Terraprobe

Consulting Geotechnical & Environmental Engineering
Construction Materials Inspection & Testing

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
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 probable detour structure are proposed. The site is located 3.7 km west of Lowther in the Township of McCrea; District of Cochrane North, 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 Corporation, under the Ministry of Transportation Ontario (MTO) Northeastern Region Assignment Number 5009-E-0020.

2 SITE DESCRIPTION & PHYSIOGRAPHY

Highway 11 crosses Crow Creek via a 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 were carried out between July 27 and August 06, 2010 and consisted of drilling and sampling four boreholes to depths ranging from 25.9 m to 31.8 m. Two boreholes (C1 & C2) were drilled at the existing bridge site and two boreholes (C3 & C4) were drilled in the vicinity of the potential detour alignment. The boreholes were numbered C1 to C4 inclusive and their approximate locations 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 (clayey) 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 were also advanced into bedrock by NQ size diamond coring techniques.

Ground water conditions in the open boreholes were observed throughout the drilling operations. The boreholes were also instrumented with a standpipe piezometer consisting of 25 mm diameter PVC pipe with a slotted screen enclosed in sand to permit longer term ground water level monitoring.

The locations and completion details of the piezometers are outlined in Table 3.1. All of the boreholes are being maintained in accordance with MOE Reg 128/03 and its amendments.

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.

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

4 LABORATORY TESTING

The recovered soil samples were subjected to Visual Identification (VI) and natural moisture content determination. Selected samples were also subjected to gradation analysis and Atterberg Limits tests. The results of the soils testing program are shown on the Record of Borehole sheets in Appendix A. The grain size distribution curves and plasticity charts are included 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 this appendix and on the "Borehole Locations and Soil Strata" drawings in Appendix C. An overall description of the stratigraphy is given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

5.1 Existing Bridge Site (Boreholes C1 & C2)

In general, the site is underlain by a flexible pavement (asphalt and sand and gravel), sand fill and native deposits of sandy silt and sand and silt, silty clay, sand and silt till and clayey silt till. These overburden soils are further underlain by bedrock consisting of metamorphic phyllite and igneous granitoid.

5.1.1 Flexible Pavement

A flexible pavement comprising of 150 mm of asphalt underlain by a layer of sand and gravel ranging in thickness from 150 mm to 170 mm was encountered. This granular layer extends to an elevation of 241.3 m below ground surface and is inferred to be in a compact state.

5.1.2 Fill – Sand

Fill material consisting of sand, trace silt, trace gravel was encountered at this site extending to a depth of 2.1 m (Elev. 239.5 m) below ground surface.

The grain size distribution plots of tested samples of the sand fill are presented in Figure B1-1. These results show a grain size distribution consisting of 0-5% gravel, 87-94% sand and 6-8% silt and clay size particles.

Standard Penetration tests in this layer gave 'N' values that ranged from 7 to 29 blows for 0.3 m. Based on these results the fill is considered to have a loose to compact relative density. The moisture content of samples of this fill ranged from 3% to 14% by weight.

5.1.3 Sandy Silt to Sand and Silt

A deposit ranging in composition from sandy silt to sand and silt was encountered in both boreholes extending to depths of 2.9 m (Elev. 238.7 m) and 3.7 m (Elev. 237.9 m) below ground surface.

Samples retrieved from this deposit were subjected to grain size distribution tests and the results are illustrated in Figure B1-2. These results show a grain size distribution consisting of 0% gravel, 23-44% sand, 44-67% silt and 10-12% clay size particles.

The blow counts from Standard Penetration tests conducted in this deposit ranged from 4 to 6 blows per 0.3 m penetration and based on these results the deposit is considered to have a loose relative density. The moisture content of samples from this stratum ranged from 17% to 19% by weight.

5.1.4 Silty Clay

A silty clay deposit was encountered at the site extending to depths ranging from 8.7 m (Elev. 232.9 m) to 9.0 m (Elev. 232.6 m) below ground surface.

The grain size distribution curves of tested samples of the silty clay are presented in Figure B1-3. These results show a grain size distribution consisting of 0-3% gravel, 1-15% sand, 55-75% silt and 24-40% clay size particles.

Samples were also subjected to Atterberg Limits tests and the results are illustrated on the plasticity chart, Figure B1-4. The index values from these tests are summarized below:

Liquid Limit:	23-29%
Plastic Limit:	12-21%
Plasticity Index:	7-11%
Natural Moisture Content:	12-38%

These values indicate low plasticity silty clay soils.

Standard Penetration tests in this stratum gave ‘N’ values that ranged from 2 to 14 blows for 0.3 m penetration. Field vane tests gave in-situ undrained shear strengths ranging from 36 kPa to in excess of 100 kPa. These values indicate that the consistency of the silty clay is generally firm to stiff with infrequent soft zones. The moisture content of samples of the silty clay ranged from 12% to 38% by weight.

5.1.5 Sand and Silt Till

A deposit of sand and silt till was encountered across this site. This deposit extends to depths ranging from 14.7 m to 17.6 m below ground surface or to elevations ranging from 226.9 m to 224.0 m.

The results of grain size distribution tests conducted on samples obtained from this till deposit are illustrated in Figure B1-5. These results show grain size distributions consisting of 6-17% gravel, 42-50% sand, 35-45% silt and 4-7% clay size particles. The field investigations also confirm the presence of random cobble and boulder inclusions in this soil matrix.

Standard Penetration tests in this deposit gave ‘N’ values that ranged from 24 to more than 100 blows per 0.3 m penetration indicating a compact to very dense relative density. The moisture content of samples from this stratum ranged from 1% to 11% by weight.

5.1.6 Clayey Silt Till

A clayey silt till deposit was encountered at the site overlying the bedrock surface. This deposit extends 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 B1-6. These results show a grain size distribution consisting of 1-9% gravel, 20-36% sand, 44-64% silt and 11-23% clay size particles. The presence of random cobble and boulder inclusions was also confirmed in this deposit by the field investigations.

Samples of the clayey silt till were also subjected to Atterberg Limits tests and the results are presented in Figure B1-7. The index values from these tests are summarized below:

Liquid Limit:	16-22%
Plastic Limit:	11-17%
Plasticity Index:	4-10%
Natural Moisture Content:	8-12%

These values indicate that the till generally consists of low plasticity clayey silt soils with occasional silty clay inclusions.

Standard Penetration tests in the clayey silt till yielded ‘N’ values ranging from 85 to more than 100 blows for 0.3 m penetration indicating a hard consistency. Moisture contents of samples of the clayey silt till range from 7% to 12% by weight.

5.1.7 Bedrock

The overburden soils described above are underlain by metamorphic phyllite and igneous granitoid bedrock. Bedrock was proved by coring in both boreholes 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

The phyllite bedrock is described as unweathered with sub-vertical foliations and its colour is generally grey. Total core recovery in this bedrock ranged from 91% to 96% and the RQD values ranged from 61% to 88%. Based on these results the rock quality is considered to be fair to good.

The granitoid bedrock is described as unweathered and its colour is bluish white. Total core recovery in this bedrock ranged from 72% to 100% and the RQD values generally ranged from 62% to 90% with an RQD of 25% in the upper run. Based on these results the rock quality is considered to be generally fair to good. The bedrock in the upper run is poor quality rock.

5.2 Detour Alignment (Boreholes C3 & C4)

In general, the site is underlain by topsoil, silty clay fill and native deposits of silty clay, sandy silt till and clayey silt till. These overburden soils are further underlain by bedrock consisting of phyllite.

5.2.1 Topsoil

Topsoil ranging from 200 mm to 300 mm thick was encountered at this site. Topsoil thickness may vary between and beyond the boreholes.

5.2.2 Fill – Silty Clay

Fill material consisting of silty clay and peat was encountered at this site extending to depths ranging from 1.4 m (Elev. 238.6 m) to 2.1 m (Elev. 237.7 m) below ground surface.

The grain size distribution curve of a sample of this fill is depicted 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 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%
Natural Moisture Content:	31%

These values are characteristic of organic soils.

Standard Penetration tests in this fill material gave ‘N’ values that ranged from 5 to 8 blows for 0.3 m penetration indicating a firm consistency. The moisture content of samples of this fill ranged from 28% to 76% by weight.

5.2.3 Silty Clay

A native silty clay deposit was encountered in both boreholes extending to depths of 7.1 m below ground surface or to elevations of 232.7 m and 232.9 m.

The grain size distribution plots of tested samples of the silty clay are presented in Figure B2-3. These results show a grain size distribution consisting of 0-1% gravel, 2-14% sand, 62-66% silt and 23-31% clay size particles.

Samples were also subjected to Atterberg Limits tests and the results are illustrated on the plasticity chart, Figure B2-4. The index values from these tests are summarized below:

Liquid Limit:	23-37%
Plastic Limit:	14-19%
Plasticity Index:	7-18%
Natural Moisture Content:	17-24%

These values are characteristic of clayey soils of low to intermediate plasticity.

Standard Penetration tests in this stratum 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 40 kPa to in excess of 100 kPa. Based on these results the silty clay has a firm to stiff consistency. The moisture content of samples of the silty clay ranged from 17% to 30% by weight.

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 results of grain size distribution tests conducted on samples of this till are illustrated in Figure B2-5. These results show grain size distributions of 5-16% gravel, 32-33% sand, 41-55% silt and 7-11% clay size particles. The field investigations also indicated that the matrix of this till contains random cobble and boulder inclusions.

Standard Penetration tests in this deposit gave 'N' values that ranged from 42 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 8% to 12% by weight.

5.2.5 Clayey Silt Till

Clayey silt till was encountered at the site overlying the bedrock surface. This deposit extends 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 B2-6. 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 field investigations also confirm the presence of random cobble and boulder inclusions in this deposit.

Samples of the clayey silt till were also subjected to Atterberg Limits tests and the results are presented in Figure B2-7. 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.

Standard Penetration tests in the clayey silt till yielded ‘N’ values of more than 100 blows for 0.3 m penetration indicating a hard consistency. Moisture contents of samples of the clayey silt till range from 7% to 15% by weight.

5.2.6 Bedrock

The overburden soils described above are underlain by metamorphic phyllite bedrock. Bedrock was proved by coring in both boreholes and the bedrock depth and top of bedrock elevations are summarized in Table 5.2.

Table 5.2 – 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 is 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 is unweathered and its 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 5.3.

Table 5.3 – Water Level Measurements

Borehole	Date	Water Levels	
		Depth (m)	Elevation (m)
Existing Bridge Site			
C1	August 06, 2010	0.2	241.4
	August 10, 2010	0.9	240.7
	September 03, 2010	0.9	240.7
C2	August 06, 2010	0.7	240.9
	August 10, 2010	0.7	240.9
	September 03, 2010	0.7	240.9
Detour Alignment			
C3	August 06, 2010	0.8(*ag)	240.6
	August 10, 2010	1.0(*ag)	240.8
	September 03, 2010	1.2(*ag)	241.0
C4	August 10, 2010	1.1(*ag)	241.1
	September 03, 2010	1.6(*ag)	241.6

*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 indicate the presence of excess hydrostatic pressure at depth in the underlying soils since the piezometric water levels are higher than the ground surface of the flood plain.

At the existing bridge the piezometric head is estimated to range between Elev. ±240.7 m and Elev. ±240.9 m. Along the detour alignment the recorded water levels are 1.2 m to 1.6 m higher than ground surface and the piezometric head ranges between Elev. ±241.0 and Elev. ±241.6 m.

All groundwater observations at this site are short term and the levels are expected to fluctuate seasonally and after severe weather events. The ground water level will also be controlled by the free water level in the creek.

5.4 Miscellaneous

The borehole locations were marked in the field by surveyors from McCormick Rankin Corporation 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 were conducted with a track mounted drill rig owned and operated by Landcore Drilling of Chelmsford, Ontario.

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

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 Rehman Abdul, P.Eng. and reviewed by Michael Tanos, P.Eng.

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GEOCRES No. 42G-33
PART 2: ENGINEERING DISCUSSION AND RECOMMENDATIONS

6 GENERAL

This report presents interpretation of the geotechnical data in the factual report and provides preliminary geotechnical design recommendations for a replacement bridge and a probable detour structure at Crow Creek located 3.7 km west of Lowther in the Township of McCrea; District of Cochrane North, Ontario.

At this site there is 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 and consequently a number of alternatives will be considered and evaluated as part of the preliminary design process.

Some of the alternatives that are being considered are:

- Undertake staged construction on Highway 11 while maintaining traffic by minor shifts to the existing alignment.
- Temporarily divert Highway 11 traffic via a detour structure, replace the existing bridge then remove the detour.
- Permanently divert Highway 11 on a new alignment and remove the existing bridge.

The discussion and recommendations presented in this report are preliminary and are based on our understanding of the project and on the limited factual data obtained in the course of the investigations. These recommendations are for planning purposes only and further investigations will be required for detail design.

7 STRUCTURE FOUNDATIONS

Existing Bridge Site (Boreholes C1 & C2)

The stratigraphy encountered at this site consists of a flexible pavement (asphalt and sand and gravel), sand fill and native deposits of sandy silt and sand and silt, silty clay, sand and silt till and clayey silt till. These overburden soils extend to depths of 22.5 m (Elev. 219.1 m) and 28 m (Elev. 213.6 m) and are further underlain by bedrock consisting of metamorphic phyllite and igneous granitoid. The ground water level at this site is estimated to be at the flood plain level i.e. Elev. 239.5 m for design purposes. Excess hydrostatic pressure exists at depth in the underlying soils and its piezometric head is estimated to range between Elev. ±240.7 m and Elev. ±240.9 m.

Detour Alignment (Boreholes C3 & C4)

The stratigraphy encountered along this alignment consists of topsoil, silty clay fill and native deposits of silty clay, sandy silt till and clayey silt till. These overburden soils extend to depths of 25.4 m (Elev. 214.6 m) and 28.2 m (Elev. 211.6 m) and are further underlain by bedrock consisting of metamorphic phyllite. The ground water level at this site is estimated to be at the flood plain level i.e. Elev. 239.5 m for design purposes. Excess hydrostatic pressure exists at depth in the underlying soils since the piezometric head is 1.2 m (Elev. 241.0 m) to 1.6 m (Elev. 241.6m) higher than 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

Spread footings are not considered to be a practical option for supporting the bridge. The geotechnical resistance of the underlying soils are low and foundation settlements will be high. Consequently, spread footings on native ground are not practical and are not 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. Therefore, this option is not a feasible and practical solution.

7.2 Augered Caissons (Drilled Shafts)

Augered caisson foundations were also considered for supporting the structure. 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 units. 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 caisson 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 are not recommended for supporting the structure.

7.3 Driven Piles

The subsurface conditions at the site are 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.

Steel tube piles were considered but were excluded due to the presence of cobbles and boulders in the till soils which would make it very difficult and impractical to drive these “high displacement” piles to the required penetration and capacity. 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

Two steel pile sections have been considered for use in the proposed foundations. Piles driven at the abutment locations and encountering effective refusal in the very dense sand and silt till, sandy silt till or the hard clayey 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 Various 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
East Abutment	C2	227.5±	Sand and Silt Till		
Detour Alignment					
West Abutment	C3	227.0±	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
East Abutment	C2	223.0±	Clayey Silt Till		
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 ± 1 m. However, to accommodate the integral abutment construction a 3.0 m long CSP will surround the pile in the silty clay to clayey silt stratum. Consequently, downdrag forces on the piles due to embankment reconstruction and the grade raise will be minimal.

Along the detour alignment, embankment construction will cause settlement of the underlying soils thereby imparting downdrag forces on piles that are installed before the embankments are constructed. Downdrag forces on piles were estimated based on compressible silty clay soils that extend to Elev. 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 preliminary design purposes.

Further investigations will be required at the detail design stage to assess the engineering properties of the silty clay deposits and provide refined estimates of the magnitude of downdrag forces.

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 . z / D [cohesionless soils] (kN/m^3)
k_s = 67 S_u/D [cohesive soils] (kN/m^3)
p_{ult} = 3 . γ . z . K_p [cohesionless soils] (kPa)
p_{ult} = 9 S_u [cohesive soils] (kPa)

where z = depth of embedment of pile (m)
D = pile width (m)
S_u = undrained shear strength (Table 7.3) (kPa)
n_h = coefficient of horizontal subgrade reaction (Table 7.3) (kN/m^3)
γ = unit weight (Table 7.3) (kN/m^3)
K_p = passive earth pressure coefficient

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

The spring constant, K, for analysis may be obtained by the expression, K = k_s x L x 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} x L x D.

Table 7.3 – Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m^3)*
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	–
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	–
Detour Alignment						
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.

7.4 Recommended Foundation

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

7.5 Frost Cover

Pile caps and footings should be provided with a minimum of 2.6 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)					
Fill – Sand	28	19	0.36	0.53	2.77
Sand and Silt to Sandy Silt	28	19	0.36	0.53	2.77
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 Alignment (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 could consist of a system of soldier piles and lagging. The soldier piles can be designed as cantilever structures or supported by employing a soil anchor system depending on the depth of soil to be retained and the performance criteria. Due to the very dense nature of the sandy silt till and the presence of cobbles and boulders, pre-augering will likely be required in order to install the piles.

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.
- Sandy Silt to Sand and Silt – Type 4 soils below the water table.
- 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 Elev. 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 soils at depth since the piezometric water levels are higher than the ground surface of the flood plain. However, excess hydrostatic pressure will not be encountered in shallow excavations extending into the underlying silty clay soils.

Excavations at the bridge site may extend into sandy silt and sand and silt soils below the ground water level. These soils will be easily disturbed by construction activity and will also yield water due to their relatively high permeability. To alleviate construction related problems we recommend that the ground water table be lowered and maintained at least 1 m below the base of the excavation. Vigorous dewatering techniques such as vacuum well pointing will be required for this undertaking.

Alternatively, (depending on the design elevations of the pile caps) it may be feasible to excavate these soils to expose the underlying more impermeable silty clay soils. For this scenario the excavation can be unwatered by installing a system of perimeter trenches designed to drain to filtered sumps from which pumping can be undertaken.

Along the detour alignment excavations will be made in relatively impermeable silty clay soils. It is anticipated that these excavations can be unwatered using a system of perimeter trenches designed to drain to filtered sumps from which pumping can be undertaken.

The pile driving operations will cause significant remoulding of the clay soils around the pile shafts thereby forming a watertight barrier that will prevent 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

11.1 Stability

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

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

Global stability analyses were conducted for 2H:1V earth fill embankments and for 1.25H:1V rock fill embankments and a target factor of safety of 1.3 was set. For earth or rock fill embankments up to 4.5 m high, factors of safety against global failure of 1.3 and greater were obtained for both long term and short term conditions. Therefore no embankment stability problems are expected. The slope stability models are included in Appendix E.

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

11.2 Settlement

At the existing bridge site the grade raise will be approximately ± 1 m based on working point elevations that range from Elev. 242.22 m to Elev. 242.41 m. At the detour bridge site working point elevations of Elev. 241.75 m suggest that approximately ± 2 m high approach embankments are required. The underlying silty clay soils at this site will therefore experience time dependent consolidation settlement due to the additional stress imposed by the embankments.

Based on limited laboratory test data, including the plasticity characteristics of the native soils, it is estimated that about 55 mm of total consolidation settlement of the silty clay soils will occur below ± 2 m high approach embankments. At the existing bridge site the estimated settlement due to a 1 m grade raise will be approximately ± 10 mm. Further investigations will be required at the detail design stage to assess the engineering properties of the silty clay deposits and determine the most likely range of settlements and their implications to embankment and bridge designs.

A maximum allowable post-construction settlement of about 25 mm would be considered acceptable for the approaches. For ± 2 m high approach embankments the estimated pre-construction settlement (about 30 mm) is likely to take up to 4 months to occur after which the remaining settlement will be equal to or less than 25 mm. Therefore other means/methods of accelerating settlement (wick drains, surcharging etc.) will not likely be required.

Approach embankments comprised of local earth fill will also settle during construction (fill compression) and this settlement is expected to be about 1% of the fill height. This settlement should be immediate in nature and essentially be complete shortly after construction is complete. For rock fill, compression is expected to be 0.5% of fill height for embankments up to 5 m high.

11.3 Embankment Construction

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

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.

12 BACKFILL TO ABUTMENTS

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

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

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

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

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

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

13 EARTH PRESSURE

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

If the support system allows yielding of the wall (unrestrained system), 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.

14 EROSION PROTECTION

We recommend rock protection (rip-rap) be used to armour areas that are susceptible to erosion. During storm events surface water can cause erosion beneath the rip-rap and movement of fines through the rip-rap blanket will occur. Therefore, a properly designed granular and fabric filter blanket would be required. The sides/ends of the filter fabric must also be anchored by burying in an anchor trench. Rip Rap/Rock Protection should be in accordance with OPSS 511, November 2008.

15 SEISMIC CONSIDERATIONS

15.1 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 0.04

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

15.2 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 ϕ = 35°; δ = 17.5° γ = 22.8 kN/m^3		OPSS Granular B Type I ϕ = 32°; δ = 16° γ = 21.2 kN/m^3		Rock Fill ϕ = 42°; δ = 21° γ = 19.0 kN/m^3	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (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

16 ADDITIONAL STUDIES

This report contains preliminary recommendations and are based on Terraprobe's interpretation of the factual information obtained from a limited number of boreholes and a visual site assessment. Detailed foundation investigations will be required at the structure/s location/s during the detail design phase of the project. The interpretation and recommendations are provided for planning purposes and feasibility studies only.

The following issues should be considered for the detailed design studies:

- Carry out the complete scope of detailed field investigations at the structure/s site/s and incorporate the data from this investigation based on the option that is carried forward.
- Depending on the option carried forward, perform detailed settlement analyses of the underlying silty clay soils especially if significant grade changes are proposed.
- Assess the potential for liquefaction of the sandy silt layer.

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TABLES

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

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

RECORD OF BOREHOLE No C-1

METRIC

G.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 07.28.10 - 07.29.10	CHECKED BY RA

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.6	Ground Surface				
241.5	150mm ASPHALT				
241.3 / 0.3	170mm FILL - Sand and Gravel, trace silt, inferred compact, brown, damp	1	SS	16	5 87 (8)
	FILL - Sand, trace silt, trace gravel, loose to compact, brown, damp to moist	2	SS	8	
		3	SS	7	
239.5 / 2.1	SANDY SILT some clay, loose, brown, wet	4	SS	6	
		5	SS	4	0 23 67 10
237.9 / 3.7	SILTY CLAY trace to some sand, trace gravel, occasional silt seams, soft to firm, grey, moist	6	SS	7	
		7	SS	3	0 4 56 40
		8	TW	PH	
		9	SS	2	1 15 58 26
232.6 / 9.0	SAND AND SILT trace clay, trace gravel, occasional cobbles and boulders, very dense, grey, damp to moist (GLACIAL TILL)	10	SS	70	commence casing and washboring
		11	SS	100/ 13cm	
		12	SS	100/ 8cm	
		13	SS	100/ 10cm	6 42 45 7

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (241 to 227); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20, 40, 60, 80, 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 10, 20, 30; UNIT WEIGHT γ kN/m³.

