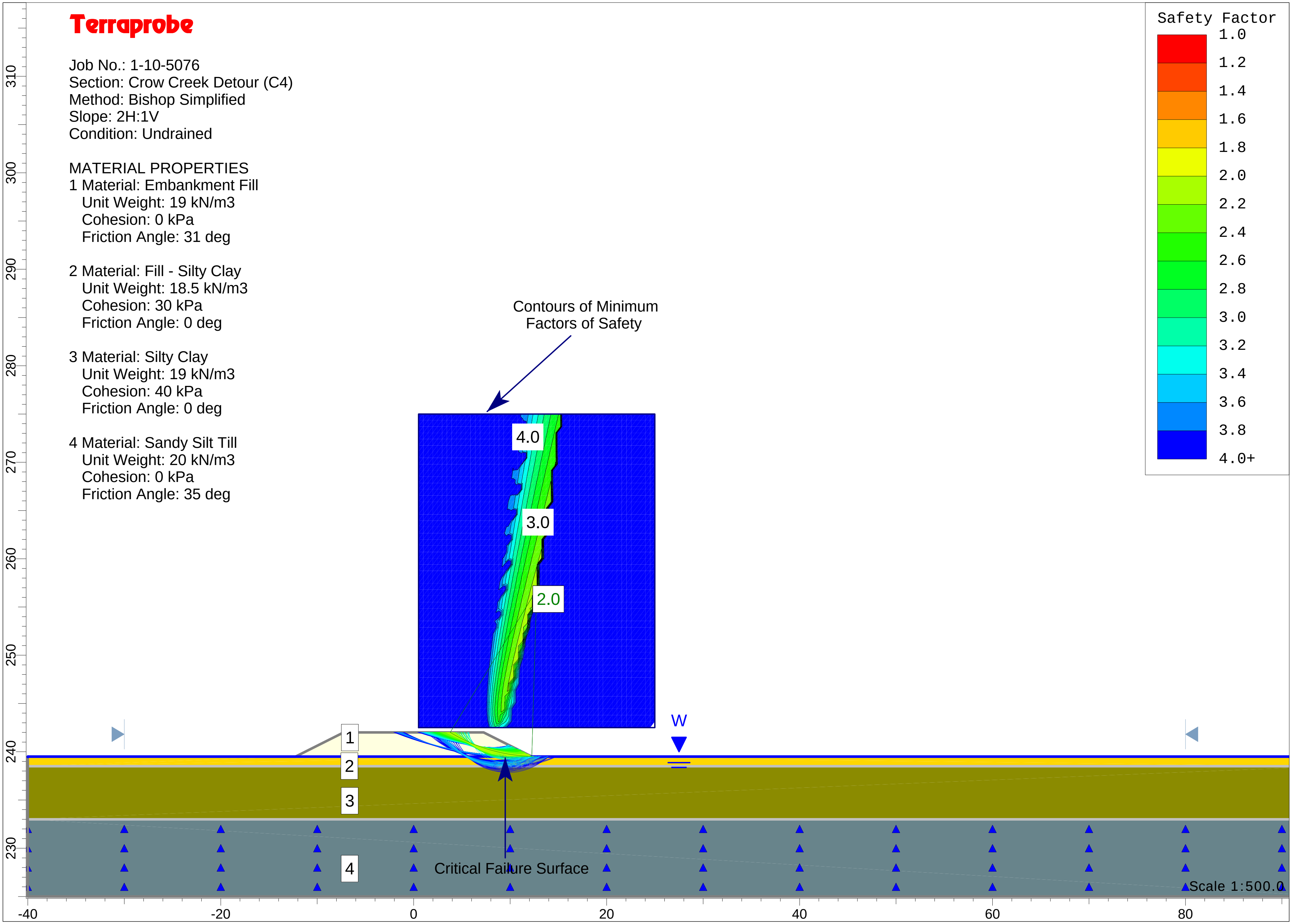
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C4)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 30 kPa
Friction Angle: 0 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum
