



Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

**FOUNDATION INVESTIGATION AND DESIGN REPORT
REMEDIAL MEASURES FOR
SOUTH EMBANKMENT SLOPE INSTABILITIES
HIGHWAY 140/CNR OVERPASS
AGREEMENT No. 2013-E-0051
G.W.P. 2044-13-00, GEOCRES No. 30L14-58**

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PART A – FOUNDATION INVESTIGATION REPORT

**REMEDIAL MEASURES FOR
SOUTH APPROACH EMBANKMENT SLOPE INSTABILITIES
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GEOCRES No. 30L14-58**



1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by D.M. Wills Associates Ltd. (D.M. Wills) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of the south approach embankment remediation of the Highway 140/CNR overpass, in Port Colborne, Ontario.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's Request for Quotation (RFQ) dated June 2011, and in Section 3.3 of D.M. Wills's *Technical Proposal* for this assignment.

This report provides compiled factual data on the subsurface conditions at the site, based on previous subsurface investigations. No new boreholes were drilled at this site by Terraprobe. The following documents are referenced in the preparation of this report:

- Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.
- Ministry of Transportation, "Foundation Investigation Report for Failure of Approach Embankments, Overhead Structure at the Crossing of Hwy. #140 and C.N.R.", W.O. 72-11025, W.P. 60-68-02, Contract 70-212, Geocres 30L14-045, dated July 17, 1972; and
- Ministry of Transportation, "Foundation Investigation Report for the Crossing of the C.N.R. Tracks and Proposed East Side Highway (Near Forkes Road)", W.O. 68-F-73, W.P. 60-68-02, District No. 4 (Hamilton), Geocres 30L14-036, dated December 4, 1968.
- Ministry of Transportation, Contract No. 98-116, W.P. No. 418-97-01.
- Ministry of Transportation, Contract No. 2010-2009, W.P. No. 2490-04-00.

2.0 SITE DESCRIPTION

The Hwy. 140/CNR overpass is located about 545± m north of the Hwy. 140/Forkes Road East intersection in Port Colborne, Ontario. The key plan on Drawing 1 provides an overview of the site location.

The south approach embankment of the overpass is approximately 300 m long and about 10 m high at the south bridge abutment. Embankment side slopes vary from about 2.5 Horizontal to 1 Vertical (2.5H:1V) to 4H:1V. Beyond the footprint area of the embankment the topography is generally flat with poor surficial drainage and vegetation consisting mainly of grass, shrubs and occasional small trees.

Hwy. 140 crosses the CNR track via a three span concrete bridge measuring about 40 m in length and, at the forward slope of the structure's south abutment is an east west oriented 3.6 m x 2.4 m concrete culvert measuring 43.5 m in length. On the east side of Highway 140, are two storm water ponds that were created by excavating the borrow material that was used to construct the bridge's approach embankments.



3.0 INVESTIGATION PROCEDURES

3.1 Current Investigation

The current investigation consisted of a desktop study of background information from MTO and a site reconnaissance. A visual inspection of the south embankment was carried out by Terraprobe on June 26, 2014 to observe and map performance related features such as cracks, settlement and unstable areas. The locations of these distress features (supplemented with photographs) are shown on Drawing 1, titled South Embankment, Existing Condition.

It is understood that embankment instabilities at the north approach embankments were rectified under Contract No. 2010-2009, G.W.P. No. 2490-04-00. Therefore, Terraprobe visually inspected the north approach embankment to assess its performance after remediation. The visual assessment concluded that there was no evidence of slope instability or guide rail movement on both the east and west slopes. The condition of the north approach embankment (with corresponding photographs) is illustrated on Drawing 2, titled North Embankment, Existing Condition.

3.2 Previous Investigations

This site was previously investigated by the Ministry of Transportation (Formerly Department of Highways Ontario and Department of Transportation and Communications) who drilled boreholes as part of the original foundation investigation for the CNR overpass structure and following the two failures of the approach embankments.

The site investigation for the CNR overpass structure and approach fills was undertaken between October 17 and November 1, 1968 and two boreholes (BH1 and BH3) were drilled in the footprint area of the south embankment. After the approach embankments failed, a site reconnaissance was carried out between July 13 to 20, 1971 and Boreholes 101, 102, 102A, 103, 103A and 104 and 105 were drilled in the area of the south embankment. A second failure occurred and additional field investigations were carried out by drilling Boreholes 208, 209, 210 and 211 during the period February 9 to 17, 1972.

During the period 1968 to 1972 a total of twelve boreholes were drilled and sampled to depths ranging from 4.6 m to 25.7 m below ground surface in the south embankment area. The borehole locations are shown on the attached Borehole Locations and Soil Strata Drawings, Drawings 3 and 4.

Samples of the overburden soils were obtained at selected intervals using a split spoon sampler in conjunction with Standard Penetration Testing (SPT). In the cohesive (clayey) deposits the undrained shear strength of the soil was measured in-situ by means of field vane tests and soil samples were also collected with thin-walled Shelby Tube samplers. Dynamic Cone Penetration tests were also performed in Boreholes 1 and 3 and the bedrock was cored by BX-size diamond coring techniques in Borehole 1.

Ground water conditions in the open boreholes were observed during the drilling operations and standpipe piezometers were installed in selected boreholes to permit longer term ground water level monitoring.



3.3 Borehole Locations

The borehole locations in MTM NAD 83 northing and easting coordinates, the ground surface elevations referenced to geodetic datum, the depths drilled and boring dates are tabulated below.

Borehole No.	MTM NAD 83 Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)	Boring Date
	Northing (m)	Easting (m)			
BH1	4,756,393.4	328,388.3	176.5	25.7	Oct. 17 – Nov. 1, 1968
BH3	4,756,418.2	328,411.9	176.9	24.5	Oct. 28 – 29, 1968
BH101	4,756,337.0	328,388.0	177.6	9.6	July 13, 1971
BH102	4,756,341.7	328,369.9	183.9	7.3	July 14 – 15, 1971
BH102A	4,756,341.7	328,369.9	183.9	13.9	July 15, 1971
BH103	4,756,344.4	328,355.7	184.3	10.8	July 15, 1971
BH 103A	4,756,344.4	328,355.7	184.3	6.9	July 19, 1971
BH104	4,756,351.4	328,388.3	177.6	4.6	July 20, 1971
BH208	4,756,278.7	328,388.8	183.7	25.3	February 9, 1972
BH209	4,756,268.4	328,352.9	180.6	9.6	February 14 – 15, 1972
BH210	4,756,357.3	328,363.4	185.1	25.3	February 15 – 16, 1972
BH211	4,756,301.4	328,374.7	178.4	9.6	February 17, 1972

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located in the physiographic region of Southern Ontario referred to as the Haldimand Clay Plain. The Haldimand Clay Plain is best described as falling into a series of parallel belts with the highest ground adjacent to the Niagara Escarpment. Generally this region is flat and poorly drained although it includes several distinctive landforms such as dunes, cobble, clay and sand beaches, limestone pavements and back-shore wetland basins¹.

The Niagara Region is underlain by a sequence of very gently south-dipping dolostones, limestones, shales and sandstones overlying Precambrian basement rock. The key elements in the bedrock geology of the region are the multiple layers of softer sedimentary limestones, shale, sandstone and dolostone.

The bedrock unit at this site consists of the Salina Formation of Upper Silurian Age². The Salina Formation consists essentially of easily weathered, grey, very finely crystalline, laminated argillaceous dolostone with grey, calcareous shale partings and gypsum veins and lenses of varying thicknesses.

4.2 Subsurface Conditions (South Embankment)

Reference is made to the Record of Borehole Sheets in Appendix A. Details of the encountered soil and rock stratigraphy are presented in this appendix and on the "Borehole Locations and Soil Strata" drawings. An overall description of the stratigraphy in the footprint area of the south embankment is

1 Chapman and Putnam, "The Physiography of South Ontario", 3rd Edition, 1984.

2 Ontario Division of Mines, "Quaternary Geology Of The Welland Area", Preliminary Map P.796, 1972.



given in the following paragraphs. However, the factual data presented in the Record of Borehole Sheets governs any interpretation of the site conditions.

The stratigraphic boundaries shown on the Record of Boreholes and on the interpreted stratigraphic profile and sections (Drawings 3 and 4) are in some instances inferred from non-continuous soil sampling and therefore represent transitions between soil types rather than exact planes of geological change. The subsurface conditions will vary between and beyond the borehole locations. The south embankment fill material may vary in elevation and material type because of subsequent maintenance treatments and repairs that were carried out by MTO at the south embankment as shown on Drawings 5 and 6.

In summary, the embankment consists of stiff to very stiff silty clay fill that is underlain by a firm to very stiff silty clay to clay deposit overlying very dense sand and gravel till and hard clayey silt till. These overburden soils are underlain by dolomite bedrock of the Salina Formation.

4.2.1 Roadway Fill

Fill material consisting of clayey silt with some sand and gravel exists in Borehole 3 extending to a depth of 1.4 m or elevation 175.5 m. A Standard Penetration test in the clayey silt fill measured an SPT 'N' value of 17 blows per 0.3 m of penetration, indicating a very stiff consistency.

4.2.2 Silty Clay Fill (Embankment Fill)

The south embankment is constructed with silty clay fill material. The locations, thicknesses and base elevations of the fill are summarized in the following table.

Borehole No.	Fill Thickness (m)	Fill Base Elevation (m)
BH102	7.3*	176.6
BH102A	7.5	176.4
BH103	6.9	177.4
BH103A	6.9*	177.4
BH208	8.3	175.4
BH209	4.4	176.2
BH210	7.2	177.9
BH211	2.6	175.8

* Borehole termination depth.

The 'N' values of Standard Penetration tests carried out in the silty clay fill varies from 4 to 52 blows per 0.3 m of penetration. Field vane tests measured in-situ undrained shear strengths ranging from more than 98 kPa to 138 kPa and unconfined compressive strength tests measured undrained shear strengths ranging from 54 kPa to 125 kPa. These results indicate that the consistency of the silty clay fill is generally stiff to very stiff.



Consolidated undrained triaxial tests were carried out on the silty clay fill by MTO in 1968 and the laboratory test results are illustrated on Figure B1 in Appendix B. The results are summarized below:

- Approximate Effective Cohesive Intercept (c') = 0 kPa to 6.2 kPa (0 psf to 130 psf); and
- Approximate Effective Angle of Internal Friction (ϕ') = 23°.

Atterberg limits tests were also carried out on seventeen samples of the silty clay fill and the results are plotted on the plasticity chart on Figures B2 to B4 in Appendix B. The results indicate that the silty clay fill is a cohesive soil of intermediate to high plasticity (CI to CH) with occasional silt inclusions. The Atterberg limits test results are summarized below.

Liquid Limit:	44% to 56 %
Plastic Limit:	24% to 30 %
Plasticity Index:	17% to 29 %
Natural Water Content:	24% to 30 %

The natural water content of thirty eight samples of the silty clay fill varies between 23% and 30% and its unit weight ranges from 19.2 kN/m³ to 20.3 kN/m³.

4.2.3 Granular 'A' (Embankment Fill)

After south embankment construction, repairs were required on many occasions and the last major rehabilitation was carried out under MTO Contract 98-116. Based on the MTO contract drawings the embankment remediation included upper slope flattening to at least 3H:1V with Granular 'A', a 200 mm earth cap and 50 mm of topsoil, seed and cover. The extent of the remediation is about 210 m long on the east slope (Sta. 15+410 to Sta. 15+620) and about 115 m long on the west slope (Sta. 15+500 to Sta. 15+615). Selected contract drawings depicting the repair details and the extent of the Granular 'A' in the embankment are provided as Drawings 5 and 6.

4.2.4 Clayey Topsoil

Below the embankment fill there exists a layer of clayey topsoil. The locations, thicknesses and base elevations of the clayey topsoil are summarized in the following table.

Borehole No.	Clayey Topsoil Thickness (m)	Clayey Topsoil Base Elevation (m)
BH102A	0.3	176.1
BH103	0.6	176.8
BH208	0.3	175.1
BH209	0.3	175.9
BH210	0.3	177.6

Standard Penetration tests in the clayey topsoil measure SPT 'N' values of 9 and 36 blows per 0.3 m of penetration. An in-situ field vane test carried out in the clayey topsoil measured an undrained shear strength of 72 kPa and unconfined compressive strength tests carried out on two samples of



the clayey topsoil measured undrained shear strengths of 78 kPa. Based on these tests the clayey topsoil is described as having a generally stiff consistency.

The natural water content of a sample of the clayey topsoil is 32%.

4.2.5 Silty Clay to Clay

The site is underlain by a cohesive silty clay to clay deposit. The locations, thicknesses and base elevations of the silty clay to clay deposit are summarized in the following table.

Borehole No.	Silty Clay to Clay Thickness (m)	Silty Clay to Clay Base Elevation (m)
BH1	23.9	152.6
BH3	22.4	153.1
BH101	9.6*	168.0
BH102A	6.1*	170.0
BH103	3.3*	173.5
BH104	4.6*	173.0
BH208	16.7*	158.4
BH209	4.9*	171.0
BH210	17.8*	159.8
BH211	7.0	168.8

* Borehole termination depth.

The 'N' values of Standard Penetration tests carried out in the silty clay to clay deposit ranges between 4 and 68 blows per 0.3 m of penetration. Field vane tests measured in-situ undrained shear strengths ranging from 50 kPa to more than 115 kPa and laboratory vane tests measured undrained shear strengths ranging from 55 kPa to 88 kPa. Unconfined compressive strength tests measured undrained shear strengths ranging from 40 kPa to 137 kPa and unconsolidated undrained triaxial tests measured undrained shear strengths varying from 50 kPa to 90 kPa. These undrained shear strength results indicate that the consistency of the silty clay to clay deposit is generally firm to very stiff.

A consolidated undrained triaxial test was carried out on the silty clay to clay deposit and the laboratory test results (parent material) are illustrated on Figure B1 in Appendix B. The results are summarized below:

- Approximate Effective Cohesive Intercept (c') = 13.8 kPa (288 psf); and
- Approximate Effective Angle of Internal Friction (ϕ') = 25°.

Samples of the silty clay to clay soils were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figure B5 in Appendix B. The test results and borehole data show a grain size distribution consisting of 0% gravel, 1% sand, 22% to 54% silt and 45% to 77% clay sized particles.

Occasional silt layers were encountered in the silty clay to clay deposit and the grain size distribution curve of a sample of the silt (Borehole 3, Sample 12) is also shown on Figure B5, Appendix B. The



grain size distribution of the silt consists of 0% gravel, 0% sand, 90% silt and 10% clay sized particles.

Atterberg limits tests were also carried out on twenty-nine samples of the silty clay to clay and the results are plotted on the plasticity chart, Figures B6 to B9 in Appendix B. The results indicate that the silty clay to clay is a cohesive deposit of generally intermediate to high plasticity (CI to CH) with occasional low plasticity clayey silt (CL) and cohesionless silt inclusions. The Atterberg limits test results are summarized below.

Liquid Limit:	16% to 33 %
Plastic Limit:	27% to 64 %
Plasticity Index:	11% to 37 %
Natural Water Content:	19% to 50 %

The natural water content of forty four samples of the silty clay to clay varies between 19% and 50% and its unit weight ranges from 17.8 kN/m³ to 20.4 kN/m³.

Consolidation tests were also performed on samples of the silty clay to clay and the results are presented in Figure B10 in Appendix B. The test results related to the south embankment soils are summarized below.

Borehole/Sample No.	Sample Depth/Elevation (m)	P _c (kPa)	C _c	e _o
BH1, Sample 10A	10.4/166.1	590	0.282	0.676
BH3, Sample 12	15.3/161.5	465	0.503	0.909

Where: P_c = Preconsolidation pressure;
C_c = Compression index; and
e_o = Initial void ratio

4.2.6 Sand and Gravel Till

In Borehole 1 the silty clay to clay deposit is underlain by a 300 mm thick layer of sand with gravel glacial till. A Standard Penetration test carried out in this sand and gravel till deposit measured an SPT 'N' value of 40 blows for 0.1 m of penetration suggesting a very dense relative density.

4.2.7 Clayey Silt Till

In Borehole 3 the silty clay to clay deposit is underlain by a 700 mm thick layer of clayey silt glacial till. A Standard Penetration test carried out in this clayey silt till deposit measured an SPT 'N' value of 70 blows for 0.15 m of penetration suggesting a hard consistency. The natural water content of a sample of this deposit is 10%.

4.2.8 Bedrock

In Borehole 1 the overburden soils are underlain by dolomite bedrock that was encountered at a depth of 24.2 m (Elev. 152.3 m) below ground surface. BX size rock coring equipment was used to core the bedrock to a depth of 1.5 m below the bedrock surface and the rock core recovery was recorded as 80%.



The bedrock is described as sound dolomite with gypsum lenses and containing occasional layers of calcareous shale.

4.3 Ground Water Levels

The ground water depths and elevations below the footprint area of the south embankment as recorded in the historical borehole logs are tabulated below.

Borehole No.	Date	Water Levels	
		Depth (m)	Elevation (m)
BH1	November 1968	0.9*	175.6
		7.6*	168.9
BH3	October 29, 1968	1.5*	175.4
BH101	July 13, 1971	1.2*	176.4
BH102	July 15, 1971	Dry	-
BH102A	July 15, 1971	8.0*	175.9
BH103	July 15, 1971	7.8*	176.5
BH 103A	July 19, 1971	Dry	-
BH104	July 20, 1971	0.6*	177.0
BH208	February 1972	Dry	-
		4.5*	179.2
BH209	February 1972	0.8*	179.8
BH210	February 1972	2.6*	182.5
BH211	February 17, 1972	3.1*	175.3

◆ Water level in borehole

● Recorded water level in standpipe piezometer.

Based on the piezometer installations and the Year 1968 readings indicated above, the ground water level in the silty clay to clay deposit (prior to fill placement) ranged from elevation 175.4 m to 175.6 m. The piezometric ground water level in the glacial till underlying the silty clay to clay deposit was measured as elevation 168.9 m. These observations would indicate that there is some downward drainage from the upper silty clay to clay stratum down into the glacial till.

Following fill placement the ground water level in the silty clay to clay deposit (Years 1971 and 1972) rose to elevations ranging from 175.9 m to 179.2 m which indicates the build-up in excess pore water pressure due to the fill loads. Water level observations in the fill material are erratic ranging from Dry in some boreholes to about elevation 182.5 m in Borehole 210.

The ground water levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year. Perched water can also be expected to occur where surficial layers of gravelly sands are underlain by relatively impermeable silty clay soils.



5.0 MISCELLANEOUS

Messrs. Hussein Ahmed, Ashkan Abouzar and Rehman Abdul carried out the site reconnaissance and visual inspections of the north and south embankments. This report was prepared by Mr. Hussein Ahmed, P.Eng., and Mr. Rehman Abdul, P.Eng., a Senior Geotechnical Engineer and Associate with Terraprobe with assistance provided by Mr. Ashkan Abouzar, P.Eng. Mr. Michael Tanos, P.Eng., Terraprobe's MTO's Designated Contact conducted an independent quality control review.

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PART B – FOUNDATION DESIGN REPORT

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6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This report addresses the Phase 1 component of this study and contains recommendations to address the rehabilitation strategy for the south embankment instability. This section of the report presents an interpretation of the factual geotechnical data and provides geotechnical design recommendations for remediation of the south approach embankment of the Hwy. 140/CNR overpass. The recommendations are based on:

- our understanding of the project based on our review of background information on the original construction, past embankment failures and remediation efforts;
- observations of the south embankment and the performance of the remediated north approach embankment during the site visit on June 26, 2014;
- our interpretation of the factual data from the previous subsurface investigations;
- back analysis to emulate the observed south embankment slope instabilities and establish geotechnical parameters for embankment remediation; and
- analysis of remediation alternatives.

The discussion and recommendations provided are intended to provide information to enable the design team to assess feasible and practical remediation options and to select a preferred alternative for the south embankment repairs.

Where comments are made on construction, they are provided to highlight those aspects that could affect the design of the project, and for which special provisions or operational constraints may be required in the Contract Documents. Those requiring information on the aspects of construction should make their own interpretation of the factual information provided, as such interpretation may affect equipment selection, proposed construction methods, scheduling and the like.

Before remediation options can be developed to address the on-going poor performance of the south embankment it is first necessary to assess and understand the potential cause(s) of the distress. The assessment has involved the following main components:

- Review of background information on the site history and the north embankment study carried out under MTO Agreement No. 2008-E-0013 (2009 study);
- Field reconnaissance to assess the south embankment distress and to assess the performance of the repaired north embankment based on the 2009 study; and
- Identification of potential mechanisms causing distress.

6.2 Site History

The site's historical embankment performance and a chronology of the key events based on our review of the information in the MTO Geocres system is outlined below.

- **October to November 1968** - MTO performs a foundation investigation for the design of the Hwy. 140/CNR overpass foundations and embankments.
- **May 1971** - the north and south embankments of the Highway 140/CNR overpass are constructed at 2H:1V side slopes up to about 9.5 m high with locally sourced native clay soil excavated from borrow pits located next to the highway alignment and north and south of the



CN rail tracks. Based on visual observations during construction, the clay material from the south pit (used to construct the south embankment) appeared to be of a higher moisture content than optimum. North embankment construction begins on May 4, 1971, followed by south embankment construction which started on May 21, 1971. During construction MTO personnel observed that the fill was placed directly on existing ground without removing the topsoil. The surface drainage in the vicinity of the embankments was generally poor at the time of placement with numerous areas of ponded surface water. The fill material placed in the lower portion of the embankment along the south approach appeared to have a higher natural water content making compaction difficult. Notes from this time suggest that the embankment fills were placed during unfavourable/wet weather conditions.

- **July 05, 1971** - instability occurs on a portion of the side slope of the south embankment when the embankment is within about 1.2 m of its final grade. The slope failure is approximately 150 m long and is described as consisting of as much as 0.6 m of subsidence at the crest with longitudinal tension cracks up to 1 m wide in the main body of the embankment (from 1.5 m to 9 m on either side of the embankment centreline). Bulging by as much as 1 m beyond the original embankment geometry is evident at the embankment toes.
- **July 09, 1971** - MTO (Mr. M. Devata) inspects the instability and also examines several test pits in the failed area. A soft, thin layer (about 0.3 m thick) of cohesive organic material is identified in the test pits at the fill material/natural subsoil interface. The tension cracks observed at the surface of the failed roadway are reported to extend down to the soft organic layer at the original ground surface. Water seepage into a test pit excavation is noted at one location.
- **August 30, 1971** - instability occurs on the north embankment approximately 1.5 months after the south embankment showed signs of distress. The degree of distress is reportedly less than the failure on the south embankment.
- **September 14, 1971** - south embankment is repaired by constructing approximately 6 m to 10 m wide mid-height berms over a length of about 175 m. The surficial organic material at the toe of the original section was also removed to a depth of about 0.6 m and the sub-excavation was backfilled and compacted with acceptable earth material.
- **September 22, 1971** - north embankment is repaired using the same scheme used to repair the south embankment.
- **October 01, 1971** – a second failure occurs at the south embankment, this time on the west side slope eventually enveloping the east side. The magnitude and extent of the subsidence, tension cracks and toe bulging are reportedly very similar to those that occurred in July 1971.
- **January 1972** – a failure occurs on the side slopes of the north embankment during the first week of January. The extent of the failure is less severe than the previous failure and it is reported that the west side of the embankment showed more distress than the east.
- **January 1973** - MTO carries out site visits to assess the performance of the repaired embankments. Tension cracks oriented parallel to the highway are observed on the embankments' upper slopes (above the mid-height berms). Localized surficial sloughing is also evident on the 2H:1V upper portions of the slope above the berm, particularly on the east and west sides of the north embankment. Seeding and mulching applied in the fall of



1972 was reportedly not successful due to lack of root development. During this time, site visits to several nearby embankments associated with the St. Lawrence Seaway Authority were also carried out. Observations indicate that where embankments are constructed with the local silty clay, those with 3H:1V side slopes appeared stable, while those with 2H:1V side slopes show signs of surficial instability similar to what was being experienced at this site.

- **April 1973** – a site visit carried out by MTO confirms similar observations to those in January 1972 i.e. localized surficial sloughing is evident in the upper portion of the embankment side slopes. Additionally, subdrains in the granular backfill behind bridge abutments are not functioning.
- **December 1990** – a site visit is carried out by MTO to assess embankment performance. Localized surficial sloughing is evident on the portion of the upper 2H:1 V slopes above the berms; particularly on the east and west sides of the north embankment. The slopes below the berm appear to be reasonably stable.
- **June 1991** – a site visit is carried out by MTO to assess embankment performance. Tension cracks are noted on both sides of the upper slopes of the north embankment. The cracks extend about 200 m north of the bridge and are oriented parallel to the roadway along the shoulders. Tension cracks (about 60 m in length and up to 20 cm wide) are also noted on the east side of the lower slopes along the edge of the berm. Surficial sloughing is observed on the upper slopes above the berm of the north embankment, particularly on the east side and localized circular failures (about 16 m wide) are observed on the east and west sides of the south abutment forward slope.
- **October 1991** – a site visit is carried out by MTO to assess embankment performance. Tension cracks are observed oriented parallel to the highway on the shoulders of the north embankment upper slopes (extending about 260 m north of the bridge) and at the south embankment (extending about 250 m south of the bridge). Surficial sloughing is noted on the upper slopes above the berm of the north embankment over a 100 m long section and, at the south embankment over a 25 m long section. Settlements of up to 80 cm are associated with the surficial failure along the upper slope of the south approach. Tension cracks are also noted on all of the lower slopes along the edges of the berms. Localized circular failures (about 16 m wide) are observed on the east and west sides of the south abutment forward slope and; on the east and west sides of the north abutment forward slopes these circular failures are about 10 m wide. Localized slope failure has occurred on the lower slope of the east side of the north embankment fill, causing bulging at the toe of the slope.
- **1991 to 1997** - although not explicitly documented in the Geocres literature, it is believed that additional remedial measures in the form of placement of granular blankets may have been carried out at select locations of the slopes (most likely within the approach areas and/or on the embankment front slopes) during this time.
- **August 1997** – a site visit is carried out by MTO to assess embankment performance. Confined to the areas above the berms and close to the top of the slope, slope distress and surficial movements have occurred. No instabilities were identified below the berms.
- **Mid-1998** - under MTO Contract No. 98-116, repairs are carried out on the south embankment. Based on the contract drawings the embankment remediation is to include



upper slope flattening to at least 3H:1V with Granular 'A', a 200 mm earth cap and 50 mm of topsoil, seed and cover. The extent of the remediation is about 210 m long on the east slope and about 115 m long on the west slope.

- **1998 to 2006** - following slope flattening at the south embankment in 1998, no additional repairs or maintenance was required on this embankment and there have been no reports of tension cracks, surficial instability or other signs of embankment distress. However, the north embankment continues to experience distress and is performing poorly as evidenced by the formation of tension cracks, surficial instability and associated sloughing near the top of embankment. These problems have required annual maintenance (once or twice per year) consisting generally of granular fill placement by end-dumping and blading/spreading on both shoulders.
- **Mid-2006** - additional repairs are carried out on the north embankment under MTO Contract No. 2006-2034. Repairs included excavation and re-grading of sloughed material on the embankment side slopes above the berm, placement of topsoil, seed and cover, guard rail removal and reinstatement, mountable curb and gutter installation along the shoulders, and construction of concrete outlets and rip-rap lined channels with geotextile at select intervals along the slope face.
- **September 2006** - within about six weeks after completing Contract No. 2006-2034, localized surficial sloughing and settlement of the new guard rail was reported along several sections of the east and west sides of the north embankment and approach.
- **August 2008** - the ministry issues MTO Consultant Agreement No. 2008-E-0013 (2009 study) to assess causes of the embankment distress and recommend remedial measures.
- **2009 to 2010** - MTO issues Contract No. 2010-2009 which includes provisions for repairing the north embankment. The embankment was repaired by slope flattening to 2.5H:1V side slopes with Granular 'A' (not more than 5% passing the No. 200 sieve) that is benched 2 m wide and 1 m high into the existing fill.
- **2014** - tension cracks and localized settlement has occurred on both the west and east sides of the highway along the south embankment over a distance of 150 m. Terraprobe is selected to provide foundation engineering services to assess causes of the embankment distress and recommend remedial measures.

6.3 Review of Existing Data

The embankments have been affected by slope instabilities ranging from large crest-to-toe failures (with associated tension cracks and slope subsidence that extended behind the embankment crests); to surficial sloughing confined to the near surface embankment slopes. Various levels of repairs were carried out such as:

- construction of wide mid-height stability berms (at both embankments);
- granular blanket overlays (near the highest portions of the embankments within the approach areas and on the front slopes);
- application of seed and mulch;
- slope flattening with granular fill; and



- slope re-grading and drainage improvement measures including installation of curb-and-gutter and construction of rip-rap lined drainage channels down the slope face(s).

Repairs consisting of deep benching with Granular 'A', a 200 mm earth cap and 50 mm of topsoil, seed and cover that were implemented at the south embankment in 1998 and the north embankment in 2010, appear to have been most effective in stabilizing large areas of the embankment with little evidence of subsequent problems. However, evidence of tension cracks and surficial sloughing on the upper embankment slopes have persisted at the south embankment outside of the area repaired under MTO Contract 98-116.

Observations of the performance of other fill embankments in the Welland area constructed with local clay sourced from the Haldimand clay plain appear to have similar problems i.e. surficial slope instability for embankments constructed with steeper than 3H:1V side slope profiles.

The previous 2009 study included a meeting and site walkover with Mr. Brian Minor, the MTO Maintenance Coordinator for Central Region Operations, Niagara. Mr. Minor had been involved with the site since about 1970 and his insights as reported in the 2009 study are provided verbatim below:

- Distress on the embankments is usually in the form of settlement of the fill materials near the embankment crests and associated movement (settlement and tilting) of the guide rails. However, slumping on the slope faces has also occurred;
- Maintenance has been required on a regular and annual or semi-annual basis at both embankments (north and south) during most of their design lives;
- Maintenance typically involves the placement (essentially end-dumping) of granular fill on the shoulders of the embankment, followed by 'blading' the fill to level it off with little to no compactive effort;
- Since the remediation (i.e. slope flattening) at the south embankment in or about 1998, no annual maintenance has been required and performance has been satisfactory; and
- Prior to remediation (i.e. slope grading and installation of curb-and-gutter and guide rail replacement) at the north embankment in 2006, settlement and/or sloughing at the upper embankment crests had become so severe that in some places the posts for the guide rail were no longer embedded in the embankment and the cables were suspending the posts.

6.4 Field Reconnaissance

The south and north approach embankments were visually inspected by Terraprobe on June 26, 2014 to observe and map performance related features such as cracks, settlement and unstable areas. The locations of these distress features supplemented with photographs, are shown on Drawings 1 and 2 titled South Embankment Existing Condition and North Embankment Existing Condition respectively. The distress features are summarized below.

South Embankment – East Side

- Sta. 15+345 to Sta. 15+360 – a 25 m long crack oriented parallel to the highway and located on the east side of the guide rail. The embankment in this area is about 2.5 m high with a 2.75H:1V side slope;



- Sta. 15+375 to Sta. 15+387 – a 12 m long crack oriented parallel to the highway and located on the west side of the guide rail i.e. on the highway shoulder. The embankment is about 3.5 m high with a 2.75H:1V side slope; and
- Sta. 15+463 to Sta. 15+475 – a 12 m long crack oriented parallel to the highway and located on the west side of the guide rail i.e. on the partially paved highway shoulder. The embankment is approximately 7.5 m to 8.0 m high with a 3.5 H:1V side slope.

South Embankment – West Side

- Sta. 15+385 to Sta. 15+465 – an 80 m long crack oriented generally parallel to the highway and meandering east and west of the guide rail. The embankment is about 5.5 m to 7.5 m and the side slopes vary from 2.5H:1V to 3.5H:1V;
- Sta. 15+450 – multiple cracks at the toe of the mid-height berm. The embankment is approximately 7.5 m high with an upper embankment side slope of 2.65H:1V and a lower embankment side slope of 3.5H:1V; and
- Sta 15+490 – the bed of the existing drainage swale is eroded.

North Embankment – East and West Sides

- The existing east and west side slopes are well vegetated and the guide rails are perfectly aligned. No signs of embankment instability such as cracks and areas experiencing sloughing/sliding were observed. The drainage swales extending from the embankment crest to its toe are performing well and there is no evidence of erosion in these areas.

6.5 Potential Failure Mechanisms

Based on our review of the background information and the field reconnaissance, four mechanisms were identified as potential contributors to the south embankment distress. These mechanisms are:

- Compression/settlement of the embankment fill;
- Settlement of the foundation soils;
- Global embankment stability; and
- Surficial embankment stability

Each mechanism was assessed and the details of the analysis, the results and the potential contribution or significance of each mechanism to the on-going distress are described in the following sections.

The data used to analyse each mechanism was obtained from existing borehole data and laboratory testing. Since the north and south embankment fill soils were obtained from borrow pits in close proximity to each other, it is logical to assume that the fill soils of both embankments will have similar engineering properties. Therefore, the laboratory test data of the north embankment fill was also incorporated into the south embankment analysis.



6.5.1 Embankment Fill Settlement

To estimate the magnitude of compression or settlement that may have occurred within the embankment fill under its own weight, an analysis was carried out at the highest section of the south embankment where cracking was observed at the crest i.e. 8 m high section at Sta. 15+475.

The total settlement was calculated using the equation:

$$S = \{\Delta e / (1 + e_0)\} \times H_0$$

where Δe = total change of void ratio from Year 1971 to 2008;

e_0 = void ratio of the embankment fill in Year 1971; and

H_0 = thickness of fill layer

Based on the fill's unit weight, specific gravity and moisture content data; the void ratio of the fill in Year 1971 was calculated and the results (Figure C1 in Appendix C) suggest a design e_0 value of 0.75. Two one-dimensional consolidation tests were carried out the north embankment clay material as part of the 2009 study and the laboratory results are provided in Figures C7 and C8, in Appendix C. Based on these laboratory results, the interpreted Year 2008 void ratio ($e_{Yr\ 2008}$) and unit weight values (provided in the following table) were used for the settlement analysis.

Embankment Height (m)	Average Unit Weight (kN/m ³)	e_0	$e_{Yr\ 2008}$
7.5	19.5	0.75	0.72

The total fill compression that would have occurred after construction of an 8 m high embankment is approximately 135 mm. Estimating the length of time required to achieve 90% of this total settlement is difficult. The length of the drainage path (single lift versus the total embankment height) is debatable and the coefficient of consolidation (c_v) values interpreted from the laboratory consolidation test results range widely from about 2.85×10^{-2} cm²/s to 5.57×10^{-2} cm²/s. Using these c_v values and assuming double drainage within the embankment fill, the time to reach 90% consolidation is about 30 to 60 days.

Since the fill was initially placed in 1971 (almost 43 years ago); it is reasonable to assume that fill settlement is complete. Since the additional fill loads from the 1998 repairs are negligible (not more than 0.5 kPa), further settlement after the 1998 construction would be negligible. Therefore, settlement due to fill compression is not considered to be a contributing factor to the observed embankment distress and the observed crack on the partially paved shoulder.

6.5.2 Foundation Soil Settlement

The magnitude and time rate of settlement that may have occurred within the foundation stratum was assessed at Sta. 15+475 (the greatest height where cracking was observed). The analysis was carried out using the commercially available program Settle3D Version 2.016 developed by Rocscience Inc.

The deformation parameters used for the analyses were established using data obtained from consolidation tests as well as empirical correlations of undrained shear strengths, laboratory index tests and soil moisture contents.



Pre-consolidation pressures provided on the e-log p curves (Figure B10) from MTO's Job No. 68-F-73 as well as estimates of pre-consolidation pressure derived from the empirical correlation suggested in the literature by Chandler (1988) were used to select a design pre-consolidation pressure. A profile of the design preconsolidation pressure versus elevation is illustrated in Figure C2, in Appendix C. The vertical effective overburden stress and the stress increase due to the embankment load is also plotted on this figure.

Values of the compression index (C_c) were obtained from Figure B10 and the recompression index (C_r) was also estimated from these e-log P curves. Estimates of C_c and C_r were also obtained using empirical correlations proposed in literature by Terzaghi and Peck (1967), Osterberg (1972), Nagaraj and Murty (1985), Lav & Ansal (2001), Kulhawy and Mayne (1990) and Das (1993). Profiles of the design lines versus elevation are shown on Figures C3 and C4, in Appendix C.