Field vane sensitivity values: 3.7, 3.4, 2.0

Continued Next Page

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

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No C-1

METRIC

G.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 07.28.10 - 07.29.10	CHECKED BY RA	

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
		14	SS	100/ 15cm	226	Jul.28 / Jul.29
		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%

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

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

G.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 07.28.10 - 07.29.10 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 (O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
			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

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 $O^{3\%}$ STRAIN AT FAILURE

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																	
213.6	28.0		Overburden, see Borehole Log C-1															
212.6	29.0		BEDROCK - PHYLLITE 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		
211.6	30.0		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		
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

G.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 07.27.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
241.6	Ground Surface						
241.5	150mm ASPHALT						
241.3 / 0.3	150mm FILL - Sand and Gravel, inferred compact, brown, damp	1	SS	29	241		
	FILL - Sand, trace silt, loose to compact, brown, dry	2	SS	9			0 94 (6)
	---- wet	3	SS	8	240		
239.5 / 2.1	SAND AND SILT some clay, loose, brown, wet	4	SS	4	239		0 44 44 12
238.7 / 2.9	SILTY CLAY trace sand, occasional gravel inclusions, firm to stiff, grey, damp to moist	5	SS	14	238		0 3 61 36
		6	SS	11	237		
		7	SS	12			0 1 75 24
		8	SS	5	236		3 7 55 35
		9	TW	PH	235		
						+ 2.3	
					234	+ 1.6	
		10	SS	6	233		
						+ 1.8	
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)	11	SS	24	232		17 44 35 4
		12	SS	117	231 / 230		
		13	SS	44	229 / 228		
		14	SS	156			9 50 37 4
226.9 / 14.7					227		

Continued Next Page

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

RECORD OF BOREHOLE No C-2

 METRIC

G.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 07.27.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT 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

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

Unable to push vane beyond 9.0m.

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

+³, ×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	C-2
Location	**Date Started**	**Completed**	**Logged By**	**Sheet**
Hwy 11, Township of McCrea, Ontario	July 27, 2010	July 27, 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
219.6	22.0		Overburden, see Borehole Log C-2															
219.1	22.5		BEDROCK - GRANITOID Unweathered, massive, bluish white, high strength.	2	CC	FV	C	SP	O	0 to 1				#1 TCR 72 SCR 61	25	NQ		
218.6	23.0			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

Foundation Design

RECORD OF BOREHOLE No C-3

METRIC

G.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 08.04.10 - 08.05.10 CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
239.8	Ground Surface							
0.0 / 239.5 / 0.3	300mm TOPSOIL							
	FILL - Silty Clay and Peat, trace sand, trace gravel, firm, dark brown / black, moist	1	SS	8			64	5 8 41 46
		2	SS	6	239			
		3	SS	5	238		76	
237.7 / 2.1	SILTY CLAY trace to some sand, trace gravel, firm to stiff, brown, moist	4	SS	10				
		5	SS	5	237			1 7 64 28
		6	SS	5	236			
		7	TW	PH	235	+3.1, +2.5		
		8	SS	1	234			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	233, 232			5 33 55 7
		10	SS	78	231			commence casing and washboring
		11	SS	100/13cm	230, 229			
		12	SS	100/13cm	228			
		13	SS	100/10cm	227, 226			
225.2 / 14.6					225			

Continued Next Page

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

ONTARIO MOT 1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ ONTARIO MOT.GDT 10/08/10

Ministry of Transportation Ontario

Foundation Design

RECORD OF BOREHOLE No C-3

METRIC

G.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 08.04.10 - 08.05.10 CHECKED BY RA

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	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	
		17	WS	-	218	12 25 43 20
					217	
		18	WS	-	216	
		19	WS	-	215	
					214	
		20	SS	149		4 29 43 24
					213	
					212	
211.6 28.2	BEDROCK - PHYLLITE unweathered below 28.9m, sub-vertical foliations, grey, medium to high strength.	1	RUN	NQ		RUN#1 TCR=46% SCR=23% RQD=0%
					211	
		2	RUN	NQ		RUN#2 TCR=90% SCR=79% RQD=29%
					210	

Continued Next Page

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

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

UNIT WEIGHT γ kN/m³

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

$O^{3\%}$ STRAIN AT FAILURE

RECORD OF BOREHOLE No C-3

METRIC

G.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 08.04.10 - 08.05.10 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

*(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: Crow Creek Bridge Replacement	Orientation: Vertical	Ground Elevation: 239.8m	Datum: Geodetic	Borehole No. C–3
Location: Hwy 11, Township of McCrea, Ontario	Date Started: August 5, 2010	Completed: August 5, 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) 1	DEPTH (m) 2	SYMBOL 3	GENERAL DESCRIPTION 4	Joint Characteristics: 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
212.1	27.7		Overburden, see Borehole Log C–3															
211.8	28.0													#1				
211.6	28.2		BEDROCK – PHYLLITE Unweathered below 28.9m, sub–vertical foliations, grey, medium to high strength. Slightly to moderately weathered from 28.2m to 28.9m. Highly fractured from 28.7m to 28.9m.	1	C	F	C	SP	SA	0 to 1				TCR 46 SCR 23	0	NQ		
					CCC	FDV	VC	SP	SA									
210.8	29.0			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

\\Terradocs\fileserver\Terraprobe Limited\All Projects\2010 Files\1-10-5001 to 5099\1-10-5076\A. Dwgs, Logs\AutoCAD\1-10-5076 CORE.dwg, DB

Ministry of Transportation
Ontario

Foundation Design

RECORD OF BOREHOLE No C-4

METRIC

G.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 08.06.10	CHECKED BY RA

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

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
240.0	Ground Surface					
239.8 / 0.2	200mm TOPSOIL					
	FILL - Silty Clay and Peat, trace sand, trace gravel, firm, dark brown, moist	1	SS	7		
		2	SS	5	239	
238.6 / 1.4	SILTY CLAY trace to some sand, trace gravel, firm to stiff, brown, moist	3	SS	8	238	
		4	SS	9	237	1 2 66 31
		5	TW	PH	236	
		6	SS	5	235	
		7	SS	4	234	
		8	SS	4	233	0 10 64 26
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)	9	SS	42	232	5 32 55 8 commence casing and washboring
		10	SS	100/15cm	231	
		11	SS	100/15cm	230, 229	
		12	SS	100/15cm	228, 227	
		13	SS	99	226	16 32 41 11
225.3 / 14.7						

Field vane sensitivity values plotted: 4.3 (at approx. 236), 3.6, 3.3 (at approx. 233). Water content value 42 plotted at 239.

Continued Next Page

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

RECORD OF BOREHOLE No C-4

METRIC

G.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 08.06.10	CHECKED BY RA

ELEV / DEPTH	DESCRIPTION	SAMPLE 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	224		5 16 62 17
		15	SS	100/ 13cm	222, 221		
		16	SS	100/ 15cm	219, 218		2 35 44 19
		17	WS	-	217, 216		
214.6 / 25.4		2	RUN	NQ	215		RUN#2 TCR=33% SCR=7% RQD=7%
	BEDROCK - PHYLLITE unweathered below 29.0m, sub-vertical foliations, grey, very low to high strength.	3	RUN	NQ	214, 213		RUN#3 TCR=59% SCR=28% RQD=9%
		4	RUN	NQ	212		RUN#4 TCR=63% SCR=34% RQD=23%
		5	RUN	NQ	211		RUN#5 TCR=98% SCR=92% RQD=65%

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

Continued Next Page

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

Ministry of Transportation
Ontario

RECORD OF BOREHOLE No C-4

METRIC

G.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 08.06.10 CHECKED BY RA

ELEV / DEPTH	SOIL PROFILE: DESCRIPTION	STRAT PLOT	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: "N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100)	PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) (10 20 30)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
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.										

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

ELEVATION (m)	DEPTH (m)	SYMBOL	GENERAL DESCRIPTION	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
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 – PHYLLITE	2	CC	FV	VC	SP	SA	0 to 1								
214.0	26.0		Unweathered below 29.0m, sub-vertical foliations, grey, very low to high strength.		CC	FV	VC	SU	SI									
			Completely weathered from 26.2m to 27.4m.	2	CC	FV	C	SU	SI	0 to 3				#3 TCR 59 SCR 28	9	NQ		
213.0	27.0		Highly weathered from 25.9m to 26.2m and 27.6m to 27.9m.															
			Slightly to moderately weathered from 25.4m to 25.9m, 27.4m to 27.6m and 27.9m to 29.0m.															
212.0	28.0		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		
					CC	FV	M	SU	SI									
211.0	29.0			2	CC	FV	M	SP	T	0 to 1				#5 TCR 98 SCR 92	65	NQ		
210.0	30.0																	
209.5	30.5		End of Core Log															

Remarks:

LEGEND:
Bedrock

\\Terradocs\fileserver\Terraprobe Limited\All Projects\2010 Files\1-10-5001 to 5099\1-10-5076\A. Dwgs, Logs\AutoCAD\1-10-5076 CORE.dwg, DB

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

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

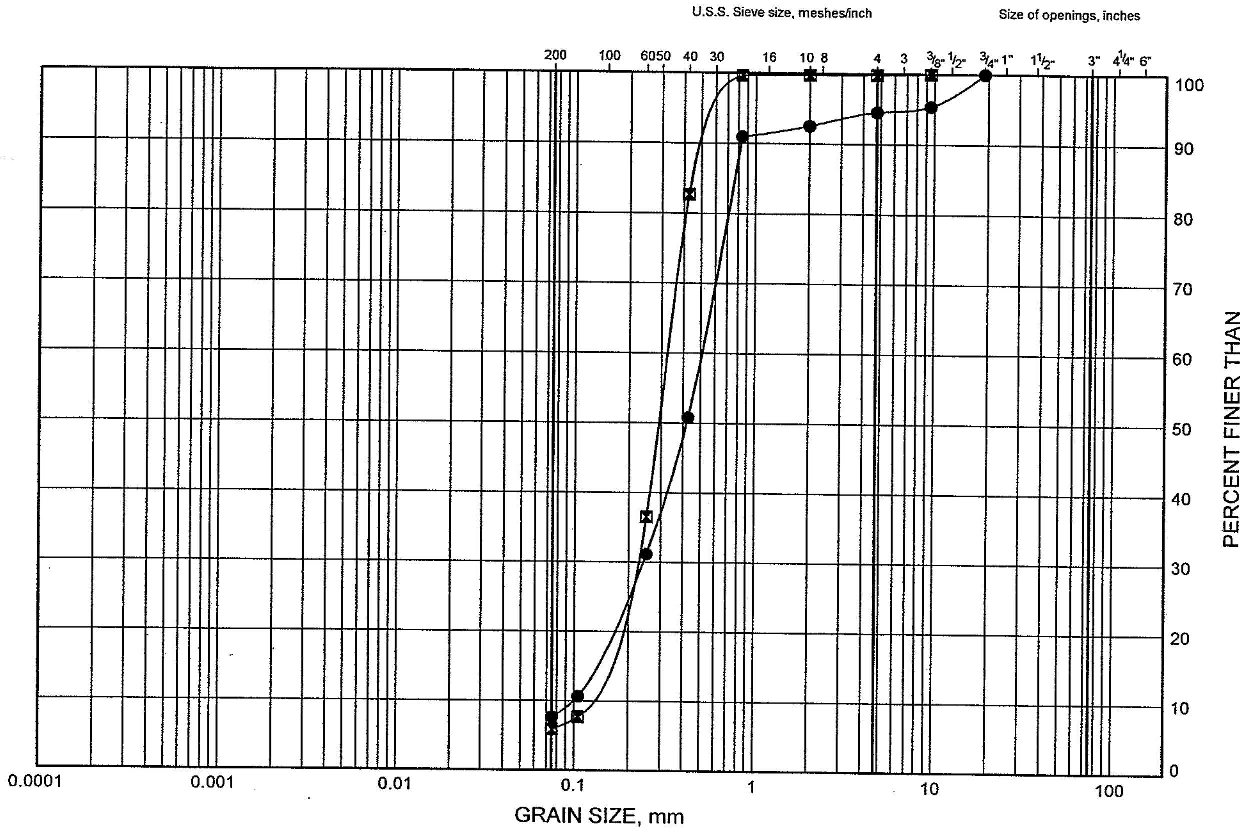
APPENDIX B

TERRAPROBE INC.

GRAIN SIZE DISTRIBUTION

FIGURE B1-1

FILL - Sand



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-1	0.5	241.1
⊠	C-2	1.0	240.6

Date October 2010

Project 1-10-5076

Prep'd DB

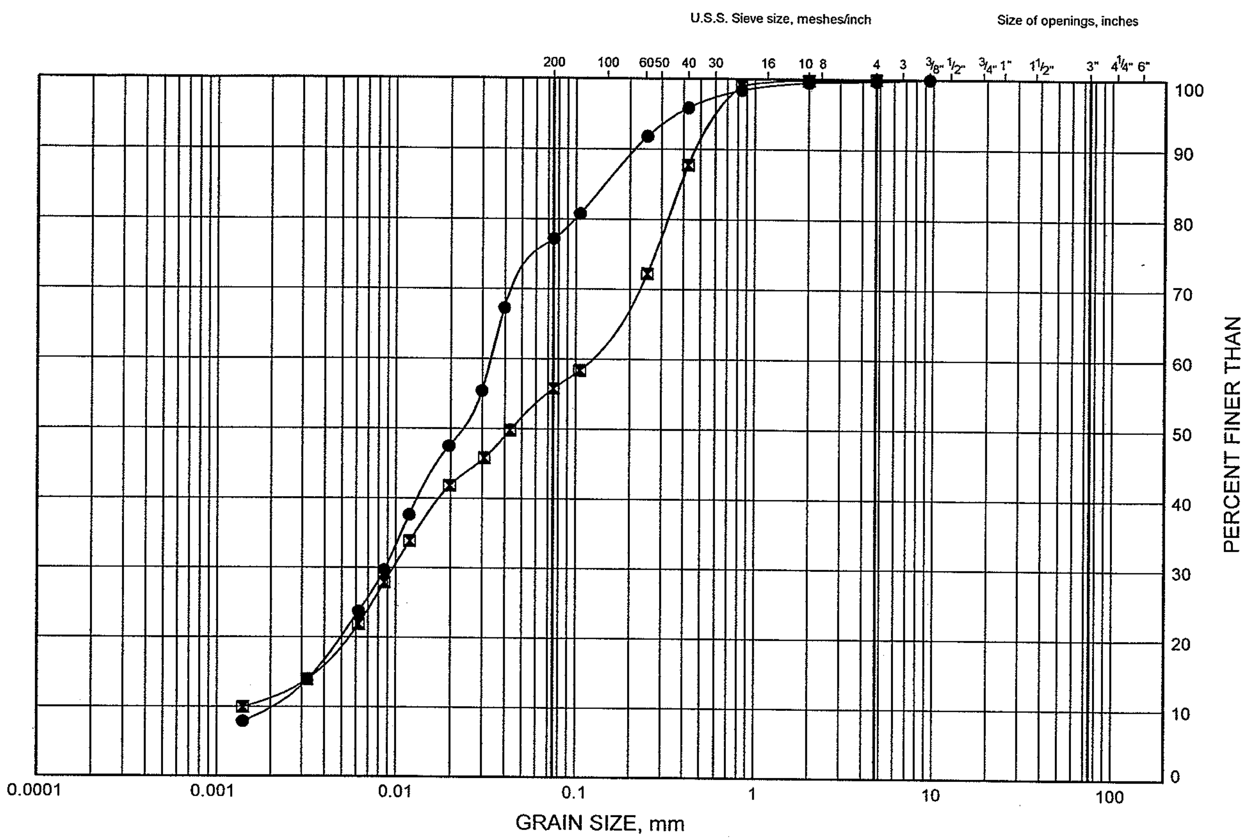
Chkd. HA

GSD 2010 MTO 1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ 10/19/10

GRAIN SIZE DISTRIBUTION

FIGURE B1-2

SANDY SILT TO SAND AND SILT



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-1	3.2	238.4
⊠	C-2	2.5	239.1

Date October 2010

Project 1-10-5076

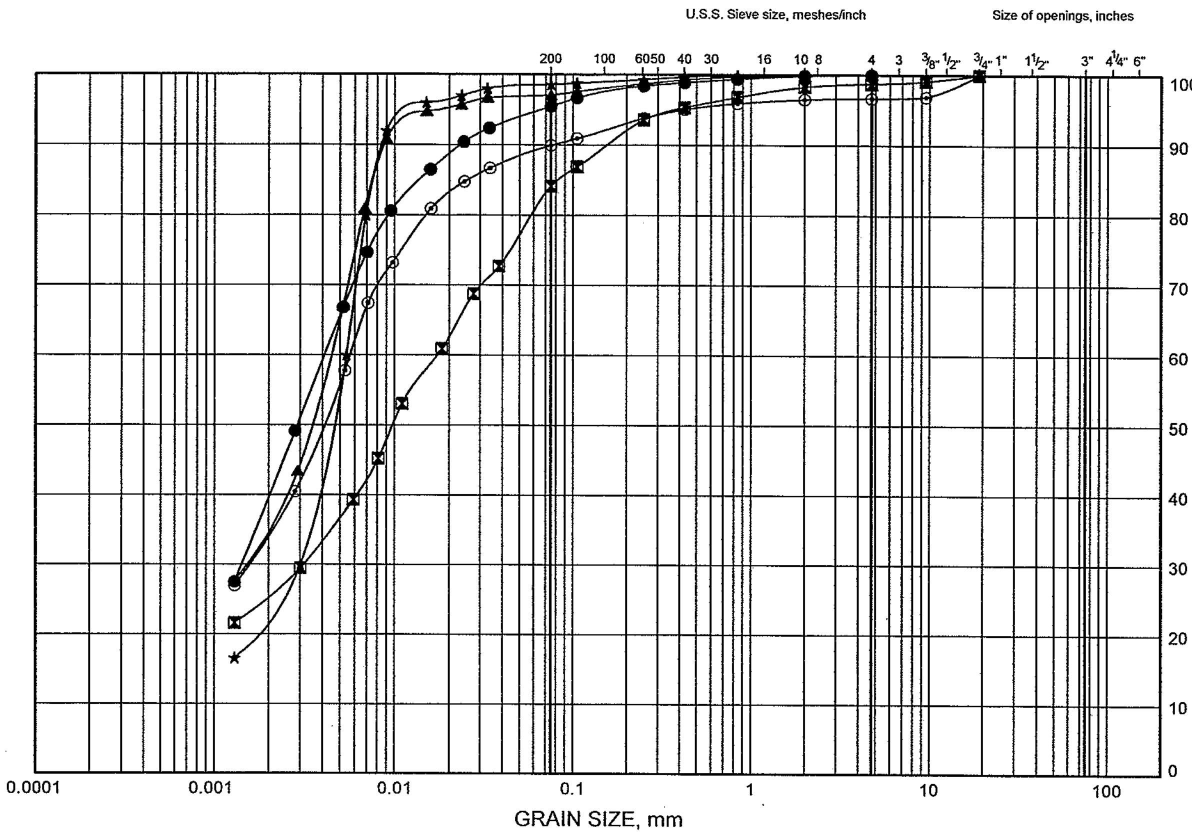
Prep'd DB

Chkd. HA

GSD 2010 MTO 1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ 10/19/10

GRAIN SIZE DISTRIBUTION	FIGURE B1-3

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-1	4.7	236.9
⊠	C-1	7.8	233.8
▲	C-2	3.2	238.4
★	C-2	4.7	236.9
⊙	C-2	5.5	236.1

Date October 2010

Project 1-10-5076

Prep'd DB

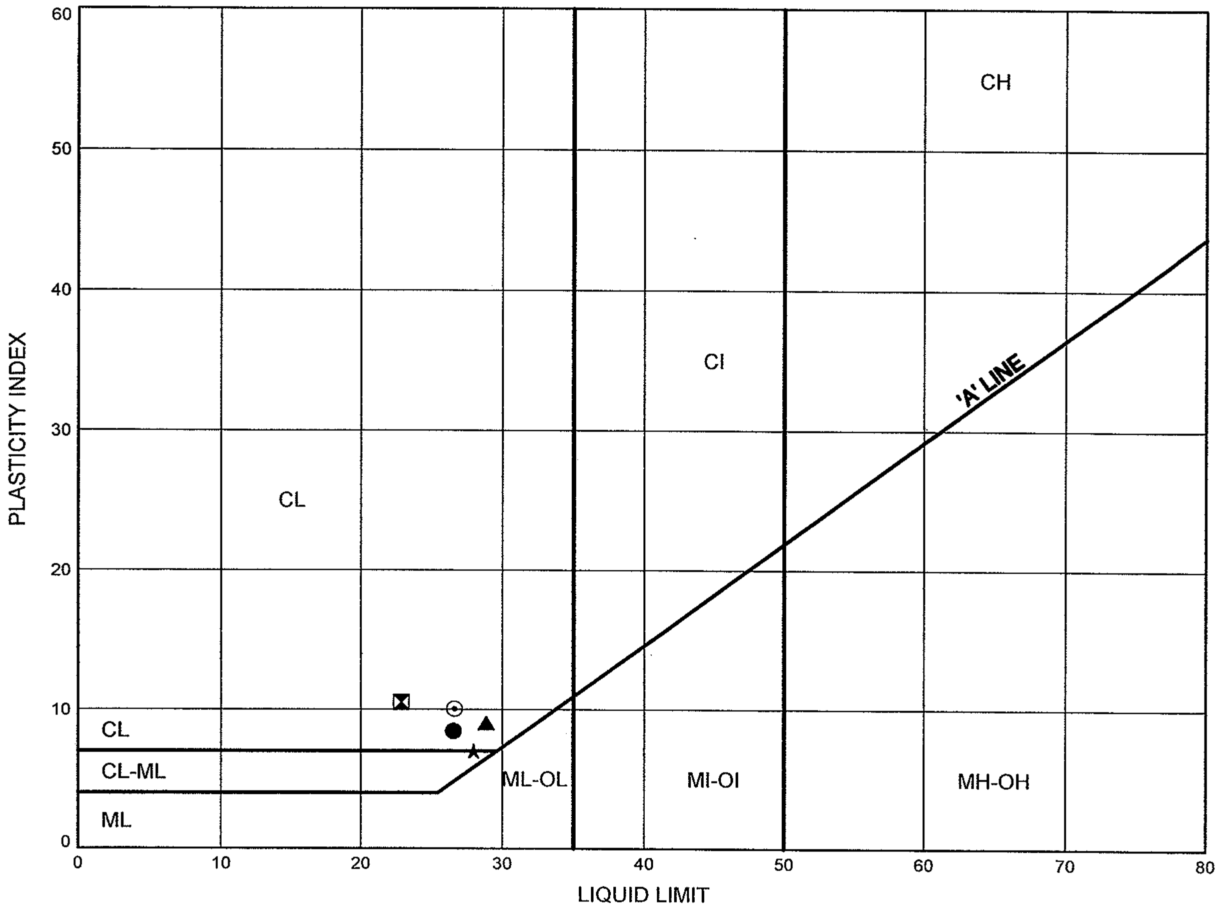
Chkd. HA

GSD 2010 MTO 1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ 10/19/10

ATTERBERG LIMITS TEST RESULTS

FIGURE B1-4

SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-1	4.7	236.9
⊠	C-1	7.8	233.8
▲	C-2	3.2	238.4
★	C-2	4.7	236.9
⊙	C-2	5.5	236.1