Factors of Safety
4.0
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Critical Failure Surface
Scale 1:500.0
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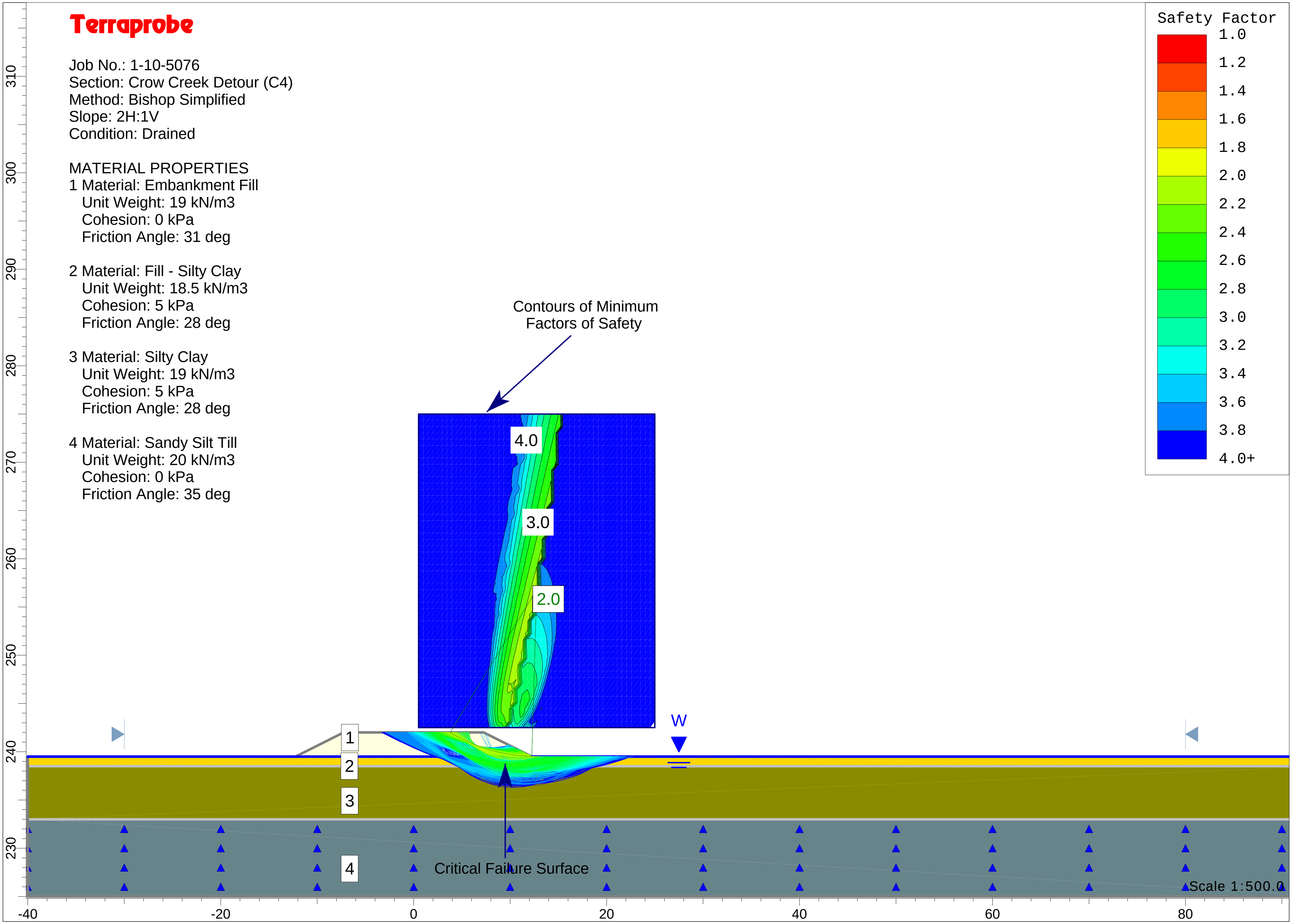

Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C4)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
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Critical Failure Surface
Scale 1:500.0
Safety Factor
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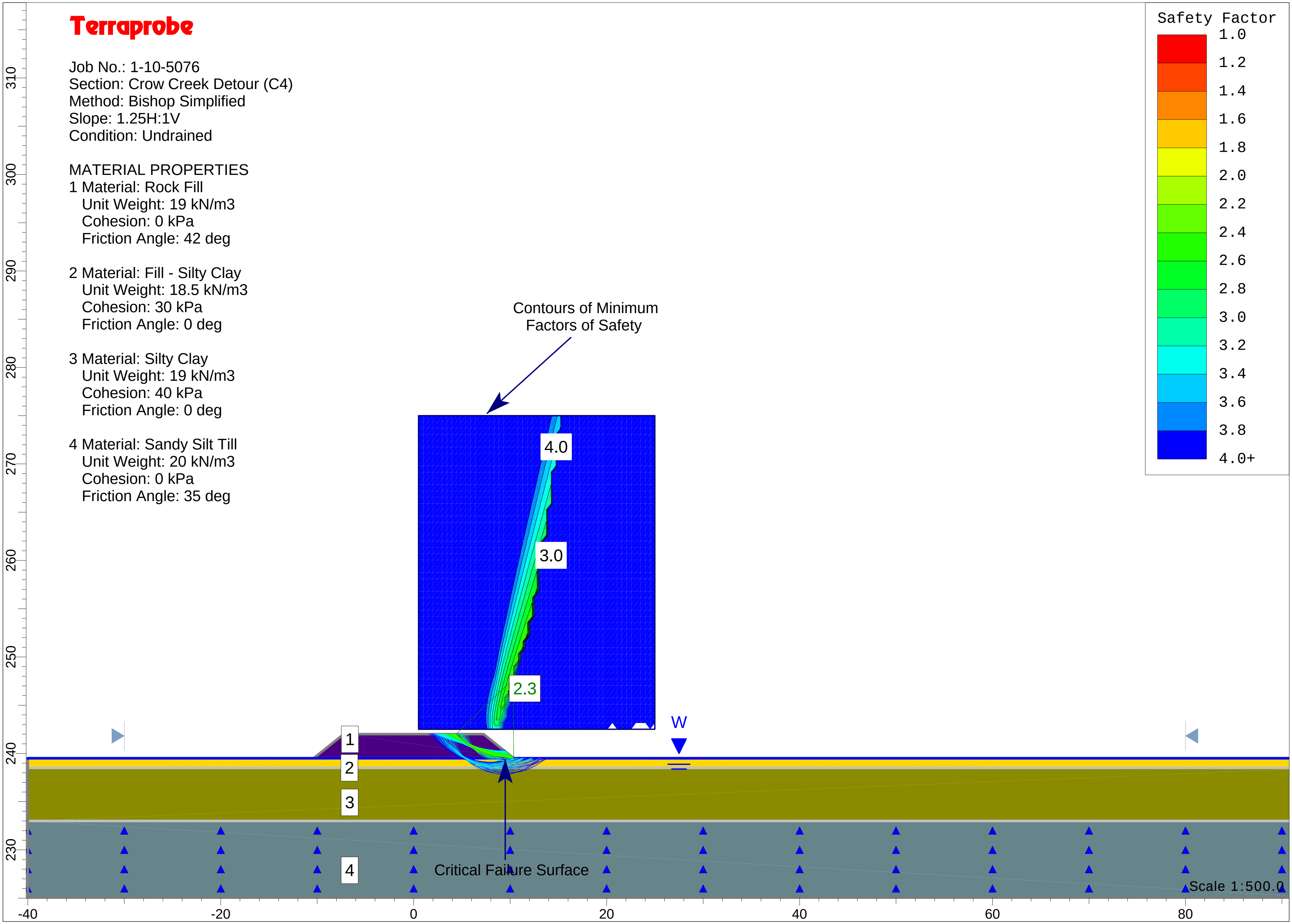
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C4)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 30 kPa