Initial void ratio (e_0) values were obtained from Figure B10 and this parameter was also estimated from empirical correlations proposed in the literature by Cozzolino (1961) and Azzouz et al. (1976). Profiles of the design lines versus elevation are shown on Figure C5, in Appendix C.

The length of time required to complete consolidation settlement of the foundation strata is a function of the value of the coefficient of consolidation of the native silty clay strata and the assumed drainage path length. Given the very stiff to hard consistency, heavily over-consolidated and likely fractured nature of the desiccated upper crust, it is reasonable to assume that consolidation/recompression will occur quickly in the crust and that the rate of consolidation will be primarily controlled by the coefficient of consolidation and thickness of the underlying firm to stiff silty clay stratum. The average coefficient of consolidation was estimated as $1.4 \times 10^{-2} \text{ cm}^2/\text{sec}$ and based on the trend of Atterberg Limits and Water Contents versus elevation as shown on Figure C6, in Appendix C; the heavily over-consolidated desiccated upper crust is estimated to extend to elevation 173 m.

The analysis (Figure C9, Appendix C) indicates that the estimated primary consolidation settlement is about 190 mm and the magnitude of secondary consolidation (creep) settlement of the silty clay foundation stratum that likely became normally consolidated due to the embankment construction is about 30 mm. After 40 years, the primary consolidation settlement is complete and, the settlement (due to creep) that remains over the life of the highway is about 5 mm, which is negligible. Based on this analysis it can be concluded that foundation settlement is not the reason for the south embankment distress.

6.5.3 Embankment Stability

Limit equilibrium slope stability analysis were performed using the commercially available program Slide Version 6.029 developed by Rocscience Inc., using the Spencer and Morgenstern-Price methods of analysis and a minimum Target Factor of Safety of 1.3 was established. Factors of safety that are less than 1.0 indicate that failure is expected and a factor of safety less than about 1.1 suggest that large deformations are likely to occur that may then lead to failure.

The global and surficial embankment stabilities were assessed at selected sections where signs of distress were observed. In this context, global stability refers to slip surfaces that extend deeper than 2 m below the existing ground surface and surficial stability refers to slip surfaces that do not extend deeper than about 1 m to 2 m below the existing ground surface.



6.5.3.1 Soil Parameters

The soil parameters of the silty clay embankment fill were selected based on the laboratory results of the 2009 study. Undisturbed and remoulded specimens of the north embankment fill were subjected to direct shear tests and, consolidated undrained triaxial tests (CIU) with pore water pressure measurements were also performed on undisturbed samples. The data from this analysis is provided in Figures D1 to D5 in Appendix D and the results of the direct shear tests in terms of peak and residual shear strengths are shown on Figure D6 in Appendix D.

The results of the north embankment fill CIU triaxial testing have been combined with the results of triaxial testing carried out by MTO on specimens of the embankment fill during their previous investigation of the site in 1972, and the data are shown on Figure D7 in Appendix D.

For analyses that use the Mohr-Coulomb failure envelope to model a soil's shear strength, best-fitting a straight line through the shear strength data is normally acceptable technique. However, this method does not capture the known non-linear trend that best represents a soil's behaviour. Furthermore, in a slope stability analysis, the range of stresses that are operative on a particular slip surface should coincide with the same stress range and soil properties defined by the Mohr-Coulomb envelope. At very low stresses, it is well accepted in literature (Lo and Morin, 1972), that the shear strength is highly non-linear and the failure envelope should pass through the origin (i.e. effective cohesion intercept, $c'=0$ kPa). This soil behaviour is very important when analysing shallow surficial slope failures.

Given the foregoing discussion on soil shear strength non-linearity, the effective shear strength of the silty clay embankment fill was defined (based on direct shear and triaxial test data) by fitting a non-linear, fully defined shear strength envelope through the data starting at the origin. The non-linear effective shear strength envelope is defined by the soil data in the following table.

Material Type	Effective Normal Stress (kPa)	Shear Stress (kPa)
Embankment Fill (Silty Clay)	0	0
	9	6.4 to 7.3
	25	17
	31	21.3
	40	23.4
	50	25.5
	70	27.6
	100	39.4
	150	62.7
	200	84

Total and effective strength parameters of the native silty clay deposit are also required for the stability analysis and the data from previous investigations by MTO in 1967 and 1972 were reviewed to arrive at reasonable design values. The undrained shear strength values obtained from field vane tests, unconfined compression tests and quick triaxial tests are plotted in Figure D8 in Appendix D along with an interpreted design C_u line.



Consolidated undrained (CIU) triaxial tests with pore pressure measurements were carried out on samples of native silty clay strata from the site and from areas close to the site (in the Welland area) and the results of the interpreted best fit, Mohr-Coulomb strength envelopes to the test data are summarized below.

Location	Location of Sample in Foundation Soil Strata	Effective Cohesion Intercept, c' (kPa)	Effective Angle of Internal Friction, ϕ' (degrees)
Hwy 140/CNR Overpass North Embankment MTO 1972 (Geocres No. 30L-45)	Unknown	14	25
Forkes Road Crossing of Proposed Welland Canal MTO 1967 (Geocres No. 30L 14-05), Line A and Line B	Ground Surface to about Elevation 171 m to 169 m (Crust)	7	24
	Below about Elevation 171 m to 169 m (below Crust)	0	25

Based on the above data, the following average effective strength data were selected for the stability analysis.

- Upper silty clay to clay – Effective cohesion = 10 kPa and Effective Angle of Internal Friction = 25°; and
- Lower silty clay to clay – Effective cohesion = 0 kPa and Effective Angle of Internal Friction = 25°.

It is also necessary to define the ground water table location for slope stability analysis. Based on the existing data, the design ground water table was set at elevation 176 m. The ground water table assessment also took into consideration the downward gradient that likely exists in the native silty clay to clay stratum and the assumption that it is unlikely that excess pore water pressure exists in the foundation soils and the embankment fills.

Tabulated below are the soil parameters used for the slope stability analyses.

Material Type	Total Stress Analysis			Effective Stress Analysis		
	ϕ (degrees)	c (kPa)	γ (kN/m ³)	ϕ' (degrees)	c' (kPa)	γ' (kN/m ³)
Embankment Fill (Silty Clay)	0	25	19.5	Shear Normal Function		19.5
Granular 'A'	35	0	21	35	0	21
Upper Silty Clay to Clay	0	100	19.5	25	10	19.5
Lower Silty Clay to Clay	0	75	19.5	25	0	19.5

6.5.3.2 Global Stability (Total and Effective Stress Analysis)

Since the embankments were constructed in 1971 (43 years ago) it is reasonable to assume that any excess pore water pressures in the embankment would have dissipated and a total stress analysis will not apply for Year 2014 conditions. However, the partially paved shoulder of Hwy. 140 North Bound Lane is cracked at Sta. 15+475, an area that was repaired under Contract 98-116 about 16 years ago. Therefore, a total stress analysis was carried out at this section to emulate conditions shortly after repairs were complete to assess if stability issues were responsible for this observed distress.



Global embankment stability effective stress analyses were also performed at selected sections where cracks were observed at the embankment crest. Tabulated below are the minimum factors of safety obtained for potential failure surfaces and the slope stability models and results are illustrated on Figures D9 and D10, in Appendix D.

Location	Minimum Factor of Safety	
	Total Stress Analysis	Effective Stress Analysis
Sta. 15+375 (East Side Slope)	Not applicable	2.2
Sta. 15+400 (West Side Slope)	Not applicable	2.0
Sta. 15+450 (West Side Slope)	Not applicable	1.8
Sta. 15+475 (East Side Slope)	1.5	1.8

The factor of safety at Sta. 15+475 for a total stress analysis exceeds the target factor of safety of 1.3, suggesting that the embankment fill's undrained shear strength is not controlling global stability and; is not considered to be the reason for the partially paved shoulder crack that exists at this location. Additionally, the effective stress analysis factors of safety for all of the sections are also greater than 1.3 implying that global stability is not a contributing factor to the observed distress.

6.5.3.3 Surficial Embankment Stability (Total and Effective Stress Analysis)

Since the embankments were constructed in 1971 (43 years ago) it is reasonable to assume that any excess pore water pressures in the embankment would have dissipated. Therefore, a total stress analysis will not apply.

As explained in the published technical literature, the surficial shallow layers of plastic clay embankments can experience dramatic strength loss over time while the global stability of the overall embankment remains unchanged. This strength loss causes shallow sloughing-type failures requiring regular maintenance. Research carried out by Zhang, Tao and Morvant (2005) indicates that the loss of strength happens in three phases:

- Shrinkage cracks form at the surface of the slope due to shrinking and swelling resulting from seasonal changes in moisture and temperature;
- Water then infiltrates the slope through the shrinkage cracks during subsequent wet seasons causing the near surface soil to become saturated; and
- The shear strength of the near surface clay then reduces due to the temporary elevated moisture content.

Laboratory studies by Kayyal and Wright (1991) and Rogers and Wright (1986) indicate that wetting of compacted soils can cause a dramatic loss in the effective cohesion/shear strength (c'), while the effective friction angle (ϕ') of the soil is virtually unchanged. The timeline can be influenced by the plasticity index of the soil, local weather conditions and the degree of compaction of the soil near the edges of the slope during construction (Zhang, Tao and Morvant, 2005, and Greenwood, Holt and Herrick, 1985).

The freeze-thaw cycle is also an influencing factor. In the spring, the surficial soils thaw from the ground surface down, become saturated and cannot drain because the underlying soils are still frozen (Andersland and Ladanyi, 2004). This condition causes a reduction in the soils shear strength



resulting in instabilities. The cyclic water infiltration and subsequent loss of shear strength can occur anywhere from months to years after construction before instabilities occur.

Effective stress parameters were used to assess the surficial embankment stability at selected sections where cracks were observed at the embankment crest; taking into consideration the expected variation in shear strength and pore pressure conditions in response to yearly wet and dry periods and the freeze-thaw cycle. To account for these seasonal variations a phreatic surface was introduced into the slope stability model to emulate the yearly wet periods and this phreatic surface was removed from the model to emulate the yearly dry periods. This analysis was carried out to assess the sensitivity of the Factor of Safety to weather events and the slope stability models and results are shown on Figures D11 to D14, Appendix D.

Tabulated below are the minimum factors of safety obtained for potential failure surfaces.

Location	Minimum Factor of Safety	
	Effective Stress Analysis (wet conditions)	Effective Stress Analysis (dry conditions)
Sta. 15+375 (East Side Slope)	0.9 to 1.0	2.0 to 2.1
Sta. 15+400 (West Side Slope)	0.9 to 1.0	1.9 to 2.0
Sta. 15+450 (West Side Slope)	0.9	1.9
Sta. 15+475 (East Side Slope)	1.3	2.0 to 2.6

It can be seen that the Factor of Safety is very sensitive to moisture in the surficial soils ranging from a value of 2.5 (for dry conditions) to values less than unity (for wet conditions). The results imply that slope failures will most likely occur under wet conditions and, failures are unlikely to occur during the dry season when the shear strength of the soils increase resulting in a corresponding increase in the factor of safety.

At Sta. 15+475 (area repaired under Contract 98-116 about 16 years ago), the factors of safety are equal to or greater than the target factor of safety of 1.3. Therefore, surficial instability is unlikely to be a contributing factor to the cracked partially paved shoulder of Hwy. 140 North Bound Lane.

6.6 Factors Contributing to Embankment Distress

Based on the settlement and stability assessment/analysis, site history, and the embankment performance after repairs to the north and south embankments under Contracts 2010-2009 and 98-116 respectively; it is our opinion that surficial instability is the most likely cause of the observed distresses. The contributing factors are:

- Mineralogy of the local soils and its inherent effect on the effective shear strength of the clay fill; and
- Effects of local climate including precipitation and wetting-and-drying cycles as well as snow melt during freezing-and-thawing cycles.

At Sta. 15+475 it is our opinion that the observed cracks in the partially paved shoulder of Hwy. 140NBL are construction related.



6.7 Remediation Options

Remediation alternatives should focus on methods that increase the surficial stability of the embankment slopes, control surface water run-off over the slope crest and down the slope, improve drainage from within the side slopes and promote deep-rooted vegetation on the slope faces.

Overviews of eight remediation schemes are provided in the following sections. The advantages, disadvantages, relative costs and risks/consequences associated with each alternative are provided in Table 1. From a foundations perspective, deep benching and slope flattening with granular material is the preferred remediation option because of its historical performance. This method was used to repair selected areas of the south embankment under Contract 98-116 and the north embankment under Contract 2010-2009. These repaired areas are performing well and “to date” there is reportedly no evidence of continued distress after the repairs were carried out.

6.7.1 Deep Benching with Granular Material (Option 1)

Adding a granular buttress to the existing embankment side slopes, from the crest of the embankment to the toe of mid-height berms or to the embankment toe, at a profile of 2.5H:1V (or flatter), is a viable alternative for mitigating the on-going surficial instability. It is recommended that the new granular fill material be keyed into the existing clay embankment by a series of deep benches to remove as much of the previously distressed/weakened material in the frost penetration zone. Benching of the new granular fill into the existing earth slopes should be carried out in accordance with the geometry illustrated on OPSD 208.010 but the benches should be constructed at least 1.0 m high and 2.0 m wide. This construction methodology is not expected to encroach into the existing travelled lane(s). Further discussion on the requirements for temporary protection systems is provided in Section 6.9.

A subdrain should be provided within the granular fill near the interface with the existing clay fill and, a mountable curb and gutter arrangement at the embankment crest is also recommended to control and divert surface water away from the top of the slope. Surface water can be directed to armoured outfalls designed to drain into roadside ditches.

Granular A or Granular B Type I material (both with not more than 5 percent passing the number 200 sieve) is recommended, and this material should be placed and compacted in accordance with the requirements of OPSS 501. This remediation scheme is illustrated on Figure E1 in Appendix E.

Global and surficial effective stress stability analyses of this proposed remediation scheme have been carried as shown on Figure E3, Appendix E. The results indicate Factors of Safety greater than 1.3.

6.7.2 Standard Benching with Granular Material (Option 2)

Benching into the existing embankment fill with new granular fill conforming to the geometry of OPSD 208.010 can also be considered. Although the standard OPSD 208.010 benching dimensions will reduce the volume of excavated material, there is a risk that a zone of weakened material will remain that could affect future embankment performance.



The subdrain and mountable curb and gutter recommendations provided in Section 6.7.1 will also apply to this repair scheme as well as the recommendations for placing and compacting the granular material. This remediation scheme is illustrated on Figure E1 in Appendix E.

Global and surficial effective stress stability analyses of this proposed alternative have been carried as shown on Figure E3. The results indicate Factors of Safety greater than 1.3.

6.7.3 Slope Flattening with Silty Clay (Option 3)

Embankment side slope flattening with locally sourced cohesive silty clay soil is a feasible option that merits consideration. However, due to the lower strength of the silty clay compared to granular material, it would be necessary to construct the side slopes at a geometry not steeper than 3.5H:1V to reduce the risk of further instabilities. It should be noted that the new silty clay fill will still be subjected to wetting/drying and freeze-thaw cycles. Therefore, the risk of localized surficial sloughing of the final slope surface (though reduced), will still exist.

Before placing the new fill, all vegetation and organic materials should be removed from the side slopes. The new fill should be keyed into the existing side slopes in accordance with OPSD 208.010 and compacted in accordance with the requirements of OPSS 501. The mountable curb and gutter recommendations provided in Section 6.7.1 will also apply to this repair scheme.

Since the field moisture content is an acceptance criteria for the use of earth borrow with more than 50% of the particles smaller than 75 μm , it may be impractical to meet the field moisture content acceptance criteria in wet weather conditions. Therefore, construction should be scheduled for the summer months when it would be easier to control the moisture content of the clay material. This remediation scheme is illustrated on Figure E1 in Appendix E.

Global and surficial effective stress stability analyses of this repair alternative have been carried as shown on Figure E4. The results indicate Factors of Safety of 1.3.

6.7.4 Plate Pile Stabilization (Option 4)

The plate pile slope stabilization method consists of installing vertical steel reinforcing elements in the slope. The sliding soil mass is resisted by a 6 mm thick and 300 mm wide steel plate welded to the upper portion of a 60 mm x 60 mm by 6 mm thick steel angle. The steel is galvanized to provide long term protection against corrosion. The plate piles are spaced approximately 1.2 m on centre parallel to the slope and staggered in rows about 2.5 m apart perpendicular to the slope, thereby forming a barrier that limits downslope movement. Some amount of soil arching occurs between the plates but the arching effect can be controlled by the pile spacing. The plate piles are driven into the slope using a small vibratory hammer to a depth of 300 mm below the slope surface so that they are not visible when the installation is completed. This remediation scheme is illustrated on Figure E1 in Appendix E.

The plate pile is an innovative low cost method for stabilizing slopes and the method stabilizes the slope in-place with minimal earthwork required. The work can be accomplished with a small excavator adapted with a driving hammer and one labourer. The installation cost ranges between \$5 and \$10 per square foot of slope surface repaired. Although a very new technology in Ontario, the



plate pile method has been utilized on several projects in California, USA in areas where shallow slides have occurred on the shoulders of highways, rural roads and behind residential properties.

6.7.5 Geogrid Reinforced Side Slope (Option 5)

The existing embankment can be partially excavated in a horizontal plane and replaced with a geogrid reinforced side slope. To minimize roadway instability, the excavation would have to be carried out in a series of strips of limited width and temporary shoring may be required. The travelled roadway section will also have to be reduced to a single lane during construction.

The geogrid reinforcement would have to be installed over a horizontal length of at least 6.5 m and should be placed at a vertical spacing of at least 500 mm. Each geogrid layer should be wrapped over the slope face and tied into the upper level to protect the slope face from erosion as well as to enhance surficial stability. These recommendations do not constitute a detail design and internal stability assessments of the reinforced slope will have to be carried out by the geogrid supplier. This remediation scheme is illustrated on Figure E2 in Appendix E.

Effective stress stability analysis of the proposed remediation scheme has been carried as shown on Figure E4. The results indicate a Factor of Safety greater than 1.3.

6.7.6 Counterfort Drains (Option 6)

Counterfort drains can be installed along the slope face of the embankment, designed to drain freely into a trench drain or drainage channel constructed at the embankment toe. The counterfort drains should be at least 1.5 m deep and spaced at 10 m to 20 m intervals. It is envisaged that this drainage system will reduce the accumulation of moisture and infiltration of water into the side slopes thereby reducing the risk of failures caused by the wetting and drying cycles. However, annual inspections and probably some maintenance will be required to monitor the installations to ensure good performance. This remediation scheme is illustrated on Figure E2 in Appendix E.

6.7.7 Slope Cover with Mass Concrete & Rock Protection (Option 7)

Covering the embankment side slopes with rock protection and mass concrete can be considered. This remediation scheme is sometimes used by MTO to treat localized surficial sloughing on slopes. It relies on the free- draining properties of the rock fill and the concrete's impermeability, to reduce the accumulation of moisture and infiltration of precipitation, thereby enhancing surficial stability.

Stripping the vegetation and topsoil from the embankment side slopes is required prior to placing the rock protection. The rock protection should be placed on the side slopes in accordance with OPSS 511 and then covered with a mass concrete blanket as shown on Figure E2 in Appendix E.

Settlement, frost heave and large temperature changes can cause concrete cracking thereby creating an opportunity for water infiltration that will create similar near surface conditions as those previously experienced, which could cause future slope instabilities. Repairing the concrete is an expensive maintenance operation and attempting to seal micro cracks is a daunting and probably impractical task.



6.7.8 Cement-Soil Mixing (Option 8)

The shear strength of clayey soils can be enhanced by cement or lime stabilization techniques. About 5% to 15% of cement (by mass) is mixed into the soil to improve its geotechnical properties. The report by Prusinski and Bhattacharja (1999) describes cement (or lime) addition to clayey soils can decrease the soils plasticity index (PI) by as much as 45% to 65% for soils with plasticity indices similar to those at this site. A reduction in plasticity index (PI) of this magnitude should be accompanied by a corresponding increase in effective friction angle (ϕ').

The construction/in-situ mixing details would require additional design and perhaps even field trials to assess the effectiveness. Mixing could be carried out at either discrete locations laid out on a grid across the face of the slope or in strips (of limited width) across the slope. This remediation scheme is illustrated on Figure E2 in Appendix E.

The suitability of using cement stabilization was assessed by laboratory tests in the 2009 study. The results indicate that a PI reduction of about 25% requires the addition of 15% of cement by mass which is a relatively low degree of improvement for such a high percentage addition of cement. The relative cost of this solution is expected to be much higher than some of the other alternatives being considered considering that an even higher percentage of cement will be required on site to achieve the desired result because of less-than-perfect site conditions such as non-uniform mixing, cement loss due to wind etc. Furthermore, the long-term performance of the treated slopes would be highly dependent on the level of Quality Assurance/Quality Control during construction.

6.7.9 Do Nothing (Option 9)

The do-nothing option could be considered provided that there are bi-annual to annual inspections and the MTO's annual maintenance budget includes money to repair unstable areas. Although it is very difficult to predict the magnitude and extent of the next failure, it is anticipated that surficial sloughing and sliding in the observed distressed areas may occur during the next freeze-thaw cycle in Year 2015. These failure depths are expected to be shallow and may extend up to about 2 m below the existing ground surface. It is imperative that these failed areas be repaired immediately to prevent further deterioration and more significant deep global failures.

6.8 Preferred Remediation Option

It is recommended that the south embankment be repaired by deep benching with granular material as described in Section 6.7.1. Benches should be cut into the existing embankment slopes starting from the toe of the upper embankment slope (i.e. top of the mid-height berm (where present)) or from the original ground surface (where the berm is not present), continuing up to within about 0.3 m of the outside edge of the partially paved asphalt shoulder.

A geotextile-wrapped perforated pipe sub-drain (150 mm diameter) should be provided at the heel of the granular portion of the slope flattening to provide longitudinal drainage. This sub-drain should be surrounded with a 0.3 m wide by 0.3 m high section of concrete sand or 19 m clear stone that is fully wrapped with a Class II non-woven geotextile with a filtration opening size of 90 microns or less. The subdrain arrangement should be designed to drain into laterals installed at a minimum spacing of 25 m.



Details of the slope remediation requirements, as described above, for a series of cross-sections along the slope are shown on Figure F1 in Appendix F. Relevant drawings and specifications pertaining to the slope remediation in Contract No. 2010-2009 (North Embankment Repair Contract) are also provided in this appendix. The extent of the slope remediation is also summarized below.

South Embankment Side Slope Location	Approximate Length of Remediation Section (m)	Approximate Station Limits
East Side Slope	70	Sta. 15+340 to Sta. 15+410
West Side Slope	20	Sta. 15+340 to Sta. 15+360
West Side Slope	140	Sta. 15+360 to Sta. 15+500

Regarding the crack in the partially paved shoulder between Sta. 15+463 and Sta. 15+475, we recommend saw cutting and removing the existing asphalt in this area. Approximately 700 mm of the underlying granular material should then be removed and the base of the repair area should be recompacted followed by reinstatement of the shoulder with Granular 'A' material. The partially paved shoulder should be repaved with the Superpave mix types recommended in the Pavement Design Report.

6.8.1 Surface Treatment

To prevent erosion of the granular material it is recommended that a 200 mm thick non-erodible soil cap be installed followed by a 50 mm thick topsoil layer to promote vegetative growth. The existing clayey embankment fill material removed during the deep benching is not recommended for use as the soil cap.

6.8.2 Construction Timing

As pointed out under Section 6.5.3.3, cyclic water infiltration and subsequent loss of shear strength can occur anywhere from months to years after construction before instabilities occur. Although the timing of the next surficial failure cannot be precisely determined, the potential risk of surficial failure associated with a deferred construction schedule is tabulated below:

Construction Year	2015	2020	2025	2035
Remediation Waiting Period (Years)	0	5	10	15
Probability of Surficial Failure	5%	50%	90%	100%

A budget should be provided for repairing any periodic embankment failures that may occur during the waiting period.

6.9 Temporary Protection

Remediation of the embankment by deep benching and slope flattening requires removing and re-installing the existing guard rail. However, excavations are not expected to encroach into the north bound and south bound lane. Therefore, temporary protection systems will not be required to support the travelled portion of the highway.



Notwithstanding the foregoing, a temporary concrete barrier conforming to OPSD 911.140 is required along the edge of the highway to protect the travelling public from the adjacent work zone and vice- versa.

7.0 CLOSURE

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

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REFERENCES

- Aubeny, C.P., and Lytton, R.L. 2004. Shallow Slides in Compacted High Plasticity Clay Slopes, ASCE, Vol. 130, No. 7, pp. 717-727.
- Bennett, B. Memorandum to Billings, M.D. 1997. *Slope Instability, Highway 140 and CN Overpass, Central Region*. MTO W.O. 97-11006, W.P. 418-97-00 Geocres Report No. 30L14-040.
- Canadian Geotechnical Society. 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06*. 2006. CSA Special Publication, S6.1 06. Canadian Standard Association.
- Chapman, L.J., and Putnam, D.F. 1984. *The Physiography of Southern*, 3rd Edition. Ontario Geological Survey, Special Volume 2. Ontario Ministry of Natural Resources.
- Darch, B.T. Devata, M. 1972. *Foundation Investigation Report for Failure of Approach Embankments Overhead Structure at the Crossing of Hwy # 140 and C.N.R. Township of Humberstone, County of Welland*. MTO W.P. 60-60-02, W.O. 72-11025 Geocres Report No. 30L14-008.
- Devata, M. Memorandum to Robertson, C.R. 1971. *Embankment Instability, South Approach Sta. 211+50 to 216+50, Hwy. #140 from 0.95 miles North of Hwy. #3 at Port Colbourne Northerly 3.32 mi. to Town Line Road*. MTO W.O. 68-F-73 Geocres Report No. 30L14-008.
- Greenwood, J. R., Holt, D. A. and Herrick, G. W. 1985. *Shallow Slips in Highway Embankments Constructed of Overconsolidated Clay. Failures in Earthworks: Proceedings of the Symposium on Failures in Earthworks*, pp. 79-92. London: Institution of Civil Engineers.
- Hutton, W., Devata, M. 1968. *Foundation Investigation Report for the Crossing of the C.N.R. Tracks and Proposed East Side Highway (Near Forkes Road) Twp. of Humberstone, Co. Of Welland, District No. 4 (Hamilton)*. MTO W.P. 60-68-02, W.J. 68-F-73, Geocres Report No. 30L14-036.
- Kayyal, M. K., and Wright, S. G. 1991. *Investigation of Long-term Strength Properties of Paris and Beaumont Clays in Earth Embankments*. Centre for Transportation Research, University of Texas at Austin, Austin, Texas.
- Kulhawy, F.H. and Mayne, P.W. 1990. *Manual on Estimating Soil Properties for Foundation Design*. EL 6800, Research Project 1493 6. Prepared for Electric Power Research Institute, Palo Alto, California.
- Lo, K.Y. and Morin, J.P. 1972. *Strength Anisotropy and Time Effects of Two Sensitive Clays*. Canadian Geotechnical Journal, Vol. 9, No. 3, pp. 261-277.
- McCormick W. and Short, R., 2006, *Cost Effective Stabilization of Clay Slopes and Failures Using Plate Piles*, International Association for Engineering Geology and the Environment 2006, Paper Number 629.
- Prusinski, J.R. and Bhattacharja, S. 1999. *Effectiveness of Portland Cement and Lime in Stabilizing Clay Soils*. Transportation Research Board 1652, Washington, D.C.



Rogers, L. E., and Wright, S. G. 1986. *The Effects of Wetting and Drying on Long-term Shear Strength Parameters for Compacted Beaumont Clays*. Centre for Transportation Research, University of Texas at Austin, Austin, Texas.

Zhang, Z., Tao, M., and Morvant, M. 2005. *Cohesive Slope Surface Failure and Evaluation*, ASCE, Vol. 131, No. 7, pp. 898-906.

Ontario Provincial Standard Specifications (OPSS)

OPSS 501	Construction Specification for Compacting.
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting.
OPSS 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material.

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching of Earth Slopes.
OPSD 911.140	Guide Rail System, Concrete Barrier, Precast I-Lock Connection, Installation Temporary and Permanent



TABLE 1
EVALUATION OF REMEDIATION ALTERNATIVES

Remediation Alternative	Option/Rank	Advantages	Disadvantages	Risk/Consequences	Relative Cost
Deep Benching with Granular ‘A’ or Granular ‘B’ Type I 2.5H:1V Side Slopes or Flatter. (1 m high & 2 m wide benches)	1	<ul style="list-style-type: none">▪ Easy to construct.▪ Weakened soil on existing slope surface is removed.▪ Granular replacement material is keyed in very well to existing embankment because of deep benches.▪ Granular ‘A’ material readily available in area.▪ Demonstrated reliable performance at the repaired North and South embankments.	<ul style="list-style-type: none">▪ Disposal of surplus embankment fill from deep benching required.▪ Slight risk of traffic interruption due to benching in existing slope especially adjacent to highway shoulder.	<ul style="list-style-type: none">▪ Very low risk of future instability issues.▪ Would induce some minor additional settlement due to increased loading on foundation soils.▪ Visual monitoring during construction may be required to manage excavations adjacent to highway shoulder.	<ul style="list-style-type: none">▪ Low to medium construction and materials cost.▪ Higher cost than Standard OPSD 208.01 Benching due to additional material required in deep benches.
Standard OPSD 208.01 Benching with Granular ‘A’ or Granular ‘B’ Type I 2.5H:1V Side Slopes or Flatter.	2	<ul style="list-style-type: none">▪ Easy to construct.▪ Minimal embankment spoil material generated from construction compared to Option 1.▪ Granular ‘A’ material readily available in area.	<ul style="list-style-type: none">▪ A portion of the existing weakened surficial soil on the existing slope will not be removed.▪ Slight risk of traffic interruption due to benching in existing slope especially adjacent to highway shoulder.	<ul style="list-style-type: none">▪ Low risk of future instability issues.▪ Localized sloughing may still occur since all of the weakened soil below slope surface may not be removed.	<ul style="list-style-type: none">▪ Low to medium construction and material costs.▪ Lower material cost than deep benching option.
Slope Flattening with Silty Clay 3.5H:1V Side Slopes or Flatter.	3	<ul style="list-style-type: none">▪ Easy to construct.▪ Minimal disruption and low risk to traffic interruption.▪ Negligible embankment spoil material generated due to construction compared to Options 1 and 2.▪ Slope flattening material may be available locally and may be less costly than granular fill.	<ul style="list-style-type: none">▪ Clayey soils may still experience some localized surficial sloughing over time.▪ Land acquisition may be necessary to accommodate the wider embankment footprint.	<ul style="list-style-type: none">▪ Some risk of localized sloughing exists since the clayey soils will continue to experience wetting and drying action with corresponding loss of shear strength.▪ Would induce minimal additional settlement due to increased loading on foundation soils	<ul style="list-style-type: none">▪ Low construction and material costs.▪ Potential for additional costs associated with land acquisition.
Plate Pile Stabilization No side slope modifications	4	<ul style="list-style-type: none">▪ Easy to install.▪ Negligible embankment spoil material generated from construction compared to Options 1 and 2.▪ No additional loads imparted to foundation soils, hence no additional settlement.▪ Innovative solution.	<ul style="list-style-type: none">▪ May be difficult to acquire the plate piles promptly and the material may have to be ordered in advance specifically for this project.▪ Plate piles must be protected to inhibit corrosion.	<ul style="list-style-type: none">▪ Low risk of future instability issues.▪ No known Ontario projects that can be used to benchmark performance.	<ul style="list-style-type: none">▪ Costs may be less than Options 1, 2 and 3.
Geogrid Reinforced Side Slopes 2.5H:1V or Flatter. (6.5 m horizontal embedment and reinstatement with silty clay backfill)	5	<ul style="list-style-type: none">▪ Very little to no new fill material required for construction.▪ Minimal clayey spoil material generated as a result of construction.▪ Geometry of existing embankment remains relatively unchanged.▪ Does not require importing and using granular material.	<ul style="list-style-type: none">▪ More difficult construction operation compared to Options 1, 2, 3 and 4.▪ Requires deeper and wider excavation, geogrid placement and backfilling in short sections to maintain embankment stability.▪ High risk of traffic interruption due to larger width of excavation into existing slope.▪ Lane closures will be required.	<ul style="list-style-type: none">▪ Low risk of future instability issues.▪ Greater QA/QC requirements during installation of geogrid.▪ Visual monitoring during construction may be required to manage excavations adjacent to highway shoulder.	<ul style="list-style-type: none">▪ Medium to high initial construction and material costs.
Counterfort Drains	6	<ul style="list-style-type: none">▪ Very little new fill material required for construction.▪ Geometry of existing embankment remains relatively unchanged.▪ Easy to construct.▪ Minimal disruption and low risk to traffic.	<ul style="list-style-type: none">▪ Disposal of some surplus embankment fill required.▪ Some annual maintenance may be required to ensure functionality of drains.	<ul style="list-style-type: none">▪ Medium risk of further surficial stability problems since this stabilization technique relies only on drainage for increased stability.▪ Annual inspections and maintenance to repair localized sloughing and/or drains may be required.▪ Potential for on-going maintenance costs.	<ul style="list-style-type: none">▪ Low to medium initial cost.▪ Potential for on-going maintenance costs.
Side Slope Cover with Mass Concrete and Rock Protection	7	<ul style="list-style-type: none">▪ Very little new fill material required for construction.▪ Geometry of existing embankment remains relatively unchanged.	<ul style="list-style-type: none">▪ More difficult to construct compared to Options 1, 2, 3 and 4.▪ Vegetative cover eliminated due to “hard lining”▪ High potential for cracking from expansion and contraction due to changes in moisture levels and temperature that could lead to moisture infiltration and future instabilities.▪ Requires importing graded rock protection.	<ul style="list-style-type: none">▪ Greater QA/QC requirements.▪ Low risk of future stability problems if concrete cracking occurs with corresponding water infiltration into underlying clayey soils.	<ul style="list-style-type: none">▪ High initial cost.

Remediation Alternative	Option/Rank	Advantages	Disadvantages	Risk/Consequences	Relative Cost
Cement Soil Mixing (Discrete Columns in Grid Arrangement)	8	<ul style="list-style-type: none">▪ Very little to no new fill material required for construction.▪ Minimal clayey spoil material generated as a result of construction.▪ Geometry of existing embankment remains relatively unchanged.▪ Does not require importing and using granular material.	<ul style="list-style-type: none">▪ More difficult construction operation compared to Options 1, 2, 3 and 4.▪ Requires specialized construction techniques.▪ Relatively high percentage of cement required to achieve desired result.	<ul style="list-style-type: none">▪ Low to medium risk of further surficial stability problems.▪ No known Ontario projects that can be used to benchmark performance.	<ul style="list-style-type: none">▪ Costs may be higher than the cost for other alternatives.
Do Noting (Continue Annual Maintenance)	9	<ul style="list-style-type: none">▪ No Initial Cost	<ul style="list-style-type: none">▪ Does not eliminate stability problems.▪ Requires providing an annual maintenance budget for emergency repairs as and when required.▪ Unable to predict when slope failures will occur.	<ul style="list-style-type: none">▪ High risk of on-going surficial stability problems.▪ Continued annual maintenance required to repair failures.	<ul style="list-style-type: none">▪ No initial cost.▪ High long term cost due to continued annual maintenance.

DRAWINGS





Crack along slope crest on the west side of Hwy 140 along south embankment looking south.



Close-up view of crack along slope crest on the west side of south embankment.