Date October 2010

Project 1-10-5076

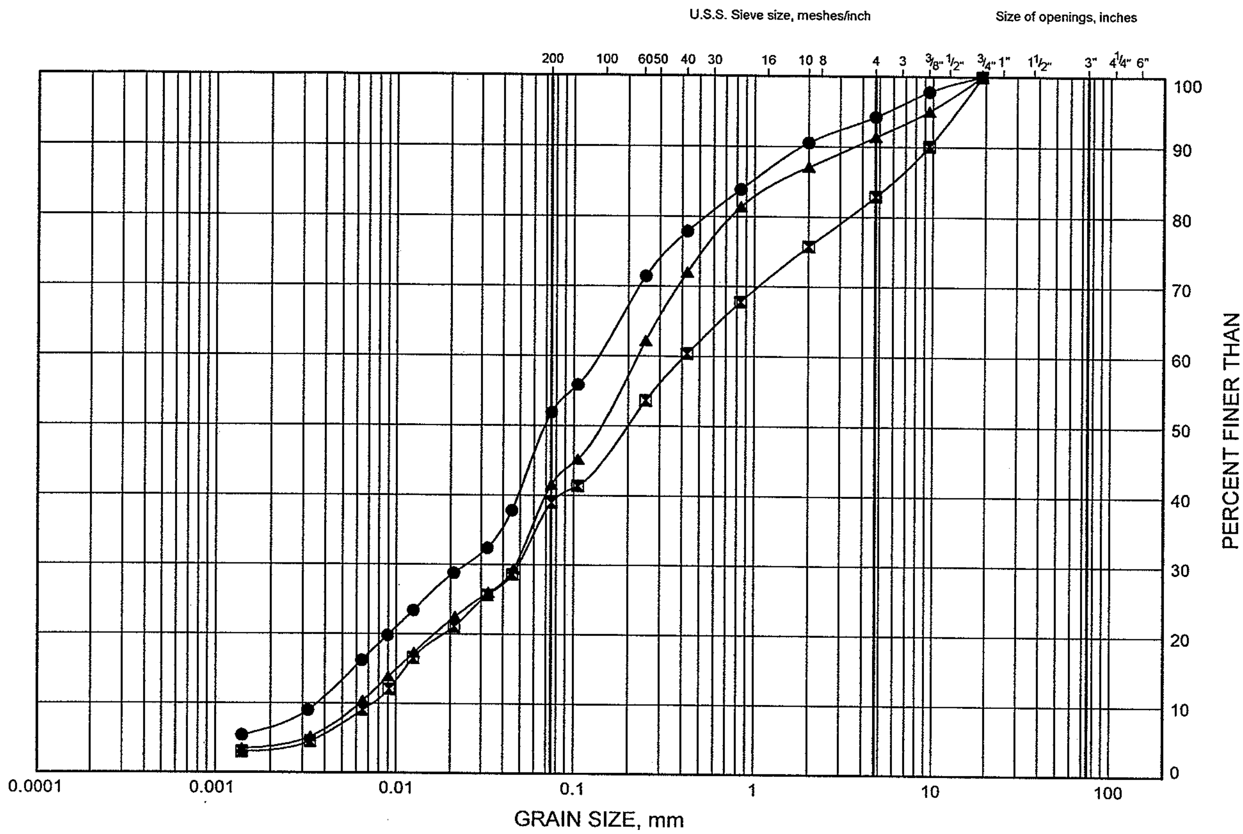
Prep'd DB

Chkd. RA

ALTR 1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ 10/12/10

GRAIN SIZE DISTRIBUTION	FIGURE B1-5

SAND AND SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-1	13.9	227.7
⊠	C-2	9.3	232.3
▲	C-2	13.9	227.7

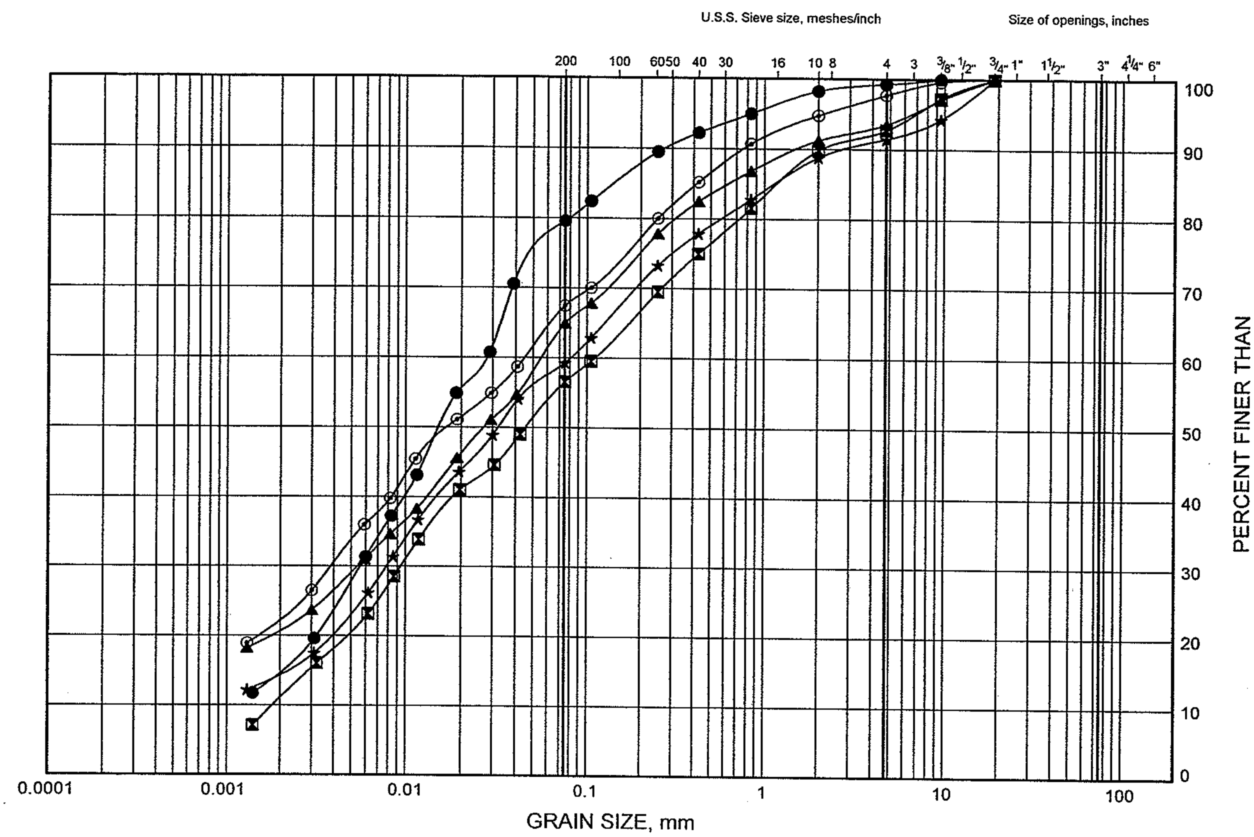
Date October 2010
Project 1-10-5076

Prep'd DB
Chkd. HA

GRAIN SIZE DISTRIBUTION

FIGURE B1-6

CLAYEY SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-1	18.5	223.1
⊠	C-1	21.5	220.1
▲	C-1	26.4	215.2
★	C-2	15.4	226.2
⊙	C-2	21.5	220.1

Date October 2010

Project 1-10-5076

Prep'd DB

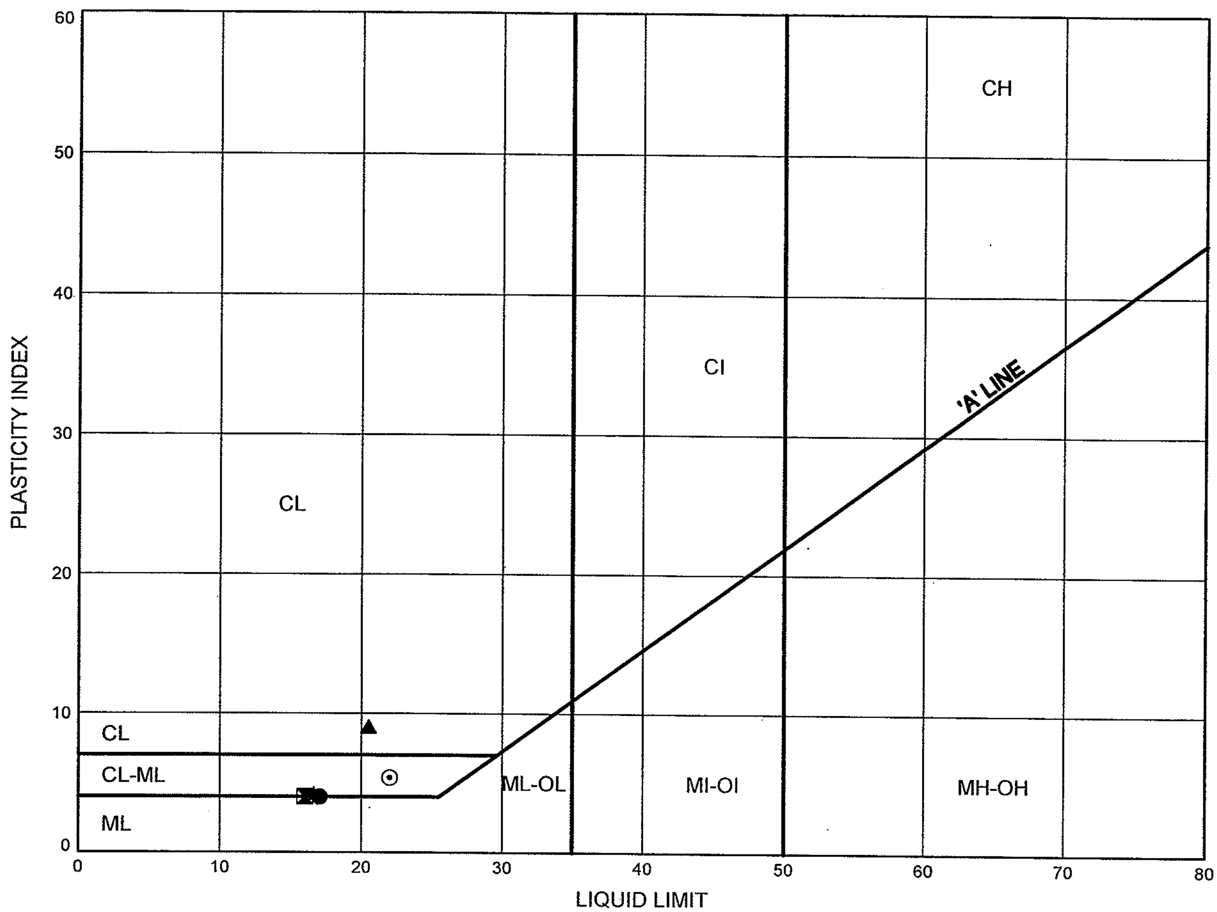
Chkd. HA

GSD 2010 MTO 1-10-5076 CROWIMOUNTCALM BRIDGE RPL.GPJ 10/19/10

ATTERBERG LIMITS TEST RESULTS

FIGURE B1-7

CLAYEY SILT TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-1	18.5	223.1
⊠	C-1	21.5	220.1
▲	C-1	26.4	215.2
★	C-2	15.4	226.2
⊙	C-2	21.5	220.1

Date October 2010

Project 1-10-5076

Prep'd DB

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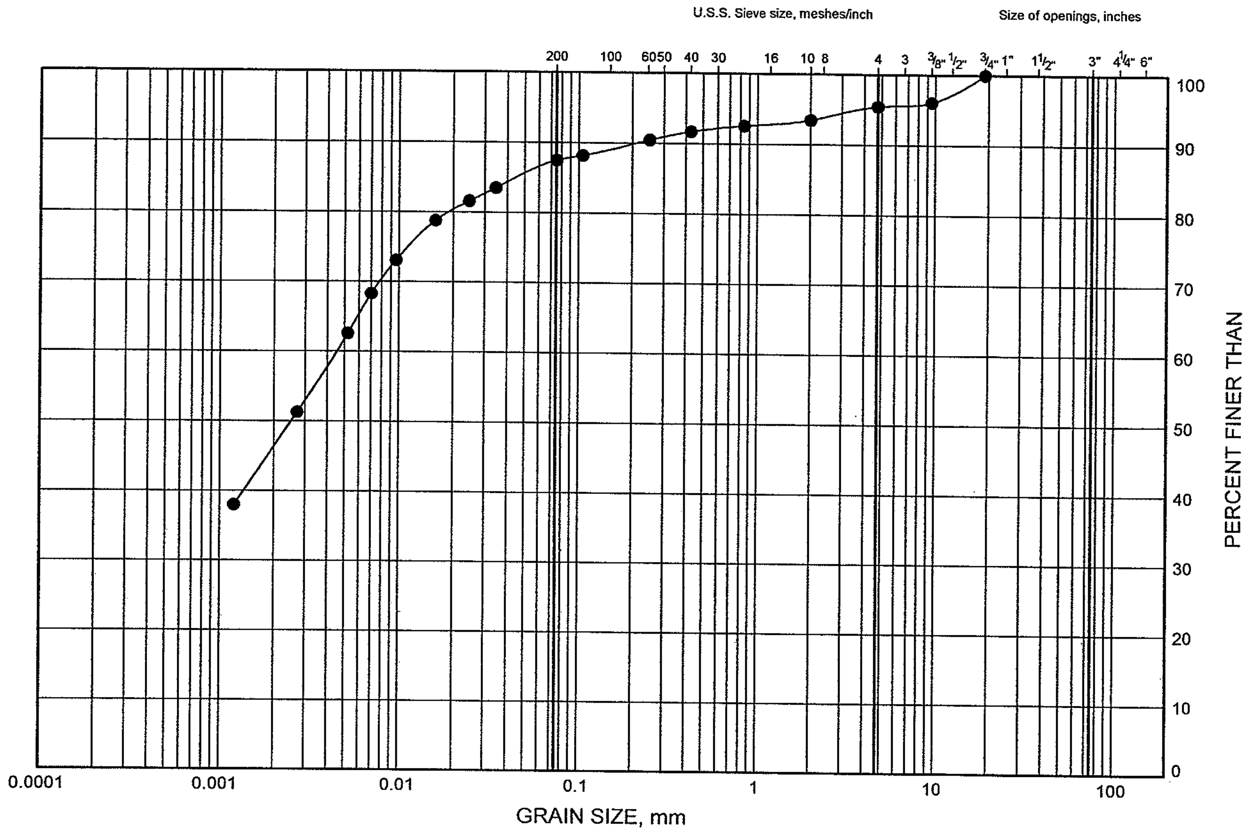
GRAIN SIZE DISTRIBUTION

FIGURE B2-1

FILL - Silty Clay



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-3	0.4	239.4

Date October 2010

Project 1-10-5076

Prep'd DB

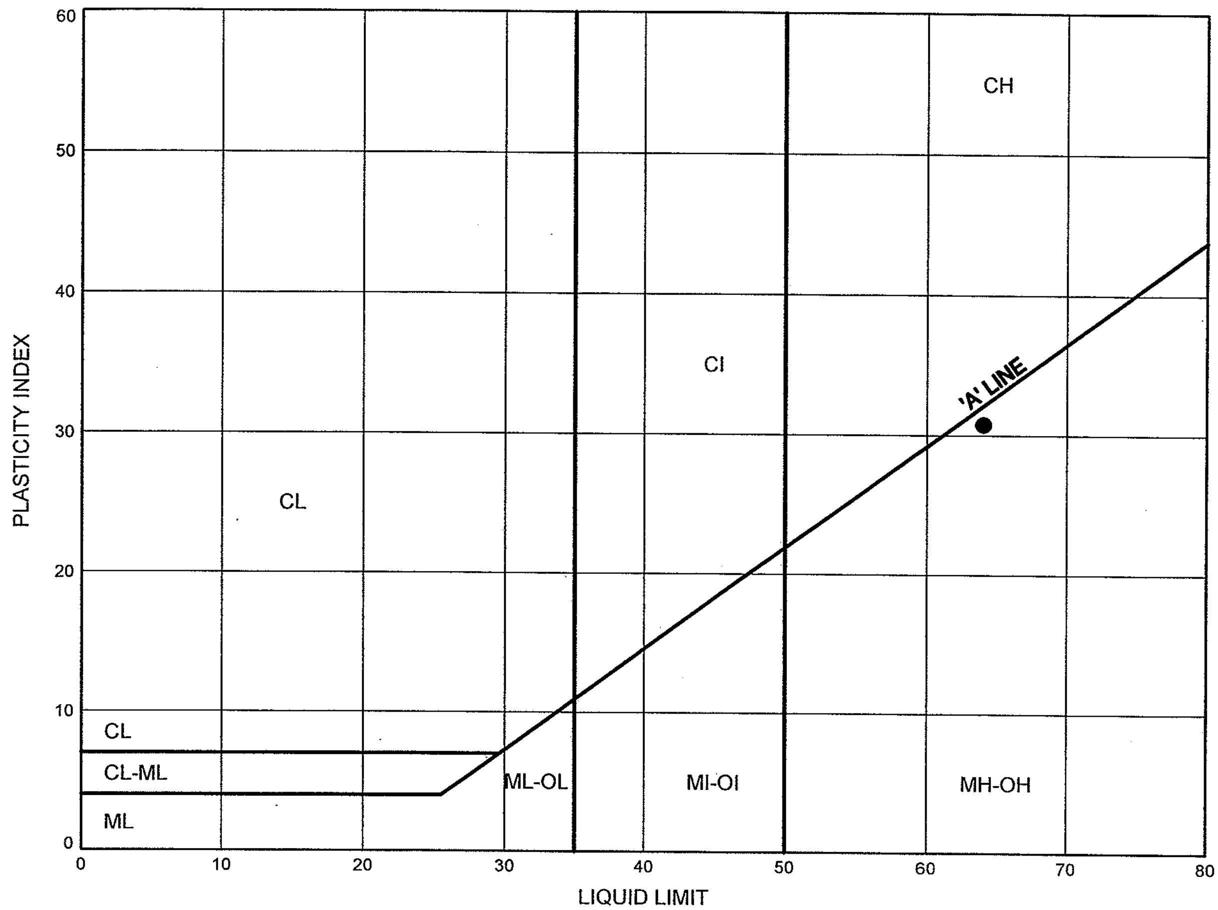
Chkd. HA

GSD 2010 MTO 1-10-5076 CROW/MOUNTCALM BRIDGE RPL.GPJ 10/19/10

ATTERBERG LIMITS TEST RESULTS

FIGURE B2-2

FILL - Silty Clay



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-3	0.4	239.4

Date October 2010

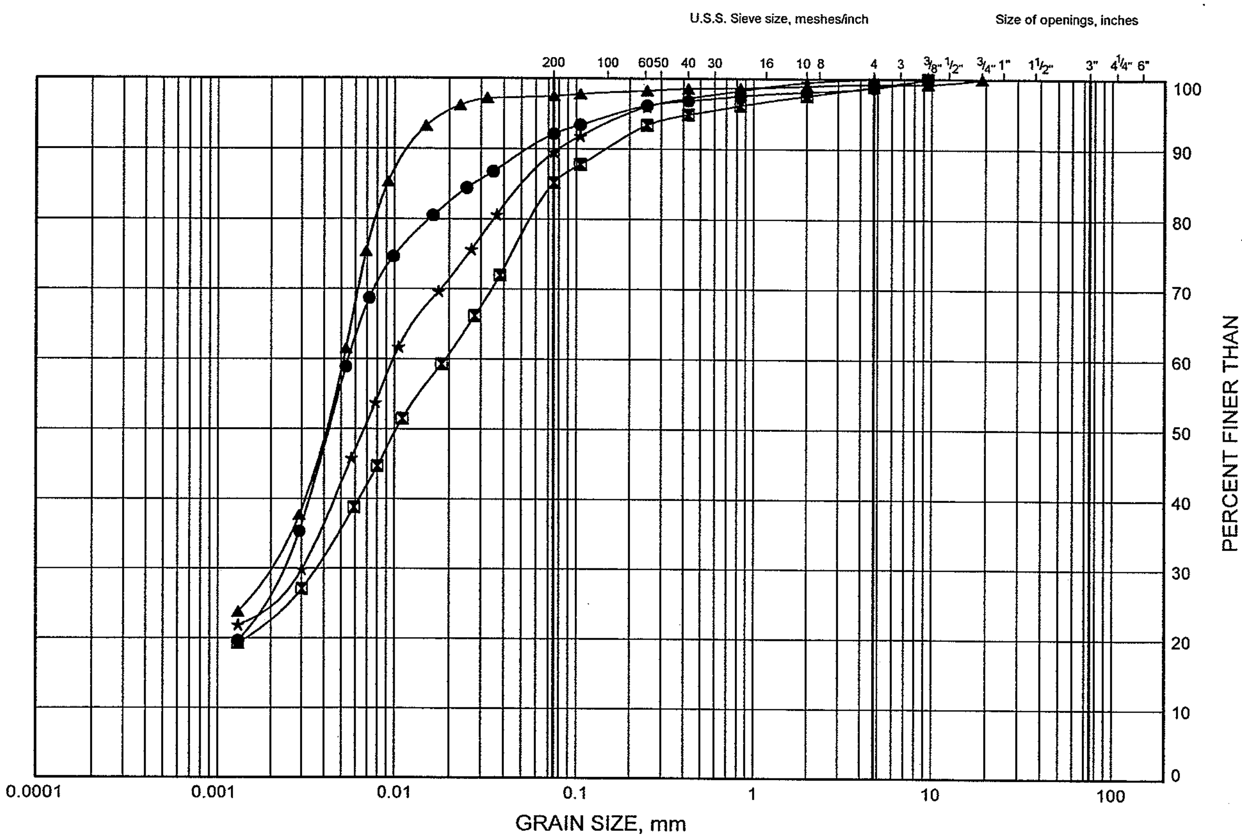
Project 1-10-5076

Prep'd DB

Chkd. RA

GRAIN SIZE DISTRIBUTION	FIGURE B2-3

SILTY CLAY



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-3	3.2	236.6
⊠	C-3	6.3	233.5
▲	C-4	2.5	237.5
★	C-4	6.3	233.7