Friction Angle: 0 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
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Critical Failure Surface
Scale 1:500.0
Safety Factor
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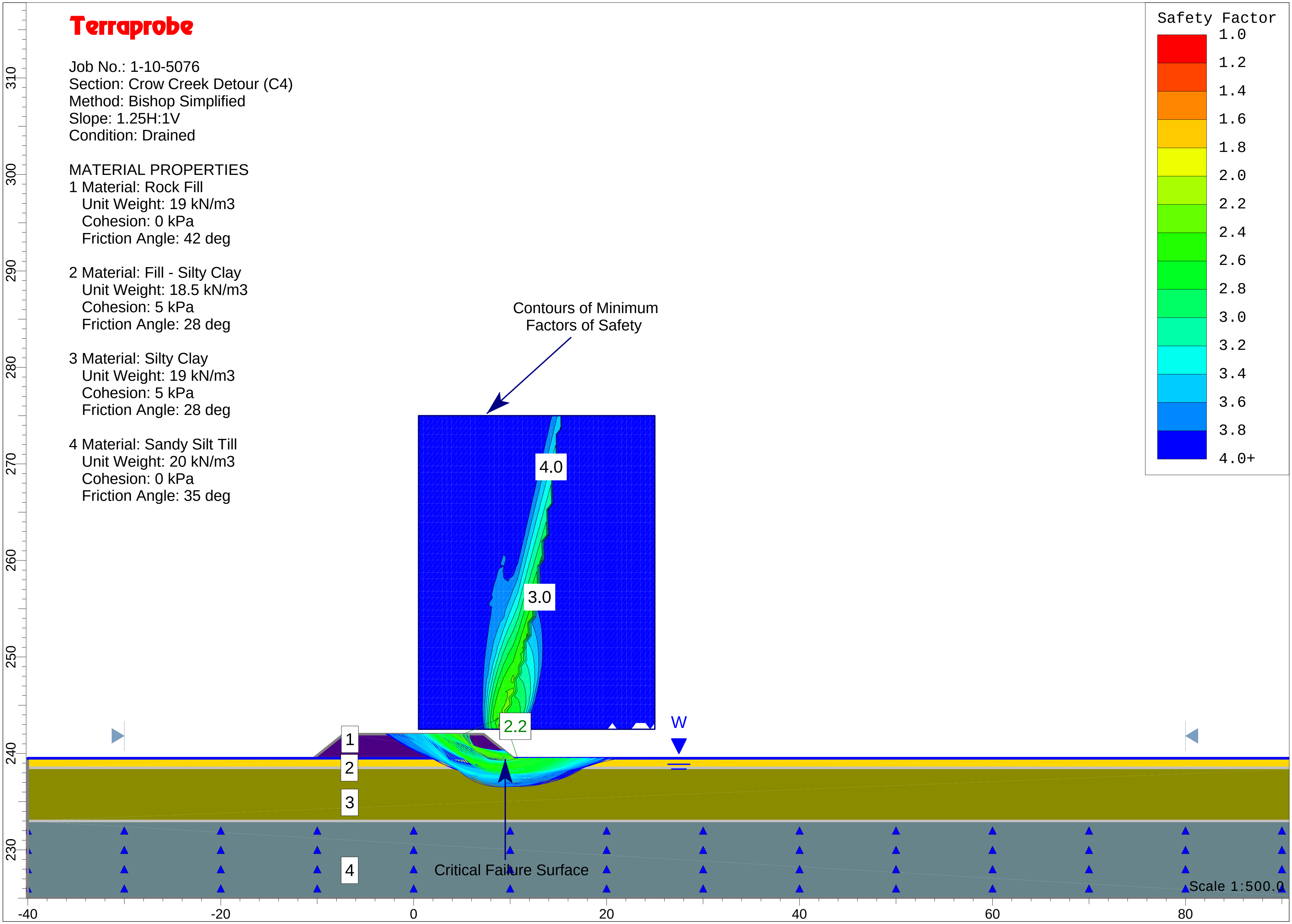
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C4)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 5 kPa