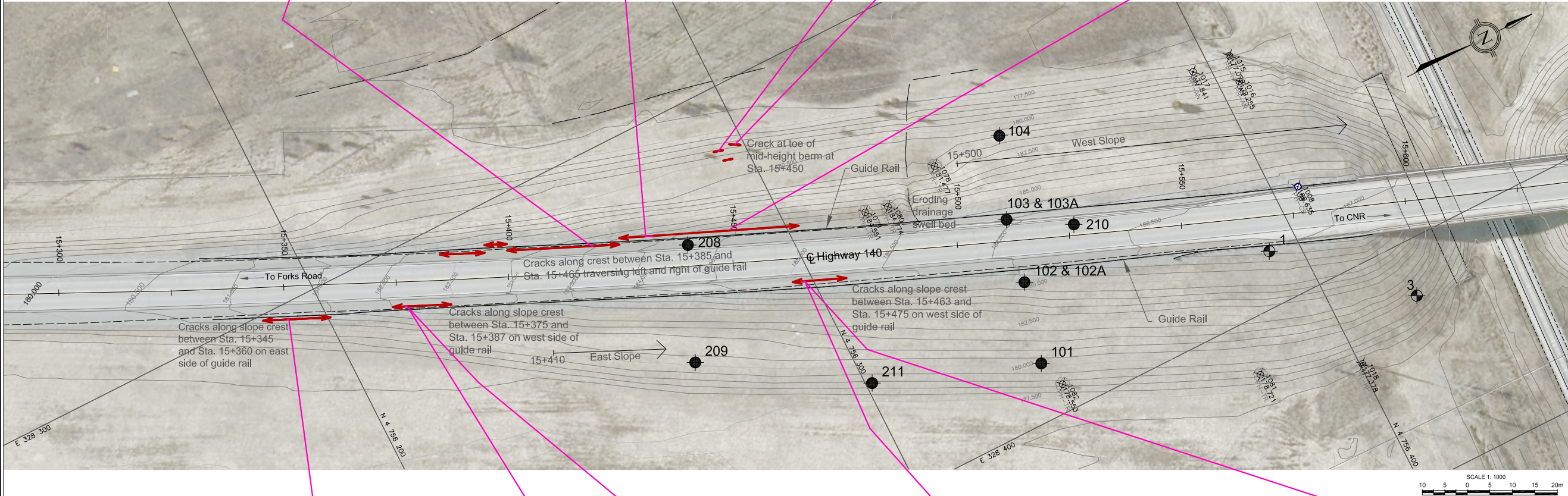
Cracks near the toe of mid-height berm on the west slope of south embankment.



General view of mid-height berm showing crack location.



Crack along slope crest about 1 m west of guide rail on the west side of south embankment.



Crack along slope crest running parallel to and on the east side of the guide rail.



Crack along slope crest running parallel to and on the west side of the guide rail.



Close-up view of crack at the slope crest at approximately Sta. 15+375.



Crack on the partially paved shoulder on the east side of south embankment looking south, located about 1.5 m west of guide rail.



Closer view of crack on the partially paved shoulder on the east side of south embankment at approximately Sta. 15+470.

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

CONT No 2008-E-0013

GWP No 2044-13-00

HWY 140 - CNR OVERPASS
South Embankment
EXISTING CONDITION

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KEY PLAN

LEGEND

- Bore Hole
- Dynamic Cone Penetration Test
- Bore Hole And Cone

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	176.5	4 756 393.4	328 388.3
3	176.9	4 756 418.2	328 411.9
101	177.6	4 756 337.0	328 388.0
102 & 102A	183.9	4 756 341.7	328 369.9
103 & 103A	184.3	4 756 344.4	328 355.7
104	177.6	4 756 351.4	328 388.3
208	183.7	4 756 278.7	328 388.8
209	180.6	4 756 268.4	328 352.9
210	185.1	4 756 357.3	328 363.4
211	178.4	4 756 301.4	328 374.7

NOTE

The locations and elevations of the boreholes were surveyed by the Central Region Engineering Surveys Section between October 1968 and February 1972. These borehole elevations will not match current ground surface elevations because of embankment remediation/repairs that have occurred after the surveys were completed.

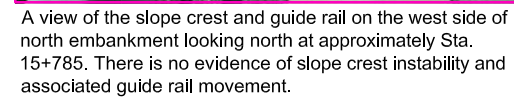
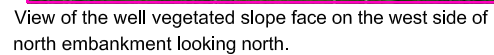
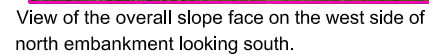
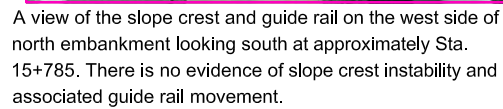
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview.

Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of OPS General Conditions

REFERENCE

Drawings provided in digital format by D.M.Wills Associates, drawing files xb-190-140-112707, xb-190-140-112708, xb-190-140-112710, x190-140, received June 19, 2014 by ftp site

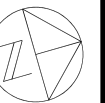
REVISIONS			
	DATE	BY	DESCRIPTION
HWY. 140	PROJECT No.	11-14-4076	DIST.
SUBM'D. HA	CHKD. RA	DATE: July 30, 2014	SITE:
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 1



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

CONT No 2008-E-0013

GWP No 2044-13-00



HWY 140 - CNR OVERPASS
North Embankment
EXISTING CONDITION

SHEET



D.M. Wills Associates Ltd.
150 Jameson Drive · Peterborough, ON · K9J 0B9
Tel: (705) 742-2297 ext. 241 · Fax: (705) 741-3568



Terraprobe Inc.

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

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



KEY PLAN

LEGEND

[illegible]

NOTE

The locations and elevations of the boreholes were surveyed by the Central Region Engineering Surveys Section between October 1968 and February 1972. These borehole elevations will not match current ground surface elevations because of embankment remediation/repairs that have occurred after the surveys were completed.

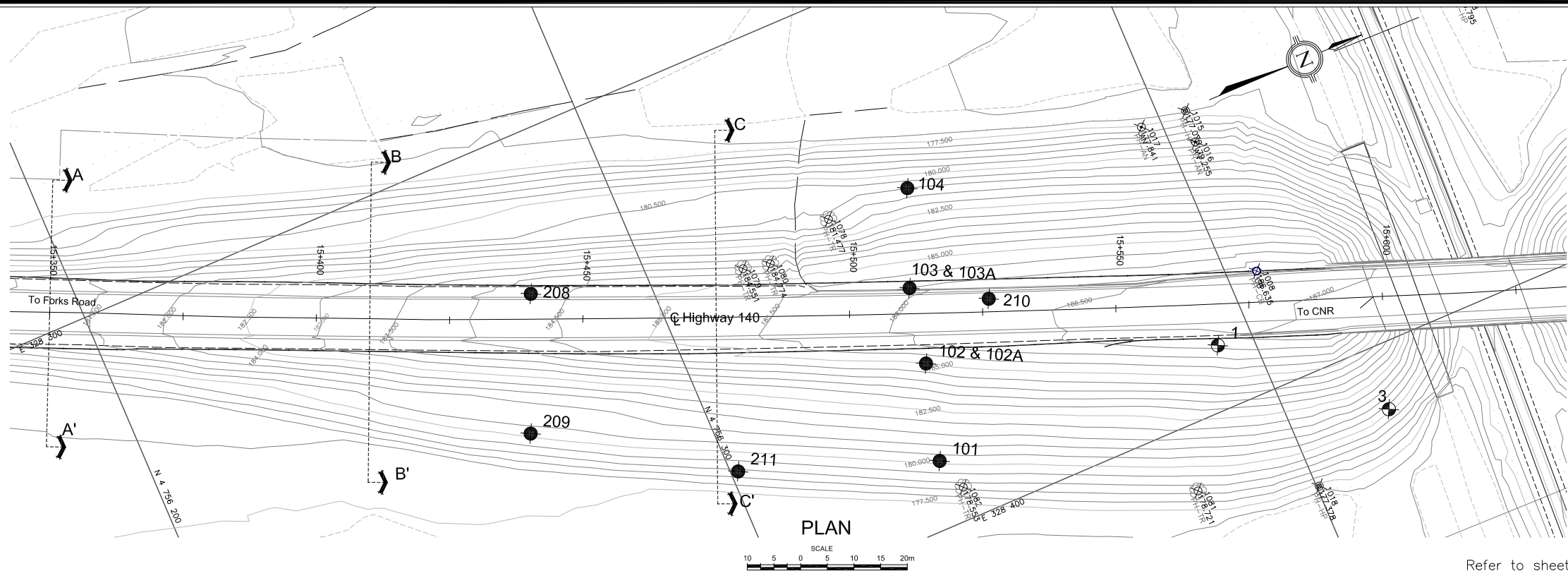
The complete foundation investigation and design report for this project and other related documents may be examined at the Materials Engineering and Research Office, Downsview. Information contained in this report and related documents are specifically excluded in accordance with Section GC 2.01 of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

REFERENCE

Drawings provided in digital format by D.M.Wills Associates,
drawing files xb-190-140-112707, xb-190-140-112708,
xb-190-140-112710, x190-140, received June 19, 2014
by ftp site

REVISIONS			
	DATE	BY	DESCRIPTION

HWY.	140	PROJECT No.	11-14-4076	DIST.
SUBM'D.	HA	CHKD. RA	DATE: July 30, 2014	SITE:
DRAWN:	KC	CHKD. RA	APPD: MT	DWG. 2



METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS UNLESS
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GWP No 2044-13-00

HWY 140 - CNR OVERPASS
South Embankment and Approach
BOREHOLE LOCATION AND SOIL STRATA



SHEET

D.M. Wills Associates Ltd.
150 Jameson Drive - Peterborough, ON - K9J 0B9
Tel: (705) 742-2297 ext. 241 - Fax: (705) 741-3568

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



KEY PLAN

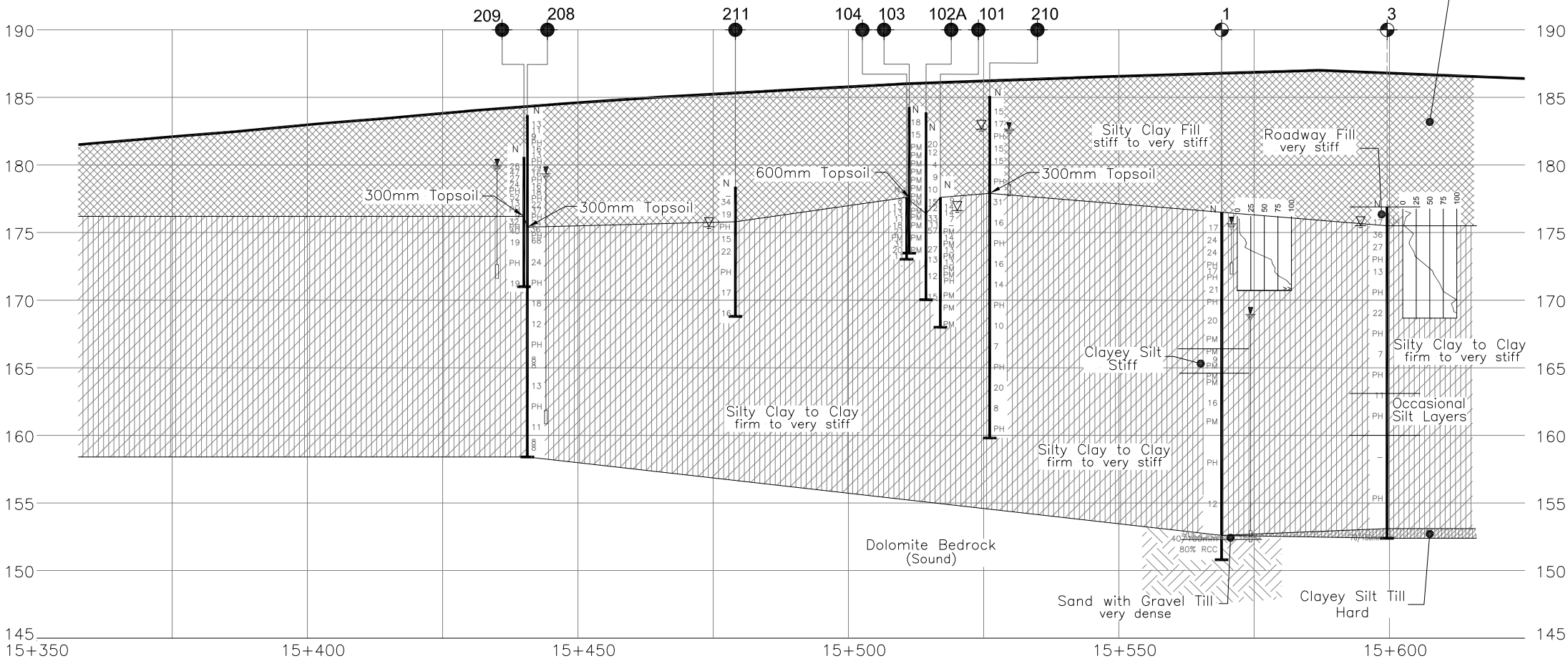
LEGEND	
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
	Blows/0.3m (Std Pen Test, 475 J/blow)
	Blows/0.3m (60' Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer
	Piezometer
	Rock Quality Designation
	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	176.5	4 756 393.4	328 388.3
3	176.9	4 756 418.2	328 411.9
101	177.6	4 756 337.0	328 388.0
102 & 102A	183.9	4 756 341.7	328 369.9
103 & 103A	184.3	4 756 344.4	328 355.7
104	177.6	4 756 351.4	328 388.3
208	183.7	4 756 278.7	328 388.8
209	180.6	4 756 268.4	328 352.9
210	185.1	4 756 357.3	328 363.4
211	178.4	4 756 301.4	328 374.7

NOTE
The locations and elevations of the boreholes were surveyed by the Central Region Engineering Surveys Section between October 1968 and February 1972. These borehole elevations will not match current ground surface elevations because of embankment remediation/repairs that have occurred after the surveys were completed.
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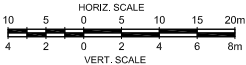
REFERENCE
Drawings provided in digital format by D.M.Wills Associates, drawing files xb-190-140-112707, xb-190-140-112708, xb-190-140-112710, x190-140, received June 19, 2014 by ftp site

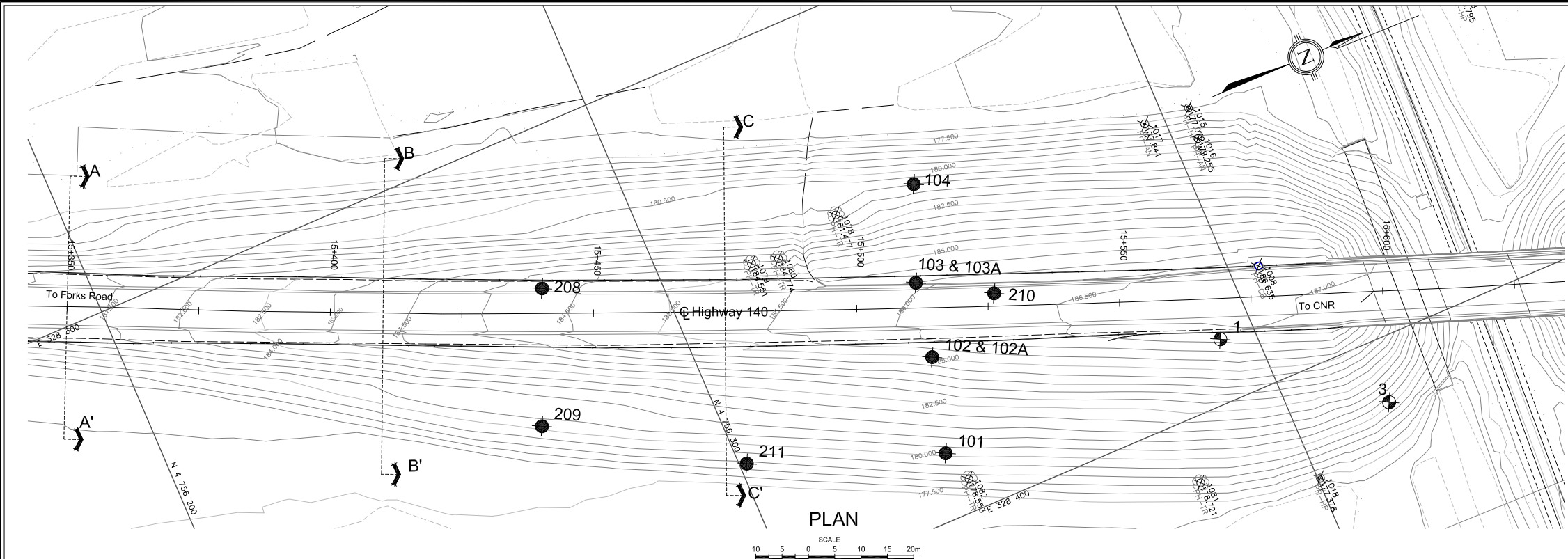
REVISIONS			
	DATE	BY	DESCRIPTION
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Refer to sheets 2 & 10 of Contract 98-116.
Embankment Repaired by benching with Granular A.
From Sta. 15+410 to Sta. 15+620 (East Slope)
From Sta. 15+500 to Sta. 15+615 (West Slope)

PROFILE HWY 140





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

GWP No 2044-13-00

HWY 140 - CNR OVERPASS
South Embankment and Approach
BOREHOLE LOCATION AND SOIL STRATA



SHEET

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



KEY PLAN

LEGEND

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

No	ELEV.	COORDINATES	
		NORTHING	EASTING
1	176.5	4 756 393.4	328 388.3
3	176.9	4 756 418.2	328 411.9
101	177.6	4 756 337.0	328 388.0
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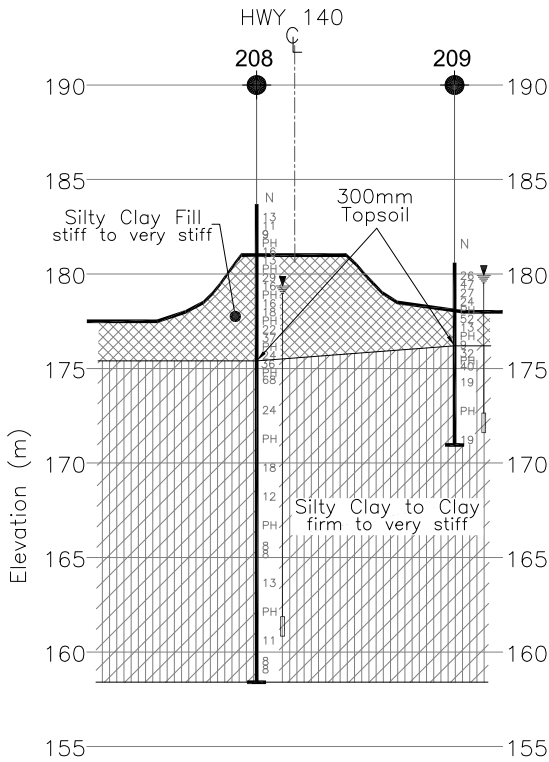
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REFERENCE

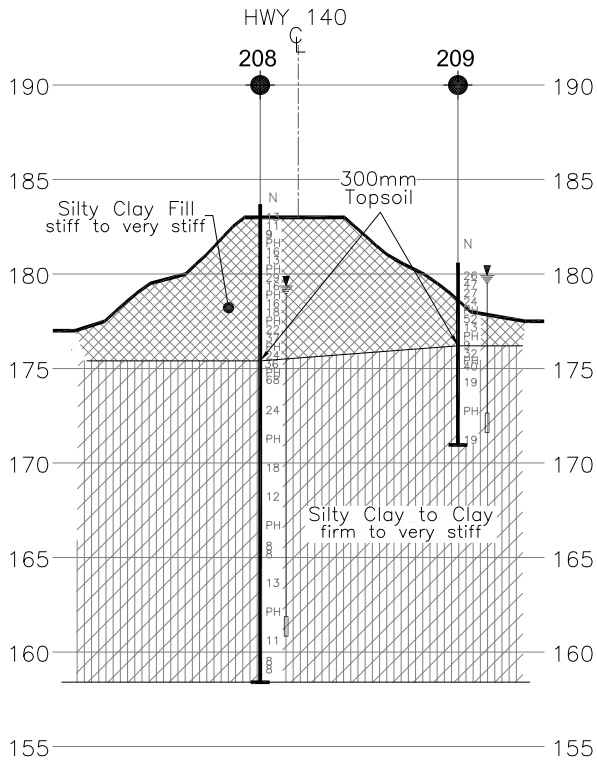
Drawings provided in digital format by D.M.Wills Associates, drawing files xb-190-140-112707, xb-190-140-112708, xb-190-140-112710, x190-140, received June 19, 2014 by ftp site

REV	DATE	BY	DESCRIPTION

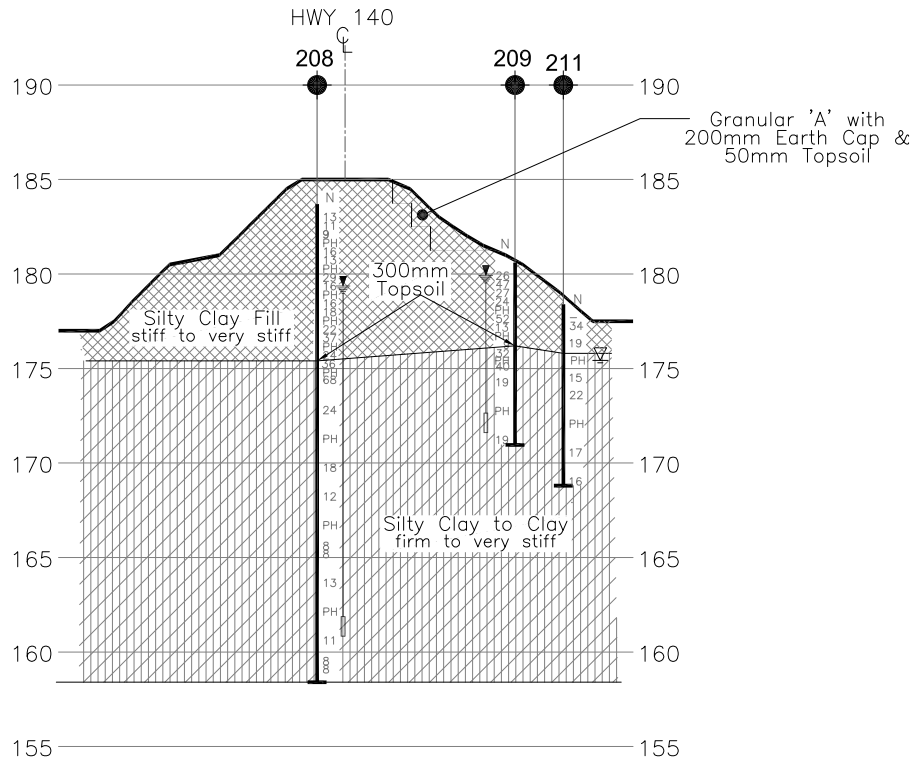
HWY. 140	PROJECT No. 11-14-4076	DIST.
SUBM'D. HA	CHKD. RA	DATE: July 30, 2014
DRAWN: KC	CHKD. RA	APPD: MT
		DWG. 4



SECTION A-A'
STA. 15+350



SECTION B-B'
STA. 15+410



SECTION C-C'
STA. 15+475

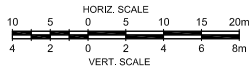


PLATE No
CONT No 98-116
WP No 418-97-01

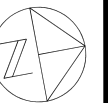
REMOVAL & NEW CONSTRUCTION
STA 15+330 TO STA 15+650

Survey _____ Revised _____



SHEET
2

HWY 140 - CNR OVERPASS
South Embankment
REPAIR DETAILS
CONTRACT 98-116



SHEET



D.M. Wills Associates Ltd.
150 Jameson Drive · Peterborough, ON · K9J 0B9
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Construction Materials Engineering, Inspection & Testing

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



KEY PLAN

REFERENCE

Ministry of Transportation, Contract No. 98-116,
WP No. 418-97-01

REVISIONS			
	DATE	BY	DESCRIPTION
HWY. 140		PROJECT No. 11-14-4076	DIST.
SUB'M'D: HA		CHK'D: RA	DATE: Aug. 2014
DRAWN: KC		CHK'D: RA	APP'D: MT
			DWG. 5

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

CONT No 98-116
WP No 418-97-01



TYPICALS

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10

GWP No 2044-13-00

HWY 140 - CNR OVERPASS
South Embankment
REPAIR DETAILS
CONTRACT 98-116

SHEET



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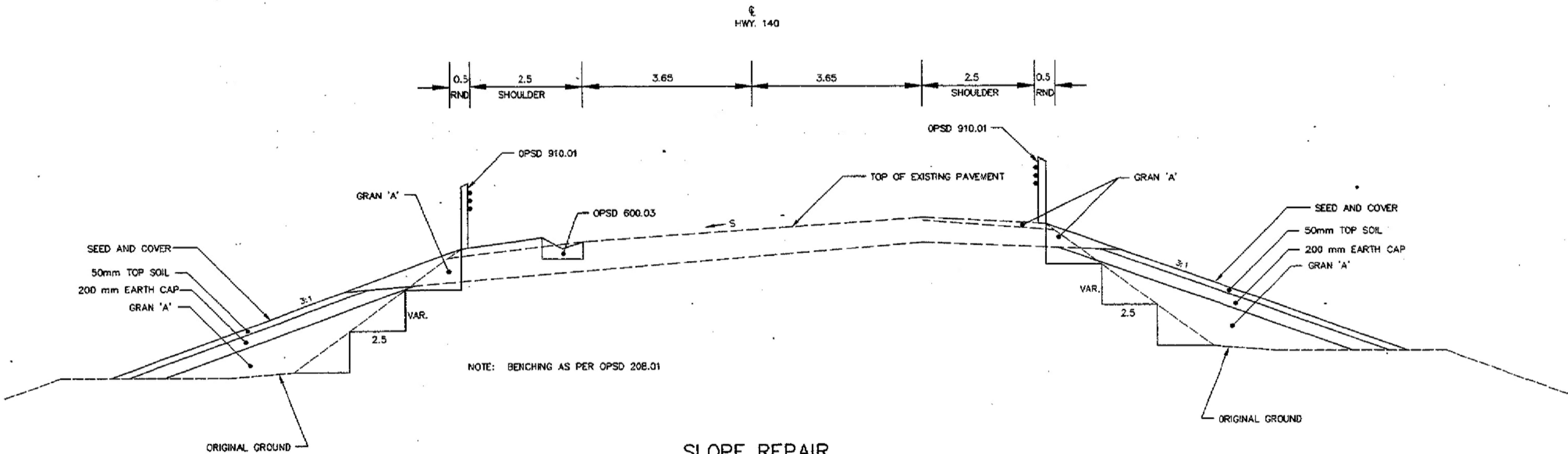


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Construction Materials Engineering, Inspection & Testing
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KEY PLAN



SLOPE REPAIR
STA 15+410 TO STA 15+620 (EAST SLOPE)
STA 15+500 TO STA 15+615 (WEST SLOPE)

NOT TO SCALE

REFERENCE

Ministry of Transportation, Contract No. 98-116,
WP No. 418-97-01

REVISIONS			
	DATE	BY	DESCRIPTION
HWY. 140	PROJECT No.	11-14-4076	DIST.
SUBM'D. HA	CHKD. RA	DATE: Aug. -, 2014	SITE:
DRAWN: KC	CHKD. RA	APPD: MT	DWG. 6

APPENDIX A

Record of Borehole Sheets



EXPLANATION OF TERMS USED IN REPORT

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

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

N (BLOWS/0.3m)	0 – 5	5 – 10	10 – 30	30 – 50	>50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND/OR STRENGTH.

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

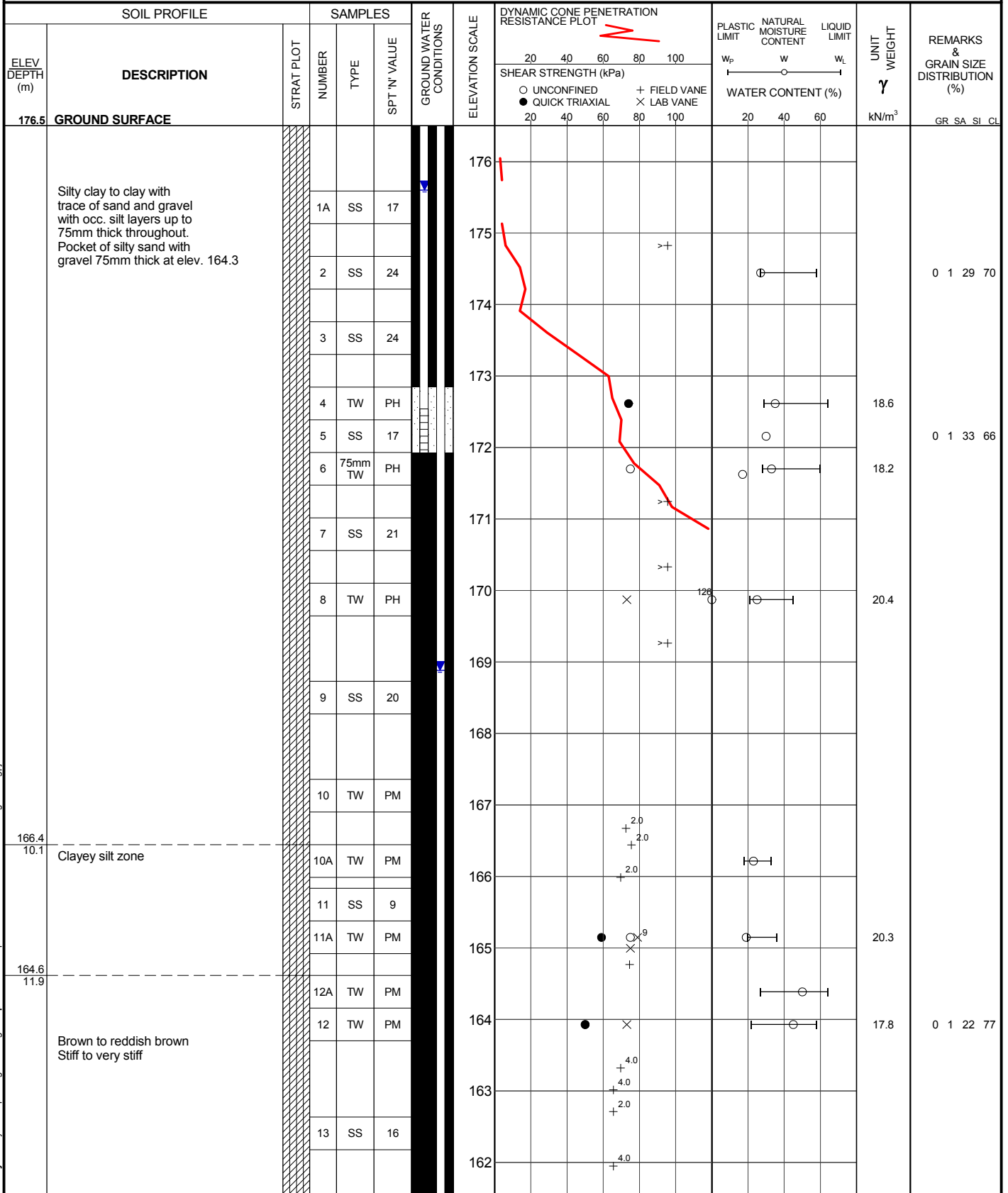
ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	e	1.0%	VOID RATIO	e_{min}	1.0%	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1.0%	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1.0%	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT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

RECORD OF BOREHOLE No 1

1 of 2

METRIC

W.P. 60-68-03 LOCATION Sta.216+50 & East Side Hwy. o/s 25' Rt. Coords: E:328388.3 N:4756393.4 ORIGINATED BY WH
DIST HWY 140 BOREHOLE TYPE CONT. FLIGHT AUGER & DIAMOND DRILL COMPILED BY WH
DATUM GEODETIC DATE 1968-10-17 - 1968-11-1 CHECKED BY



Continued Next Page

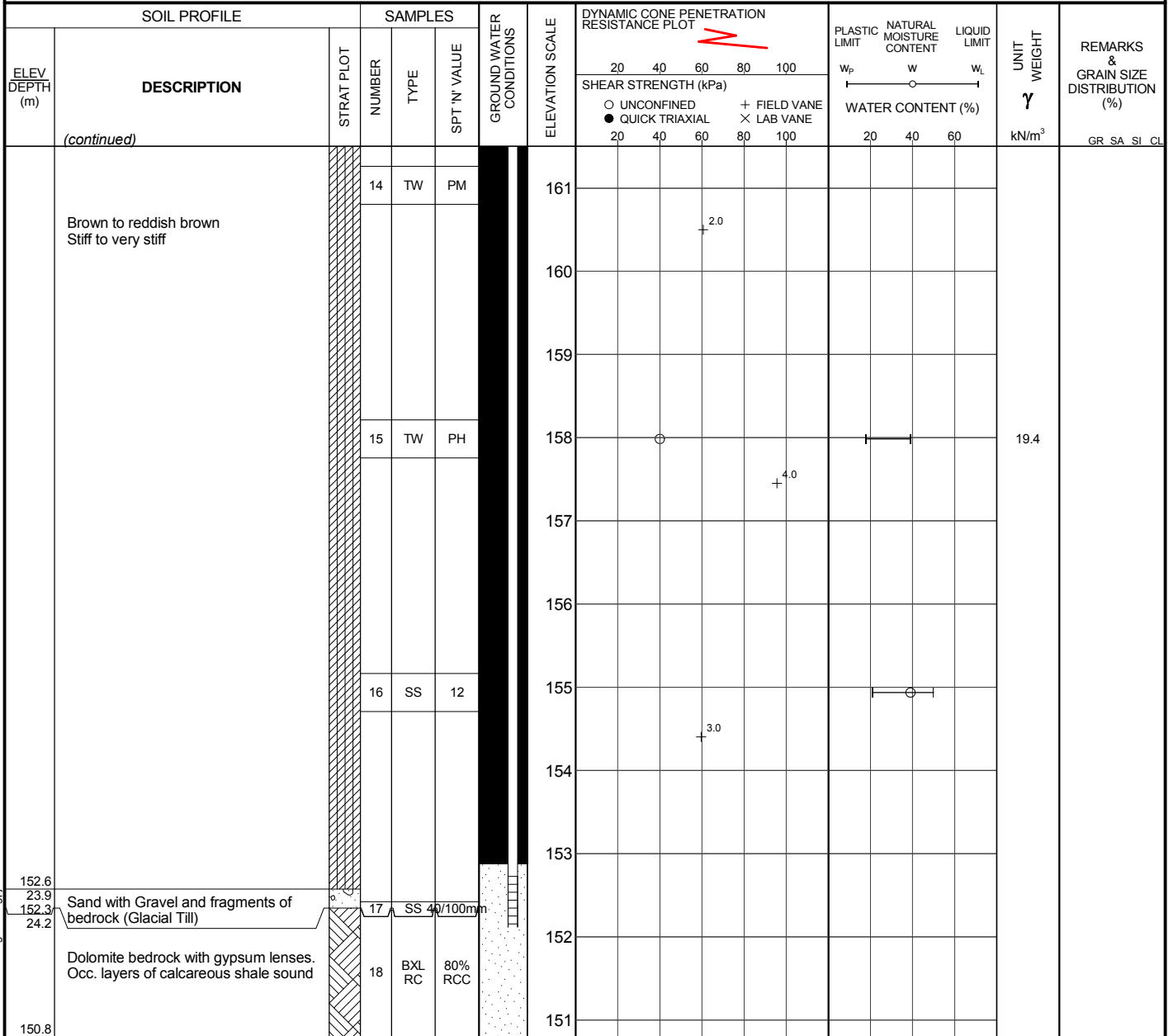
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1

2 of 2

METRIC

W.P. 60-68-03 LOCATION Sta.216+50 & East Side Hwy. o/s 25' Rt. Coords: E:328388.3 N:4756393.4 ORIGINATED BY WH
DIST HWY 140 BOREHOLE TYPE CONT. FLIGHT AUGER & DIAMOND DRILL COMPILED BY WH
DATUM GEODETIC DATE 1968-10-17 - 1968-11-1 CHECKED BY



END OF BOREHOLE

RECORD OF BOREHOLE No 3

1 of 2

METRIC

W.P. 60-68-03 LOCATION Sta 217+53 & East Side Hwy. O/S 73' Rt. Coords: E:328411.9 N:4756418.2 ORIGINATED BY WH
DIST HWY 140 BOREHOLE TYPE CONT. FLIGHT AUGER COMPILED BY WH
DATUM GEODETIC DATE 1968-10-28 - 1968-10-29 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE								
176.9	GROUND SURFACE												
175.5	Roadway Fill Clayey Silt with some sand & gravel, very stiff, dark grey		1	SS	17		176						
174	Silty clay to clay with trace of sand occ. very thin grey silt seams containing clear gypsum crystals above elev. 168.9m.		2	SS	36		175						
			3	SS	27		174						
			4	TW	PH		173						
			5	SS	13		172						
			6	TW	PH		171						
			7	SS	22		170						
			8	3"TW	PH		169						
			9	SS	7		168						
			10	3"TW	PH		167						
			11	SS	11		166						
163.1	Occasional silt layers up to 75mm thick.						165						
163.8							164						
							163						
							162						