Date October 2010

Project 1-10-5076

Prep'd DB

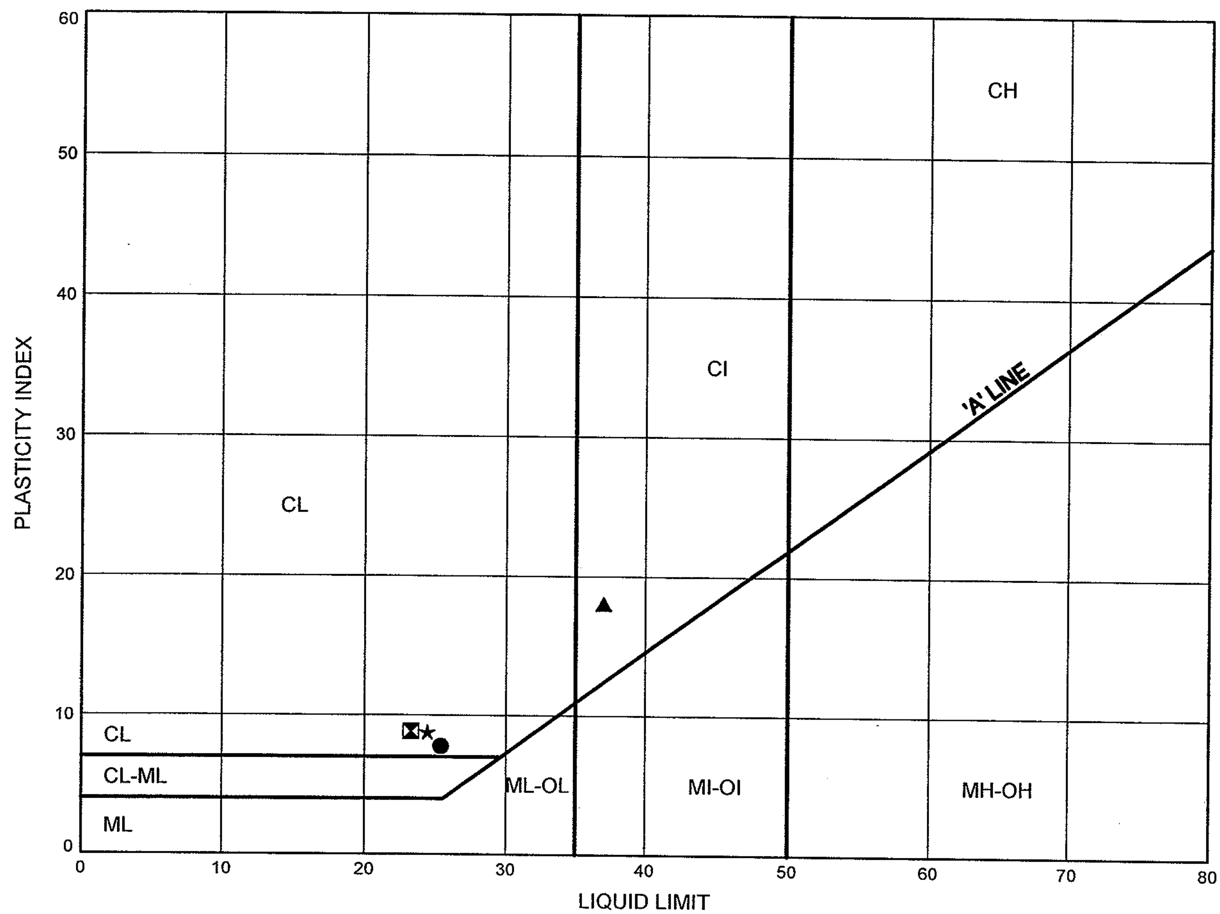
Chkd. HA

GSD 2010 MTO .1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ 10/19/10

ATTERBERG LIMITS TEST RESULTS

FIGURE B2-4

SILTY CLAY



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-3	3.2	236.6
⊠	C-3	6.3	233.5
▲	C-4	2.5	237.5
★	C-4	6.3	233.7

Date October 2010

Project 1-10-5076

Prep'd DB

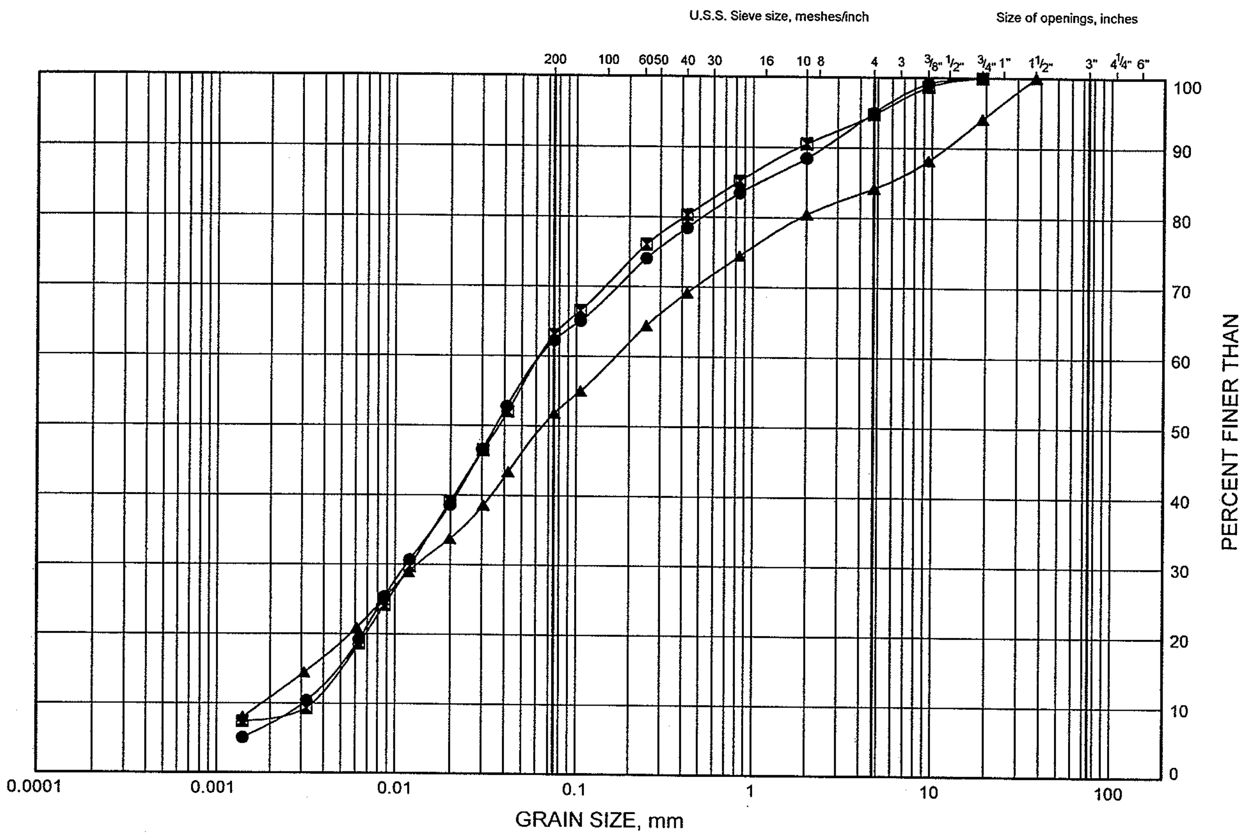
Chkd. RA

ALTR 1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ 10/12/10

GRAIN SIZE DISTRIBUTION

FIGURE B2-5

SANDY SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-3	7.8	232.0
⊠	C-4	7.8	232.2
▲	C-4	13.9	226.1

Date October 2010
Project 1-10-5076

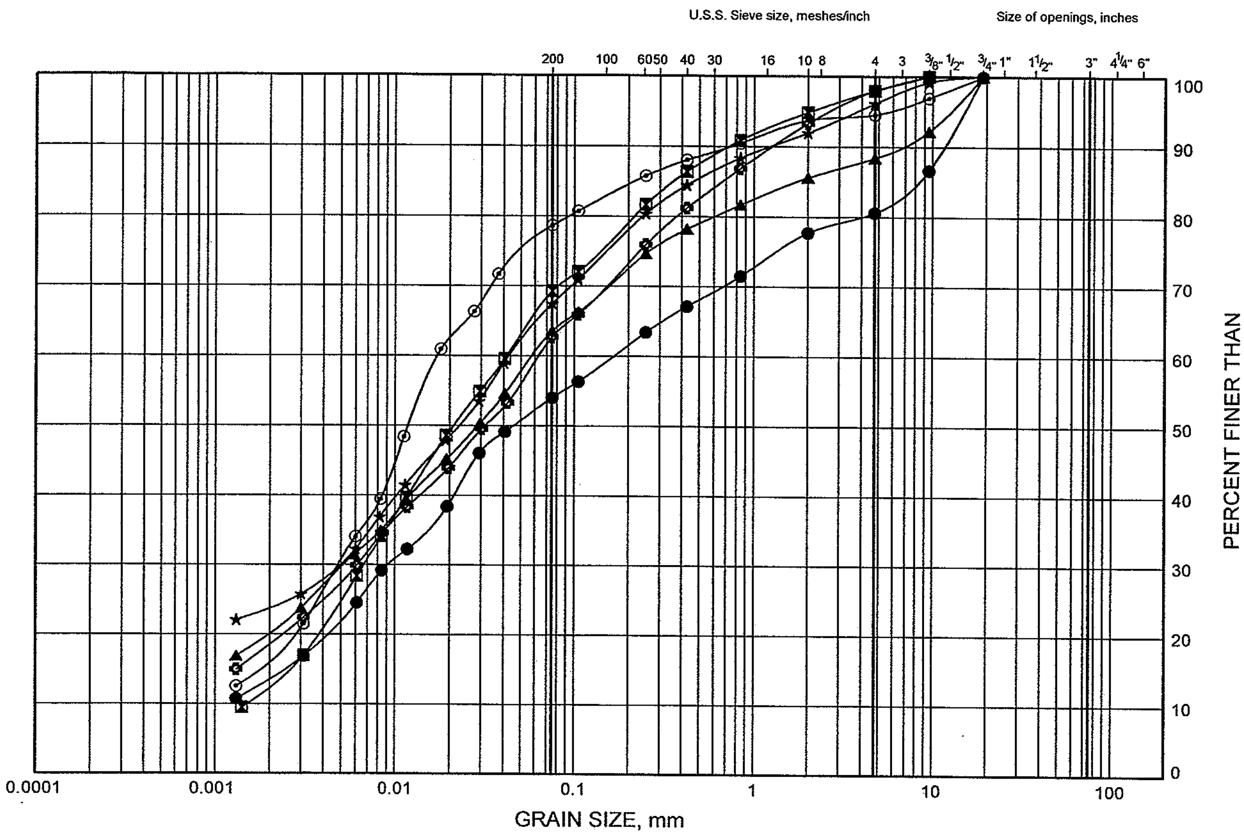
Prep'd DB
Chkd. HA

GSD 2010 MTO 1-10-5076 CROWMOUNTCALM BRIDGE RPL.GPJ 10/19/10

GRAIN SIZE DISTRIBUTION

FIGURE B2-6

CLAYEY SILT TILL



SILT and CLAY	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE SIZE
FINE GRAINED	SAND			GRAVEL		

SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-3	15.4	224.4
⊠	C-3	18.5	221.3
▲	C-3	22.1	217.7
★	C-3	26.3	213.5
⊙	C-4	15.4	224.6
✪	C-4	21.5	218.5

Date October 2010

Project 1-10-5076

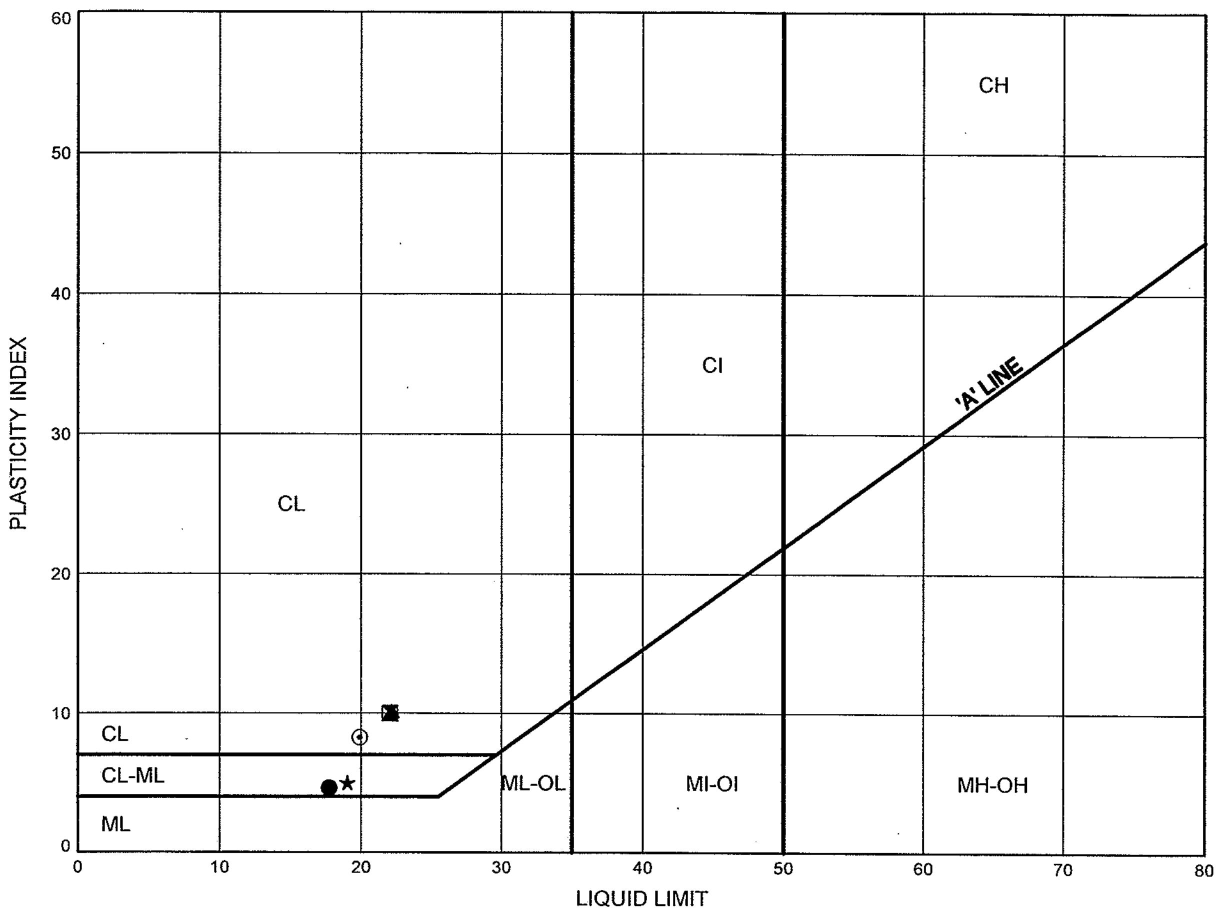
Prep'd DB

Chkd. HA

ATTERBERG LIMITS TEST RESULTS

FIGURE B2-7

CLAYEY SILT TILL



SYMBOL	BOREHOLE	DEPTH (m)	ELEVATION (m)
●	C-3	18.5	221.3
⊠	C-3	22.1	217.7
▲	C-3	26.3	213.5
★	C-4	15.4	224.6
⊙	C-4	21.5	218.5

Date October 2010

Project 1-10-5076

Prep'd DB

Chkd. RA

ALTR 1-10-5076 CROWIMOUNTCALM BRIDGE RPL.GPJ 10/12/10

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

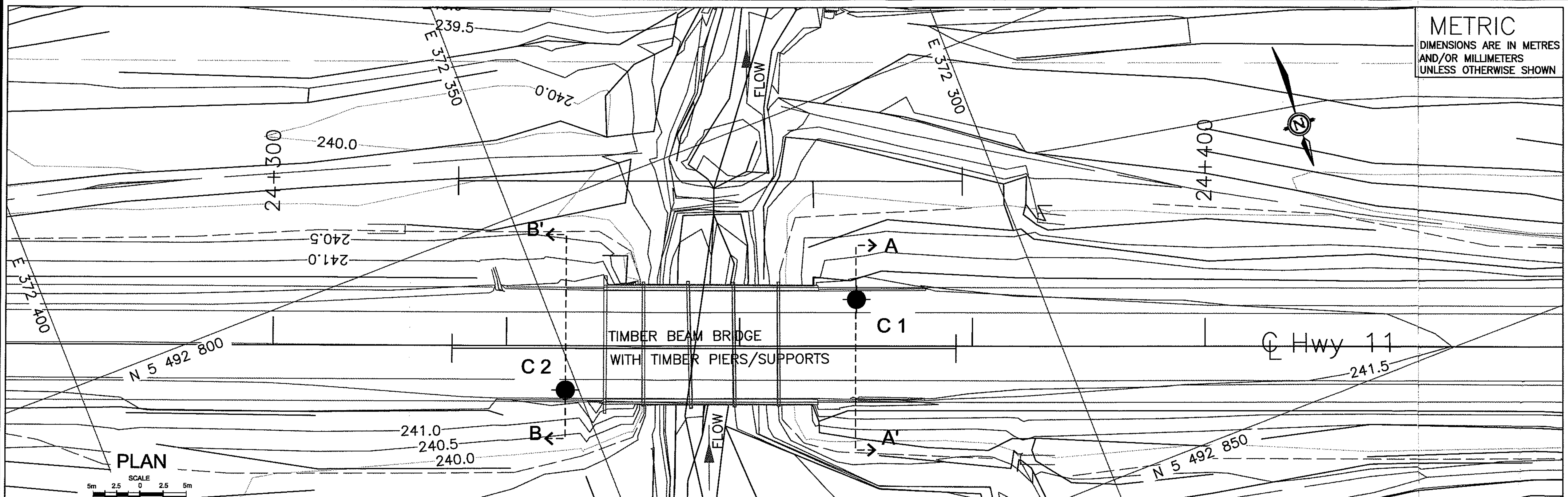
CONT No
GWP No 5233–06–00

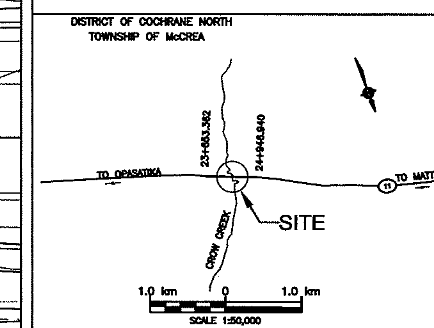
CROW CREEK BRIDGE
BOREHOLE LOCATION AND SOIL STRATA

SHEET 1 OF 4

McCORMICK RANKIN CORPORATION — MRC

Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
10 Bram Court - Brampton Ontario L6W 3R6 (905) 796-2650



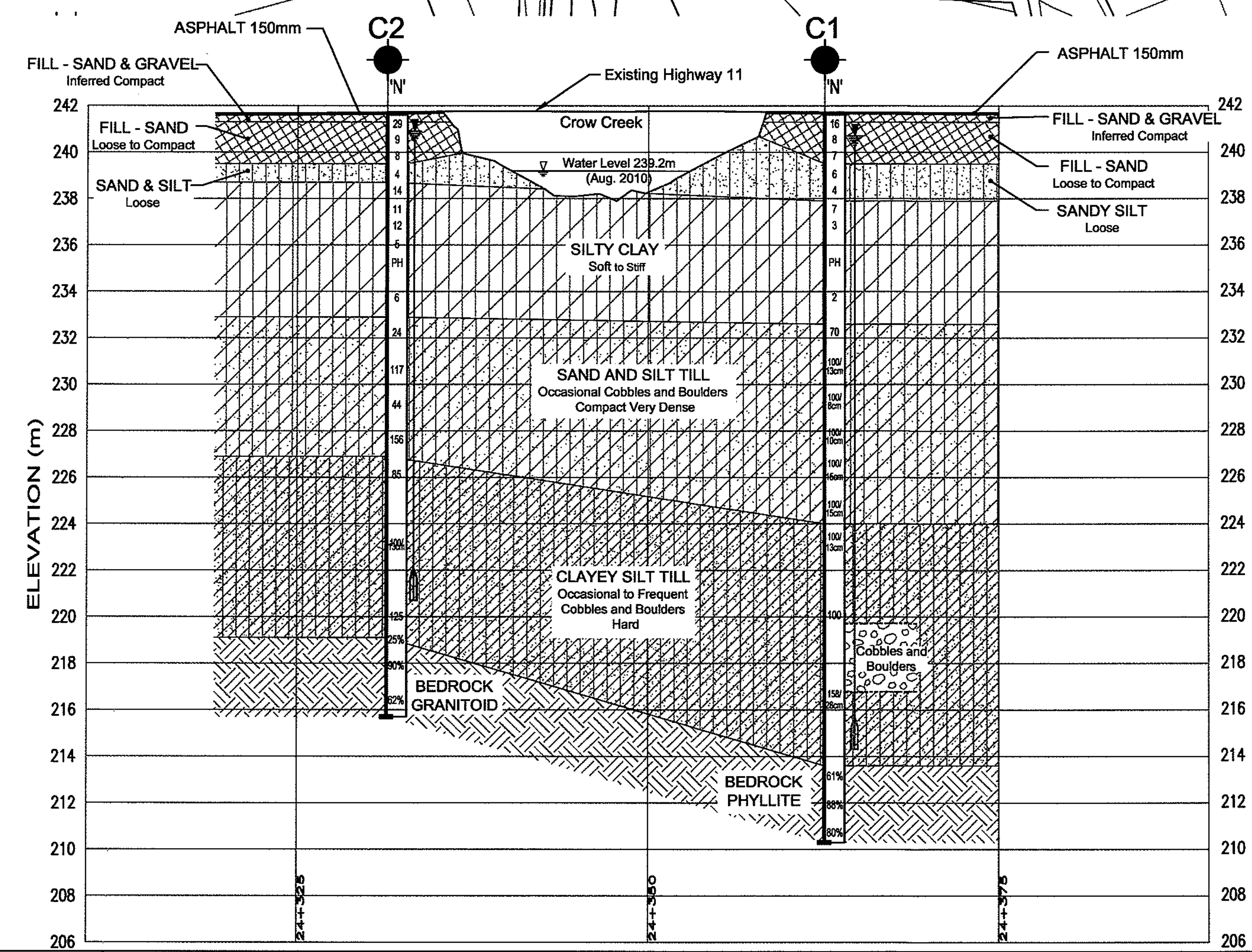


KEY PLAN

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 (SEPT. 2010)
▮	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES NORTHING	COORDINATES EASTING
C1	241.6	5 492 821.8	372 318.7
C2	241.6	5 492 819.4	372 351.3