Friction Angle: 28 deg
4 Material: Sandy Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
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Critical Failure Surface
Safety Factor
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Scale 1:500.0
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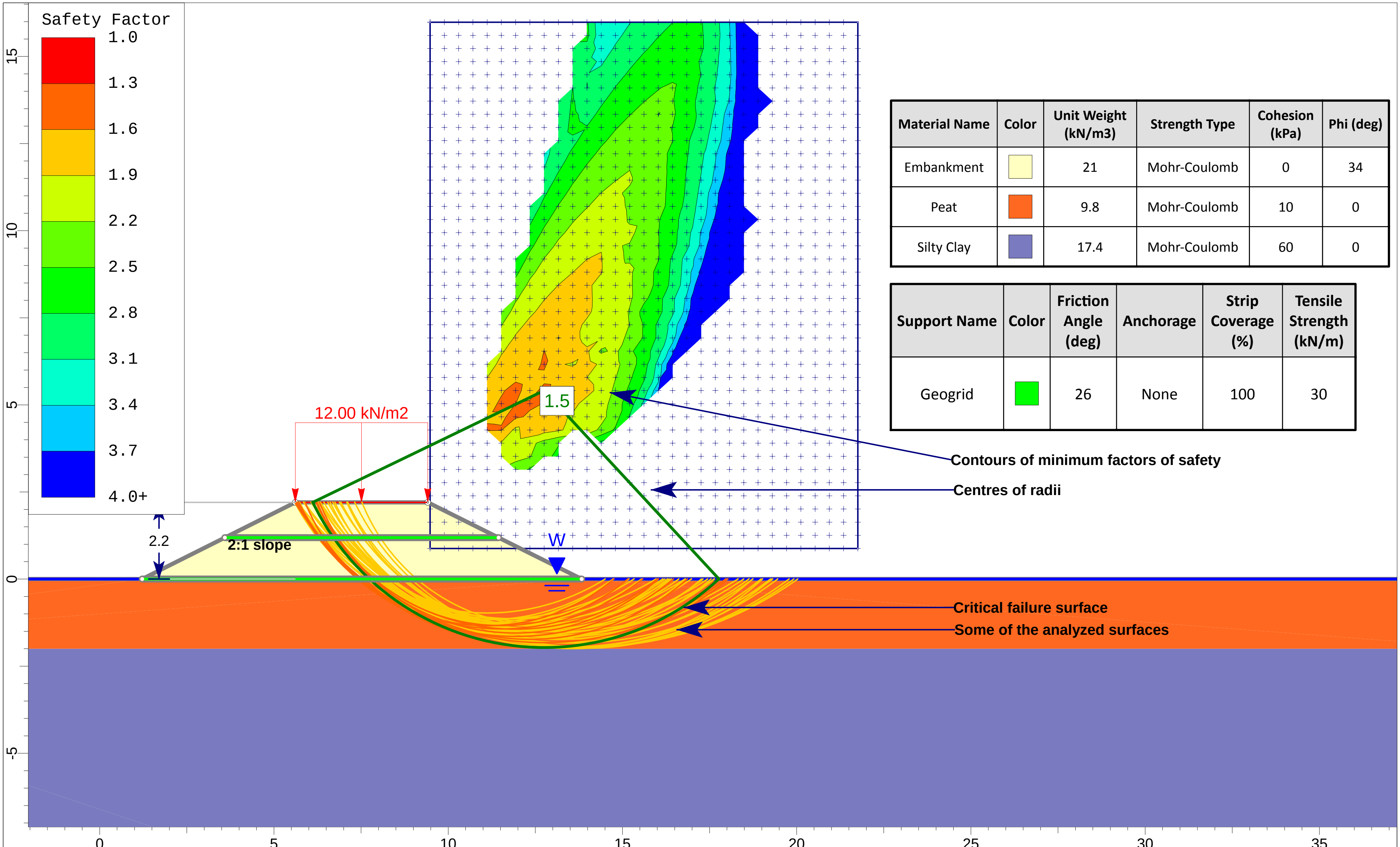
APPENDIX F