Continued Next Page

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

RECORD OF BOREHOLE No 3

2 of 2

METRIC

W.P. 60-68-03 LOCATION Sta 217+53 & East Side Hwy. O/S 73' Rt. Coords: E:328411.9 N:4756418.2 ORIGINATED BY WH
DIST HWY 140 BOREHOLE TYPE CONT. FLIGHT AUGER COMPILED BY WH
DATUM GEODETIC DATE 1968-10-28 - 1968-10-29 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					W _p W W _L				
								20 40 60 80 100					20 40 60				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)				
	(continued)																
	Occssional silt layers up to 75mm thick.		12	3"TW	PH		161								0 0 90 10		
160.0							160										
16.9							159										
	Brown to reddish brown		13	SS	-		158										
							157										
	Firm to very stiff						156										
			14	TW	PH		155										
							154										
153.1							153										
23.8	Glacial Till, Clayey silt with sand & gravel, Hard, Brown																
152.4			15	SS	70/150mm												
24.5																	

END OF BOREHOLE

Probable Bedrock

library: library - terraprobe gint - md.gib report: mto-terraprobe soil file: 11-14-4076 bh logs rev2.gpj

RECORD OF BOREHOLE No 101

1 of 1

METRIC

W.P. 60-68-02 LOCATION C.N.R & Forkes Road. Sta. 214+60 32' Rt. Hwy. 140 Coords: E:328388 N:4756337 ORIGINATED BY S.A
DIST HWY 140 BOREHOLE TYPE NX CASING COMPILED BY W.V.U
DATUM GEODETIC DATE 1971-11-23 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)										WATER CONTENT (%)		
								○ UNCONFINED ● QUICK TRIAXIAL										+ FIELD VANE × LAB VANE		
177.6	GROUND SURFACE						20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL				
176.7 0.9	(Reddish - Brown) Silty clay to clay, trace of sand. (Grey - Brown) Stiff to very stiff.		1	SS	9		177													
			2	SS	14		177													
			3	SS	14		176													
			4	SS	7		176													
			5	TW	-		175										20.0			
			6	TW	PM		175										20.3			
			7	SS	14		174													
			8	TW	PM		174										19.4			
			9	SS	13		173													
			10	TW	PM		173										19.9			
			11	SS	13		172													
			12	TW	PM		172										18.3			
			13	TW	PM		172										18.9			
			14	TW	PH		171										19.2			
							171													
							170													
							169													
168.0			17	TW	PM		168													

END OF BOREHOLE


library: library - terraprobe gint - md.gib report: mto-terraprobe soil file: 11-14-4076 bh logs rev2.gpj

RECORD OF BOREHOLE No 102

1 of 1

METRIC

W.P. 60-68-02 LOCATION Forkes Road & C.N.R., Sta. 214+60 32' Rt. Hwy. 140 Coords: E:328369.9 N:475630.0 ORIGINATED BY S.A.
 DIST HWY 140 BOREHOLE TYPE NX-AX CASING (DRY BORING) COMPILED BY W.V.U.
 DATUM GEODETIC DATE 1971-7-14 - 1971-7-15 CHECKED BY O.E.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
								20 40 60 80 100					W _p W W _L				
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
183.9	GROUND SURFACE							20 40 60 80 100					20 40 60				GR SA SI CL
176.6	Silty clay to clay, trace of sand (Fill) (grey - Brown) Stiff to very stiff.		1	SS	10		183										
			2	SS	8												
			3	SS	7		182										
			4	TW	PM												
			5	TW	PM		181								19.5		
			6	TW	PM										19.2		
			7	TW	PM		180								19.5		
			8	TW	PM										19.5		
			9	TW	PM		179								19.9		
			10	TW	PM										19.4		
			11	TW	PM		178								20.3		
			12	TW	PM										19.9		
			13	TW	PM												
			14	SS	----		177								19.4		

END OF BOREHOLE

RECORD OF BOREHOLE No 102A

1 of 1

METRIC

W.P. 60-68-02 LOCATION Forkes Road & C.N.R., Sta. 214+60 32' Rt. Hwy. 140 Coords: E:328369.9 N:475634.0 ORIGINATED BY S.A.
 DIST HWY 140 BOREHOLE TYPE NX COMPILED BY W.V.U.
 DATUM GEODETIC DATE 1971-7-15 CHECKED BY O.E.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
							20	40	60	80	100	W _p	W	W _L			
183.9	GROUND SURFACE																
	Silty clay to clay, trace of sand - Fill. Reddish - Brown. Stiff to very Stiff																
			1	SS	20								○				
			2	SS	12								○				
			3	SS	4								○				
			4	SS	9								○				
			5	SS	10								○				
			6	SS									○				
			7	SS	15												
			8	SS	12												
176.4																	
7.5	Clayey Topsoil		9	SS	33												
176.1			10	SS	11												
7.8			11	SS	57												
	Silty clay to clay, trace of sand. Occasional silt and sand layers up to 75mm thick. Stiff to very stiff																
			12	SS	27												
			13	SS	13												
			14	SS	12												
			15	SS	15												
170.0																	
13.9																	
END OF BOREHOLE																	


library: library - terraprobe gint - md.gib report: mto-terraprobe soil file: 11-14-4076 bh logs rev2.gpj

RECORD OF BOREHOLE No 103

1 of 1

METRIC

W.P. 60-68-02 LOCATION Forkes Road & C.N.R., Sta. 214+50 18' Lt. Hwy. 140 Coords: E:328355.7 N:47563.4
 DIST HWY 140 BOREHOLE TYPE NX CASING ORIGINATED BY S.A
 DATUM GEODETIC DATE 1971-7-15 COMPILED BY W.V.U
 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					WATER CONTENT (%)									
								20 40 60 80 100					W _p	W	W _L							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					WATER CONTENT (%)									
184.3	GROUND SURFACE							20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL					
	Clay to silty clay, trace of sand - Fill. Reddish - Brown. Stiff to very Stiff					▽	184										20.0					
			1	SS	18		183															
			2	SS	15		182															
			3	TW	PM																	
			4	TW	PM		181															
5	TW	PM																				
6	TW	PM	180																			
7	TW	PM	179																			

RECORD OF BOREHOLE No 103A

1 of 1

METRIC

W.P. 60-68-02 LOCATION Forkes Road & C.N.R., Sta. 214+50 18' Lt. Hwy. 140 Coords: E:328355.7 N:4756340
 DIST HWY 140 BOREHOLE TYPE NX CASING ORIGINATED BY S.A
 DATUM GEODETIC DATE 1971-7-19 COMPILED BY W.V.U
 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
								20 40 60 80 100					W _p W W _L				
													○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE				
184.3	GROUND SURFACE							20	40	60	80	100	20	40	60	kN/m ³	GR SA SI CL
177.4	Clay to silty clay, trace of sand - fill Reddish - brown Stiff to very stiff						184										DRY
			1	SS	21		183										
							182										
			2	SS	12		181										
							180										
			3	SS	16		179										
							178										
			4	SS	15												

RECORD OF BOREHOLE No 104

1 of 1

METRIC

W.P. 60-68-02 LOCATION Forkes Road & C.N.R., Sta. 214+50 86' Lt. Hwy. 140 Coords: E:328388.3 N:475635.0 ORIGINATED BY S.A
 DIST HWY 140 BOREHOLE TYPE NX CASING COMPILED BY W.V.U
 DATUM GEODETIC DATE 1971-7-20 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
								20	40	60	80	100	W _p	W	W _L		
177.6	GROUND SURFACE																
173.0	Clay to silty clay, trace of sand, Reddish - brown, Stiff to very stiff		1	SS	8		177										
			2	SS	13												
			3	SS	13												
			4	SS	11												
			5	SS	18												
			6	SS	21												
			7	TW	PM												
			8	SS	31												
			9	SS	20												
			10	SS	11												

END OF BOREHOLE

METRIC[illegible][illegible]

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

RECORD OF BOREHOLE No 208

2 of 2

METRIC

W.P. 60-68-02 LOCATION Hwy. #140 Sta. 212+00 o/s 15' Lt. Coords: E:328388.8 N:4756278.7 ORIGINATED BY R.R.B
DIST HWY 140 BOREHOLE TYPE C.M.E. AUGERING AND NX CASING WASHBORE COMPILED BY R.R.B
DATUM GEODETIC DATE 1972-2-9 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					W _p W W _L				
								20 40 60 80 100					WATER CONTENT (%)				
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE						
	(continued)																
	Silty clay to clay, trace of sand (occasional seams of silt and sand up to 75mm thick throughout) (Grey - Brown) stiff to very stiff.		24	SS	12												
			25	TW	PH												
			26	SS	8												
			27	SS	13												
			28	TW	PH												
			29	SS	11												
			30	SS	8												
158.4																	







END OF BOREHOLE

RECORD OF BOREHOLE No 209

1 of 1

METRIC

W.P. 60-68-02 LOCATION Hwy. #140 Sta. 212+00 o/s 75' Rt. Coords: E:328352.9 N:4756268.4 ORIGINATED BY R.R.B
DIST HWY 140 BOREHOLE TYPE C.M.E. AUGERED COMPILED BY R.R.B
DATUM GEODETIC DATE 1972-2-14 - 1972-2-15 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100												
								SHEAR STRENGTH (kPa)												
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE	WATER CONTENT (%)								
180.6	GROUND SURFACE																GR SA SI CL			
176.9 3.7	Silty clay to clay (Fill) Reddish - brown Very stiff to hard.		1	SS	26		180													
			2	SS	47		179													
			3	SS	27		178													
			4	SS	24		178													
			5	TW	PH		178													
			6	SS	52		178													
			7	SS	13		177													
176.2 4.4	Softened Zone. Firm		8	TW	PH			177												
			9	SS	9			176												
175.9 4.7	Clayey Topsoil		10	SS	32				176											
			11	TW	PH				175											
			12	SS	40				175											
			13	SS	19				174											
			14	TW	PH				173											
			15	SS	19				172											
			171.0 9.6																	



END OF BOREHOLE

RECORD OF BOREHOLE No 210

1 of 2

METRIC

W.P. 60-68-02 LOCATION Hwy. #140 Sta. 215+00 o/s 9' Lt. Coords: E:328363.4 N:4756357.3 ORIGINATED BY R.R.B
DIST HWY 140 BOREHOLE TYPE COMPILED BY R.R.B
DATUM GEODETIC DATE 1972-2-15 - 1972-2-16 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL						
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100							W _p	W	W _L			
								SHEAR STRENGTH (kPa)										WATER CONTENT (%)		
								○ UNCONFINED		● QUICK TRIAXIAL		+ FIELD VANE						× LAB VANE		
185.1	GROUND SURFACE																			
	Silty clay to clay, trace of sand and gravel (Fill) Reddish - brown Stiff to very stiff.		1	SS	15															
			2	SS	17															
			3	TW	PH						125									
			4	SS	15															
			5	SS	15					133										
										133										
			6	TW	PH															
177.9																				
7.2	Clayey Topsoil	~ ~																		
177.6																				
7.5	Silty clay to clay, trace of sand and gravel. (occasional seams of silt and sand up to 75mm thick) Stiff to very stiff.		7	SS	31															
			8	SS	16															
			9	TW	PH															
			10	SS	16															
			11	SS	14															

Continued Next Page

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

RECORD OF BOREHOLE No 210

2 of 2

METRIC

W.P. 60-68-02 LOCATION Hwy. #140 Sta. 215+00 o/s 9' Lt. Coords: E:328363.4 N:4756357.3 ORIGINATED BY R.R.B
DIST HWY 140 BOREHOLE TYPE COMPILED BY R.R.B
DATUM GEODETIC DATE 1972-2-15 - 1972-2-16 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100					W _p W W _L				
								SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
	(continued)																
159.8	Silty clay to clay, trace of sand and gravel. (occasional seams of silt and sand up to 75mm thick) Stiff to very stiff.		12	TW	PH				+	×			○	20.3			
			13	SS	10												
			14	SS	7												
			15	TW	PH												
			16	SS	20												
			17	SS	8												
			18	TW	PH												
													</				



END OF BOREHOLE

RECORD OF BOREHOLE No 211

1 of 1

METRIC

W.P. 60-68-02 LOCATION Hwy. #140 Sta. 213+36 o/s 100' Rt. Coords: E:328374.7 N:4756301.4 ORIGINATED BY R.R.B
DIST HWY 140 BOREHOLE TYPE C.M.E. AUGER COMPILED BY R.R.B
DATUM GEODETIC DATE 1972-2-17 CHECKED BY O.E

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
								20 40 60 80 100					w _p w w _L				
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
178.4	GROUND SURFACE																
175.8 2.6	Silty clay to clay, trace of sand and gravel (Fill) Very stiff.		1	SS	-		178										
			2	SS	34		177										
			3	SS	19		176										
	4	TW	PH	175													
	5	SS	15	174													
	6	SS	22	173													
	7	TW	PH	172													
	8	SS	17	171													
	9	SS	16	170													
	168.8																

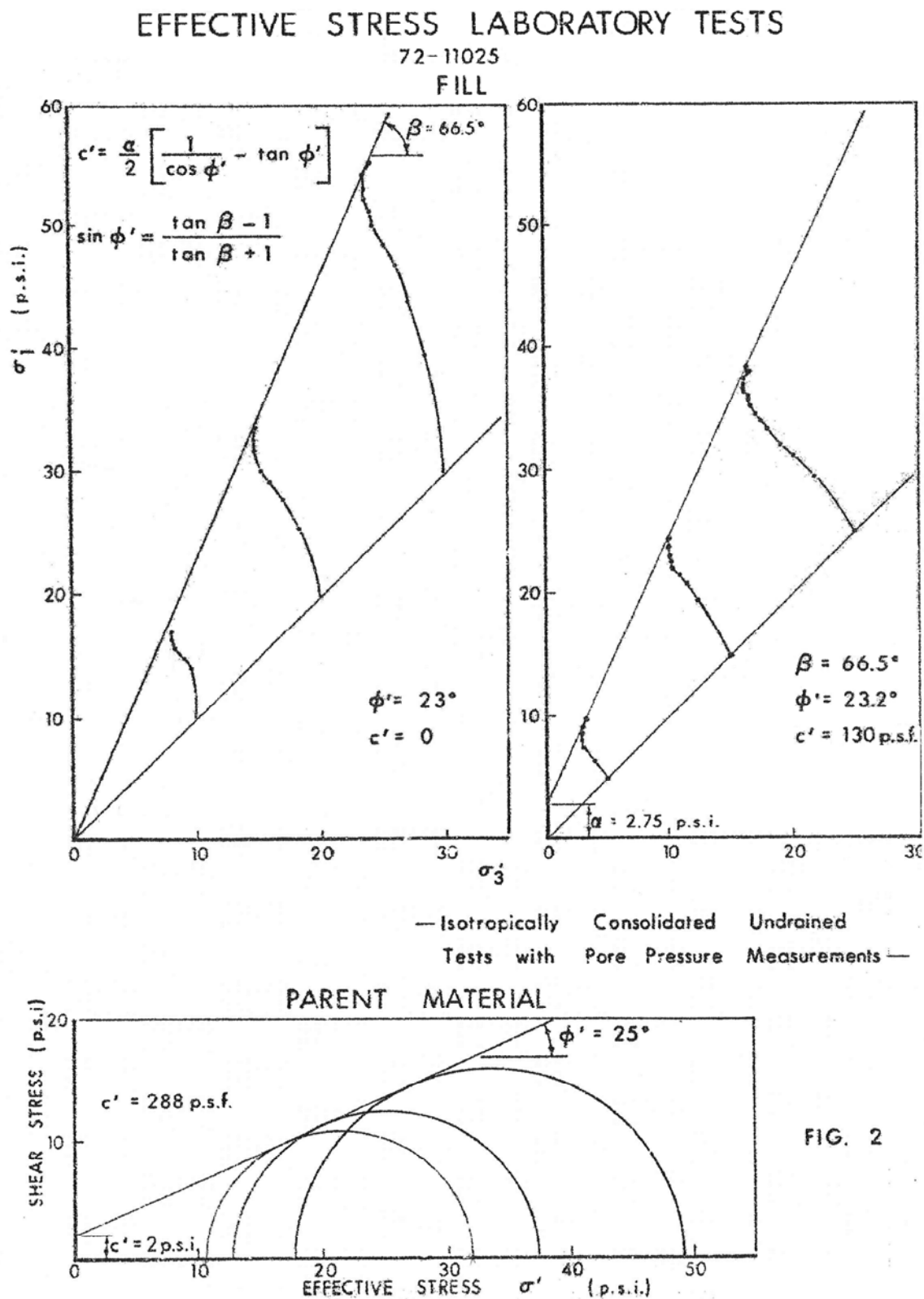
END OF BOREHOLE

library: library - terraprobe gint - md.gib report: mto-terraprobe soil file: 11-14-4076 bh logs rev2.gpj

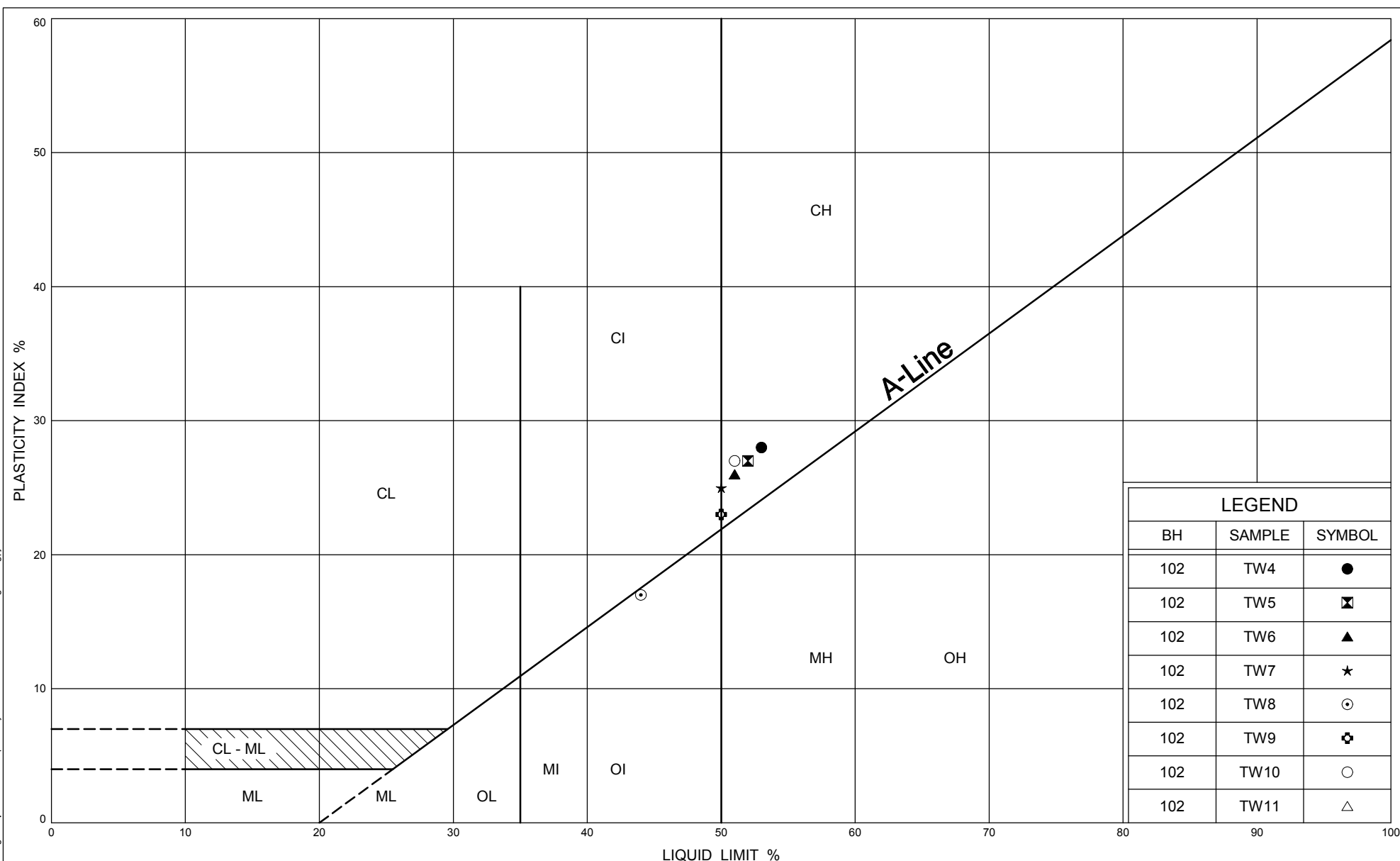
APPENDIX B

Laboratory Test Results





library: library - terraprobe gint - md.glb report: mto-terra-plasticity chart file: 11-14-4076 bh logs rev2.gpl



Ministry of
Transportation

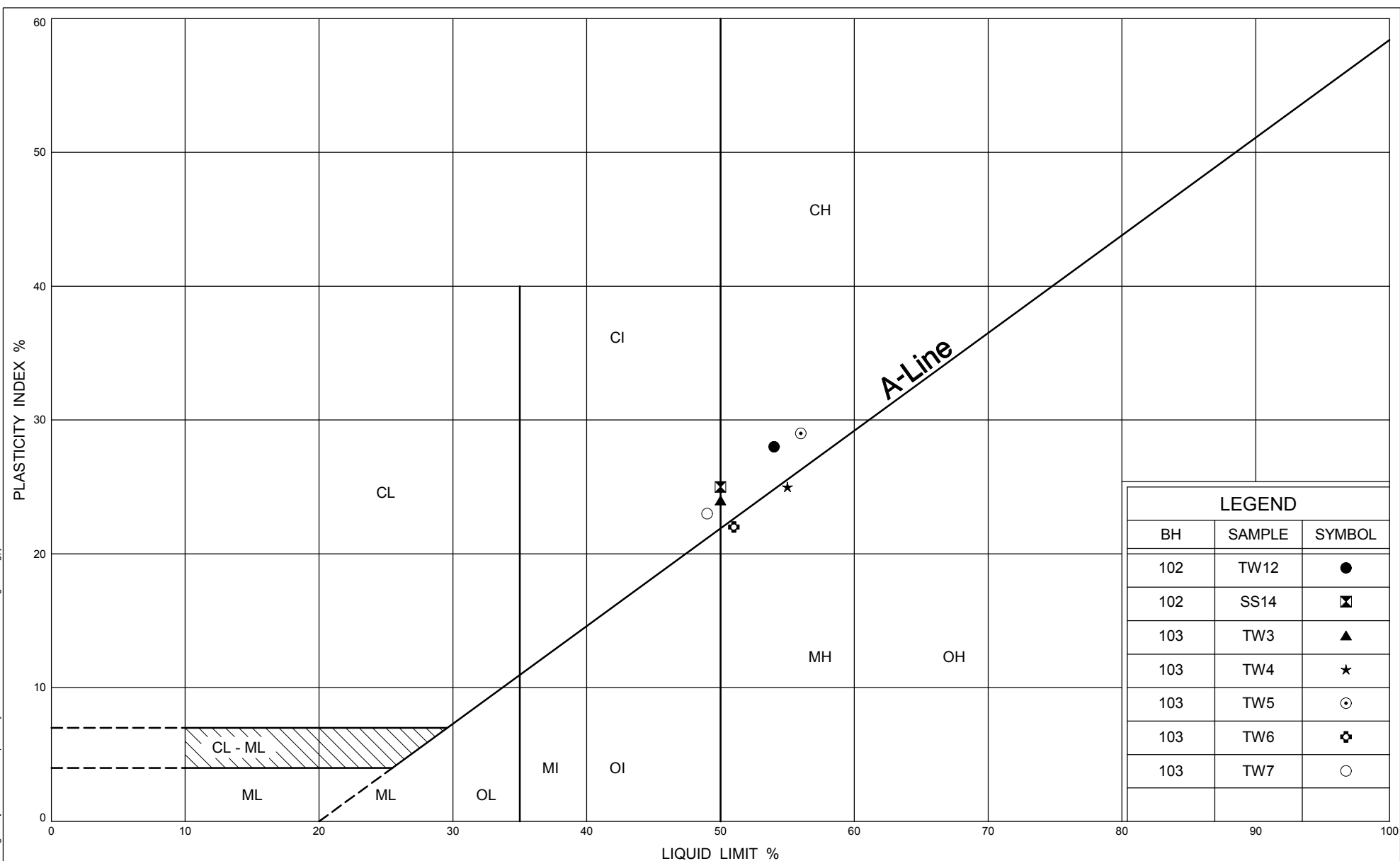
PLASTICITY CHART SILTY CLAY TO CLAY FILL

FIG No B2

W P 60-68-02

Highway 140-CNR Overpass

library: library - terraprobe gint - md.glb report: mto-terra-plasticity chart file: 11-14-4076 bh logs rev2.gpl



Ministry of
Transportation

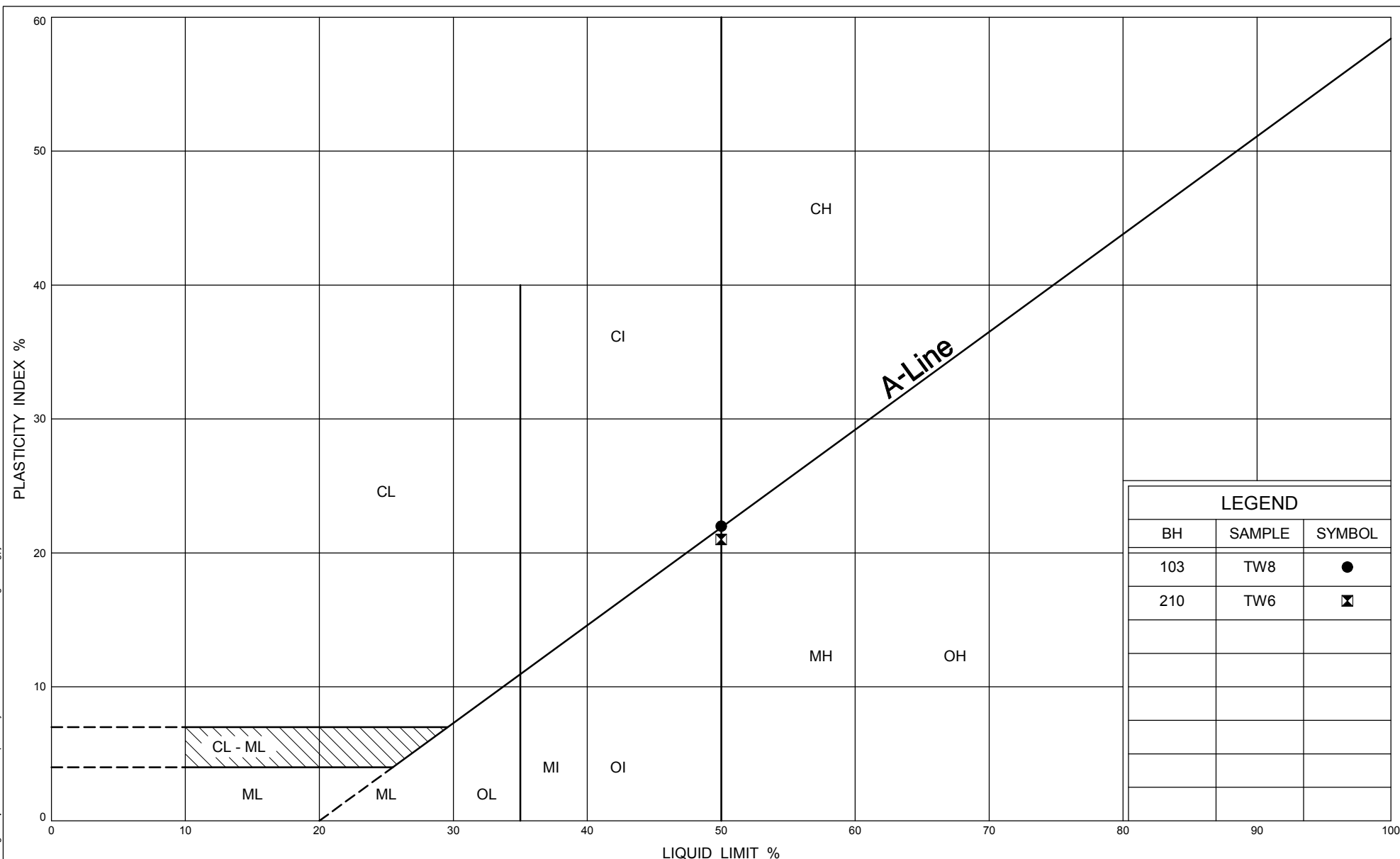
PLASTICITY CHART SILTY CLAY TO CLAY FILL

FIG No B3

W P 60-68-02

Highway 140-CNR Overpass

library: library - terraprobe.gint - md.glb report: mto-terra-plasticity chart file: 11-14-4076 bh logs rev2.gpl



Ministry of
Transportation

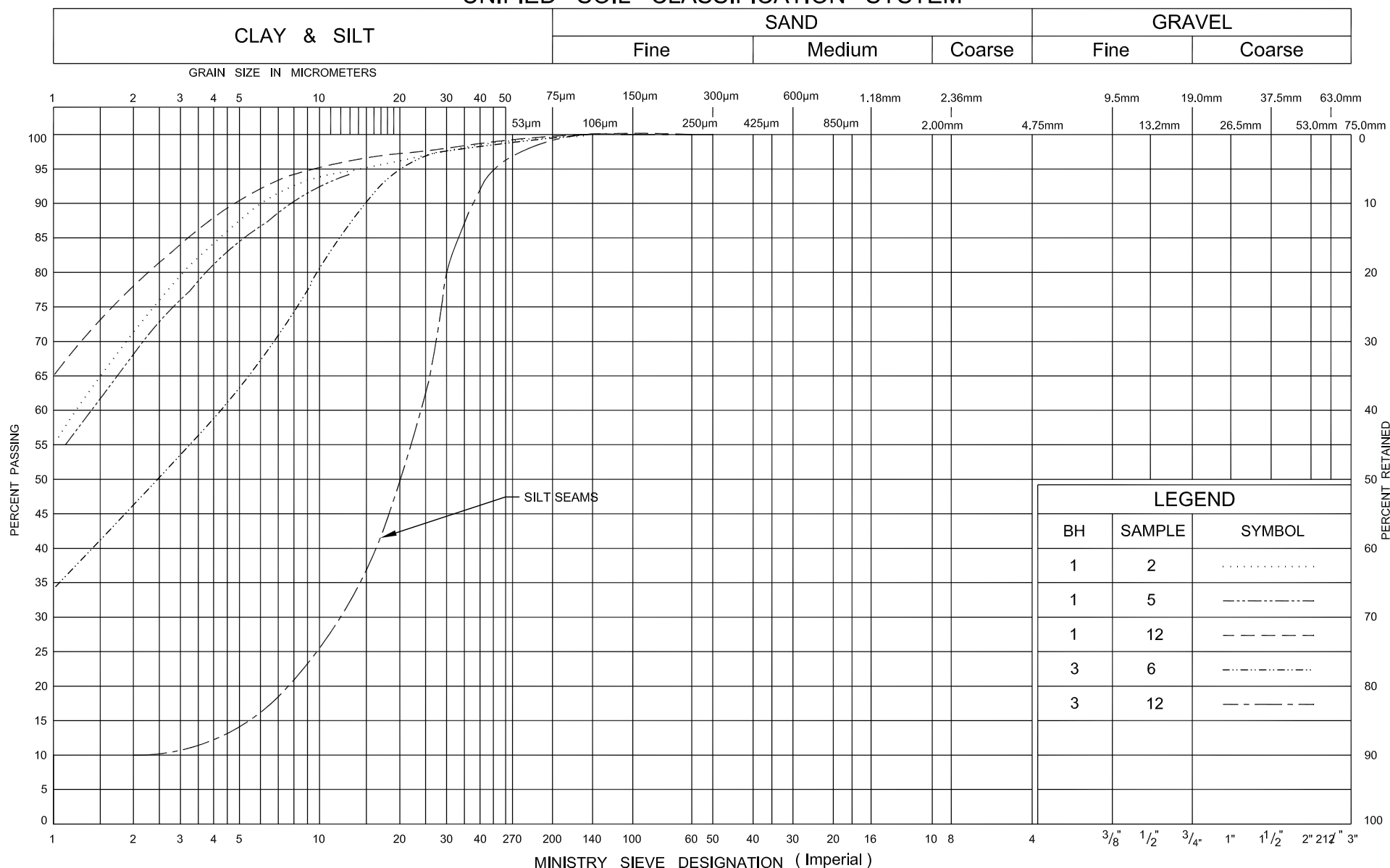
PLASTICITY CHART SILTY CLAY TO CLAY FILL

FIG No B4

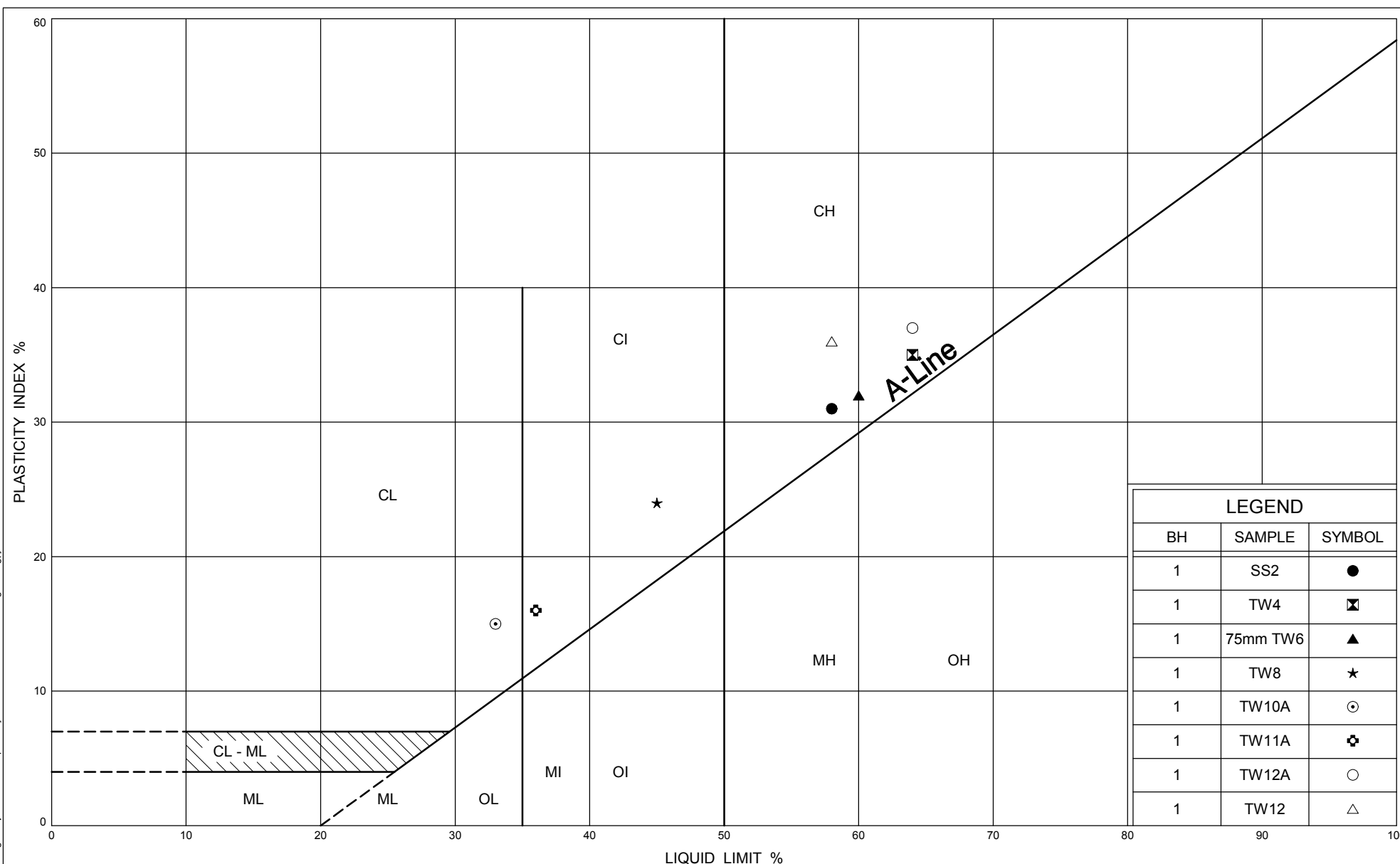
W P 60-68-02

Highway 140-CNR Overpass

UNIFIED SOIL CLASSIFICATION SYSTEM



library: library - terraprobe.gint - md.glb report: mto-terra-plasticity chart file: 11-14-4076 bh logs rev2.gpl