PROFILE ℄ HIGHWAY 11

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

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
02·03·2011
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
March 02/2011
PROVINCE OF ONTARIO

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 R.A	CODE CHBDC2006	LOAD	DATE MARCH 2011
DRAWN K.C	CHK R.A	STRUCT 39W–055	GEOCRES 42G–33

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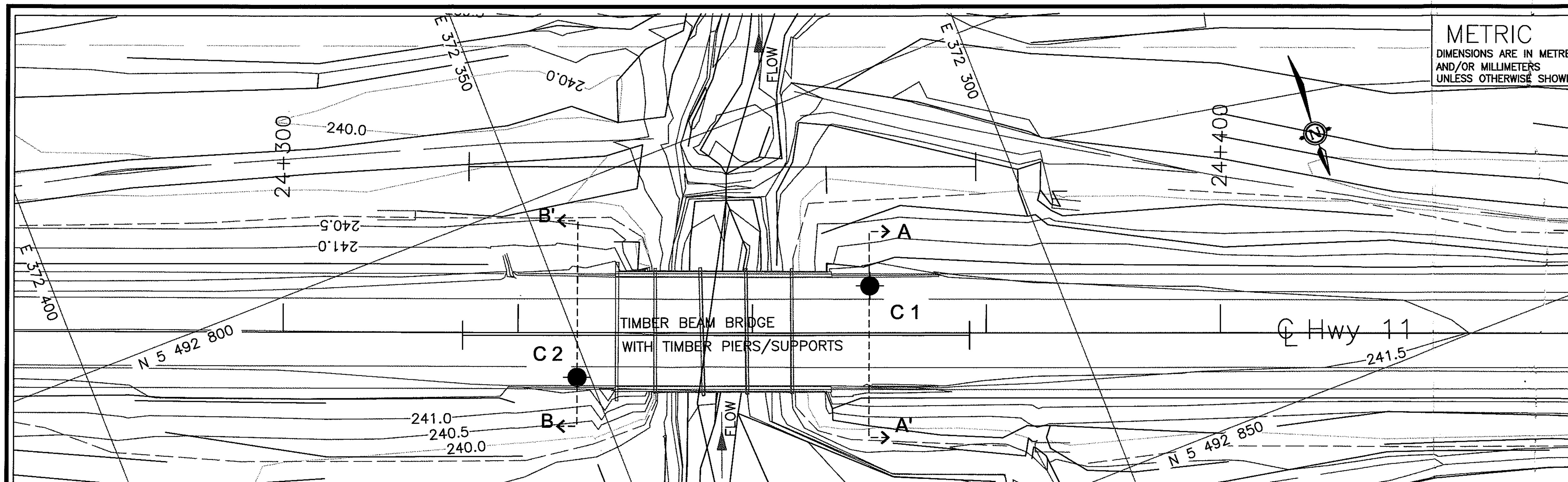
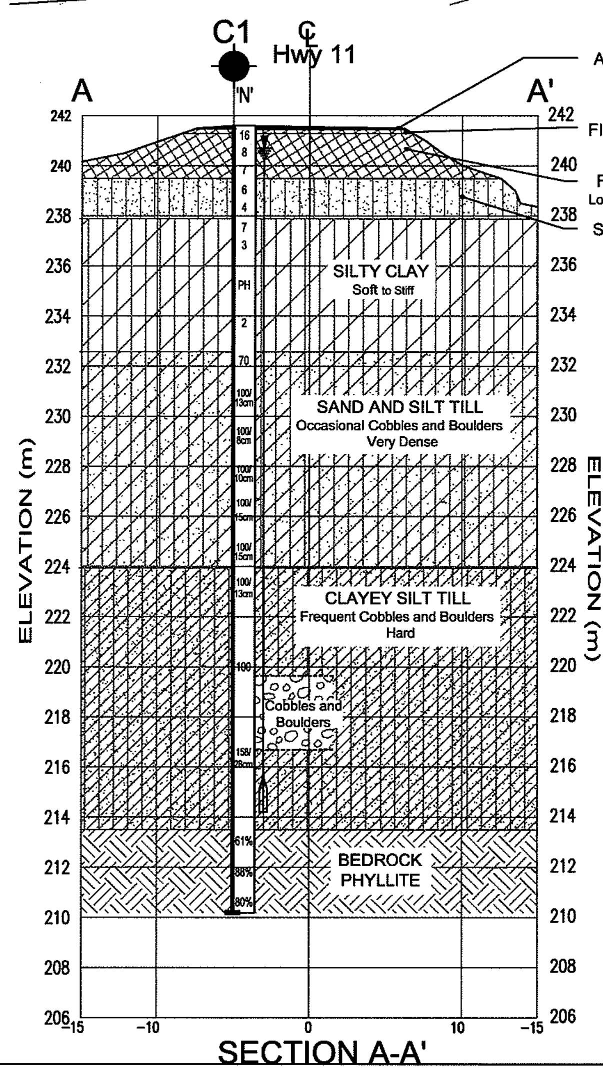
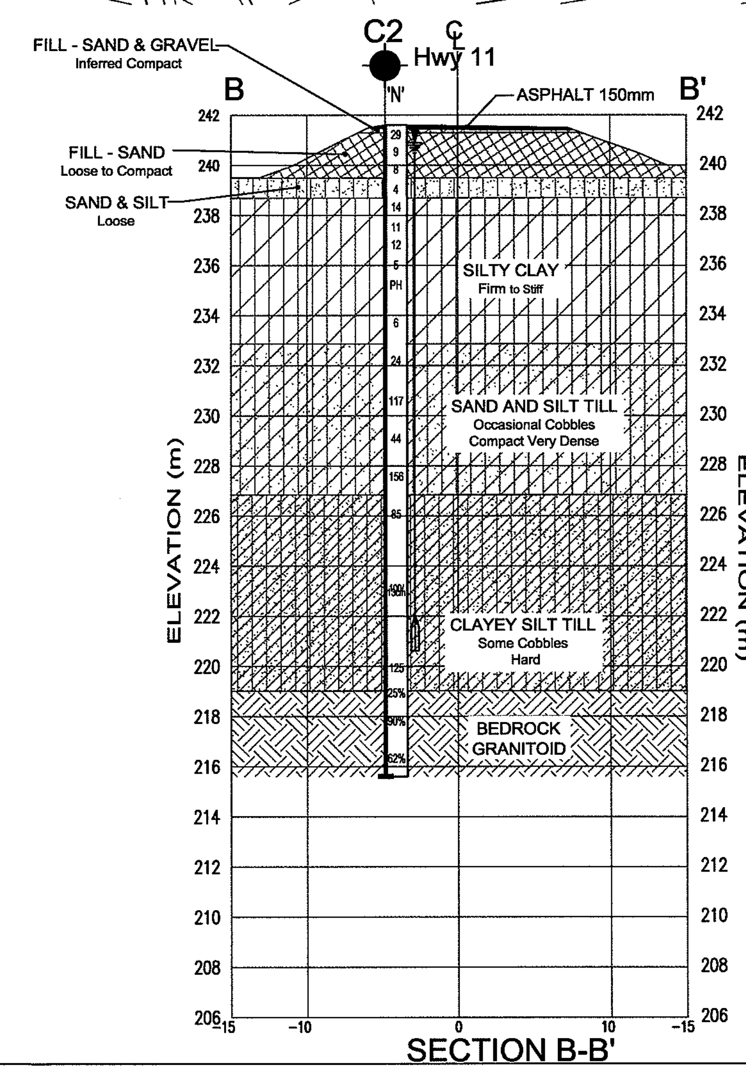
METRIC
DIMENSIONS ARE IN METRES AND/OR MILLIMETERS UNLESS OTHERWISE SHOWN



PLAN



SECTION A-A'



SECTION B-B'

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
02.03.2011
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
March 02/2011
PROVINCE OF ONTARIO

HORIZONTAL SCALE 5m 2.5 0 2.5 5m

VERTICAL SCALE 5m 2.5 0 2.5 5m

CONT No
GWP No 5233-06-00

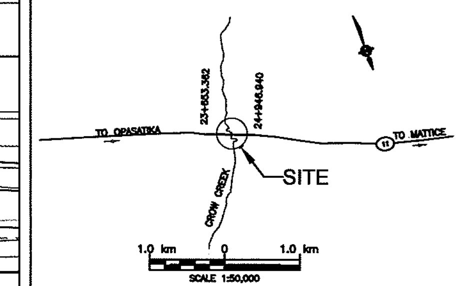
CROW CREEK BRIDGE
BOREHOLE LOCATION AND SOIL STRATA

SHEET 2 OF 4

McCORMICK RANKIN CORPORATION
MRC

Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
10 Bram Court - Brampton Ontario L6W 3R6 (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 (SEPT. 2010)
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES NORTHING	COORDINATES EASTING
C1	241.6	5 492 821.8	372 318.7
C2	241.6	5 492 819.4	372 351.3

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 R.A	CODE CHBDC2006	LOAD	DATE MARCH 2011
DRAWN K.C	CHK R.A	STRUCT 39W-055	GEOCRES 42G-33

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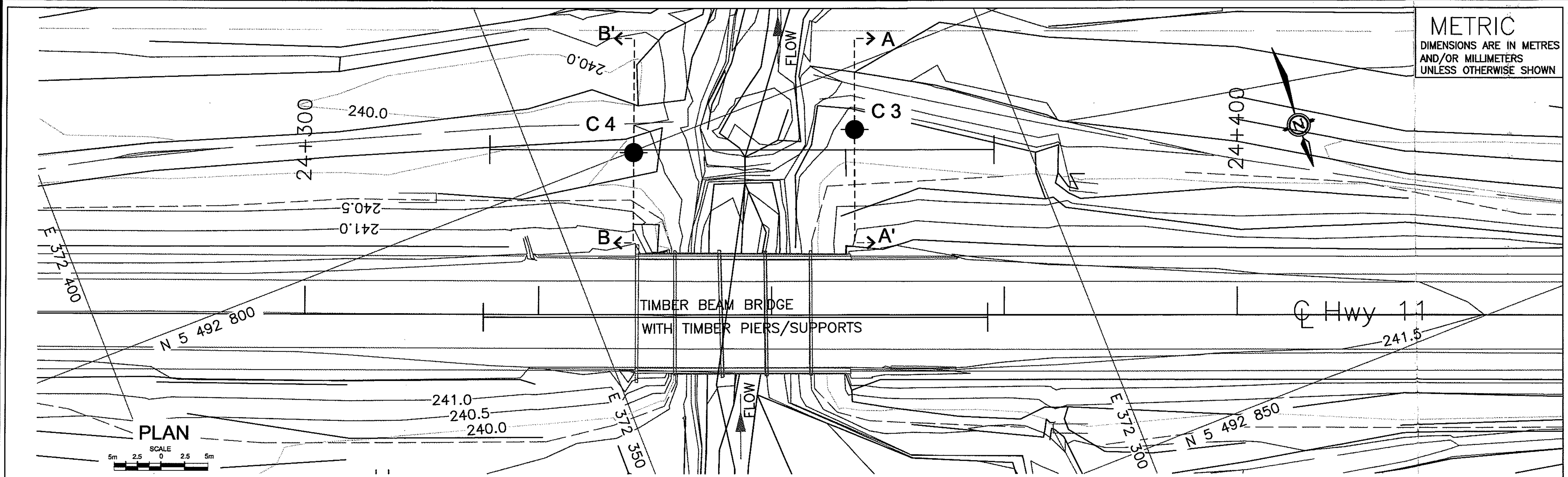

METRIC
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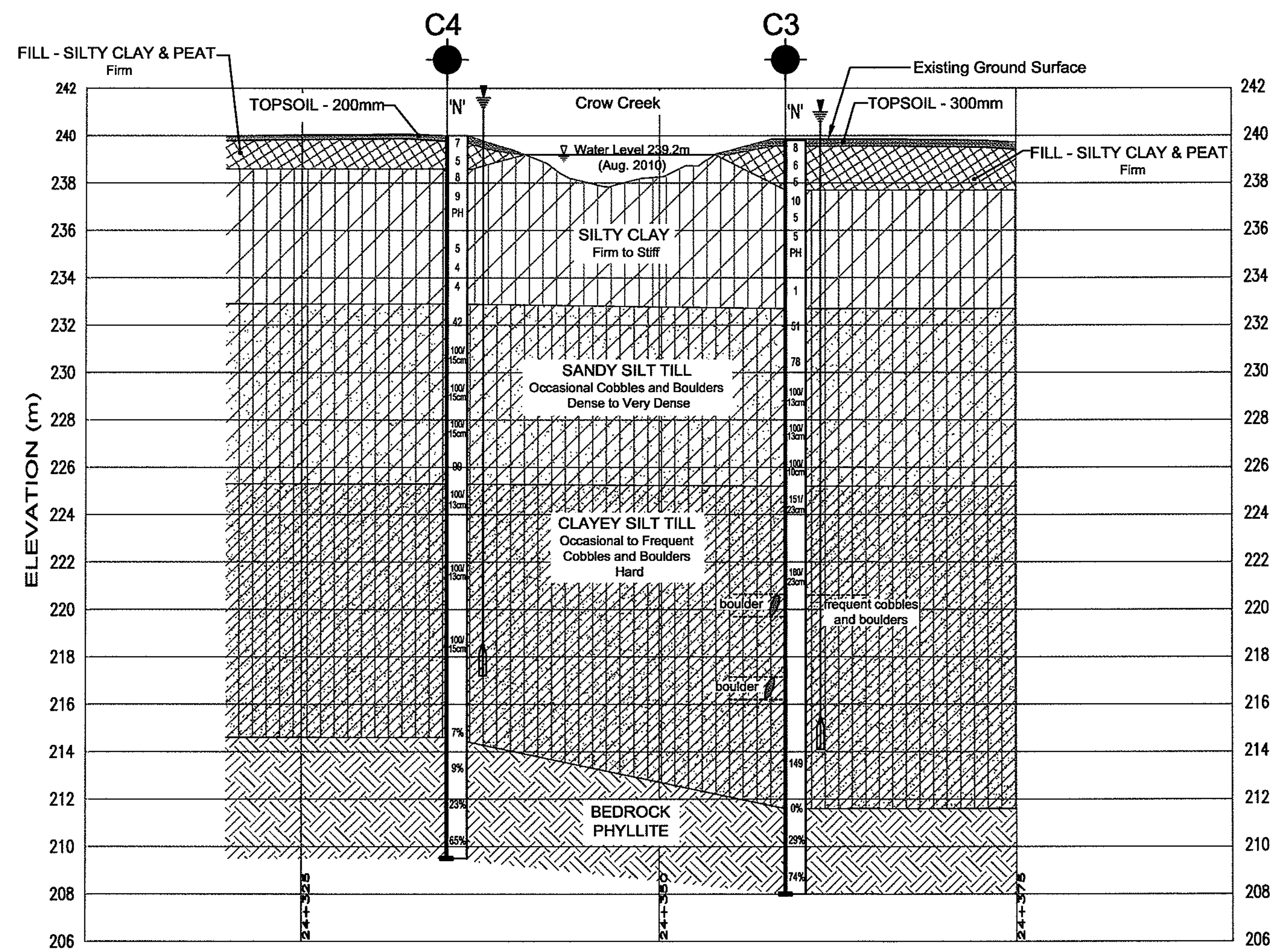


PROFILE - DETOUR ALIGNMENT

HORIZONTAL SCALE
5m 2.5 0 2.5 5m

VERTICAL SCALE
5m 2.5 0 2.5 5m

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
02.03.2011
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
March 02/2011
PROVINCE OF ONTARIO

CONT No
GWP No 5233-06-00

CROW CREEK DETOUR
BOREHOLE LOCATION AND SOIL STRATA

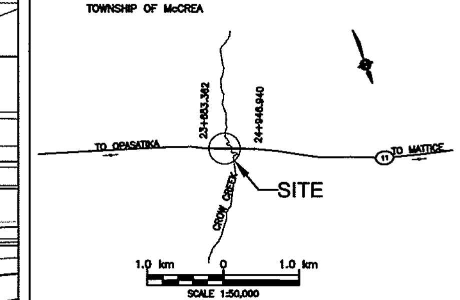
SHEET 3 OF 4

McCORMICK RANKIN CORPORATION

MRC

Terraprobe Inc.
Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
10 Bram Court - Brampton Ontario L6W 3R6 (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 (SEPT. 2010)
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

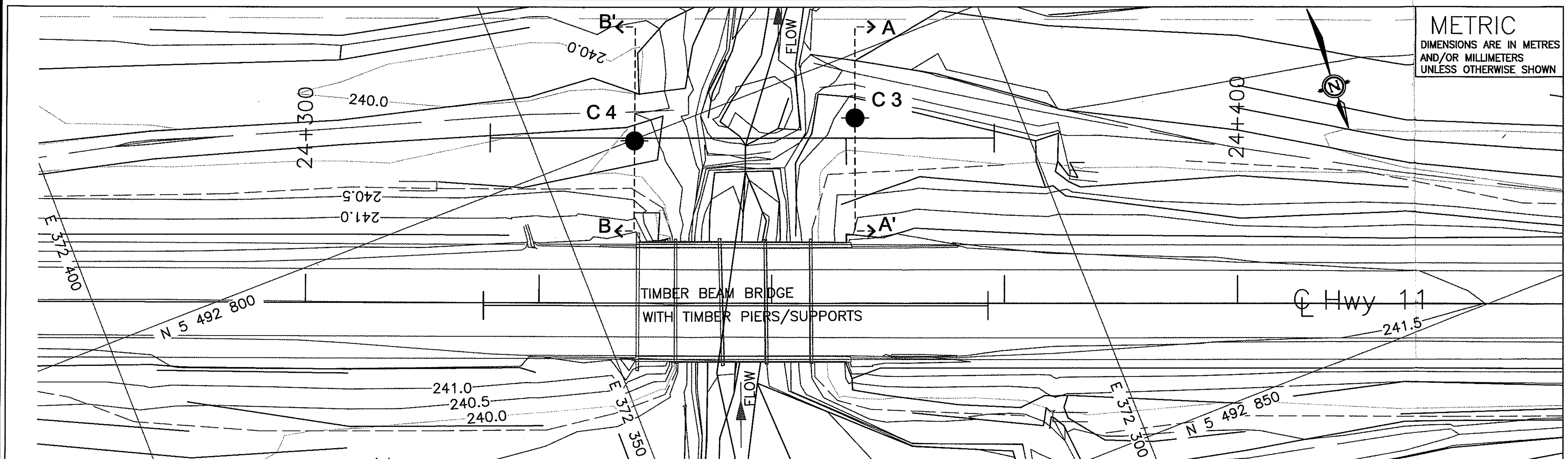
No	ELEV.	COORDINATES	
		NORTHING	EASTING
C3	239.8	5 492 806.5	372 316.7
C4	240.0	5 492 800.2	372 339.6

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.