TERRAPROBE INC.



Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Embankment		21	Mohr-Coulomb	0	34
Peat		9.8	Mohr-Coulomb	10	0
Silty Clay		17.4	Mohr-Coulomb	60	0

Support Name	Color	Friction Angle (deg)	Anchorage	Strip Coverage (%)	Tensile Strength (kN/m)
Geogrid		26	None	100	30

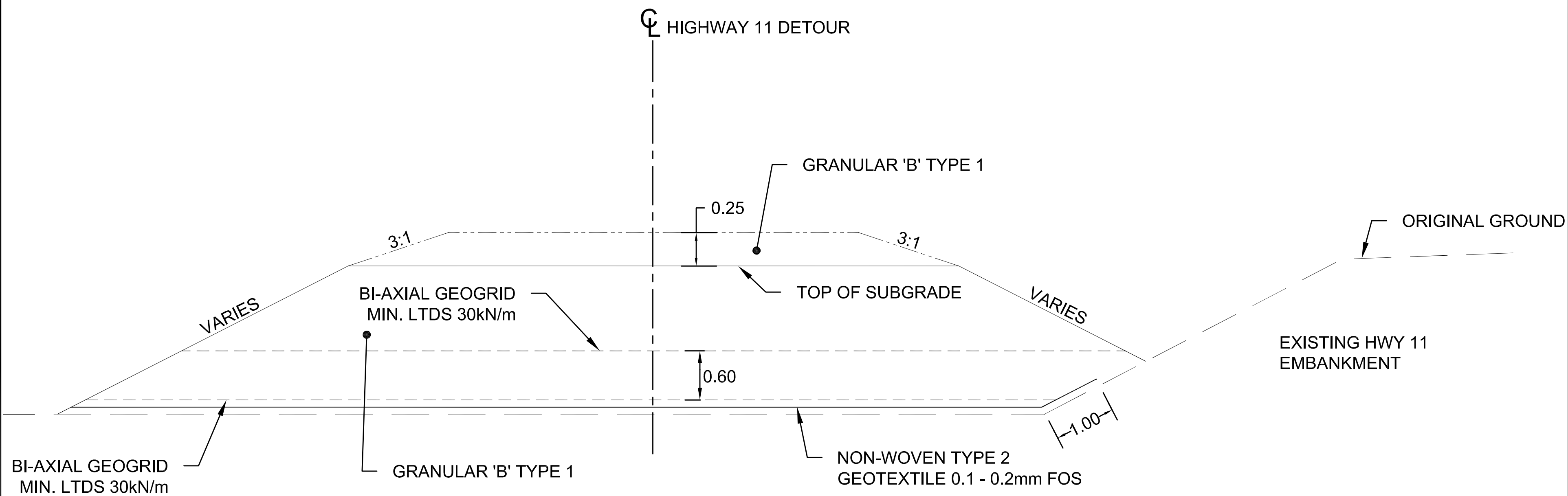
Terraprobe
Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

Project	**11-10-5076 Crow Bridge Temporary Bypass**
Analysis Description	2 to 1 side slopes, undrained analysis, two rows of geotextile
Drawn By MD	*Scale* 1:150
Reference	n/a
Date	04/16/12
File Name	11-10-5076 crow bypass - 2to1 undrained 2 rows geotext.slim

SLIDEINTERPRET 6.015

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: CROW CREEK GEOTEXTILE/GEOGRID DETAIL PRELOAD IN FILL	FIGURE : G1
	File No.: 11-10-5076	

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.

Special Provision

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.

4.0 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.

5.0 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

6.0 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.