Ministry of
Transportation

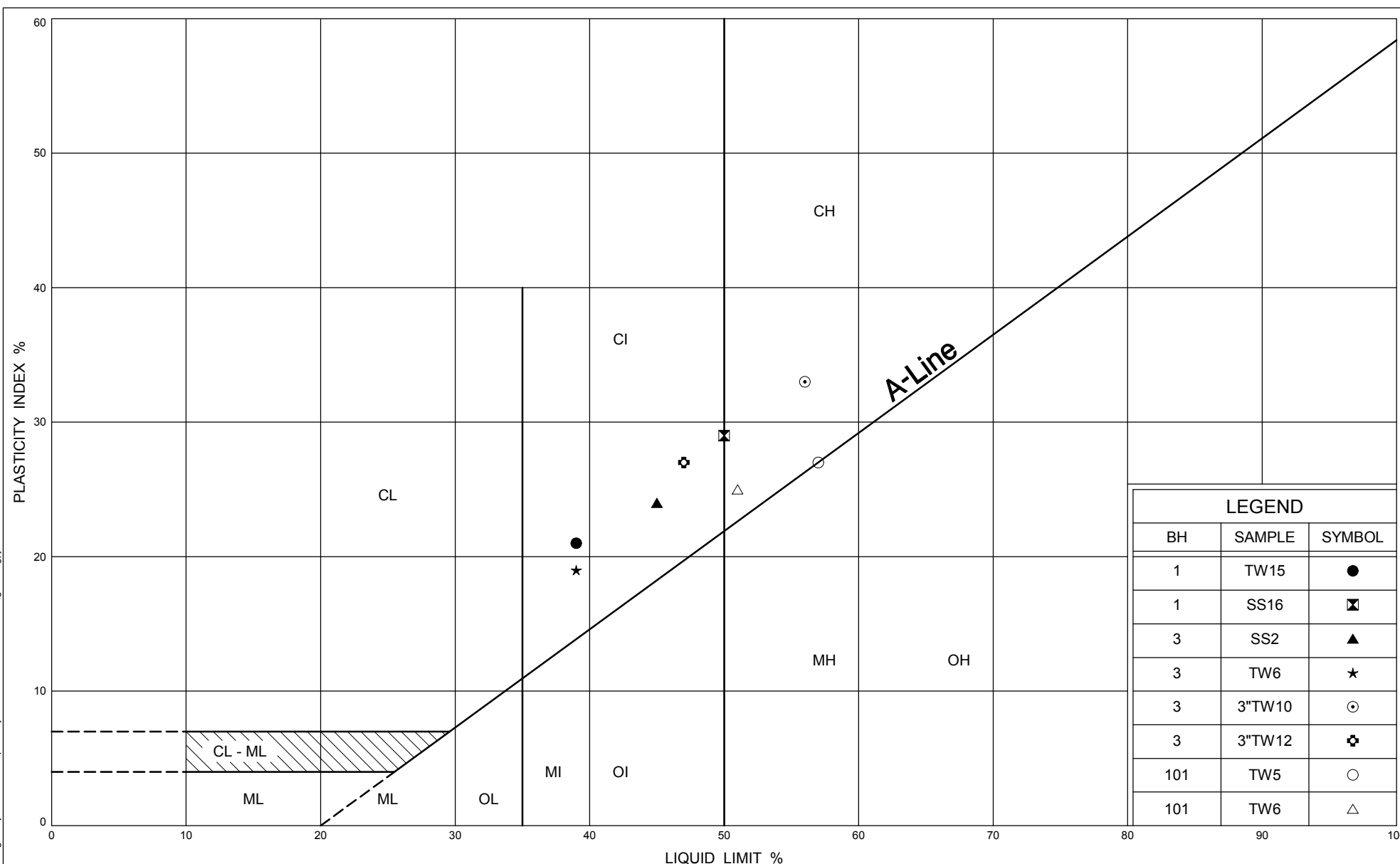
PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B6

W P 60-68-03

Highway 140-CNR Overpass

library: library - terraprobe.gint - md.glb report: mto-terra-plasticity chart file: 11-14-4076 bh logs rev2.gpl



Ministry of
Transportation

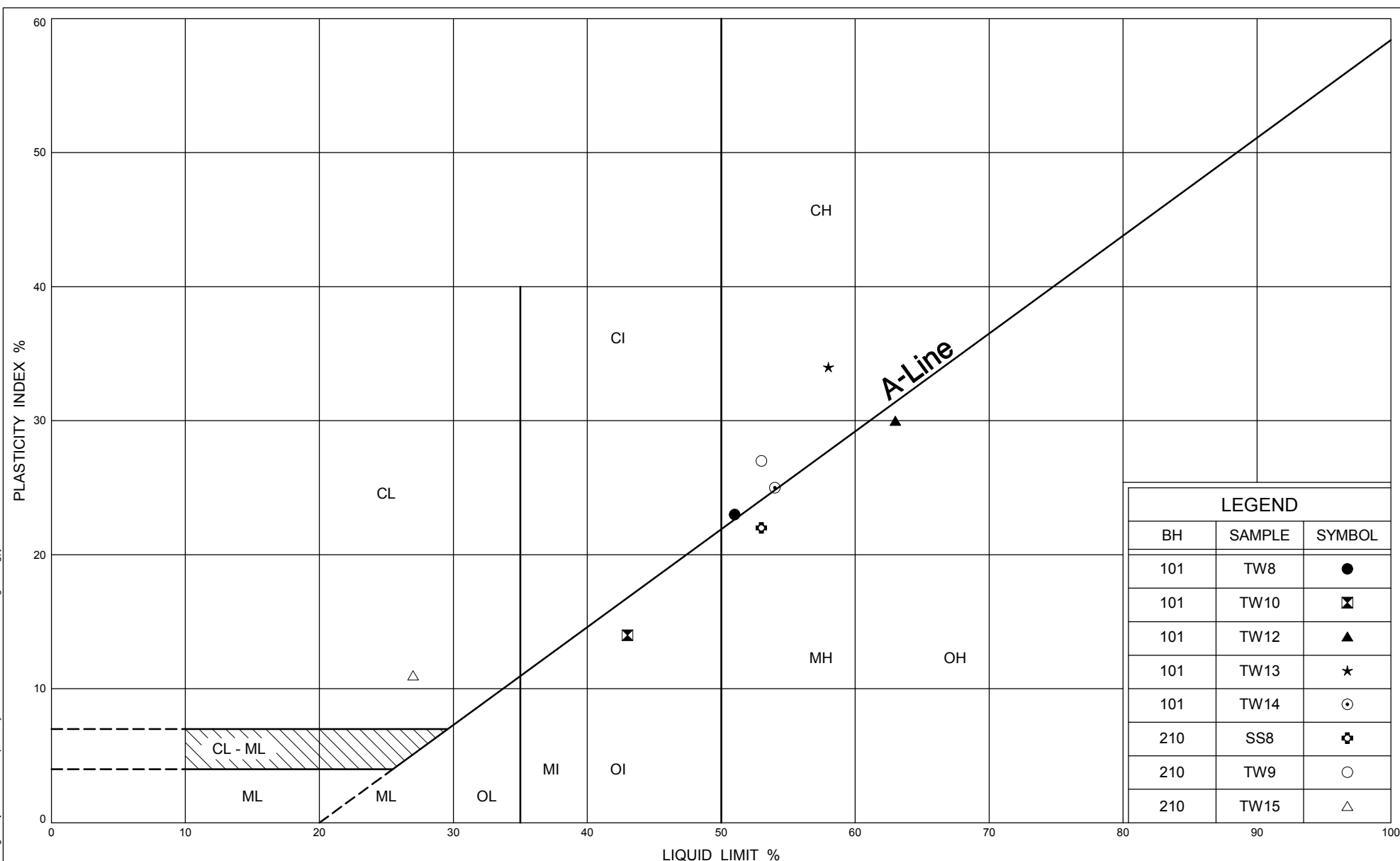
PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B7

W P 60-68-02 & 60-68-03

Highway 140-CNR Overpass

library: library - terraprobe.gint - md.glb report: mto-terra-plasticity chart file: 11-14-4076 bh logs rev2.gpl



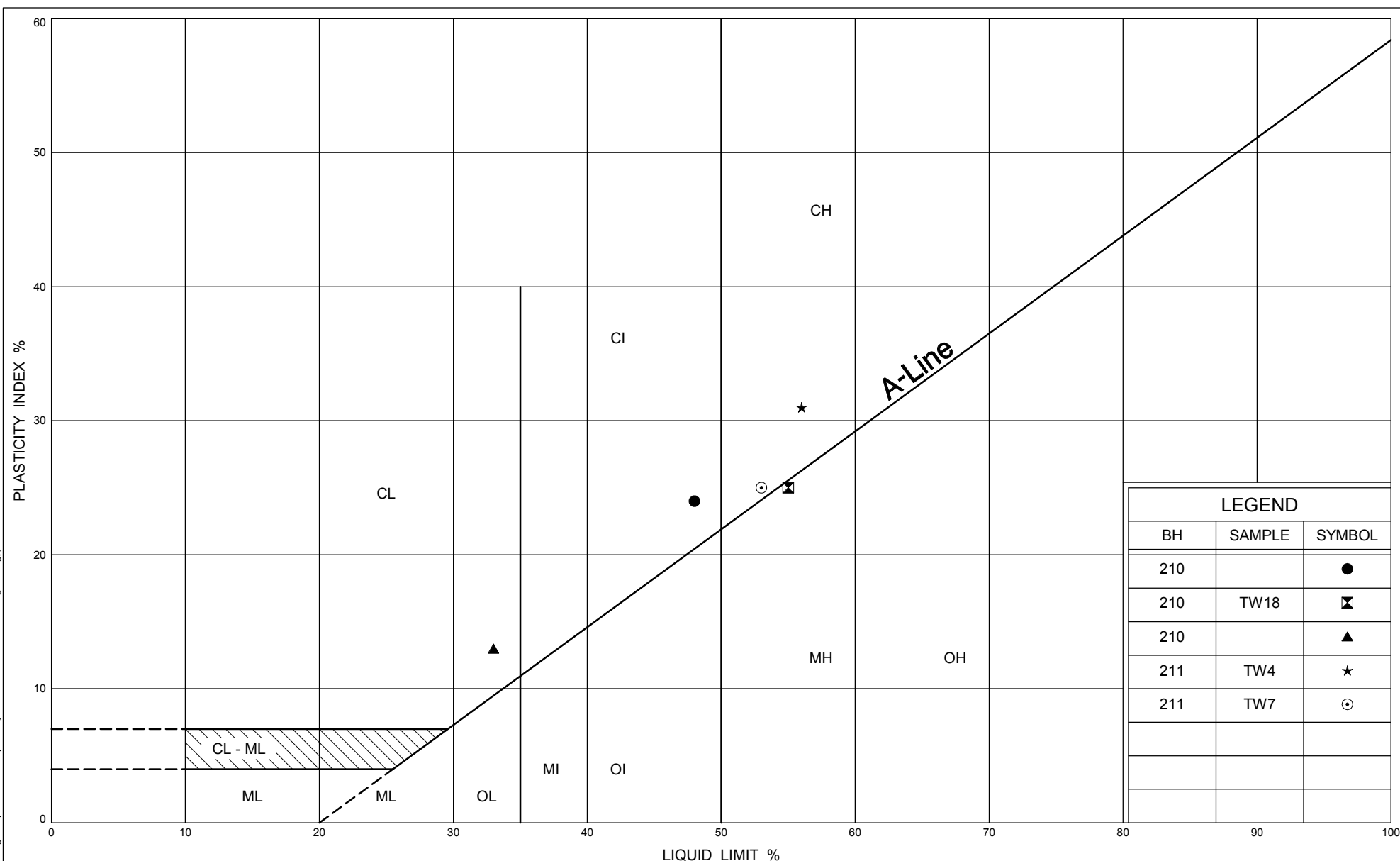
Ministry of
Transportation

PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B8

W P 60-68-02

Highway 140-CNR Overpass



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Transportation

PLASTICITY CHART SILTY CLAY TO CLAY

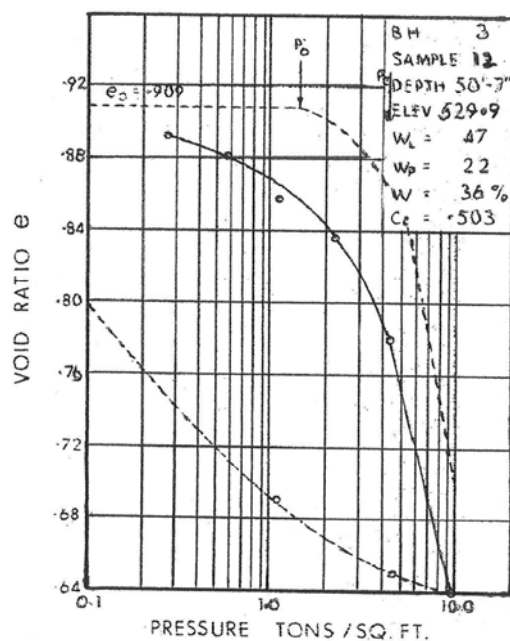
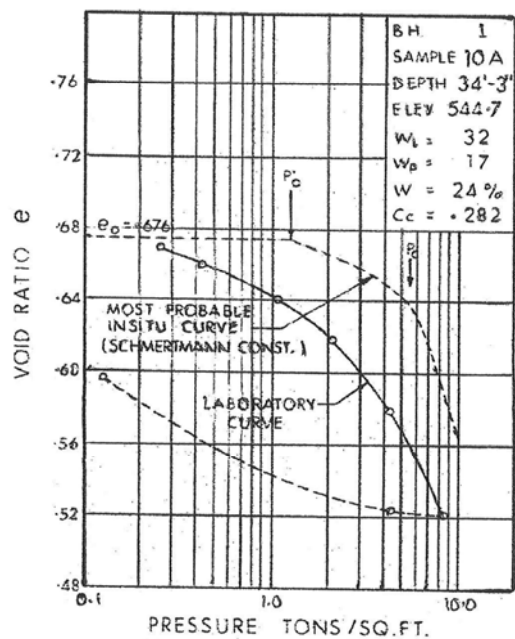
FIG No B9

W P 60-68-02

Highway 140-CNR Overpass

VOID RATIO - PRESSURE CURVES

JOB NO. 68-F-73



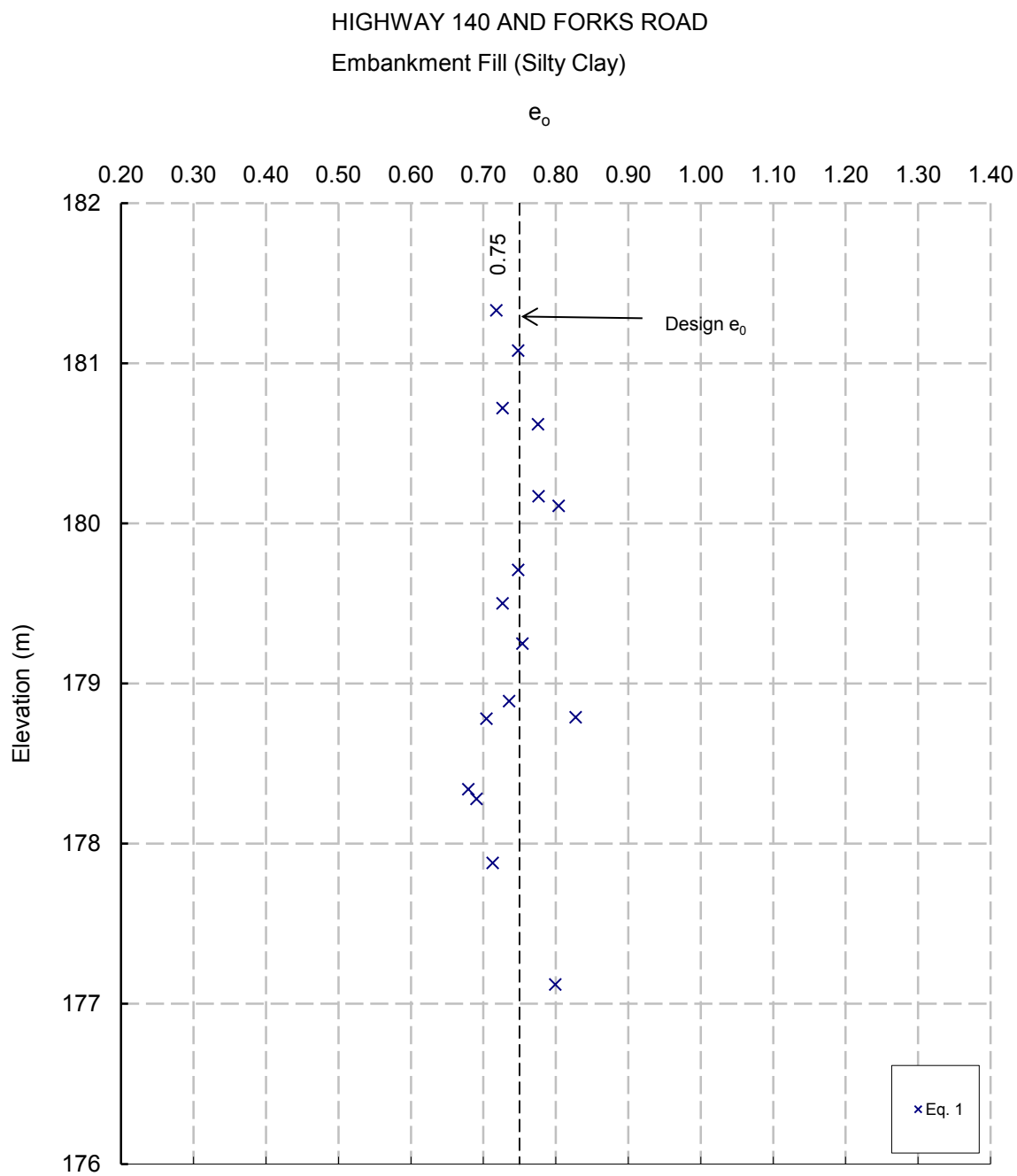
APPENDIX C

Data Interpretation & Analysis (Settlement)



1971 VOID RATIO - EMBANKMENT FILL

FIGURE C1



Eq. 1

$$e_o = (G_s * \gamma_w * (1 + \omega) / \gamma) - 1$$

Unit-Weight Relationship

Project No. : 11-14-4076

Date : August, 2014



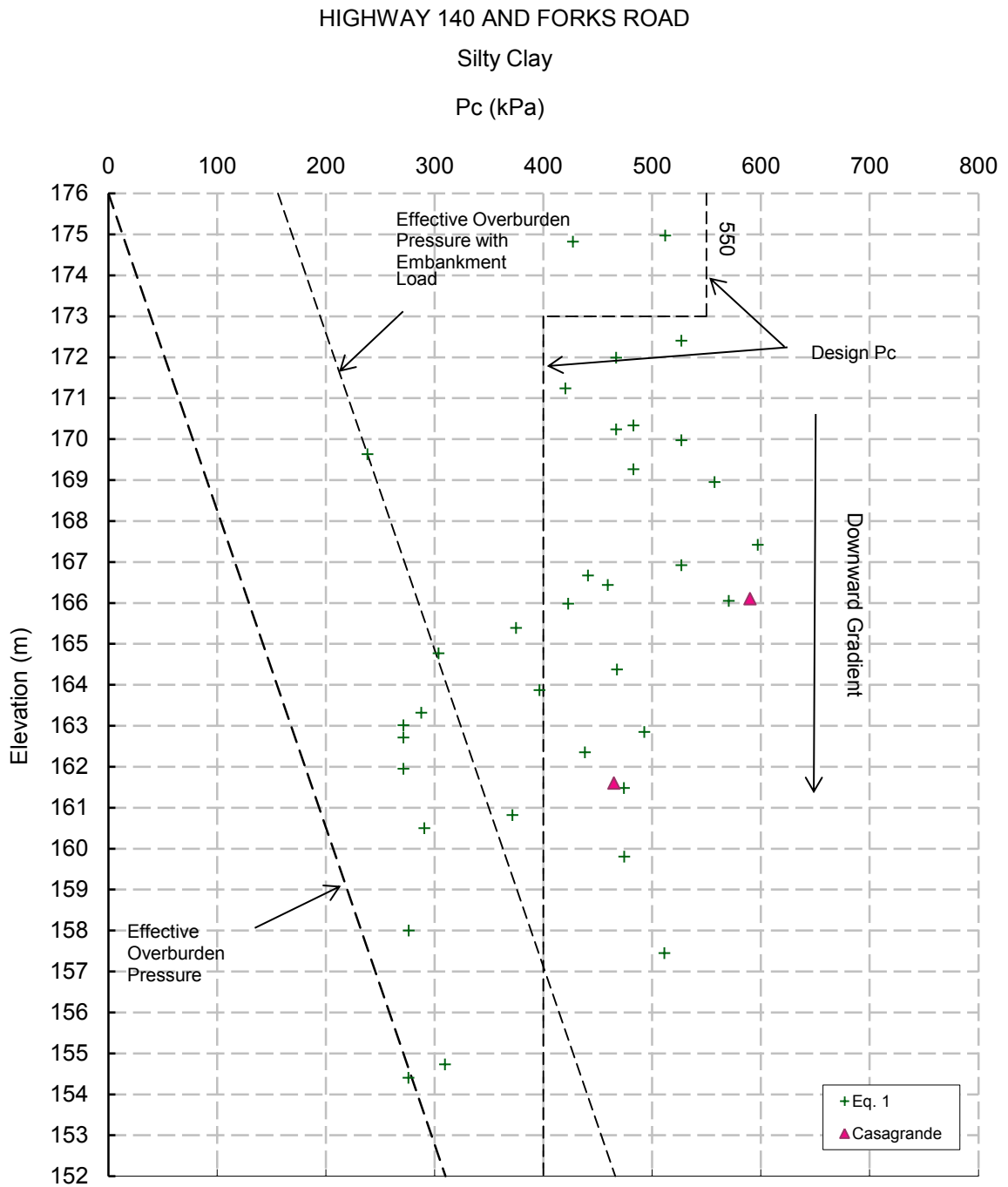
Terraprobe Inc.

Prepared by : AA

Checked by : RA

PREDICTED AND MEASURED PRECONSOLIDATION STRESS

FIGURE C2



Eq. 1 $P_c = C_u / (0.11 + 0.0037 * I_p)$

Chandler (1988)

Project No. : 11-14-4076

Date : August, 2014



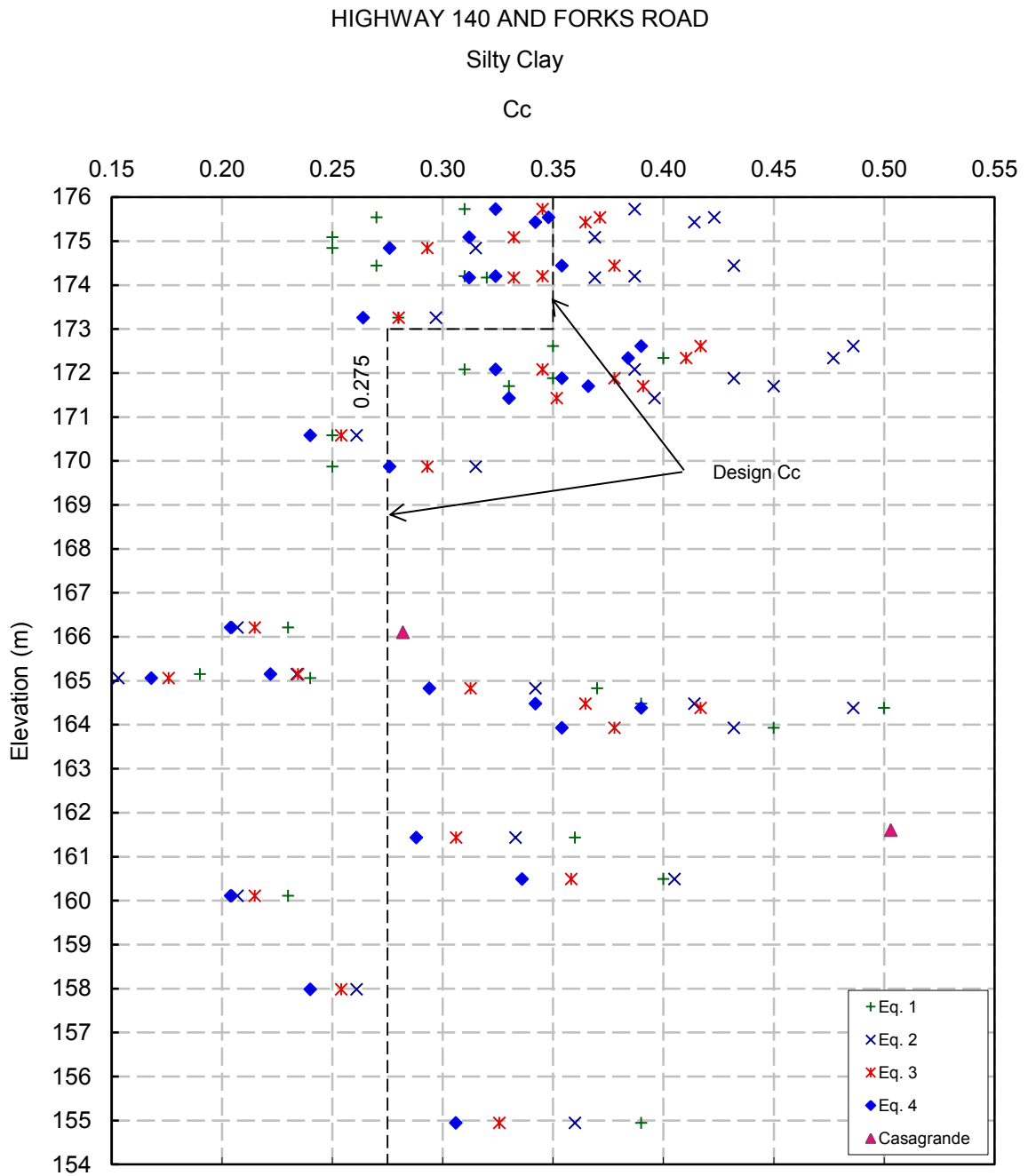
Terraprobe Inc.

Prepared by : AA

Checked by : RA

PREDICTED AND MEASURED COMPRESSION INDEX

FIGURE C3



Eq. 1	$Cc = 0.01 * \omega$	Osterberg (1972)
Eq. 2	$Cc = 0.009 * (LL - 10)$	Terzaghi & Peck (1967)
Eq. 3	$Cc = 0.2343 * LL * Gs$	Nagaraj & Murty (1985)
Eq. 4	$Cc = 0.006 * (LL + 1)$	Lav & Ansal (2001)

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

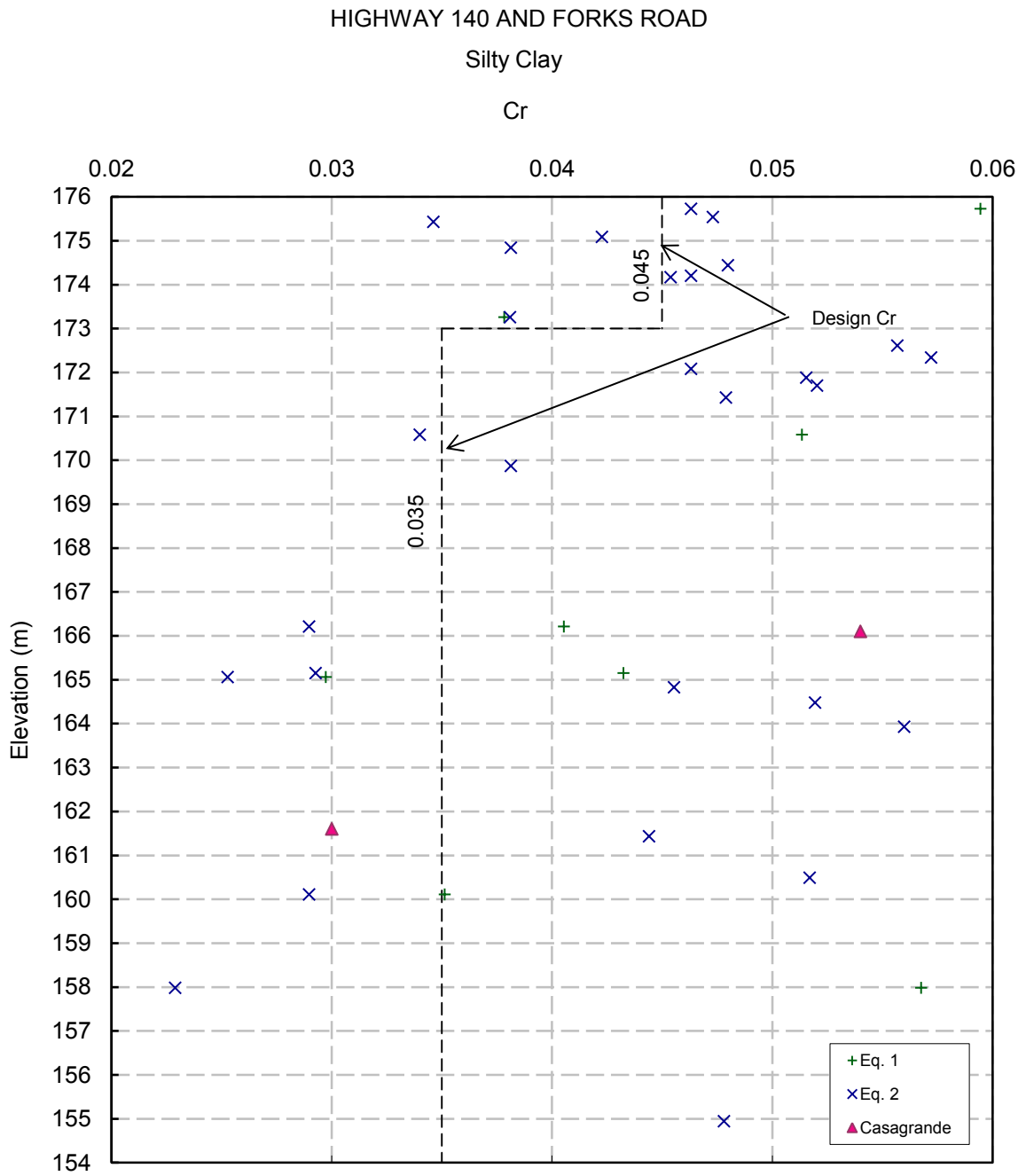


Terraprobe Inc.

Prepared by : AA
Checked by : RA

PREDICTED AND MEASURED RECOMPRESSION INDEX

FIGURE C4



Eq. 1

$$Cr = I_p / 370$$

Kulhawy & Mayne (1990)

Eq. 2

$$Cr = C_c / 5 \sim C_c / 10$$

Das (1993)

Project No. : 11-14-4076

Date : August, 2014



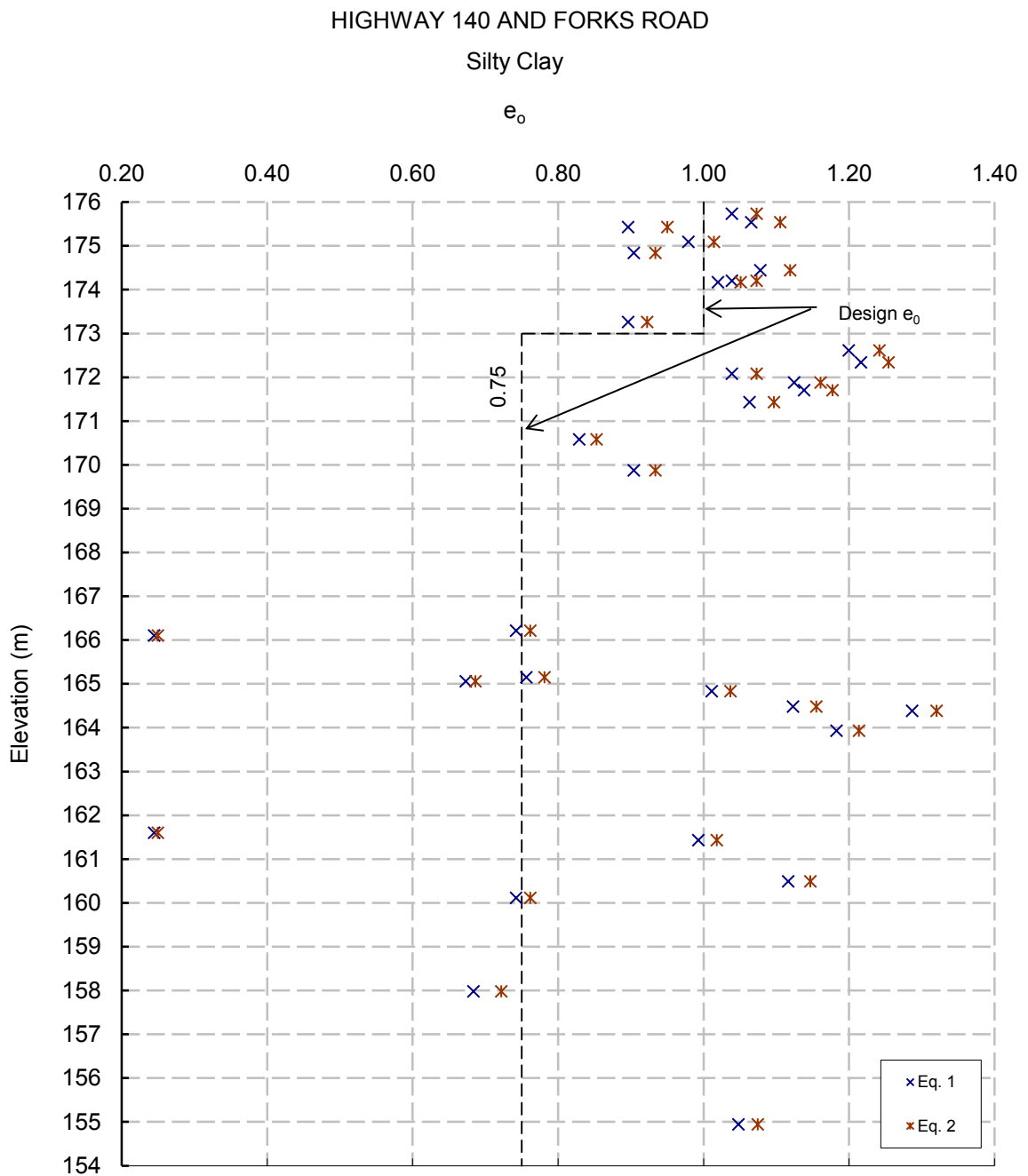
Terraprobe Inc.