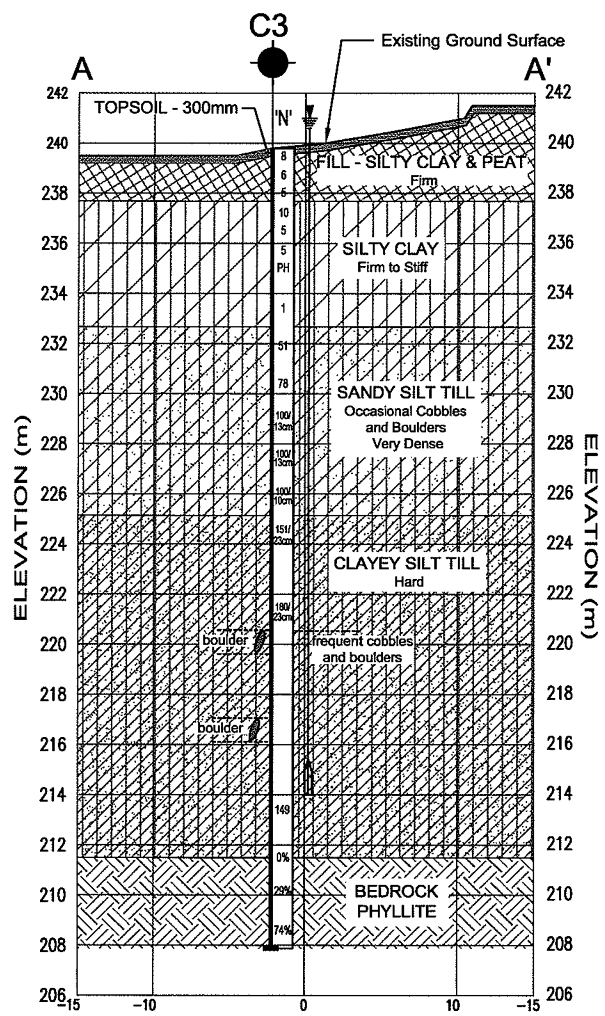
REVISIONS		
DATE	BY	DESCRIPTION

DESIGN R.A	CODE CHBDC2006	LOAD	DATE MARCH 2011
DRAWN K.C	CHK R.A	STRUCT 39W-055	GEOCRES 42G-33



PLAN

SCALE 5m 2.5 0 2.5 5m



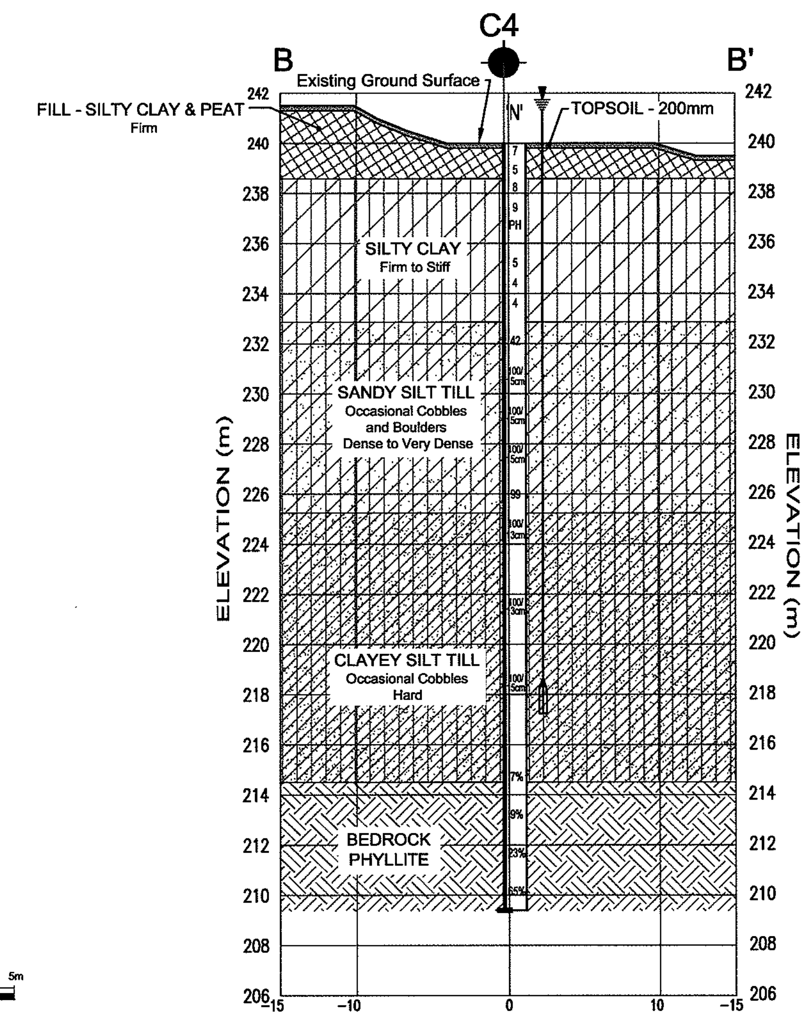
SECTION A-A'

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
02.03.2011
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
March 02/2011
PROVINCE OF ONTARIO

HORIZONTAL SCALE 5m 2.5 0 2.5 5m

VERTICAL SCALE 5m 2.5 0 2.5 5m



SECTION B-B'

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

CONT No

GWP No 5233-06-00

CROW CREEK DETOUR

BOREHOLE LOCATION AND SOIL STRATA

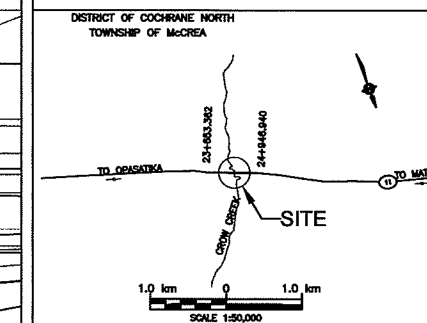
SHEET 4 OF 4

McCORMICK RANKIN CORPORATION

MRC

Terraprobe Inc.

Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
10 Bram Court - Brampton Ontario L6W 3R6 (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 (SEPT. 2010)
ǀ	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV.	COORDINATES NORTHING	COORDINATES EASTING
C3	239.8	5 492 806.5	372 316.7
C4	240.0	5 492 800.2	372 339.6

NOTE

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REVISIONS

DATE	BY	DESCRIPTION

DESIGN R.A	CODE CHBDC2006	LOAD	DATE MARCH 2011
DRAWN K.C	CHK R.A	STRUCT 39W-055	GEOCRES 42G-33

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APPENDIX D

TERRAPROBE INC.

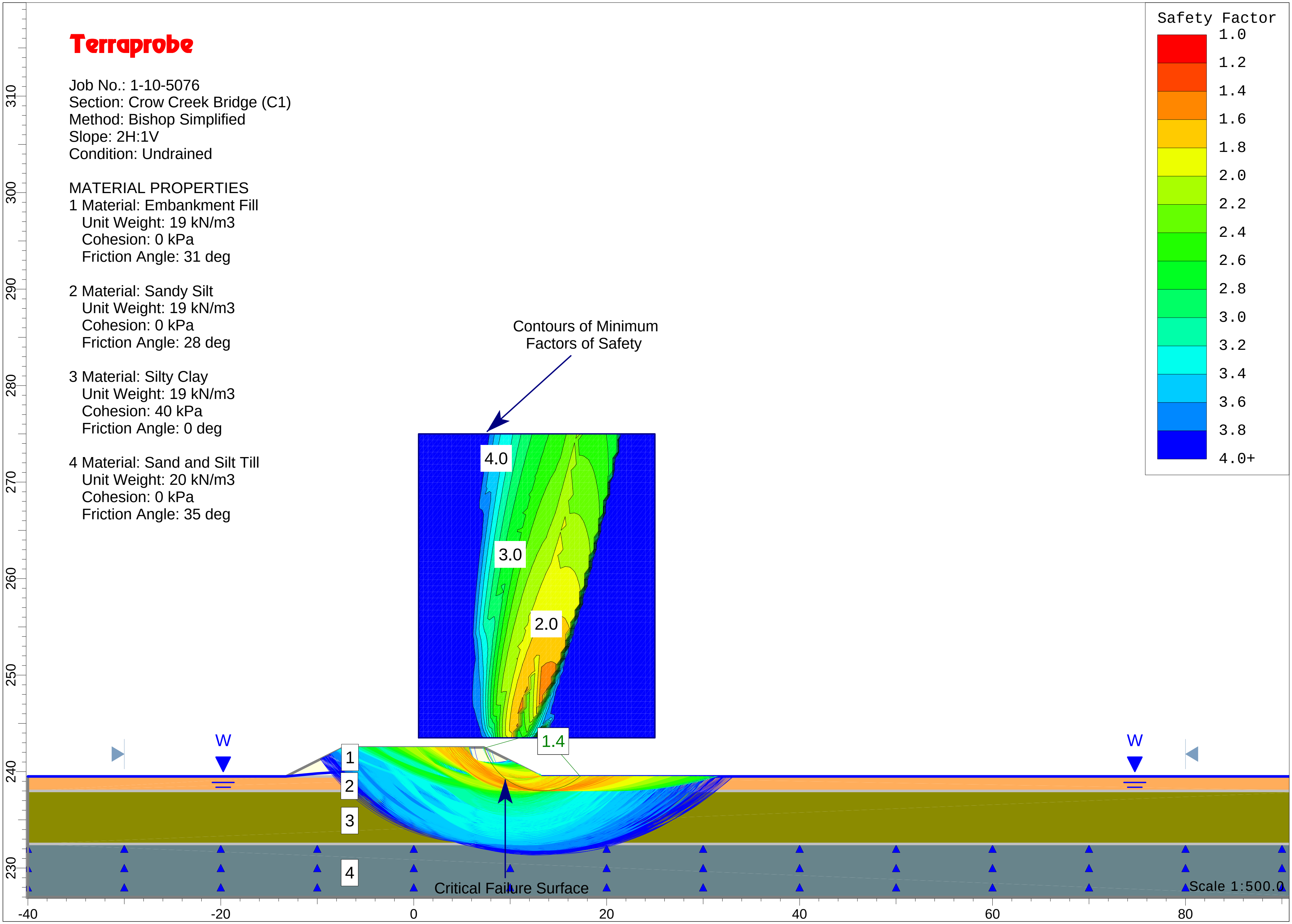
COMPARISON OF FOUNDATION ALTERNATIVES FOR EACH FOUNDATION ELEMENT

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 ALIGNMENT				
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
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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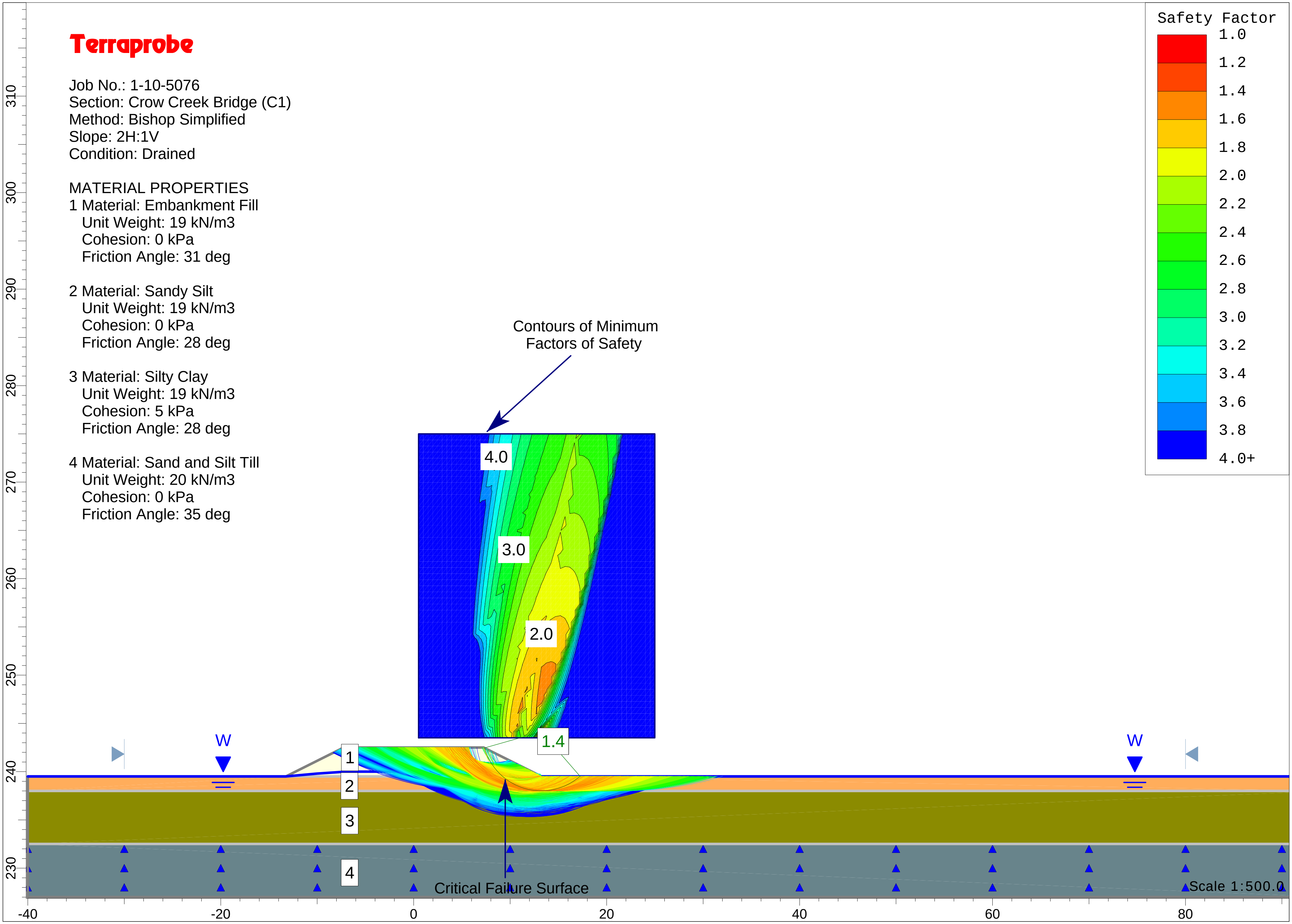

Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C1)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 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
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
1.4
W
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
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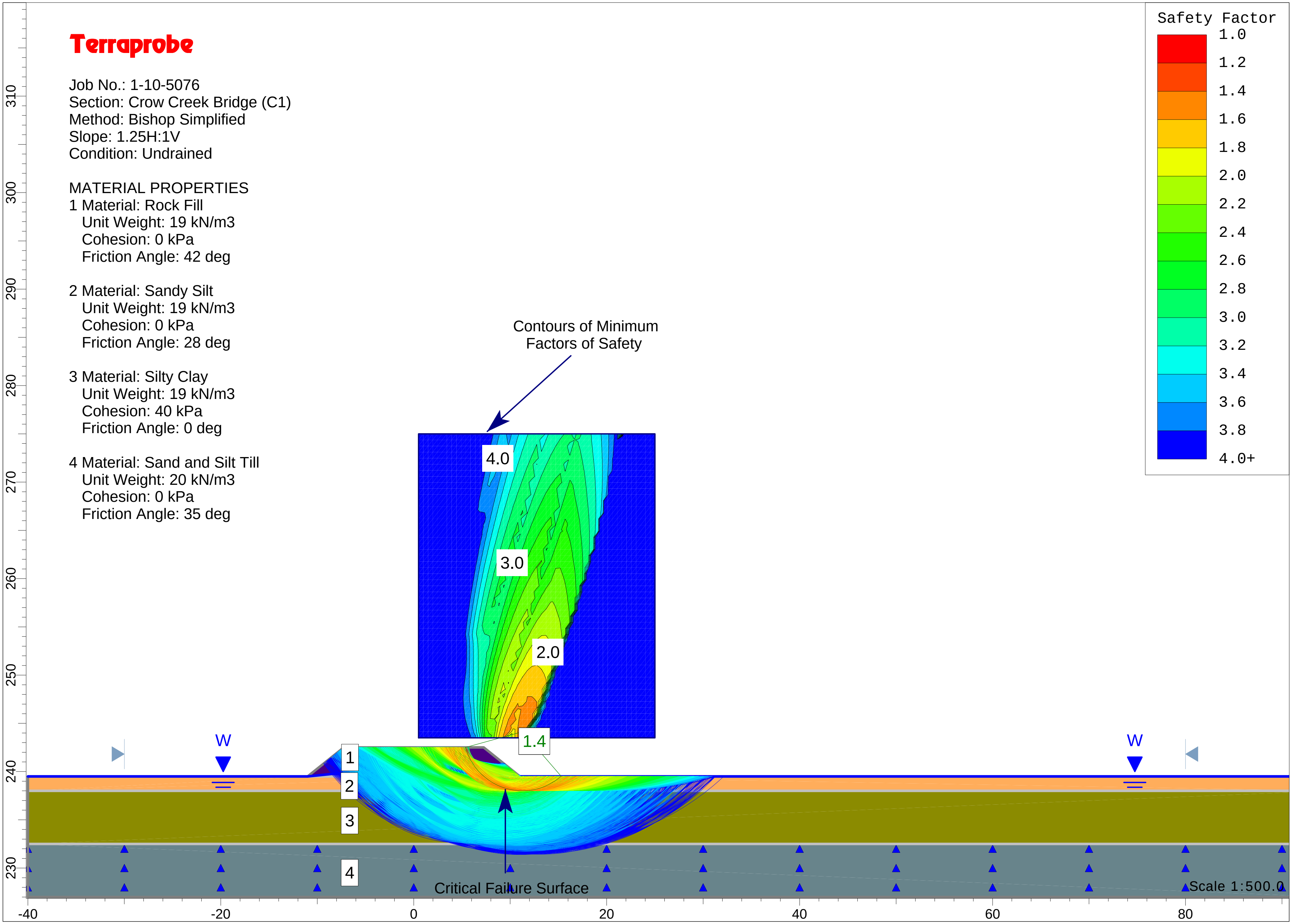
60
80


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
Critical Failure Surface
W
W
1
2
3
4
Safety Factor
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1.2
1.4
1.6
1.8
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4.0+
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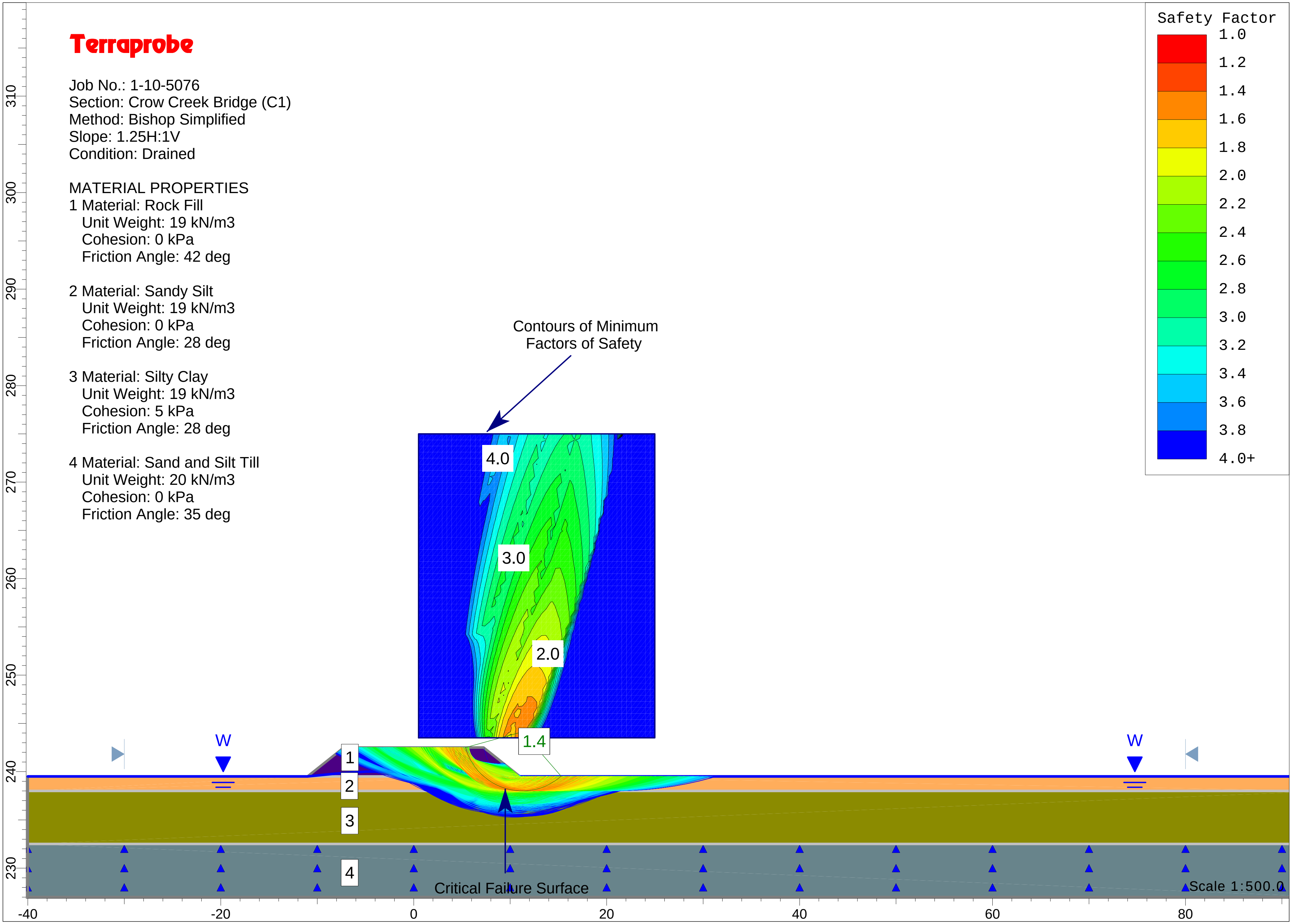
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C1)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 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
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+
W
W
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2
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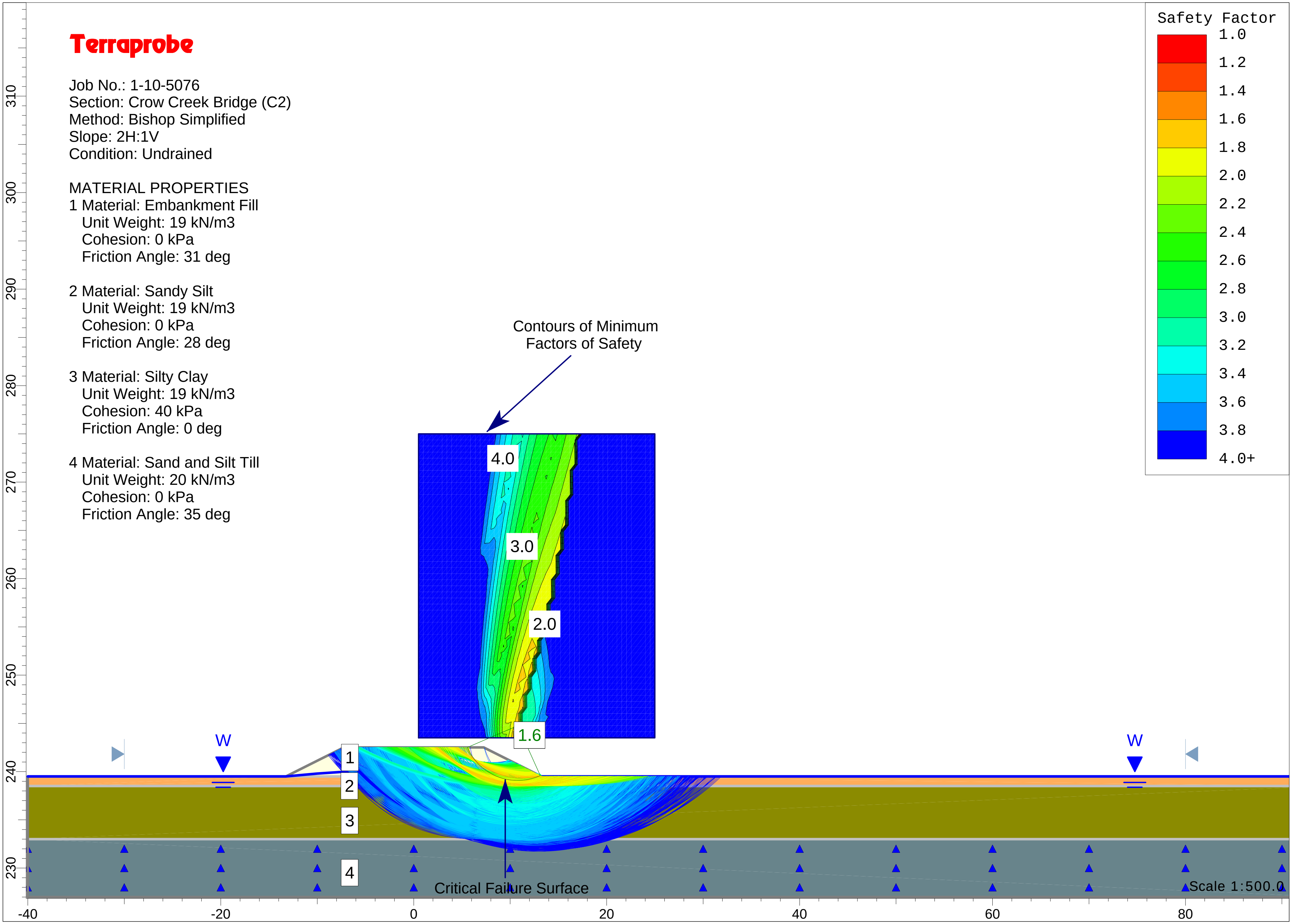

Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction 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
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3.2
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3.6
3.8
4.0+
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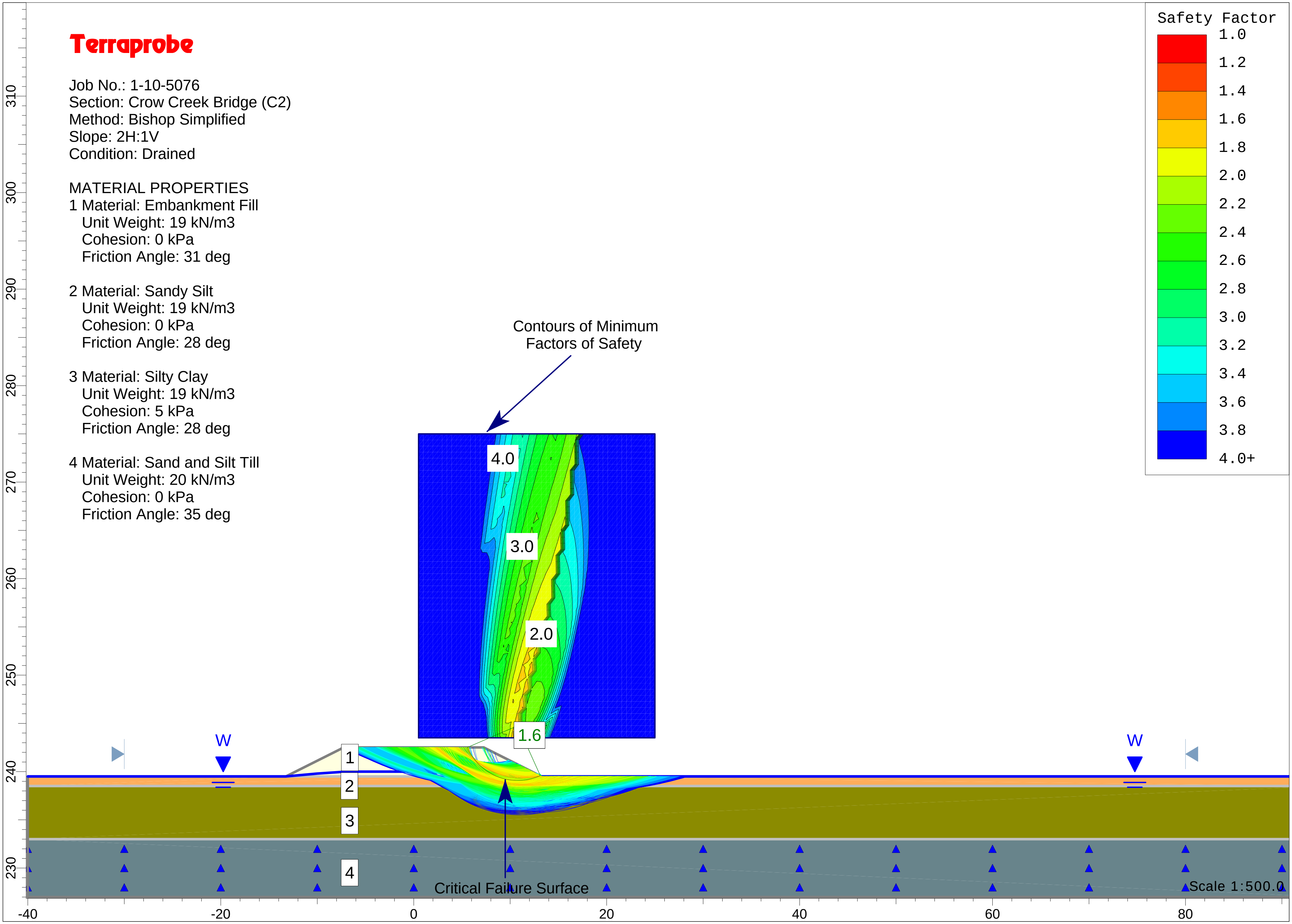

Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 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
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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+
Scale 1:500.0
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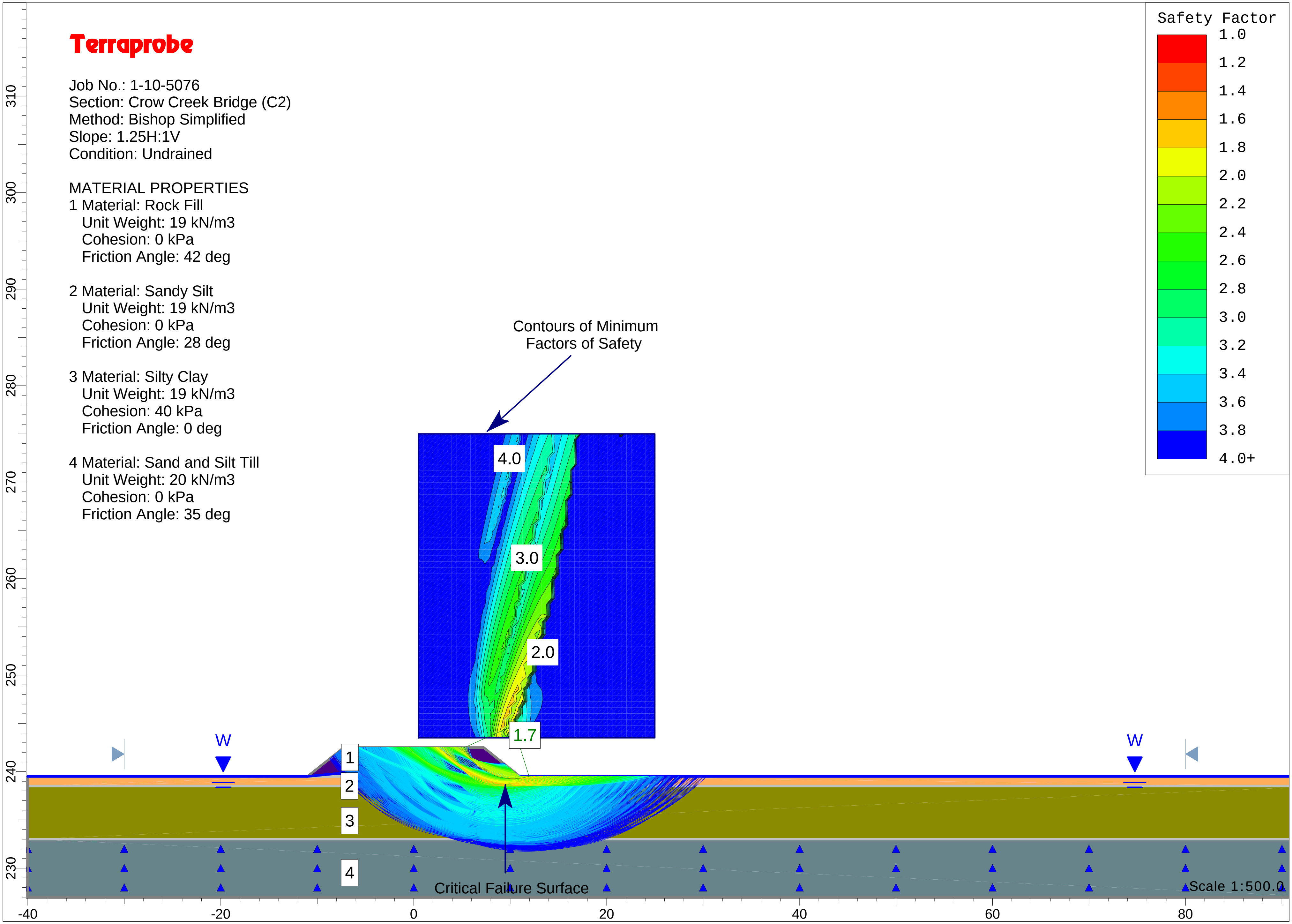
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 28 deg
3 Material: Silty Clay
Unit Weight: 19 kN/m3
Cohesion: 40 kPa
Friction Angle: 0 deg
4 Material: Sand and Silt Till
Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
Contours of Minimum Factors of Safety
4.0
3.0
2.0
1.7
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
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3.2
3.4
3.6
3.8
4.0+
Scale 1:500.0
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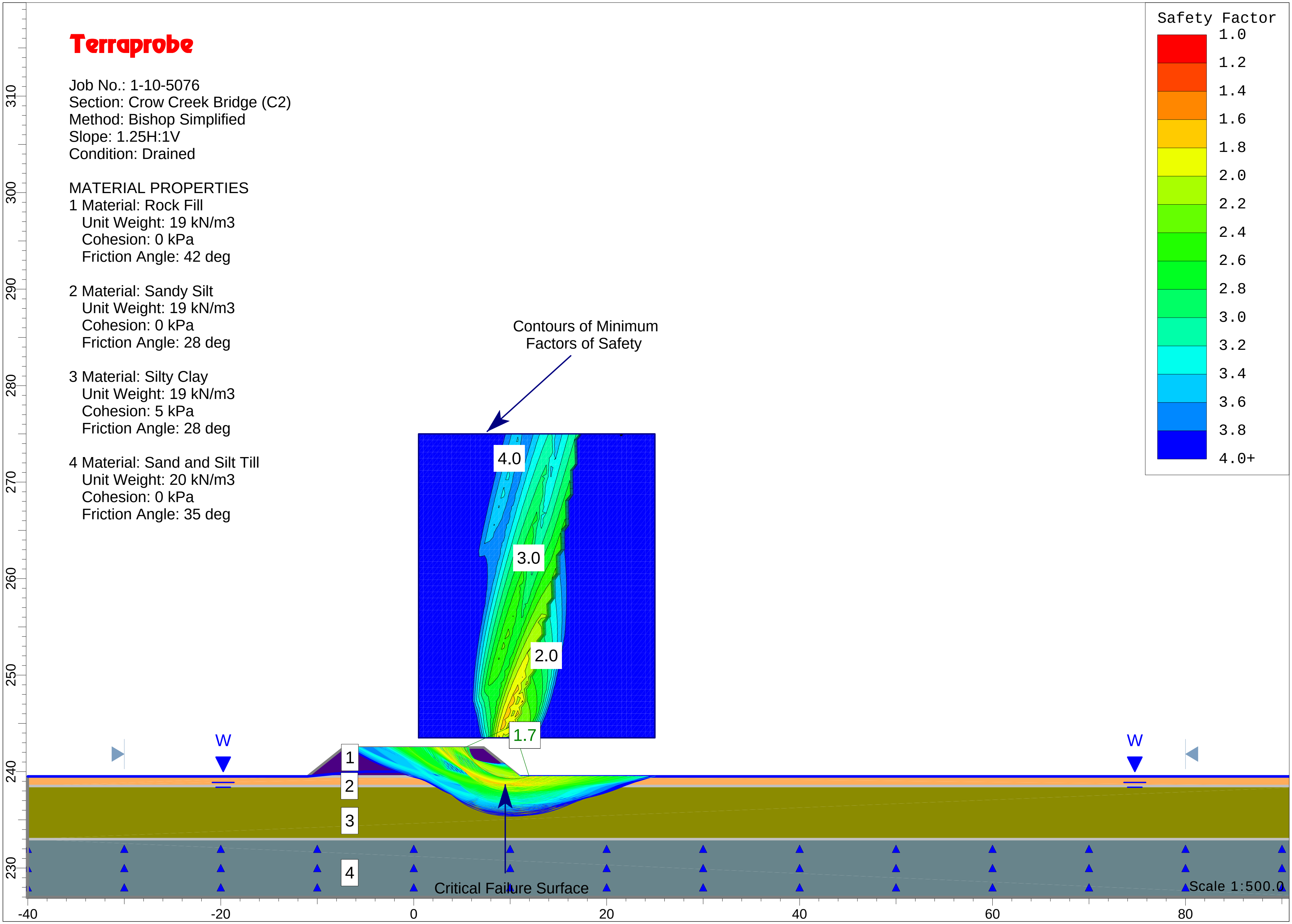
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Bridge (C2)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Sandy Silt
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 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
1.7
W
W
1
2
3
4
Critical Failure Surface
Scale 1:500.0
Safety Factor
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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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300
290
280
270
260
250
240
230
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-20
0
20
40
60
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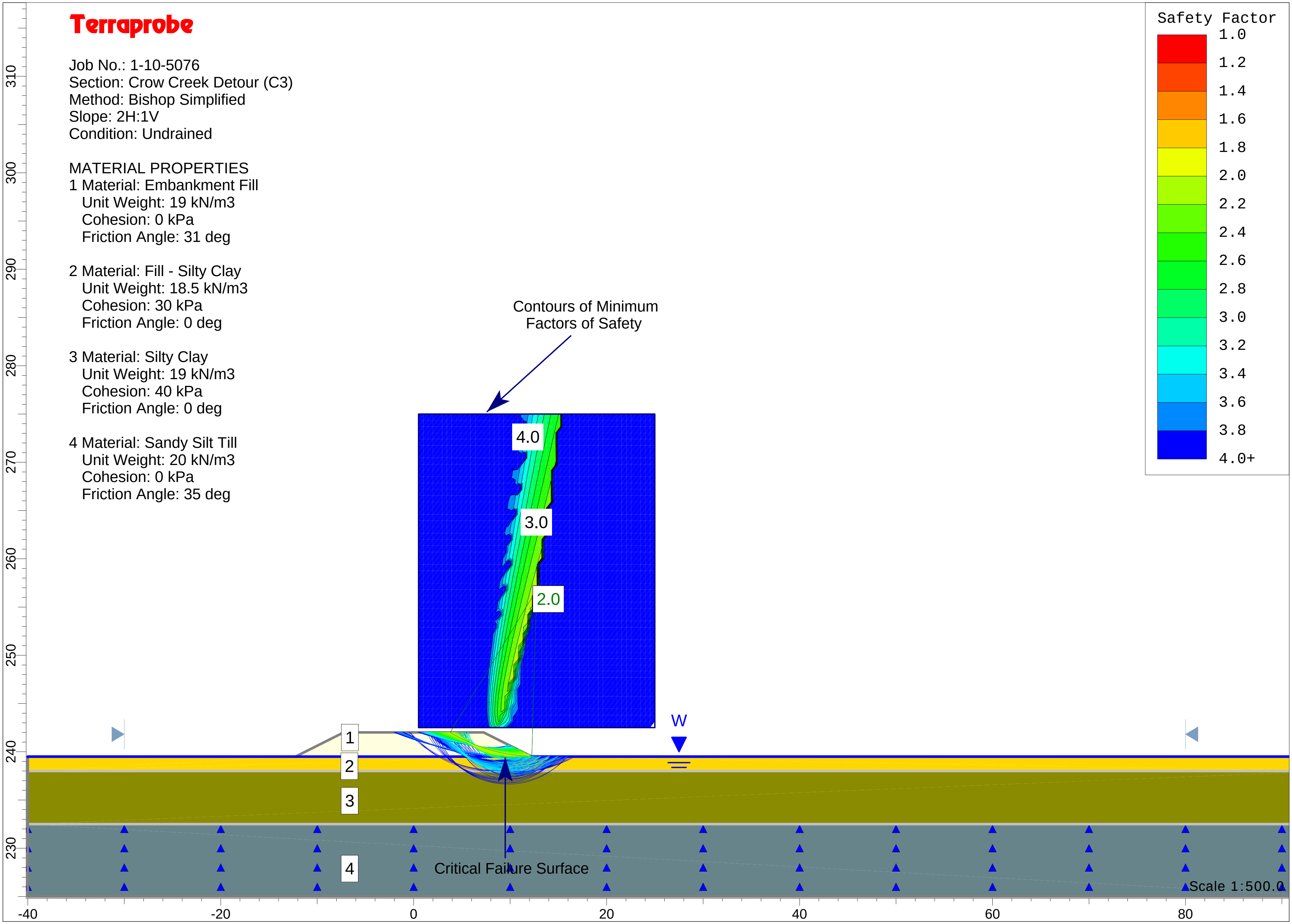
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 30 kPa
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
3.8
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60
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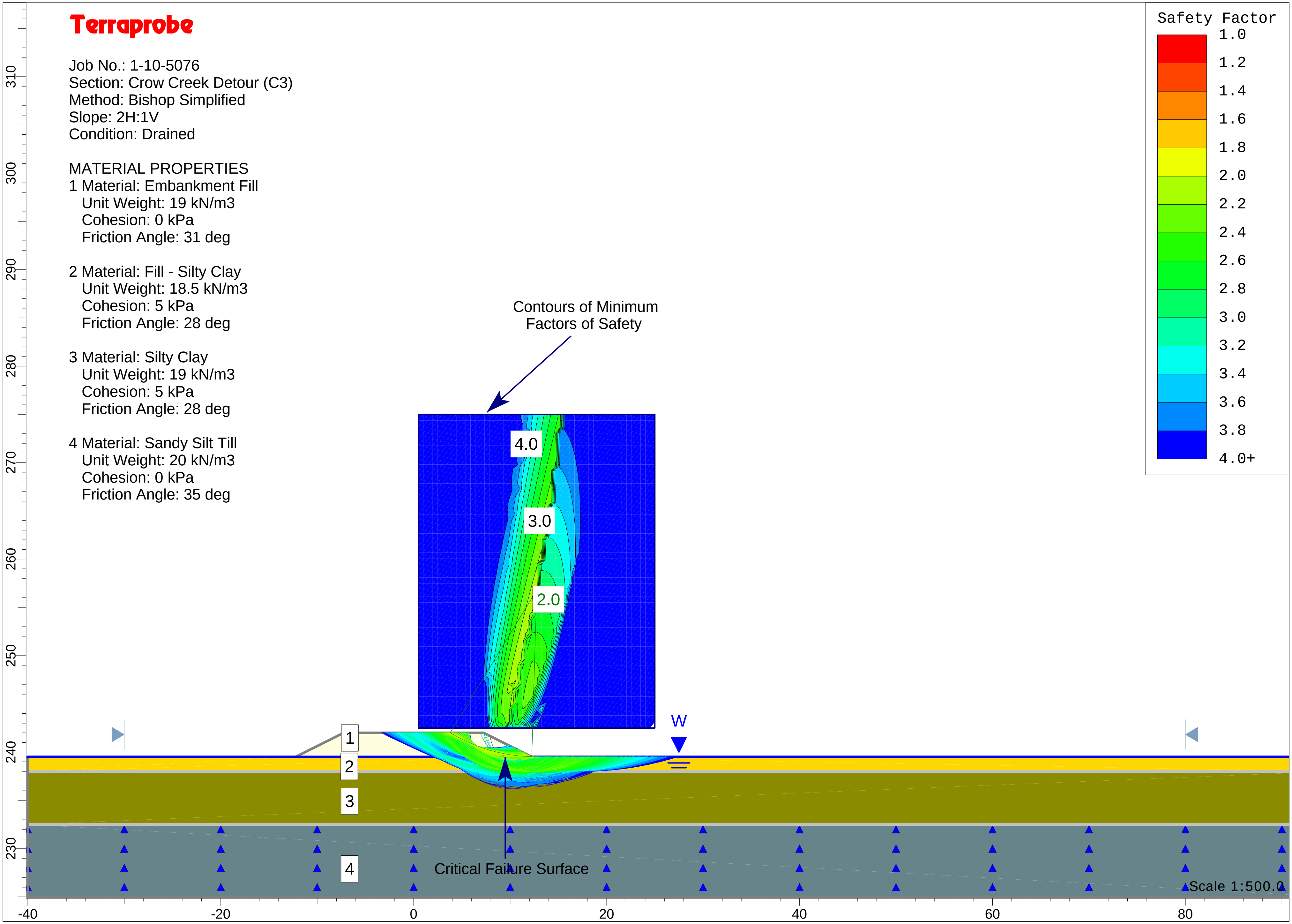
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 2H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Embankment Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 31 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 5 kPa
Friction 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
4.0+
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60
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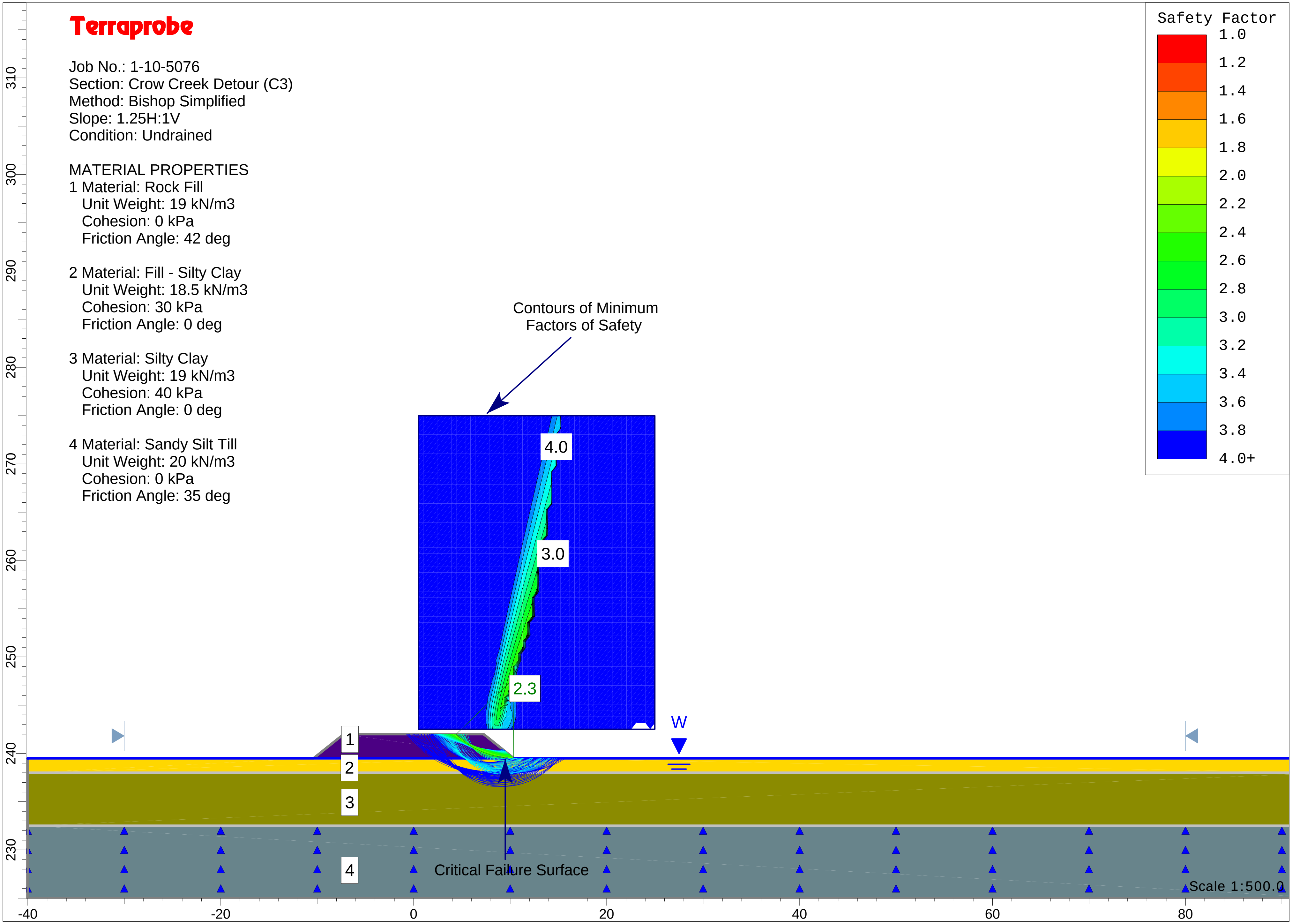
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 30 kPa
Friction 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
4.0
3.0
2.3
Critical Failure Surface
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+
Scale 1:500.0
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290
280
270
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-40
-20
0
20
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60
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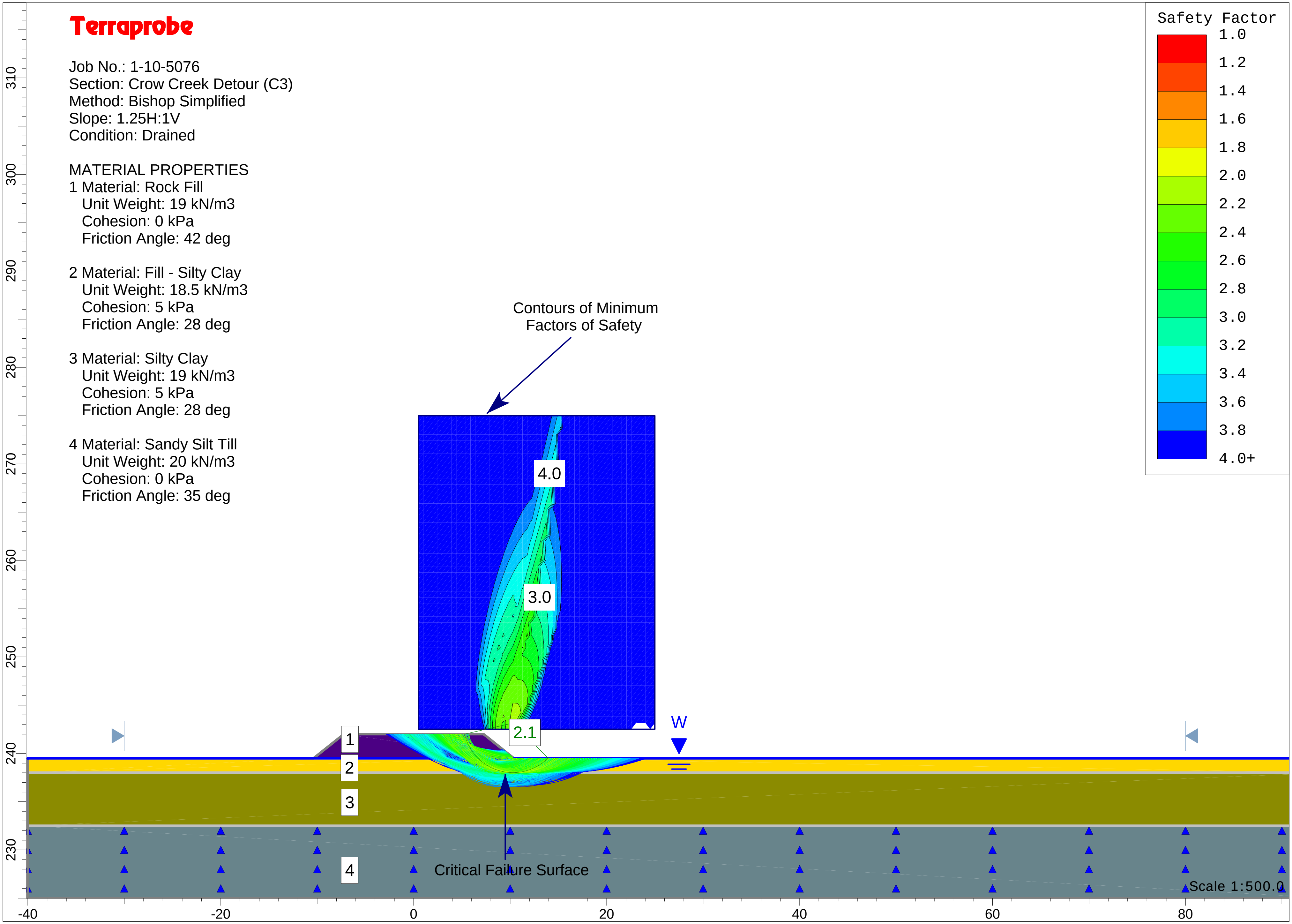
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C3)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
Cohesion: 5 kPa
Friction 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
2.8
3.0
3.2
3.4
3.6
3.8
4.0+
Scale 1:500.0
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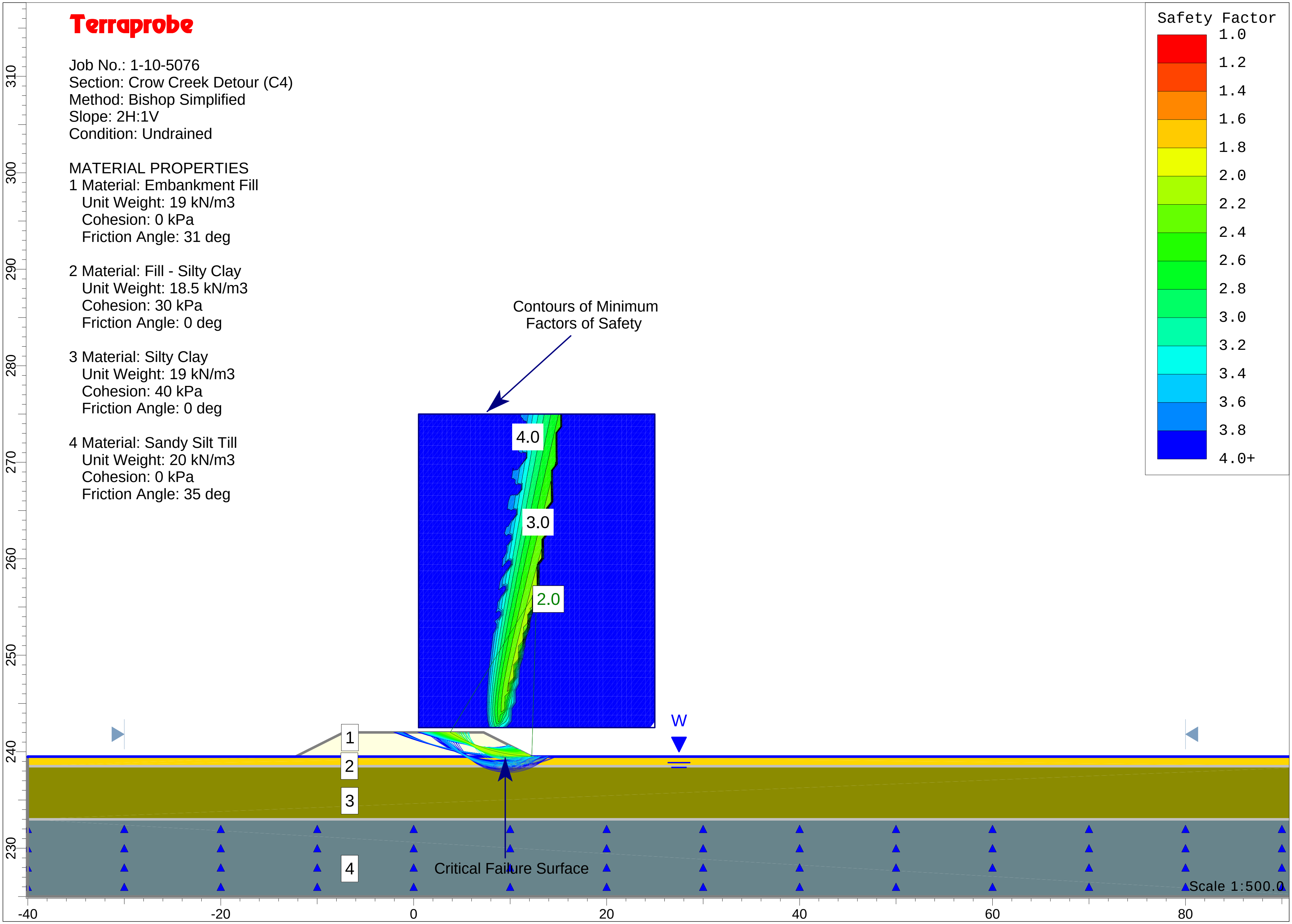
300
290
280
270
260
250
240
230
-40
-20
0
20
40
60
80