Prepared by : AA

Checked by : RA

PREDICTED AND MEASURED VOID RATIO

FIGURE C5



Eq. 1 $e_o = (Cc - 0.256) / 0.43 + 0.84$

derived from Cozzolino (1961)

Eq. 2 $e_o = Cc / 0.40 - 0.001 * \omega + 0.25$

derived from Azzouz et al. (1976)

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Project No. : 11-14-4076

Date : August, 2014



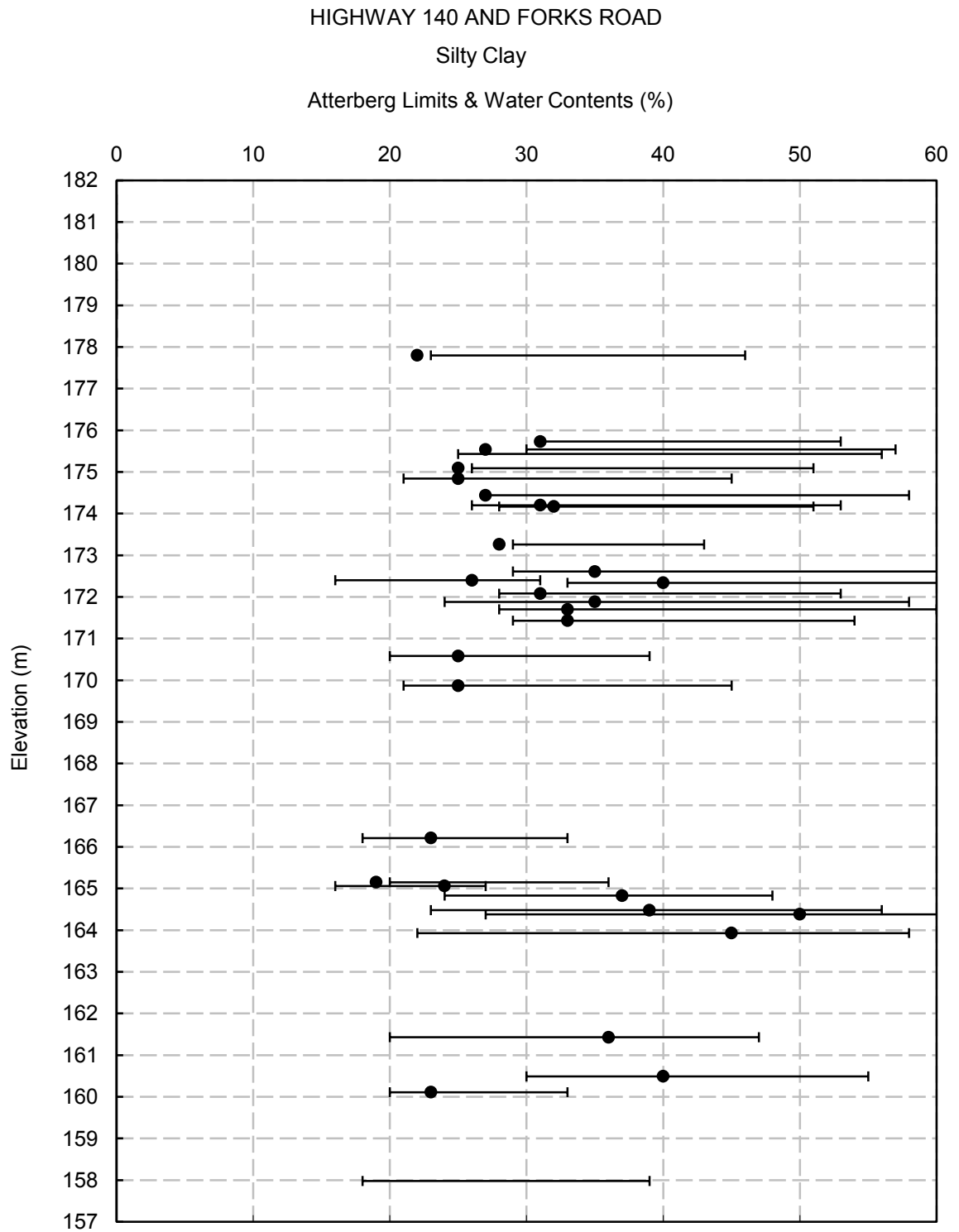
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Prepared by : AA

Checked by : RA

ATTERBERG LIMITS AND WATER CONTENTS

FIGURE C6



Z:\1-Project Files\11-Geo\2014\11-14-4076 Hwy 140 Port Colbourne\J. Eng Analysis\Appendix C Spread Sheets\Appendix C1 to C6.xls

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



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Prepared by : AA
Checked by : RA

OEDOMETER CONSOLIDATION SUMMARY

FIGURE C7
Sheet (1 of 4)

OEDOMETER CONSOLIDATION SUMMARY					FIGURE C12		
Clay Fill					(Sheet 1 of 4)		
SAMPLE IDENTIFICATION							
Project Number	08-1111-0031			Sample Number	3		
Borehole Number	08-1			Sample Depth, m	1.52-2.13		
TEST CONDITIONS							
Test Type	Standard			Load Duration, hr	24		
Oedometer Number	9						
Date Started	11/25/2008						
Date Completed	12/09/2008						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	1.90			Unit Weight, kN/m ³	19.94		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m ³	15.90		
Area, cm ²	31.47			Specific Gravity, measured	2.79		
Volume, cm ³	59.89			Solids Height, cm	1.106		
Water Content, %	25.45			Volume of Solids, cm ³	34.80		
Wet Mass, g	121.80			Volume of Voids, cm ³	25.09		
Dry Mass, g	97.09			Degree of Saturation, %	98.5		
TEST COMPUTATIONS							
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	1.903	0.721	1.903				
4.72	1.972	0.784	1.938	swell			
9.59	1.927	0.743	1.950	7	1.15E-01	4.88E-03	5.50E-05
19.20	1.922	0.738	1.925	101	7.77E-03	2.73E-04	2.08E-07
38.88	1.908	0.725	1.915	86	9.04E-03	3.74E-04	3.31E-07
77.88	1.884	0.704	1.896	37	2.06E-02	3.23E-04	6.53E-07
155.81	1.854	0.677	1.869	26	2.85E-02	2.02E-04	5.65E-07
314.85	1.821	0.647	1.837	35	2.05E-02	1.09E-04	2.19E-07
622.26	1.776	0.606	1.798	20	3.43E-02	7.68E-05	2.58E-07
1244.69	1.720	0.555	1.748	34	1.91E-02	4.73E-05	8.83E-08
2489.96	1.651	0.493	1.685	21	2.87E-02	2.92E-05	8.20E-08
1244.69	1.671	0.511	1.661				
314.85	1.723	0.558	1.697				
77.88	1.780	0.610	1.752				
19.37	1.834	0.658	1.807				
4.72	1.872	0.693	1.853				
Note: k calculated using cv based on t ₉₀ values.							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	1.87			Unit Weight, kN/m ³	20.54		
Sample Diameter, cm	6.33			Dry Unit Weight, kN/m ³	16.16		
Area, cm ²	31.47			Specific Gravity, measured	2.79		
Volume, cm ³	58.91			Solids Height, cm	1.106		
Water Content, %	27.11			Volume of Solids, cm ³	34.80		
Wet Mass, g	123.41			Volume of Voids, cm ³	24.11		
Dry Mass, g	97.09						
Prepared By: LFG				Golder Associates		Checked By: MM	

Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario"
Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

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



Terraprobe Inc.

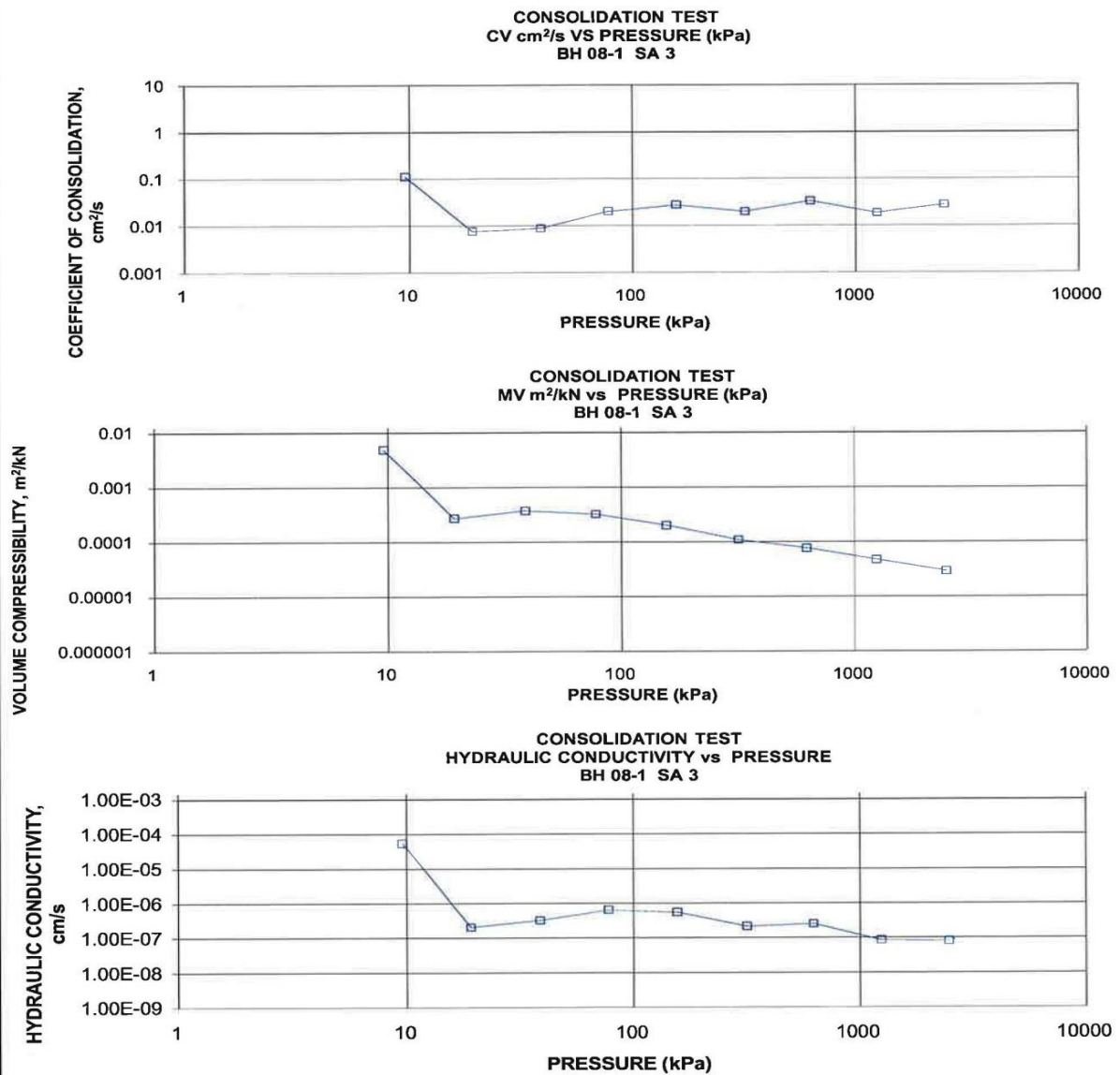
OEDOMETER CONSOLIDATION SUMMARY

FIGURE C7
Sheet (2 of 4)

OEDOMETER CONSOLIDATION SUMMARY

ClayFill

FIGURE C12
(Sheet 2 of 4)



Project No. 08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario"
Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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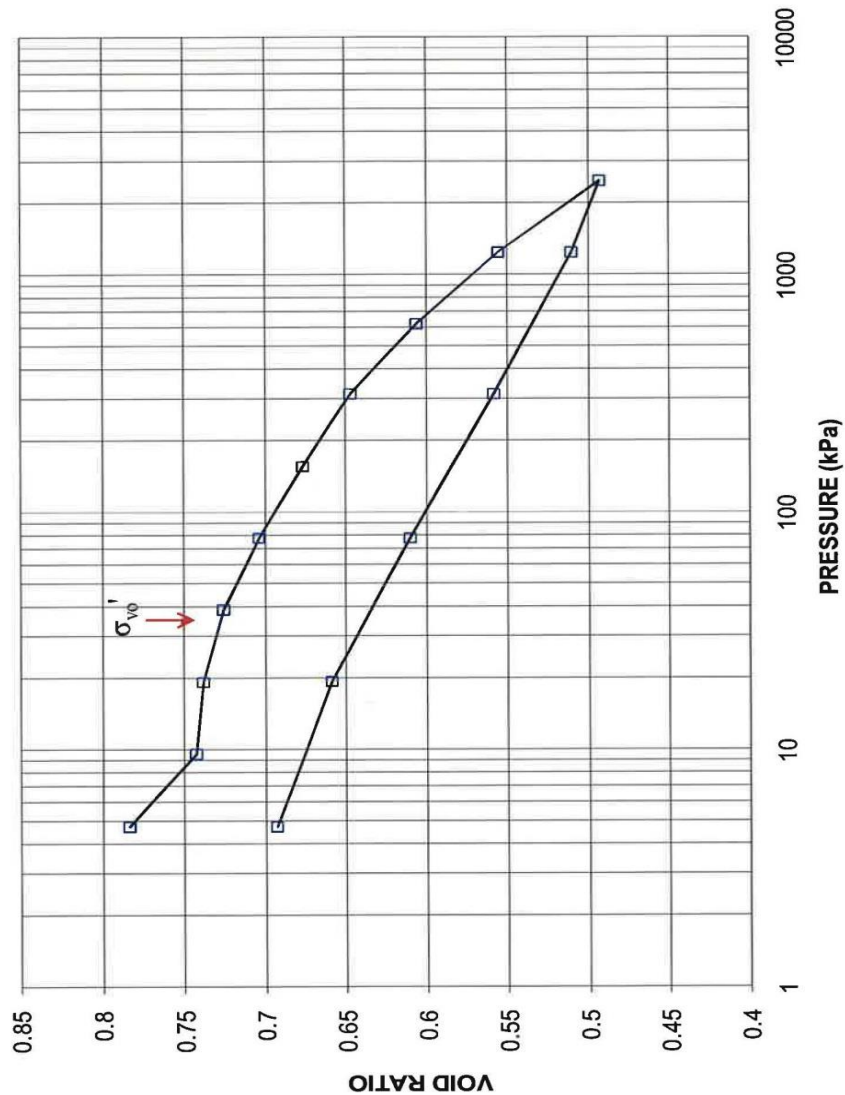
CONSOLIDATION TEST

FIGURE C7
Sheet (3 of 4)

CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

FIGURE C12
(Sheet 3 OF 4)

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 08-01 SA 3



Project No. 08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario"
Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



Terraprobe Inc.

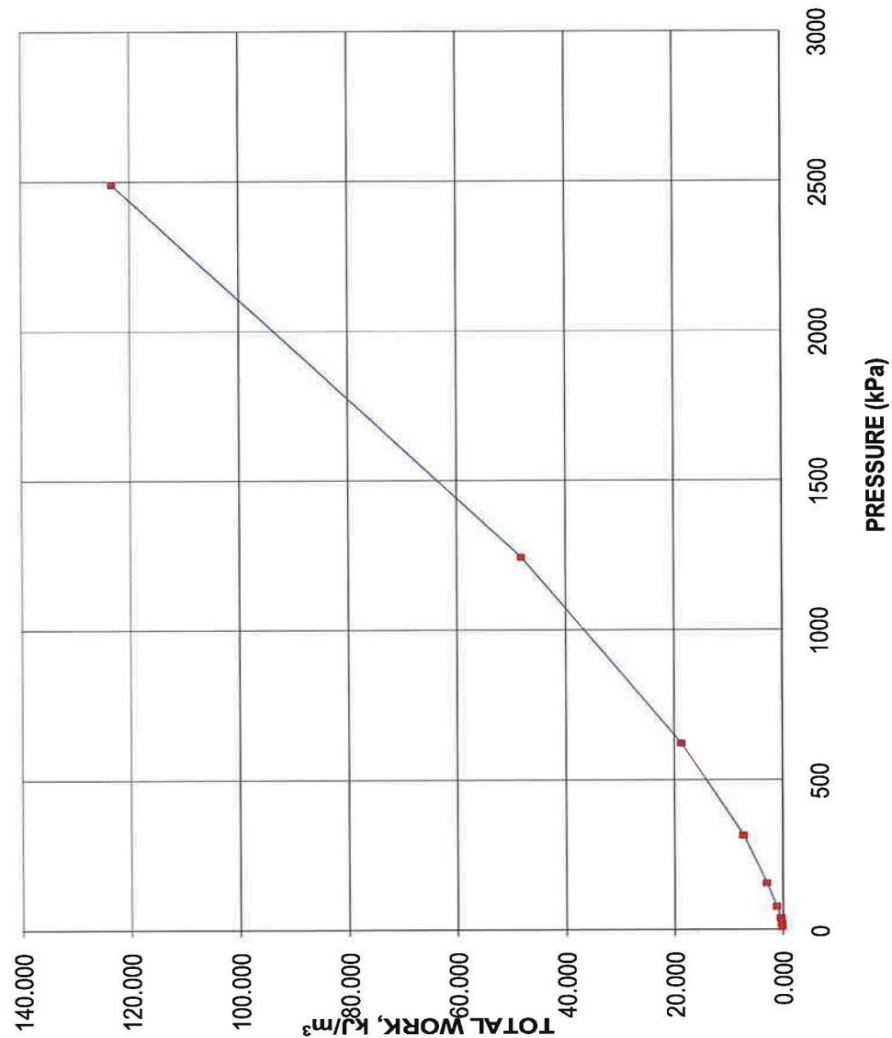
CONSOLIDATION TEST

FIGURE C7
Sheet (4 of 4)

CONSOLIDATION TEST TOTAL WORK VS. PRESSURE

FIGURE C12
(Sheet 4 of 4)

**CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs PRESSURE
BH 08-1 SA 3**



Project No. 08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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OEDOMETER CONSOLIDATION SUMMARY

FIGURE C8
Sheet (1 of 4)

OEDOMETER CONSOLIDATION SUMMARY				FIGURE C13			
Clay Fill				(Sheet 1 of 4)			
SAMPLE IDENTIFICATION							
Project Number	08-1111-0031	Sample Number	7				
Borehole Number	08-2	Sample Depth, m	6.1-6.7				
TEST CONDITIONS							
Test Type	Standard	Load Duration, hr	24				
Oedometer Number	3						
Date Started	11/12/2008						
Date Completed	11/27/2008						
SAMPLE DIMENSIONS AND PROPERTIES - INITIAL							
Sample Height, cm	2.54	Unit Weight, kN/m ³	19.35				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.15				
Area, cm ²	31.62	Specific Gravity, measured	2.80				
Volume, cm ³	80.28	Solids Height, cm	1.401				
Water Content, %	27.67	Volume of Solids, cm ³	44.30				
Wet Mass, g	158.38	Volume of Voids, cm ³	35.98				
Dry Mass, g	124.05	Degree of Saturation, %	95.4				
TEST COMPUTATIONS							
Pressure kPa	Corr. Height cm	Void Ratio	Average Height cm	t ₉₀ sec	cv, cm ² /s	mv m ² /kN	k cm/s
0.00	2.539	0.812	2.539				
4.76	2.536	0.810	2.537	2	6.82E-01	2.57E-04	1.72E-05
9.56	2.536	0.810	2.536	2	6.82E-01	8.21E-06	5.48E-07
19.52	2.545	0.816	2.540	2	6.84E-01	-3.64E-04	-2.44E-05
38.75	2.545	0.816	2.545	16	8.58E-02	6.14E-06	5.17E-08
77.55	2.530	0.806	2.537	18	7.58E-02	1.49E-04	1.11E-06
154.89	2.494	0.780	2.512	24	5.57E-02	1.83E-04	1.00E-06
309.65	2.447	0.746	2.471	12	1.08E-01	1.20E-04	1.26E-06
619.21	2.386	0.703	2.417	41	3.02E-02	7.76E-05	2.30E-07
1239.25	2.305	0.645	2.346	89	1.31E-02	5.15E-05	6.61E-08
2477.06	2.207	0.575	2.256	128	8.43E-03	3.12E-05	2.58E-08
1239.25	2.227	0.589	2.217				
309.65	2.297	0.639	2.262				
77.55	2.375	0.695	2.336				
19.52	2.453	0.751	2.414				
4.76	2.505	0.788	2.479				
Note: k calculated using cv based on t ₉₀ values. Sample swelled under 38.75kPa							
SAMPLE DIMENSIONS AND PROPERTIES - FINAL							
Sample Height, cm	2.51	Unit Weight, kN/m ³	19.87				
Sample Diameter, cm	6.35	Dry Unit Weight, kN/m ³	15.36				
Area, cm ²	31.62	Specific Gravity, measured	2.80				
Volume, cm ³	79.21	Solids Height, cm	1.401				
Water Content, %	29.40	Volume of Solids, cm ³	44.30				
Wet Mass, g	160.52	Volume of Voids, cm ³	34.90				
Dry Mass, g	124.05						
Prepared By: LFG				Golder Associates		Checked By: MM	

Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario"
Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



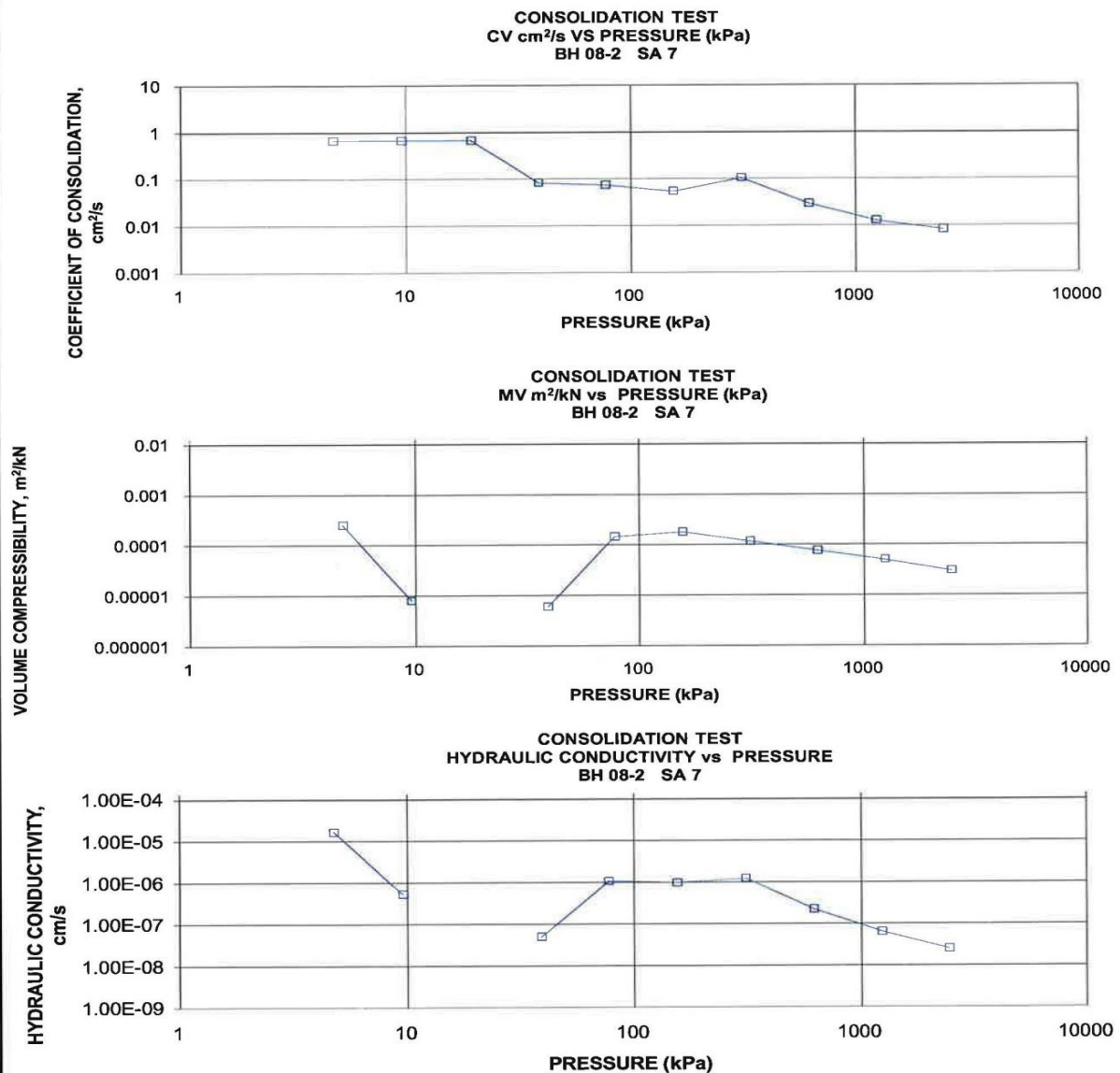
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OEDOMETER CONSOLIDATION SUMMARY

FIGURE C8
Sheet (2 of 4)

OEDOMETER CONSOLIDATION SUMMARY Clay Fill

FIGURE C13
(Sheet 2 of 4)



Project No. 08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario"
Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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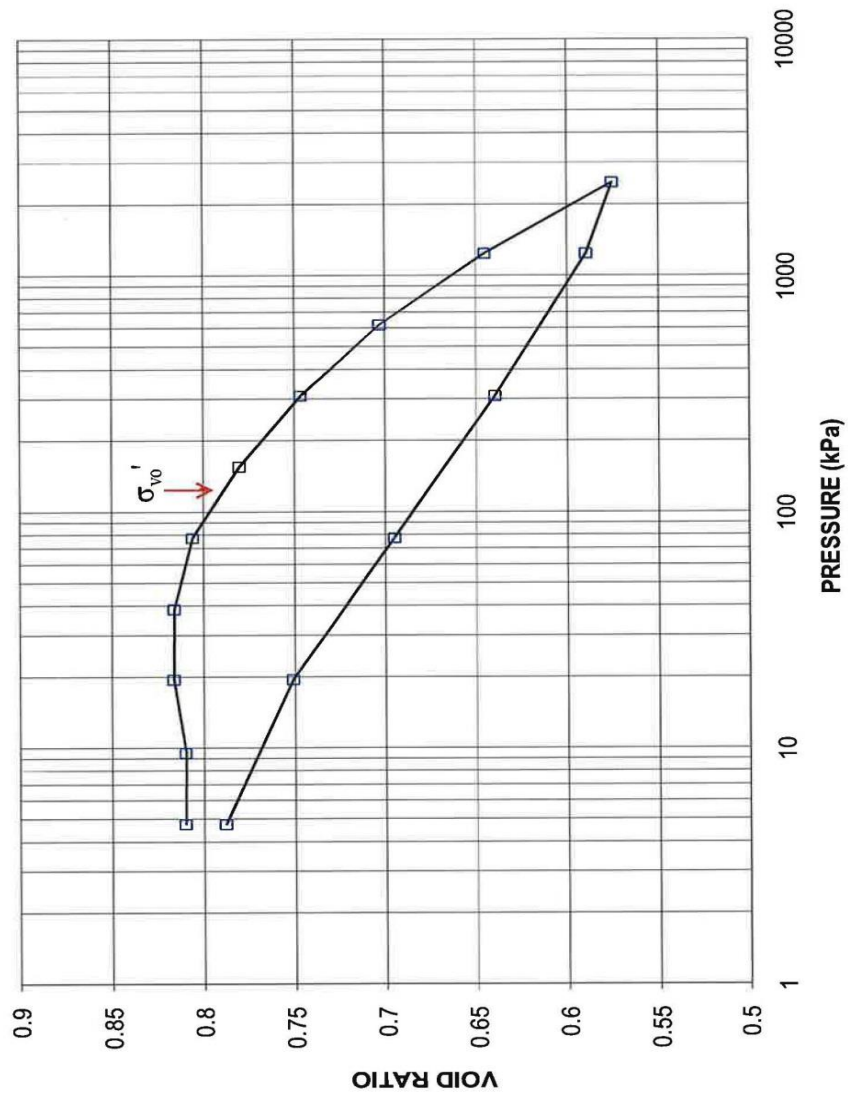
CONSOLIDATION TEST

FIGURE C8
Sheet (3 of 4)

CONSOLIDATION TEST VOID RATIO VS. LOG PRESSURE

FIGURE C13
(SHEET 3 OF 4)

CONSOLIDATION TEST
VOID RATIO vs PRESSURE
BH 08-02 SA 7



Project No.08-1111-0031

Prepared By: LFG

Golder Associates

Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario"
Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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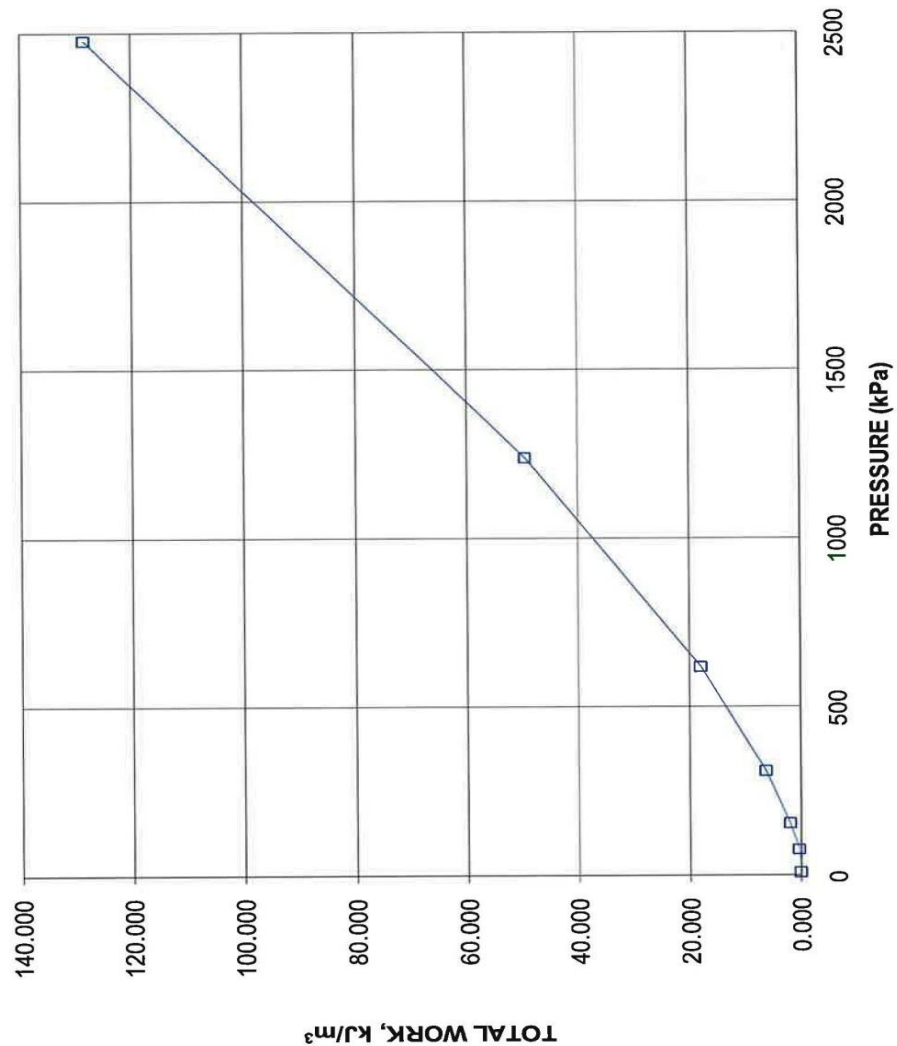
CONSOLIDATION TEST

FIGURE C8
Sheet (4 of 4)

CONSOLIDATION TEST TOTAL WORK VS. PRESSURE

FIGURE C13
(SHEET 4 OF 4)

CONSOLIDATION TEST
TOTAL WORK, kJ/m³ vs PRESSURE
BH 08-02 SA 7



Project No.08-1111-0031

Prepared By: LFG

Golder Associates

Checked By: MM

Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario"
Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

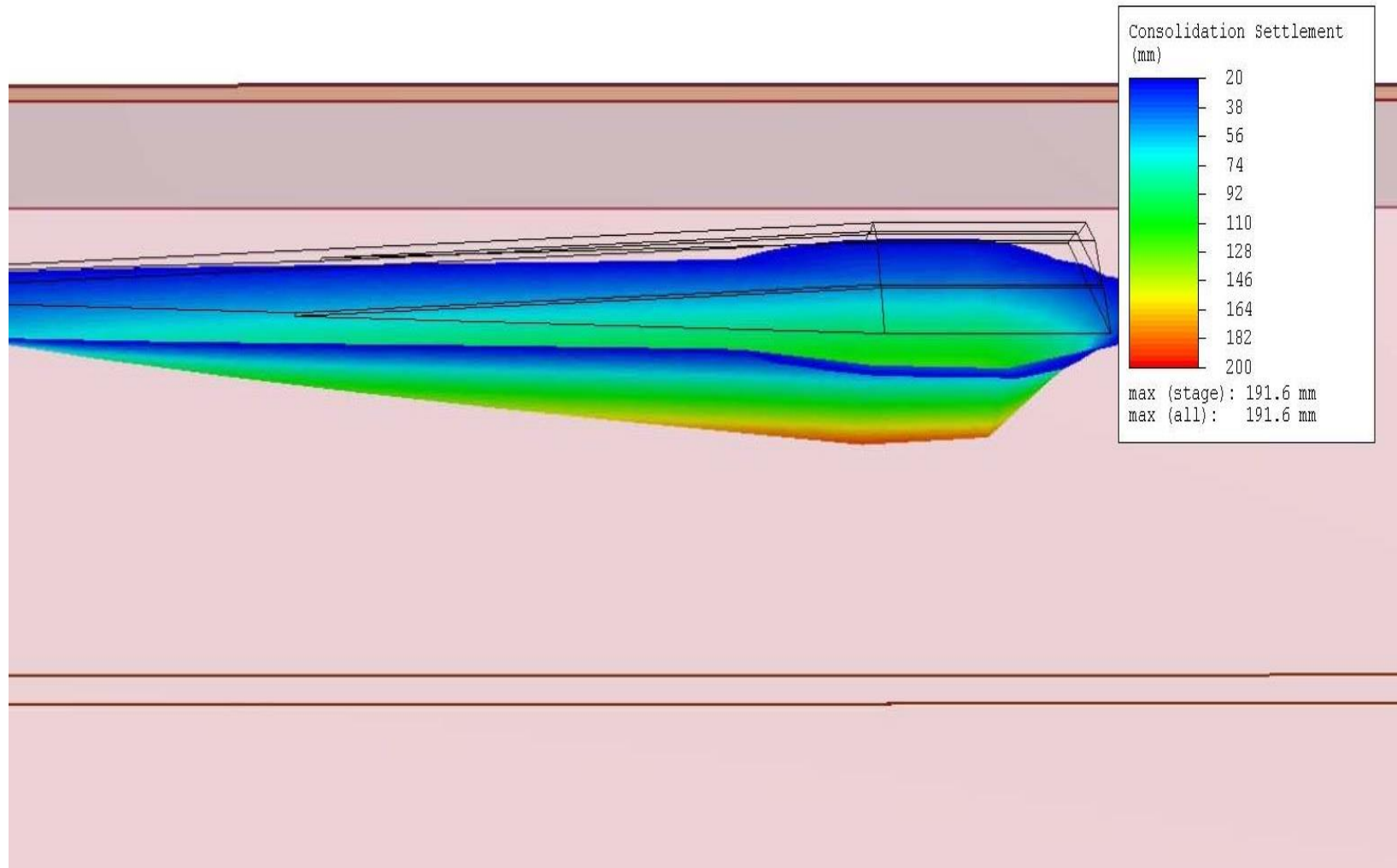
Date : August, 2014



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PRIMARY CONSOLIDATION SETTLEMENT

FIGURE C9
Sheet (1 of 2)



Project No. : 11-14-4076

Date : August, 2014



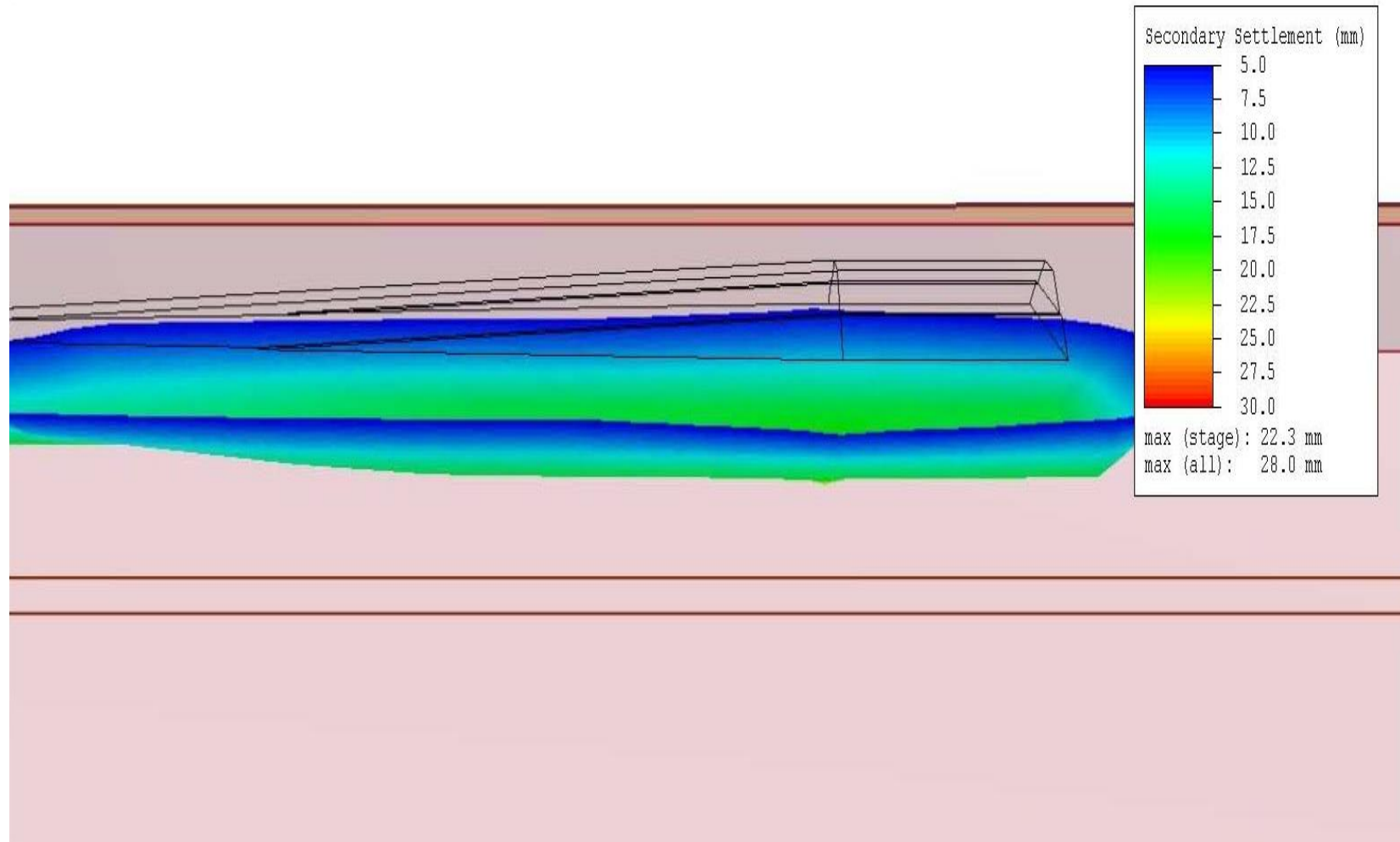
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Designed by : AA