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
3.0
2.0
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
2.8
3.0
3.2
3.4
3.6
3.8
4.0+
Scale 1:500.0
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280
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260
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240
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-40
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0
20
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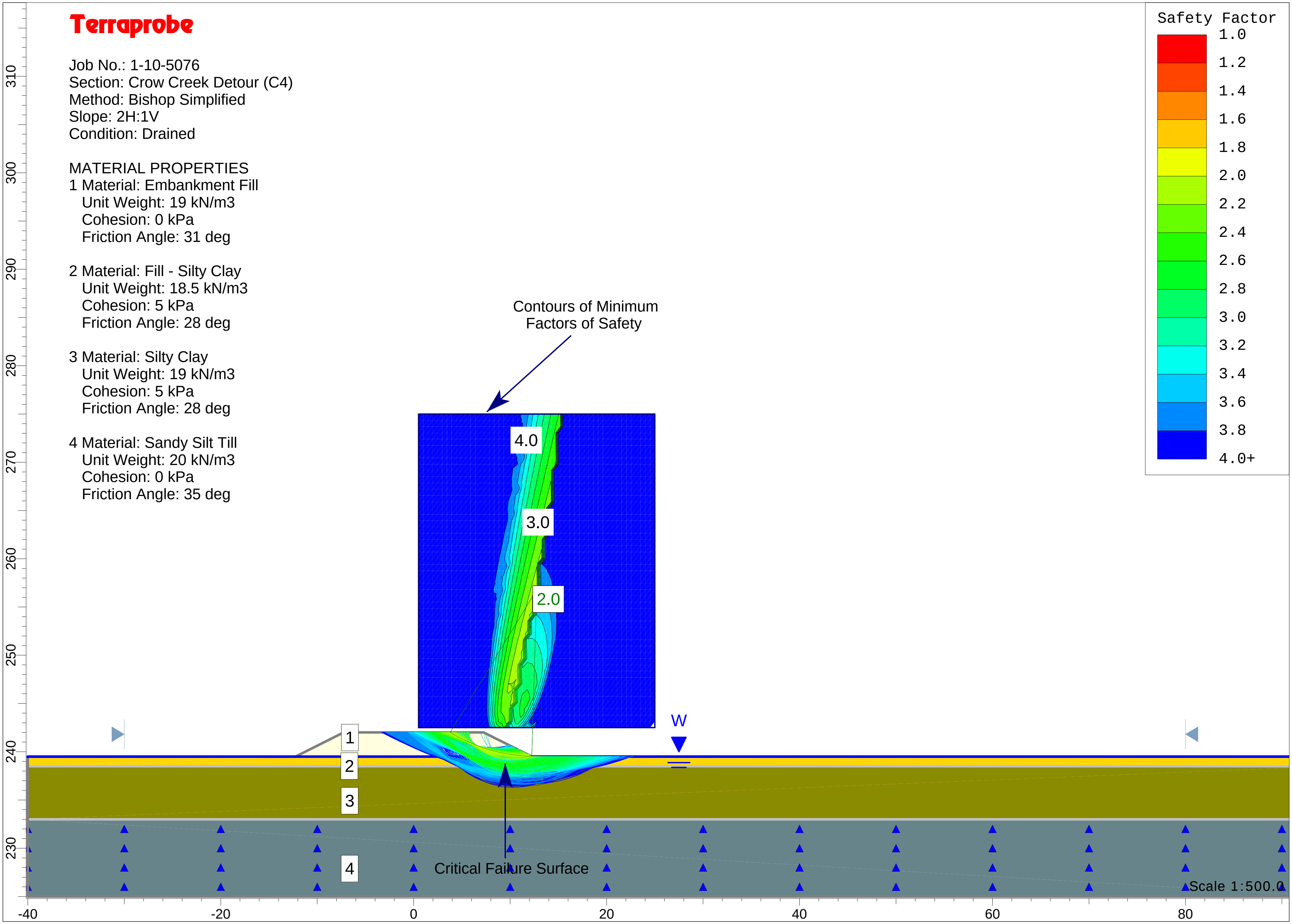
80


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
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
4.0+
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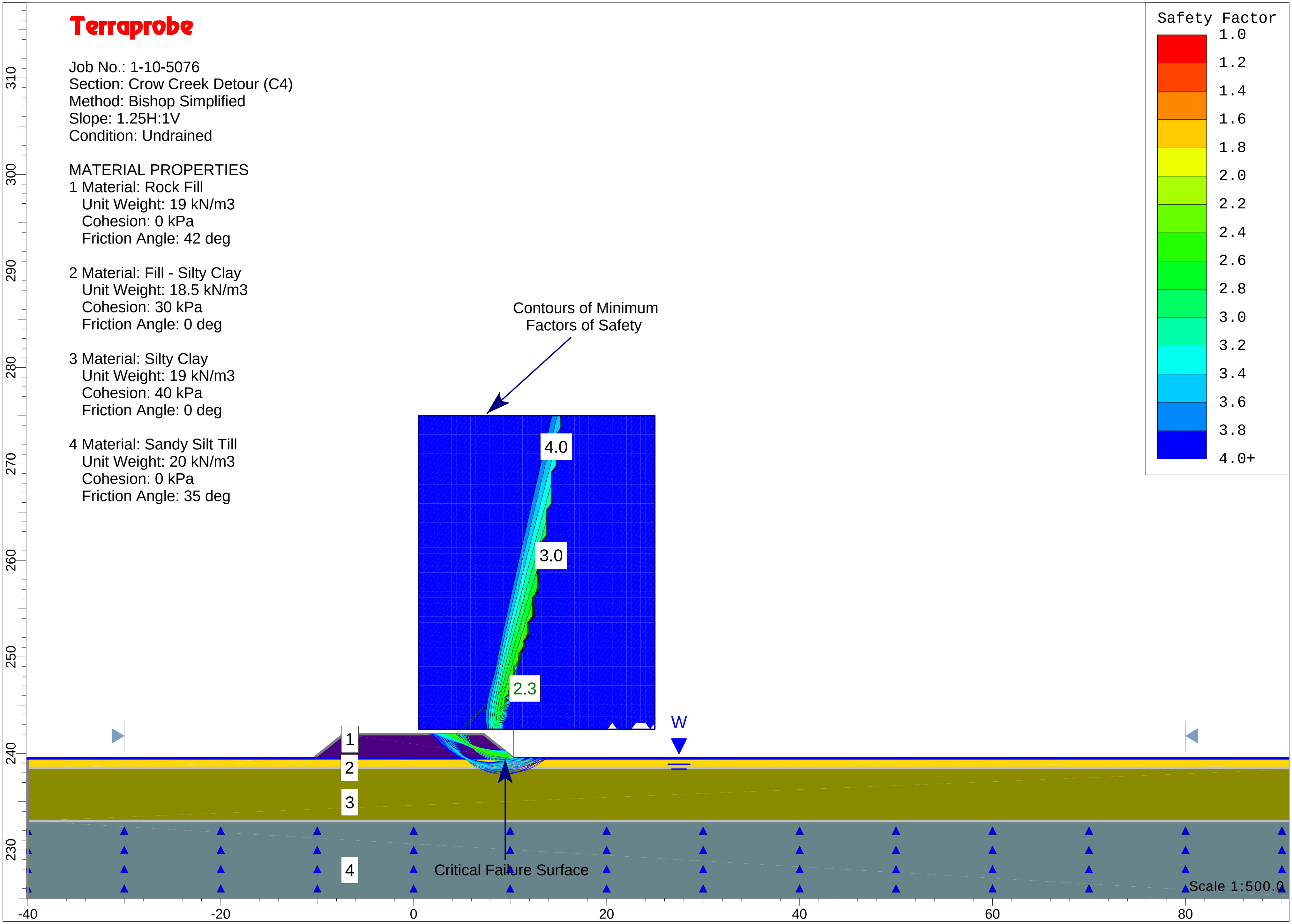
Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C4)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Undrained
MATERIAL PROPERTIES
1 Material: Rock Fill
Unit Weight: 19 kN/m3
Cohesion: 0 kPa
Friction Angle: 42 deg
2 Material: Fill - Silty Clay
Unit Weight: 18.5 kN/m3
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Friction Angle: 0 deg
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Unit Weight: 20 kN/m3
Cohesion: 0 kPa
Friction Angle: 35 deg
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Safety Factor
1.0
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2.2
2.4
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2.8
3.0
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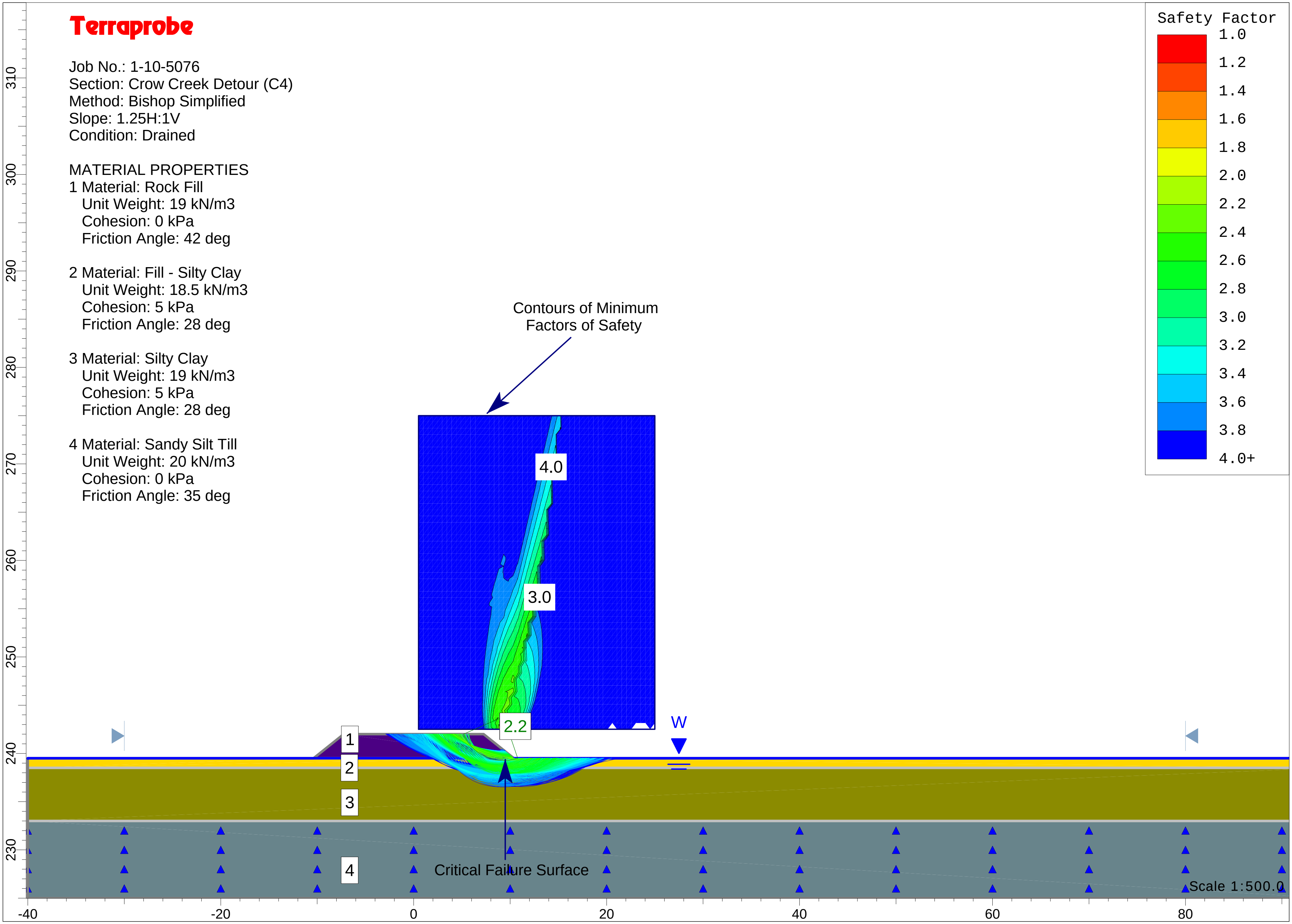

Terraprobe
Job No.: 1-10-5076
Section: Crow Creek Detour (C4)
Method: Bishop Simplified
Slope: 1.25H:1V
Condition: Drained
MATERIAL PROPERTIES
1 Material: Rock Fill
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Friction Angle: 35 deg
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