Checked by : RA

SECONDARY CONSOLIDATION SETTLEMENT

FIGURE C9
Sheet (2 of 2)



Project No. : 11-14-4076

Date : August, 2014



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Designed by : AA

Checked by : RA

APPENDIX D

Data Interpretation & Analysis (Stability)



FIGURE D1
Sheet (1 of 3)

CONSOLIDATED DRAINED DIRECT SHEAR TEST		FIGURE C5 (Sheet 1 of 3)	
Undisturbed Clay Fill			
TEST STAGE	A	B	C
TEST PIT NUMBER	8	8	8
SAMPLE NUMBER	2	2	2
SAMPLE DEPTH, (m)	0.7-0.9	0.7-0.9	0.7-0.9
SAMPLE HEIGHT, (mm)	25	25	25
SAMPLE LENGTH, (mm)	60	60	60
WATER CONTENT, BEFORE TEST, (%)	27.4	26.2	26.0
NORMAL (CONSOLIDATION) STRESS, (kPa)	10	20	40
WATER CONTENT, AFTER TEST, (%)	33.1	31.0	30.2
DISPLACEMENT RATE, mm/min	0.0048	0.0048	0.0048
TIME TO FAILURE, min	82	325	350
PEAK SHEAR STRESS, (kPa)	12.24	18.67	28.95
HORIZONTAL DISPLACEMENT AT PEAK, (mm)	0.39	1.56	1.68
RESIDUAL SHEAR STRESS, (kPa)	8.64	14.36	21.80
HORIZONTAL DISPLACEMENT AT RESIDUAL, (mm)	14.03	11.49	8.98
DRY DENSITY, initial, Mg/m ³	1.58	1.60	1.60
WET DENSITY, initial, Mg/m ³	2.01	2.02	2.02
TEST NOTES:			
Date:	11/20/2008	Prepared By:	LFG
Project No.	08-1111-0031	Checked By:	MM
Golder Associates			

Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

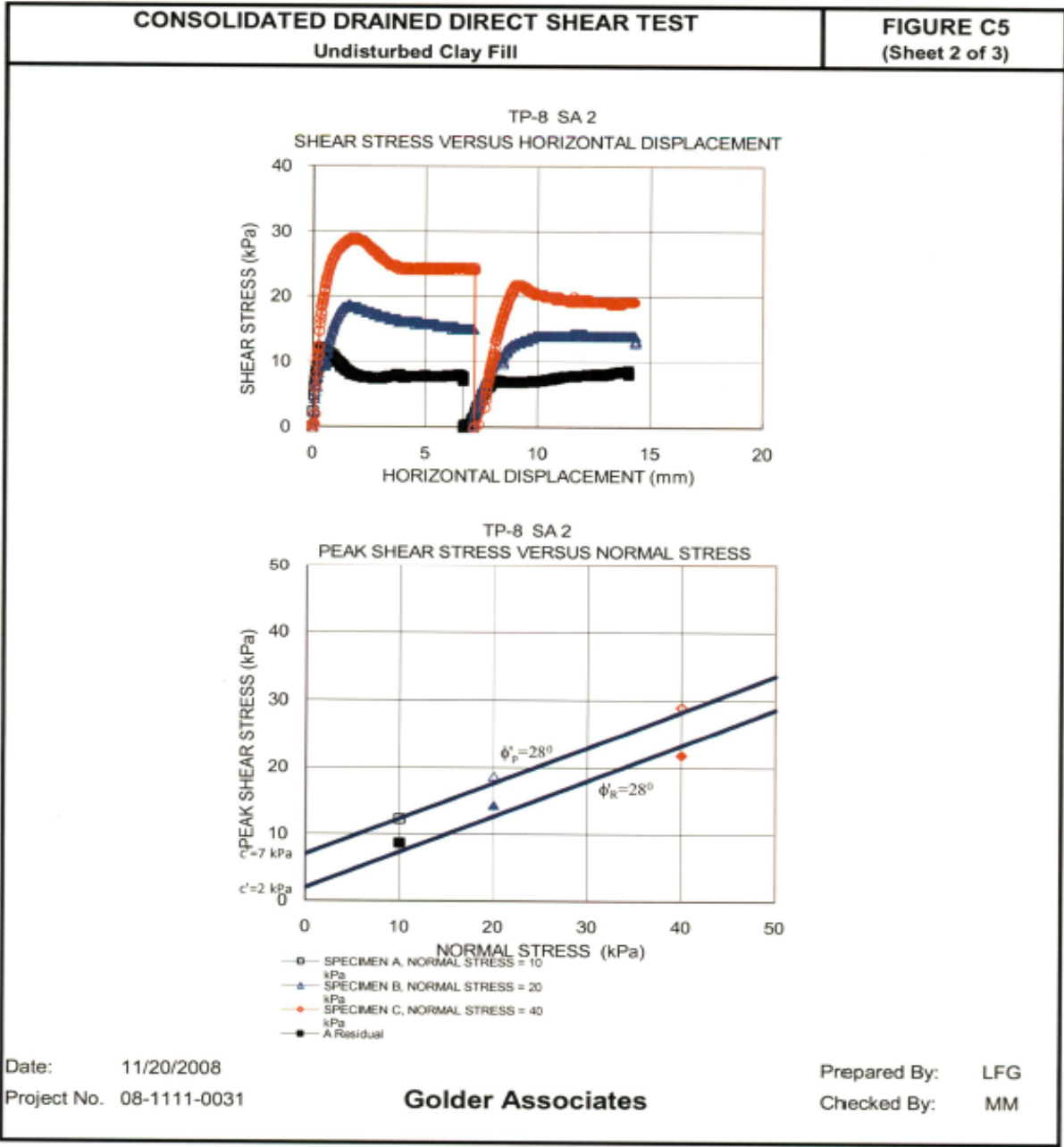
Date : August, 2014



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CONSOLIDATED DRAINED DIRECT SHEAR TEST

FIGURE D1
Sheet (2 of 3)



Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

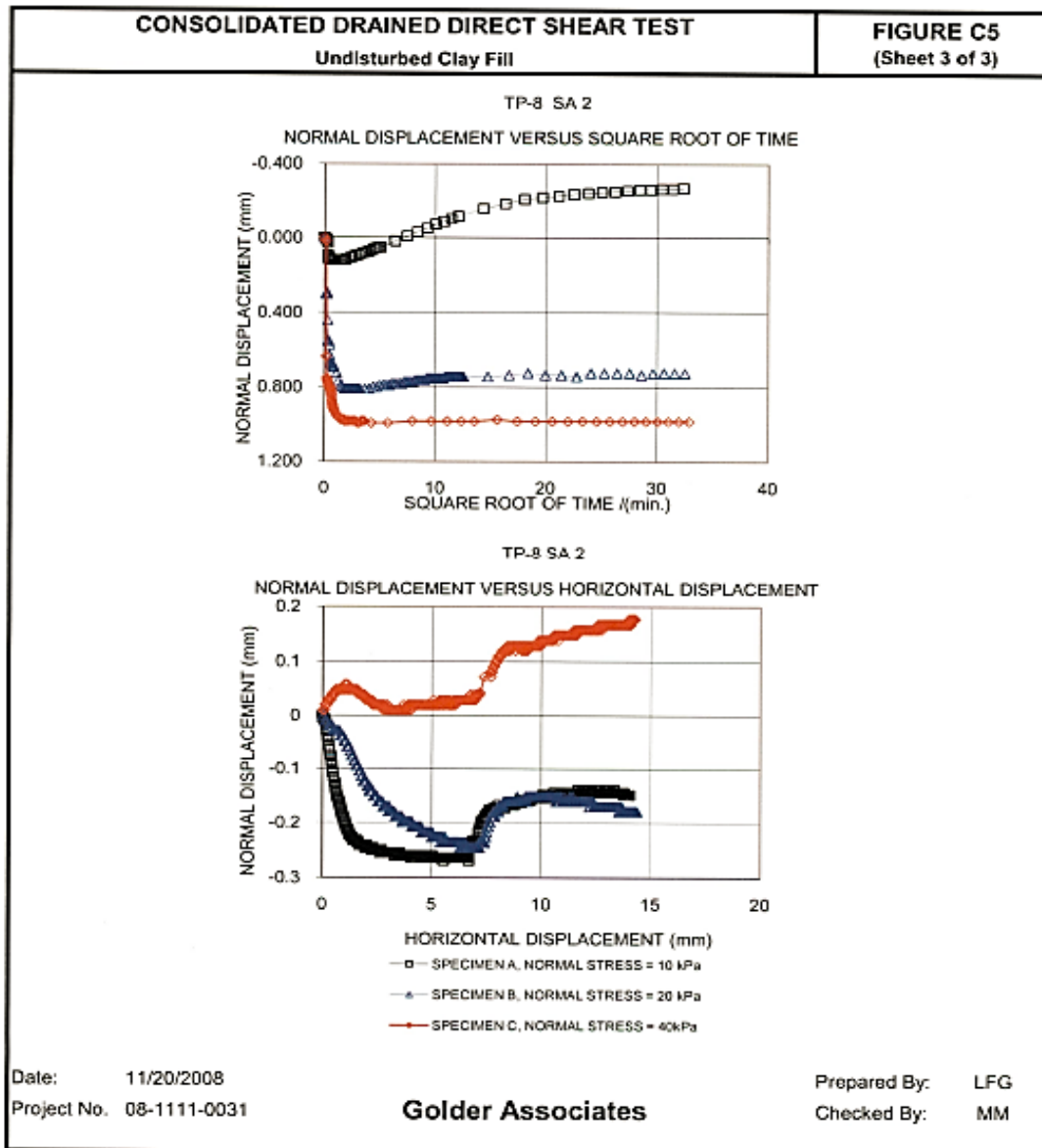
Date : August, 2014



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CONSOLIDATED DRAINED DIRECT SHEAR TEST

FIGURE D1
Sheet (3 of 3)



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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FIGURE D2
Sheet (1 of 3)

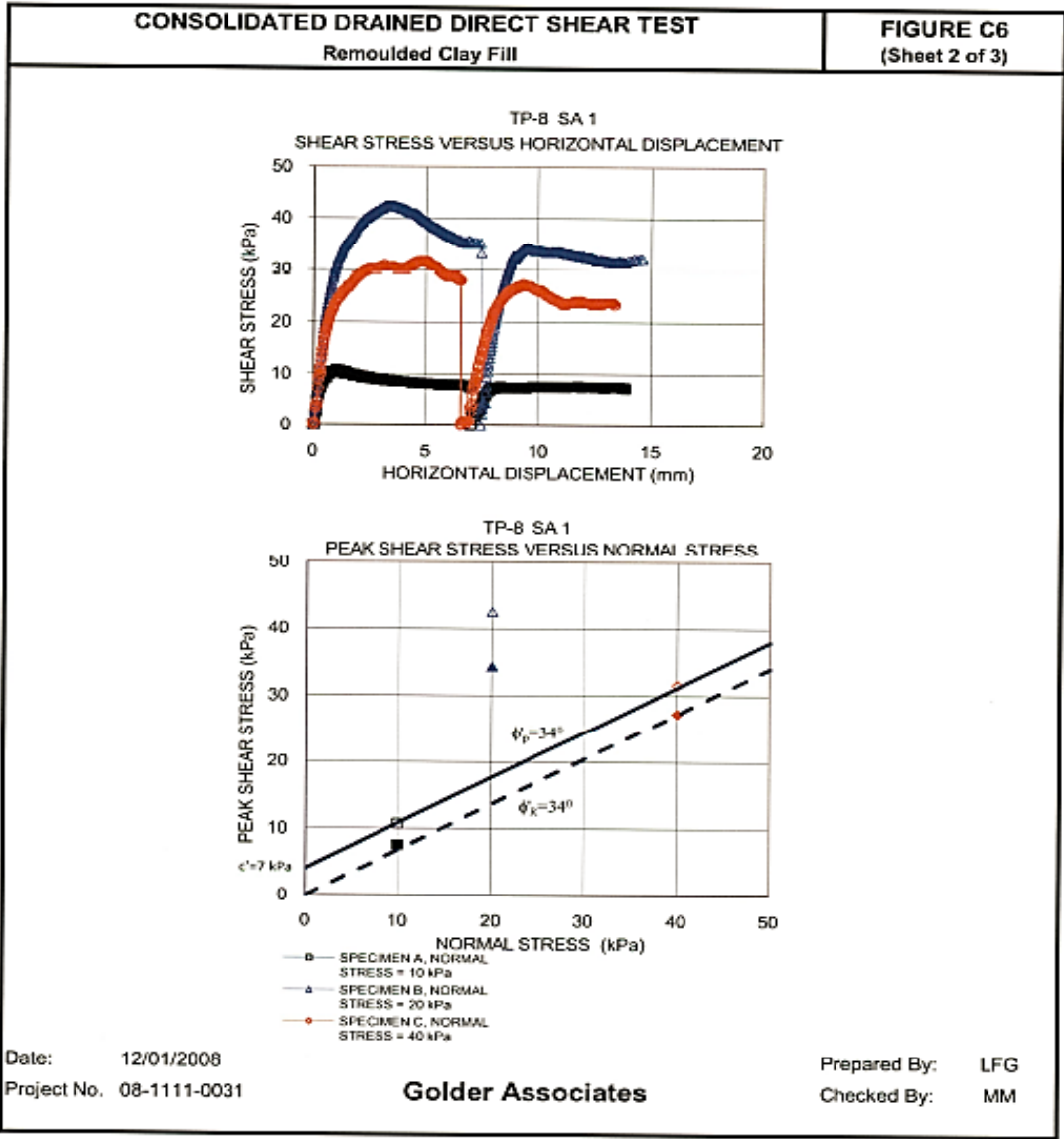
Reference	<p>Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.</p>
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CONSOLIDATED DRAINED DIRECT SHEAR TEST

FIGURE D2
Sheet (2 of 3)



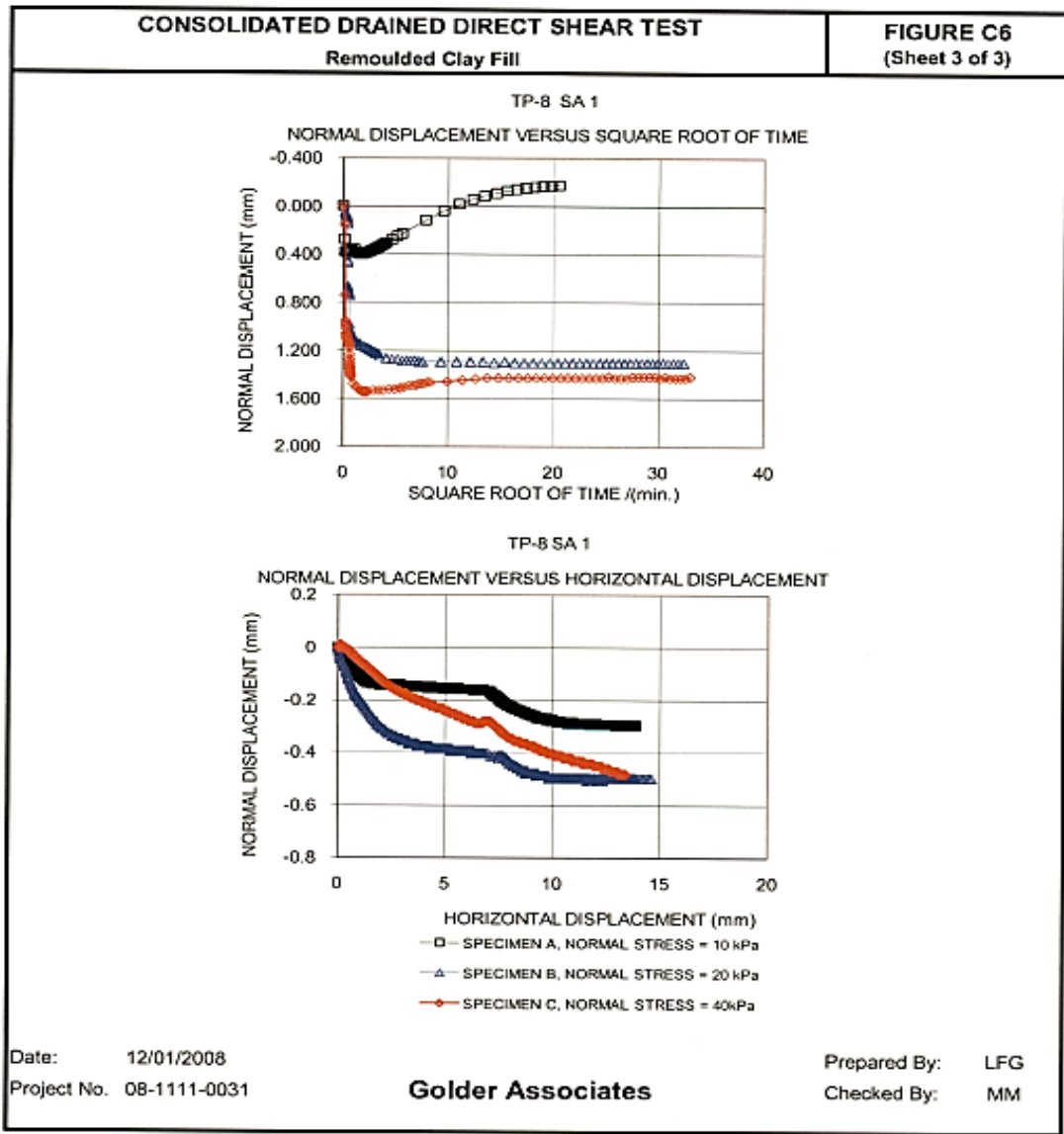
Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

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



CONSOLIDATED DRAINED DIRECT SHEAR TEST

FIGURE D2
Sheet (3 of 3)



Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

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



CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D3
Sheet (1 of 4)

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill		FIGURE C7 (Sheet 1 of 4)	
TEST STAGE	A	B	C
BOREHOLE NUMBER	08-2	08-2	08-2
SAMPLE	7	7	7
SPECIMEN DIAMETER, cm	5.01	4.99	4.99
SPECIMEN HEIGHT, cm	10.14	10.16	10.15
WATER CONTENT BEFORE CONSOLIDATION, %	31.4	30.8	28.2
CELL PRESSURE, σ_3 , kPa	645.0	395.0	010.0
BACK PRESSURE, kPa	625.0	345.0	485.0
PORE PRESSURE PARAMETER "B"	0.96	0.99	0.99
CONSOLIDATION PRESSURE, σ_c , kPa	20.0	50.0	125.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	0.6	6.3	5.8
WATER CONTENT AFTER CONSOLIDATION, %	31.1	27.0	24.7
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5	0.5
TIME TO FAILURE, DAYS	1	1	1
WATER CONTENT AFTER TEST, %	28.5	29.5	26.6
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	51.1	53.0	192.4
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	17.8	19.4	5.1
MAX EFFECTIVE PRINCIPAL STRESS			
RATIO, (σ_1 / σ_3) MAXIMUM	3.8	2.6	2.9
DEVIATOR STRESS AT (σ_1 / σ_3) MAXIMUM, kPa	36.8	39.7	181.0
AXIAL STRAIN AT (σ_1 / σ_3) MAXIMUM, %	1.6	3.3	4.0
PORE PRESSURE PARAMETER, A_f , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	-0.20	0.23	0.11
PORE PRESSURE PARAMETER, A_f , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	0.18	0.63	0.17
NATURAL WATER CONTENT, %	23.8	24.1	22.6
DRY DENSITY, Mg/m ³	1.66	1.66	1.69
FILTER DRAINS USED, y/n	y	y	y
TEST NOTES:			
CHANGED RATE OF STRAIN, %/hr	-	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-	-
FAILURE PLANE NUMBER	1.0	1.0	1.0
ANGLE OF FAILURE, DEGREES	65.0	55.0	65.0
<div> <div>Date: 02/27/2009</div> <div>Project No. 08-1111-0031</div> </div> <div align="center"> Golder Associates </div> <div> <div>Prepared By: MM</div> <div>Checked By: MK</div> </div>			

Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

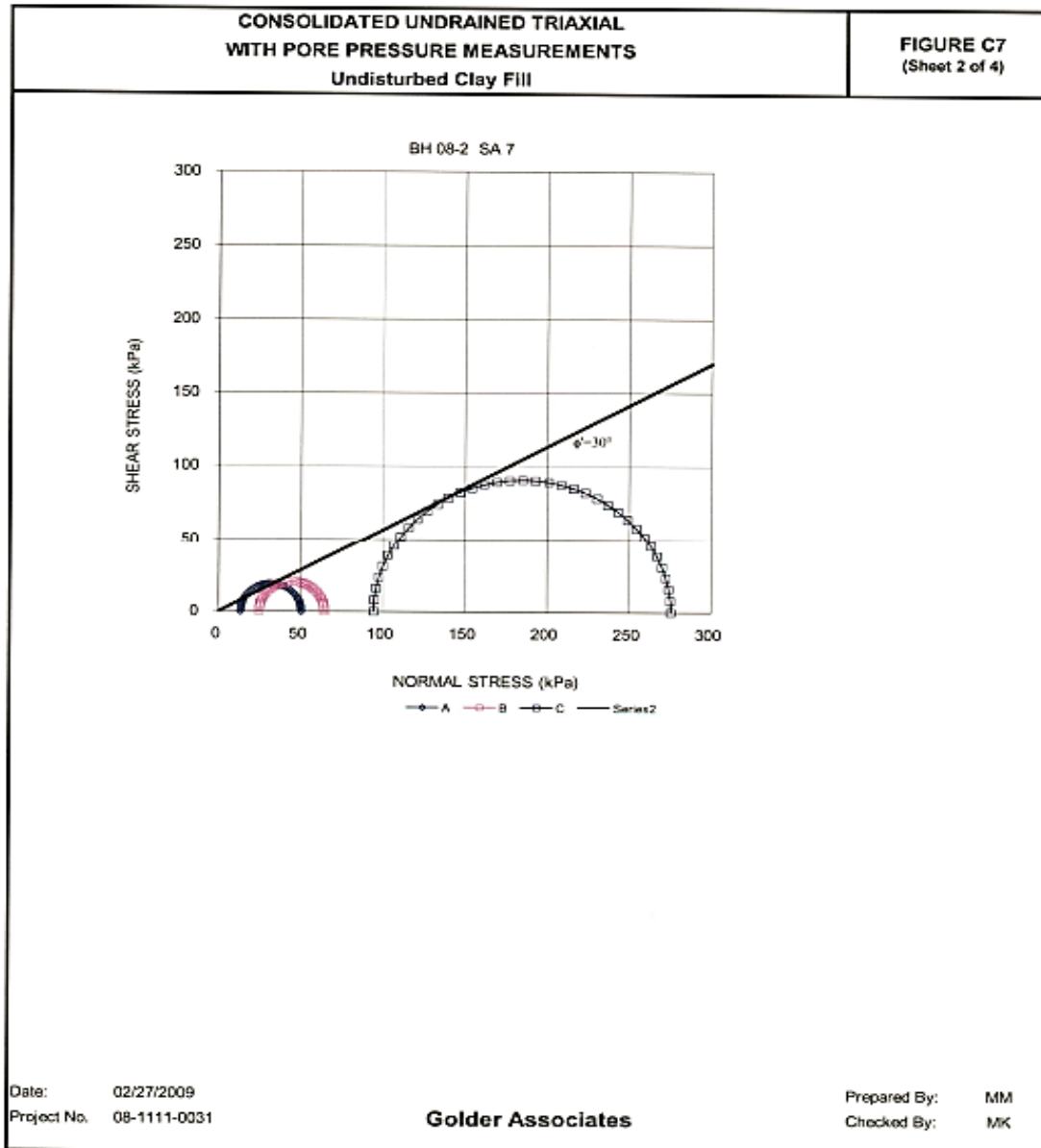
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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D3
Sheet (2 of 4)



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

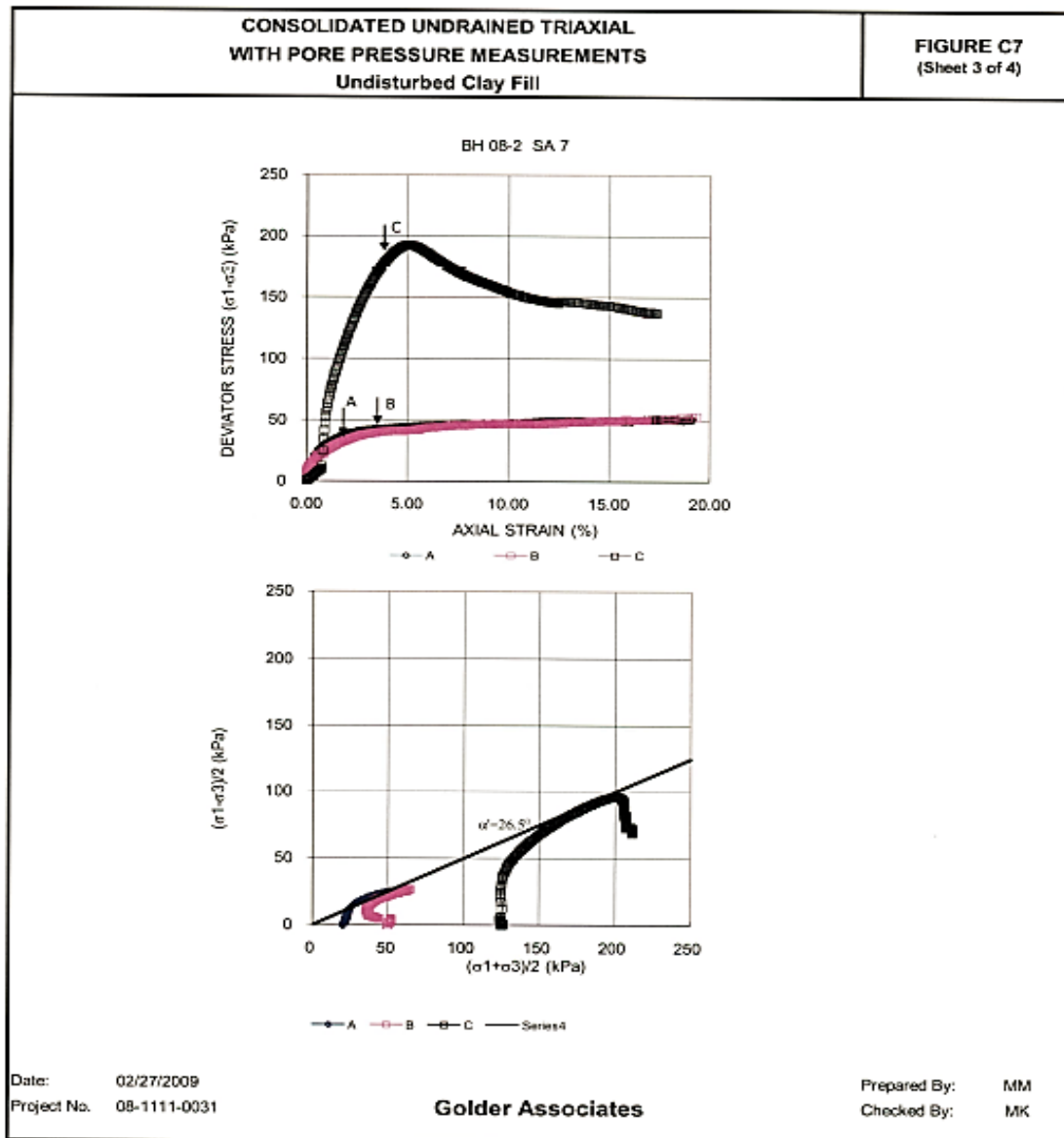
Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D3
Sheet (3 of 4)



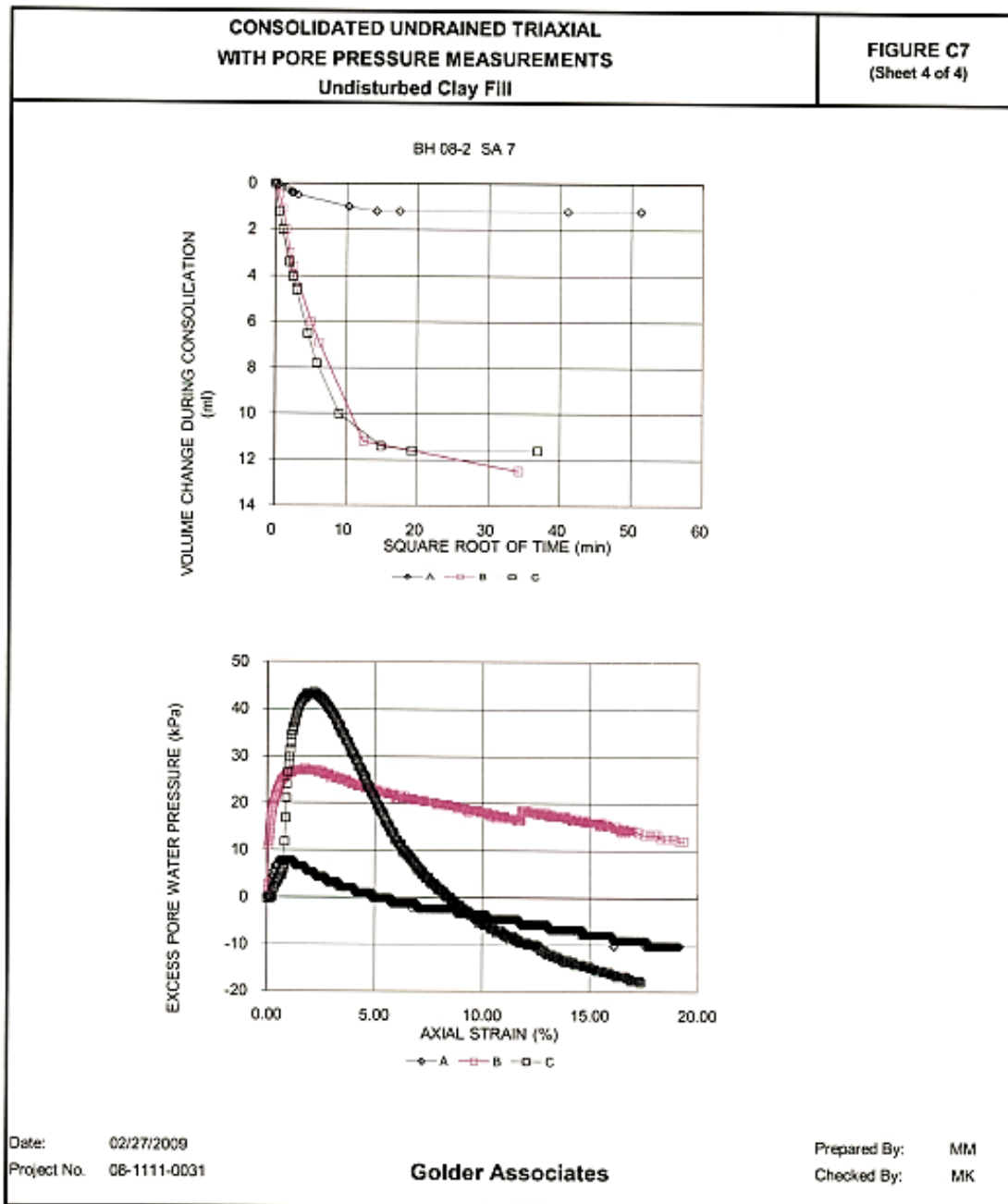
Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

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



CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D3
Sheet (4 of 4)



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D4
Sheet (1 of 4)

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill		FIGURE C8 (Sheet 1 of 4)
TEST STAGE	A	B
TEST PIT NUMBER	3	3
SAMPLE	3	3
SPECIMEN DIAMETER, cm	5.09	5.00
SPECIMEN HEIGHT, cm	10.17	10.17
WATER CONTENT BEFORE CONSOLIDATION, %	29.8	32.8
CELL PRESSURE, σ_3 , kPa	220.0	165.0
BACK PRESSURE, kPa	205.0	135.0
PORE PRESSURE PARAMETER "B"	0.99	0.96
CONSOLIDATION PRESSURE, σ_c , kPa	15.0	30.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	1.4	3.1
WATER CONTENT AFTER CONSOLIDATION, %	28.9	30.7
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5
TIME TO FAILURE, DAYS	1	1
WATER CONTENT AFTER TEST, %	30.8	30.6
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	52.6	57.8
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	12.2	10.8
MAX EFFECTIVE PRINCIPAL STRESS		
RATIO, (σ_1 / σ_3) MAXIMUM	5.4	3.7
DEVIATOR STRESS AT (σ_1 / σ_3) MAXIMUM, kPa	26.3	50.1
AXIAL STRAIN AT (σ_1 / σ_3) MAXIMUM, %	2.4	4.5
PORE PRESSURE PARAMETER, Af, AT $(\sigma_1 - \sigma_3)$ MAXIMUM	-0.06	0.06
PORE PRESSURE PARAMETER, Af, AT (σ_1 / σ_3) MAXIMUM	0.34	0.22
NATURAL WATER CONTENT, %	28.8	31.8
DRY DENSITY, Mg/m ³	1.53	1.46
FILTER DRAINS USED, y/n	y	y
TEST NOTES:		
CHANGED RATE OF STRAIN, %/hr	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-
FAILURE PLANE NUMBER	1.0	1.0
ANGLE OF FAILURE, DEGREES	55.0	65.0
Date: 12/29/2008 Project No. 08-1111-0031 <div align="center">Golder Associates</div>		Prepared By: MM Checked By: RO

Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

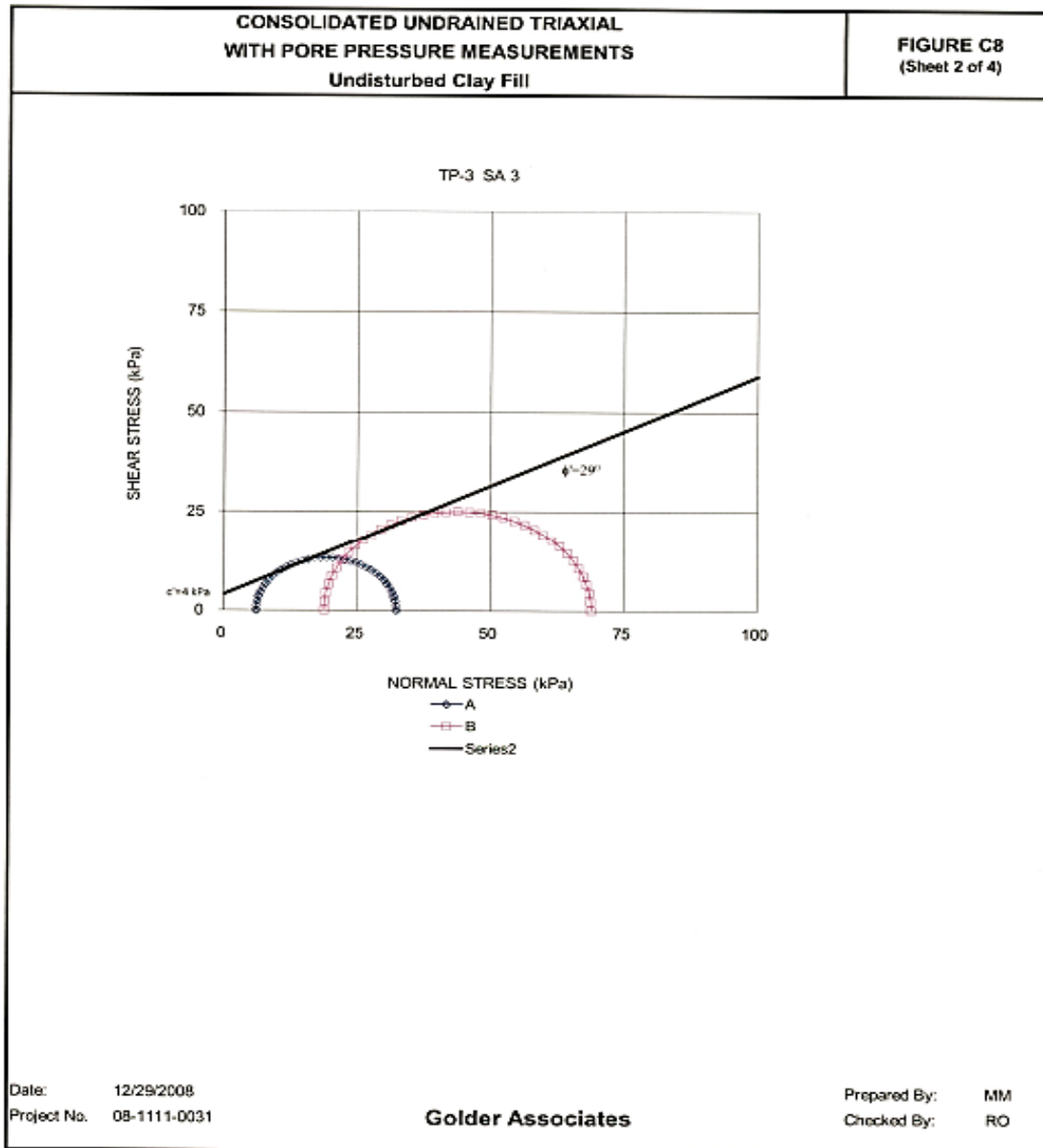
Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D4
Sheet (2 of 4)



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

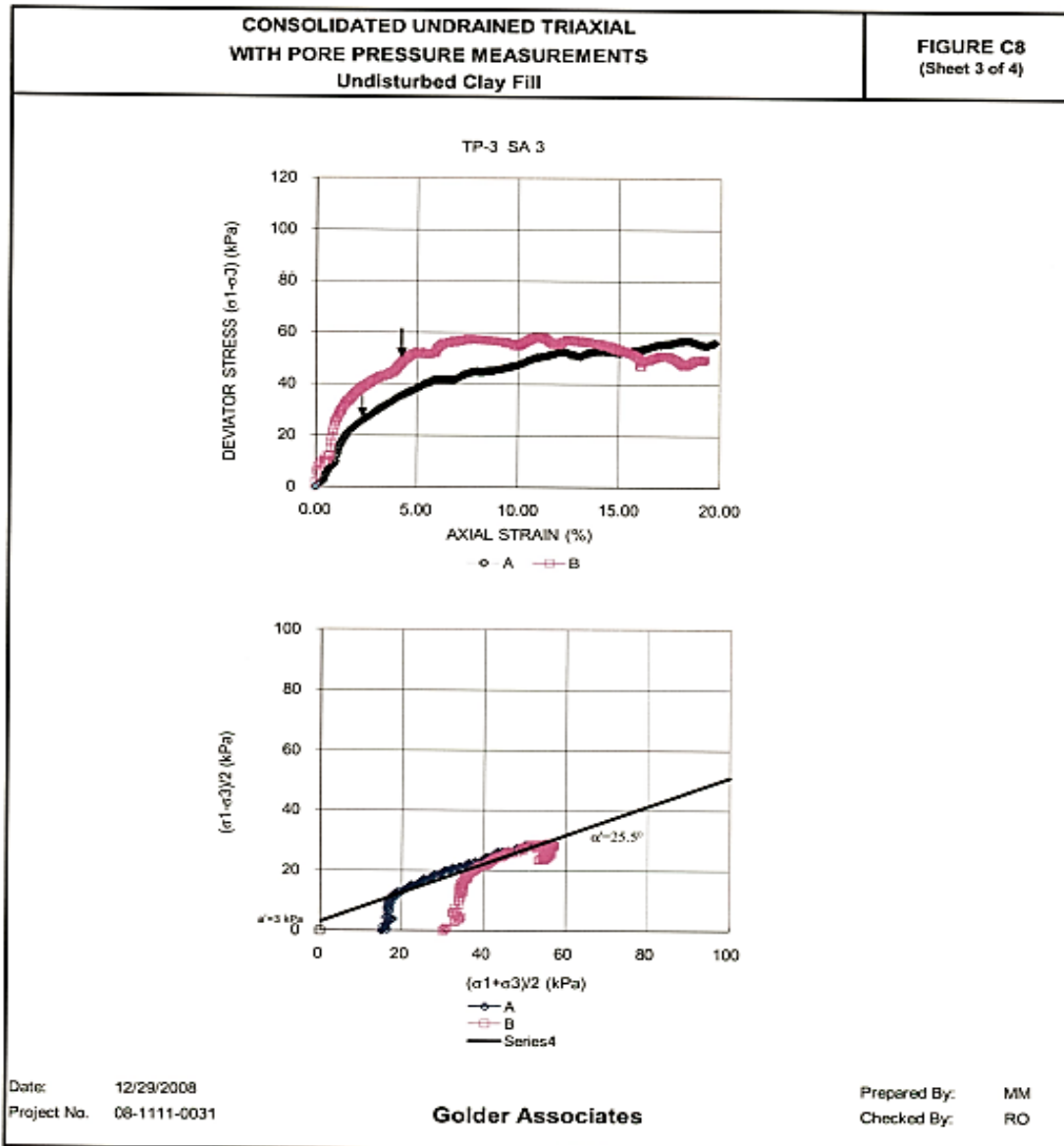
Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D4
Sheet (3 of 4)



Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

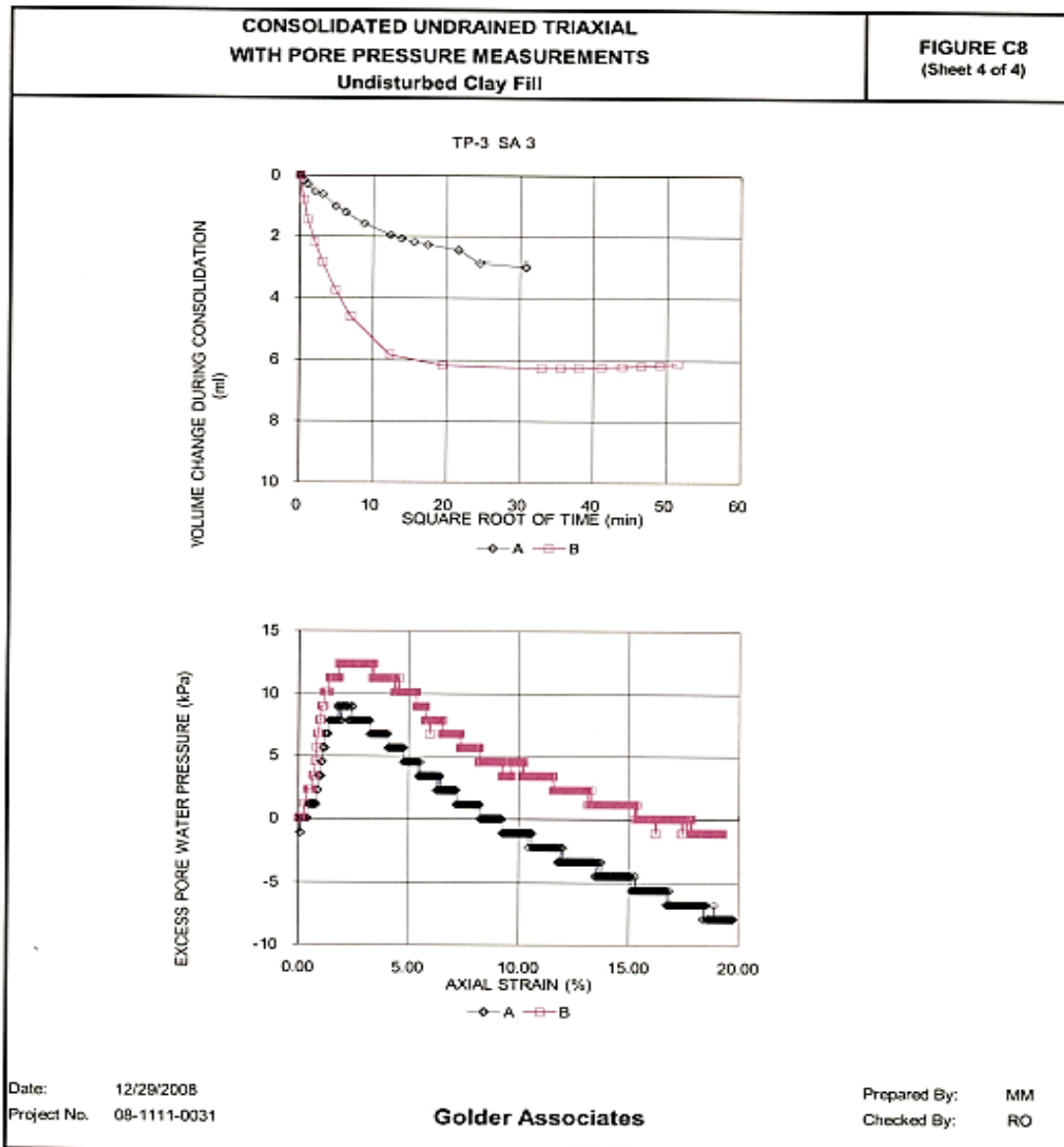
Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D4
Sheet (4 of 4)



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D5
Sheet (1 of 4)

CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS Undisturbed Clay Fill		FIGURE C9 (Sheet 1 of 4)
TEST STAGE	A	B
BOREHOLE NUMBER	08-3	08-3
SAMPLE	3	3
SPECIMEN DIAMETER, cm	5.03	5.00
SPECIMEN HEIGHT, cm	10.15	10.10
WATER CONTENT BEFORE CONSOLIDATION, %	30.7	31.6
CELL PRESSURE, σ_3 , kPa	355.0	370.0
BACK PRESSURE, kPa	345.0	345.0
PORE PRESSURE PARAMETER "B"	0.99	0.96
CONSOLIDATION PRESSURE, σ_c , kPa	10.0	25.0
VOLUMETRIC STRAIN DURING CONSOLIDATION, %	0.5	2.4
WATER CONTENT AFTER CONSOLIDATION, %	30.4	30.1
AVERAGE RATE OF STRAIN, %/hr	0.5	0.5
TIME TO FAILURE, DAYS	1	1
WATER CONTENT AFTER TEST, %	29.7	29.4
MAX. DEVIATOR STRESS, $(\sigma_1 - \sigma_3)$, kPa	49.7	51.9
AXIAL STRAIN AT $(\sigma_1 - \sigma_3)$ MAXIMUM, %	14.2	6.2
MAX EFFECTIVE PRINCIPAL STRESS		
RATIO, (σ_1 / σ_3) MAXIMUM	6.5	3.7
DEVIATOR STRESS AT (σ_1 / σ_3) MAXIMUM, kPa	18.2	42.8
AXIAL STRAIN AT (σ_1 / σ_3) MAXIMUM, %	1.3	3.0
PORE PRESSURE PARAMETER, A_f , AT $(\sigma_1 - \sigma_3)$ MAXIMUM	-0.29	0.05
PORE PRESSURE PARAMETER, A_f , AT (σ_1 / σ_3) MAXIMUM	0.37	0.21
NATURAL WATER CONTENT, %	25.7	26.1
DRY DENSITY, Mg/m ³	1.58	1.59
FILTER DRAINS USED, y/n	y	y
TEST NOTES:		
CHANGED RATE OF STRAIN, %/hr	-	-
AXIAL STRAIN WHERE RATE OF STRAIN WAS CHANGED, %	-	-
FAILURE PLANE NUMBER	1.0	1.0
ANGLE OF FAILURE, DEGREES	55.0	60.0
Date: 01/25/2008		Prepared By: MM
Project No. 08-1111-0031		Checked By: RO
Golder Associates		

Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

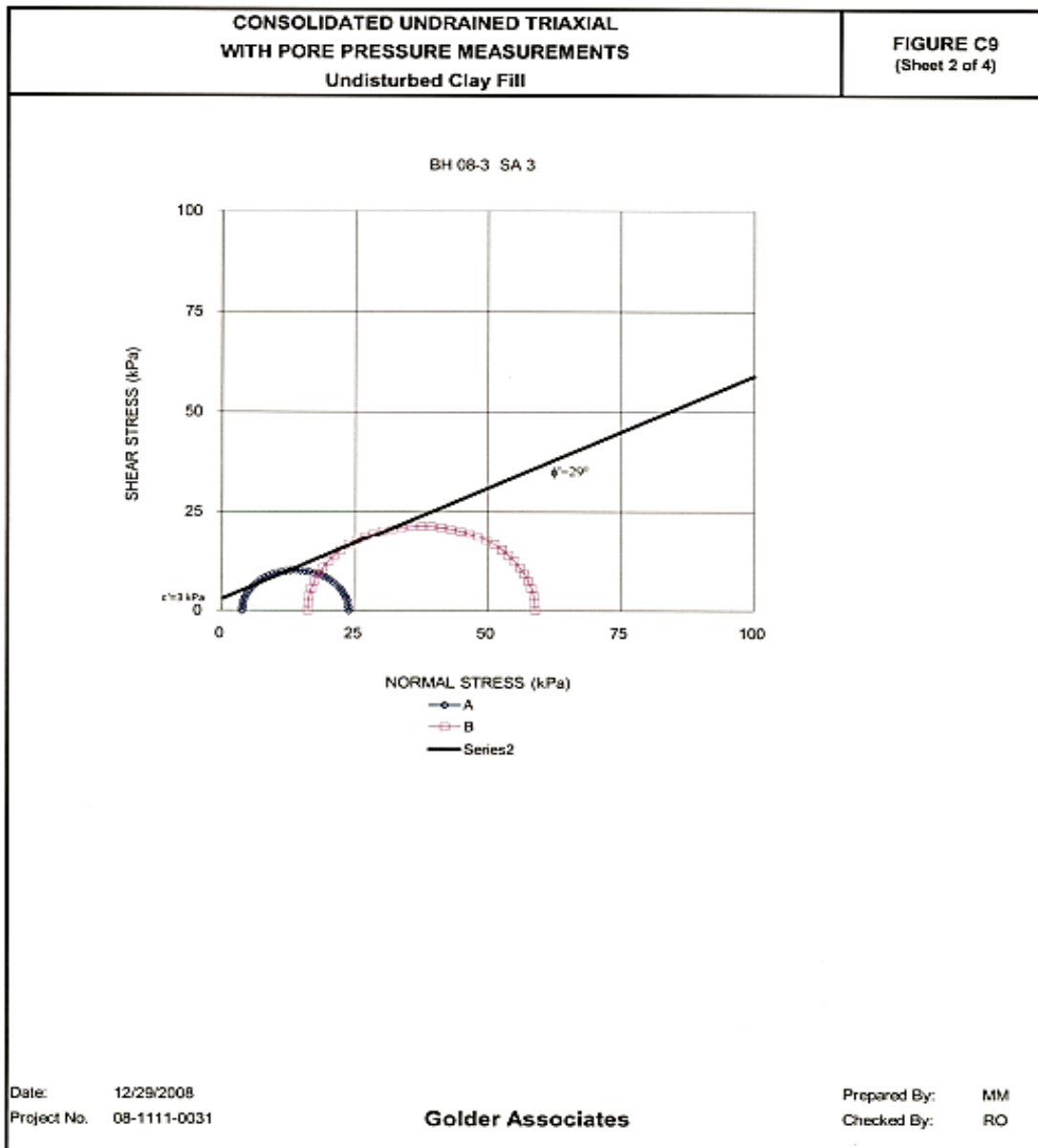
Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D5
Sheet (2 of 4)



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

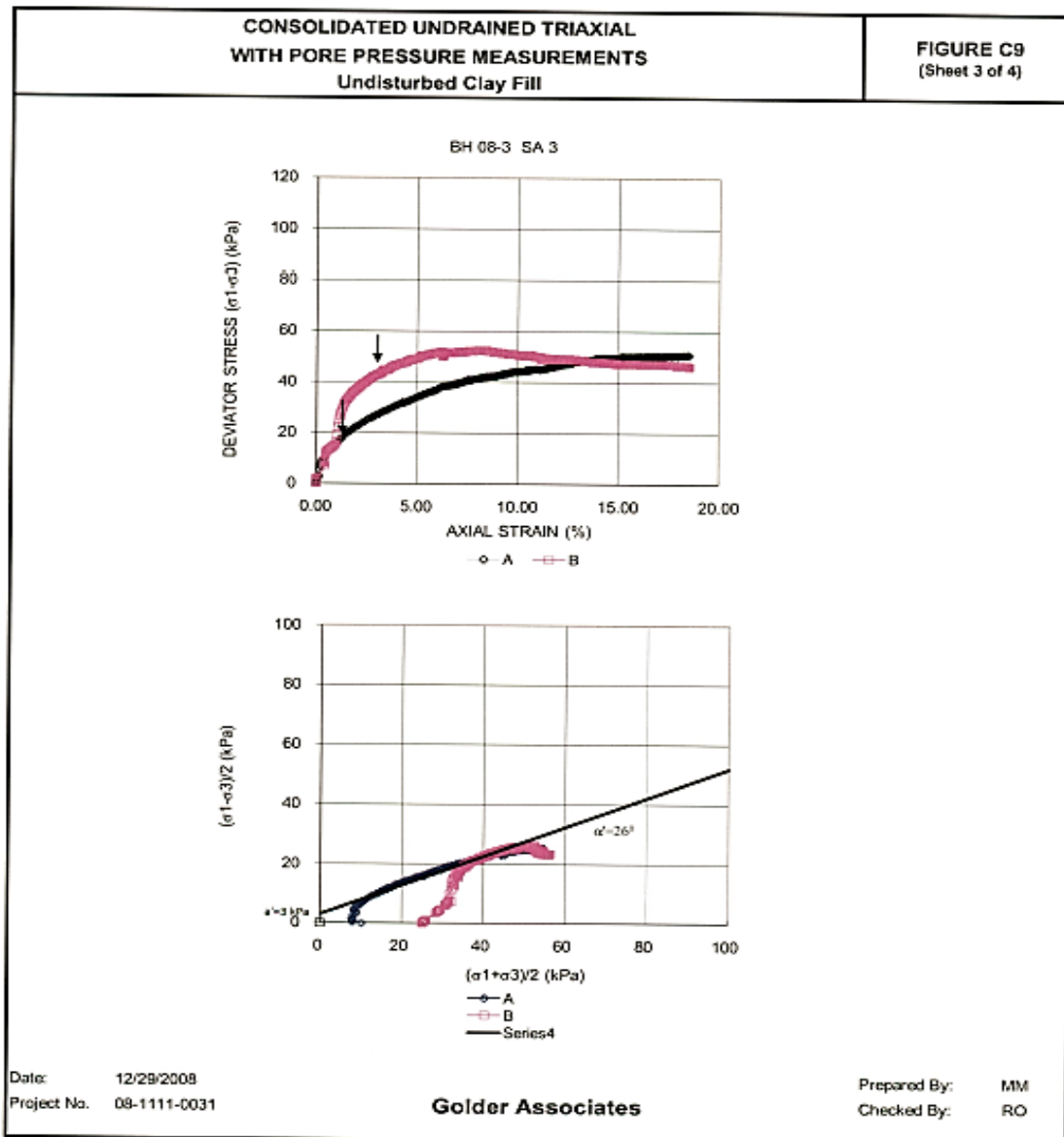
Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D5
Sheet (3 of 4)



Reference Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

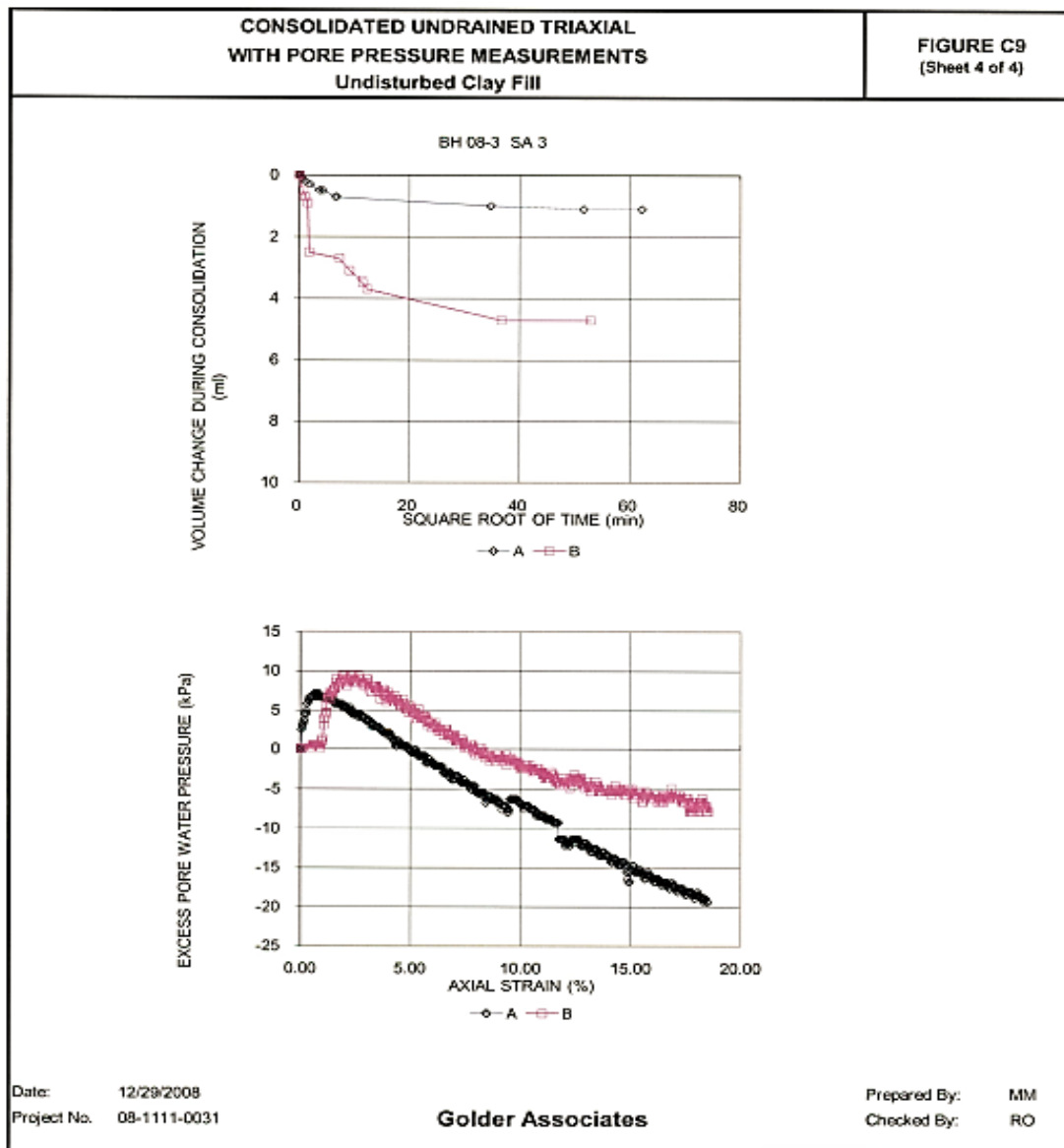
Date : August, 2014



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CONSOLIDATED UNDRAINED TRIAXIAL WITH PORE PRESSURE MEASUREMENTS

FIGURE D5
Sheet (4 of 4)



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

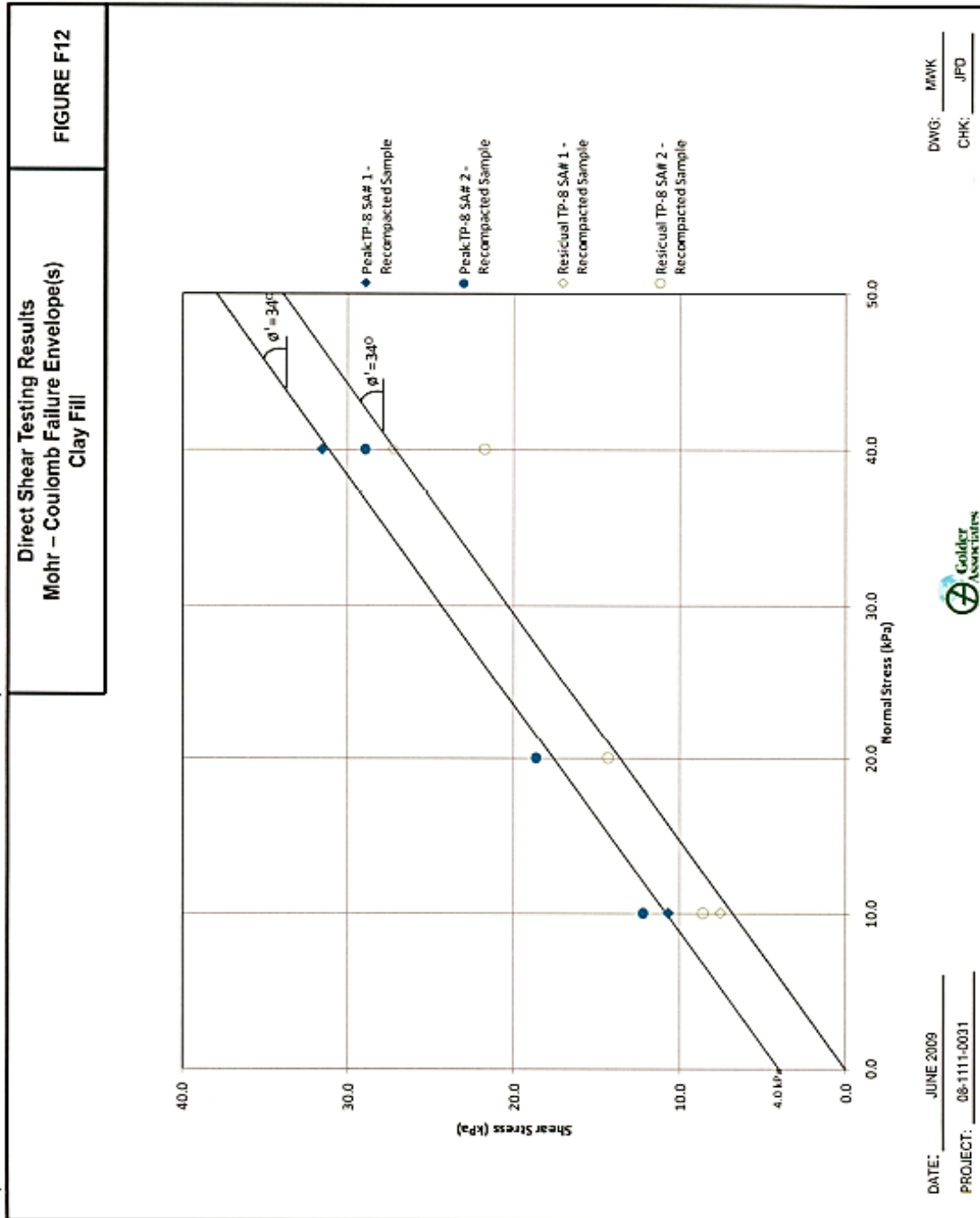
Date : August, 2014



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DIRECT SHEAR TEST RESULTS MOHR-COULOMB FAILURE ENVELOPES

FIGURE D6



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

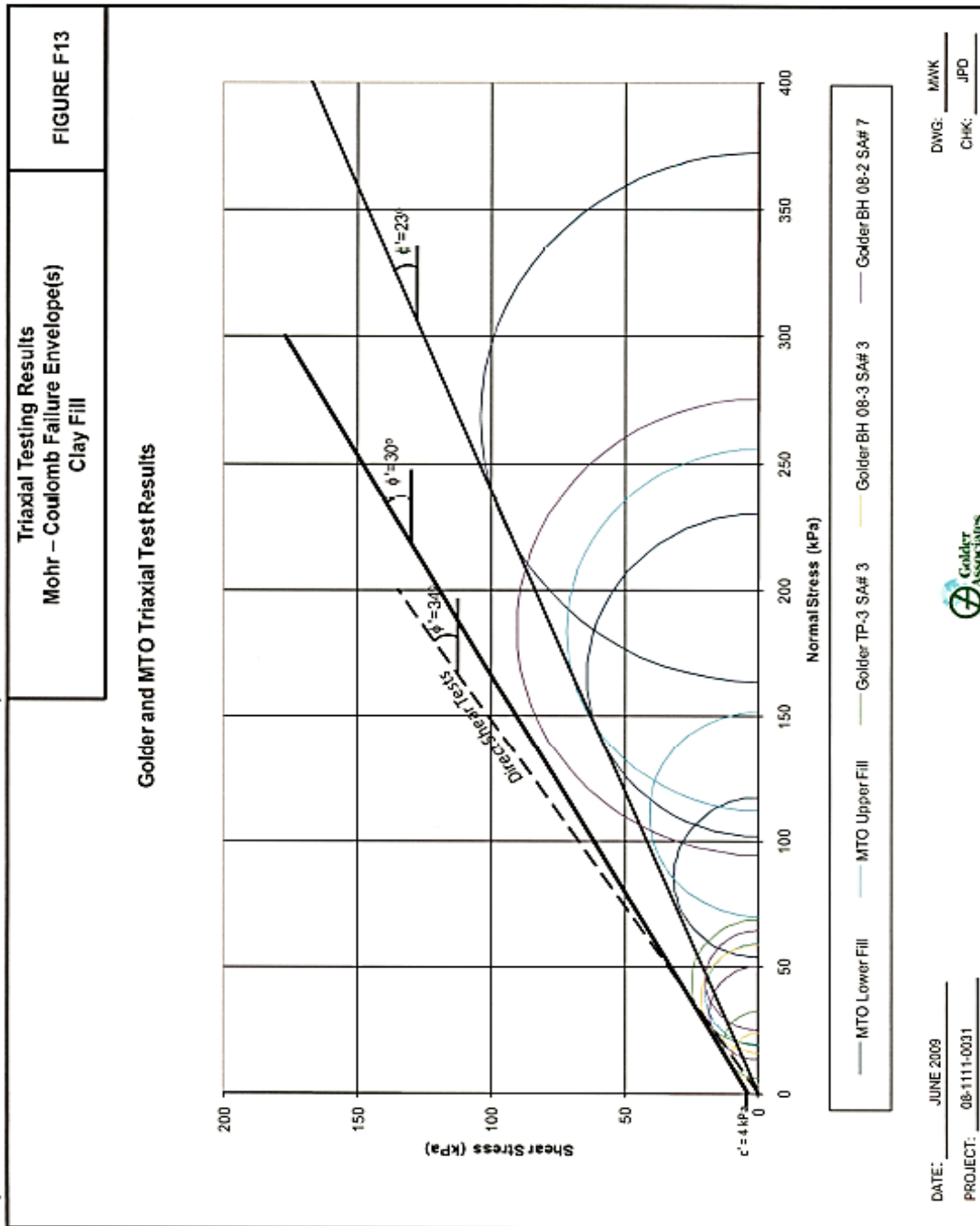
Date : August, 2014



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TRIAxIAL TEST RESULTS MOHR-COULOMB FAILURE ENVELOPES

FIGURE D7



Reference

Golder Associates Ltd., "Foundation Investigation and Design Report, Rehabilitation of North Embankment and Approach, Highway 140/CNR Overpass, Port Colborne, Ontario" Geocres No. 30L14-50, Report No. 08-1111-0031, dated August, 2009.

Project No. : 11-14-4076

Date : August, 2014



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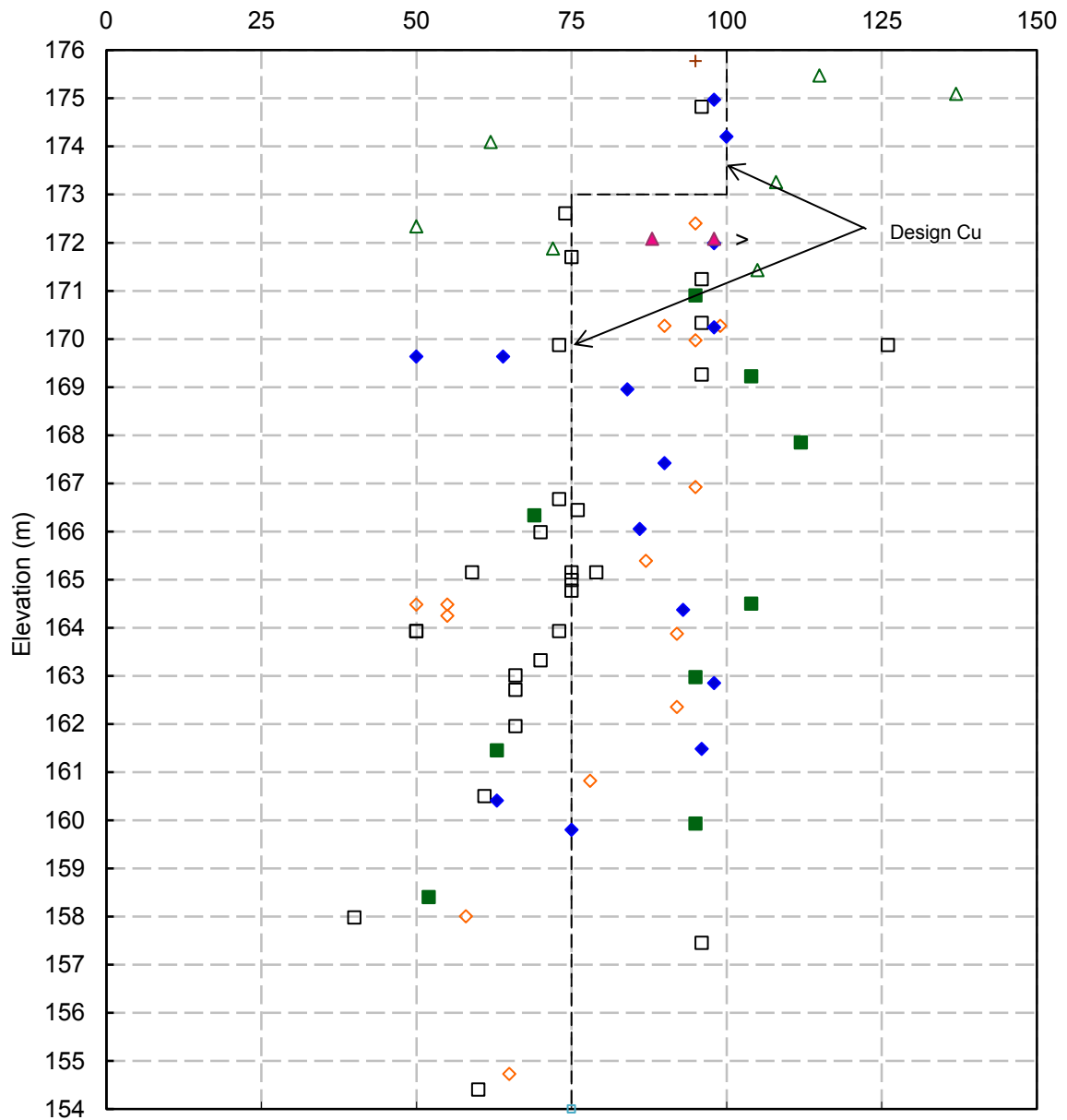
UNDRAINED SHEAR STRENGTH

FIGURE D8

HIGHWAY 140 AND FORKS ROAD

Silty Clay

Cu (kPa)



□ 1 ◇ 3 △ 101 × 102 * 103 + 104 ■ 208 ◆ 210 ▲ 211

Project No. : 11-14-4076

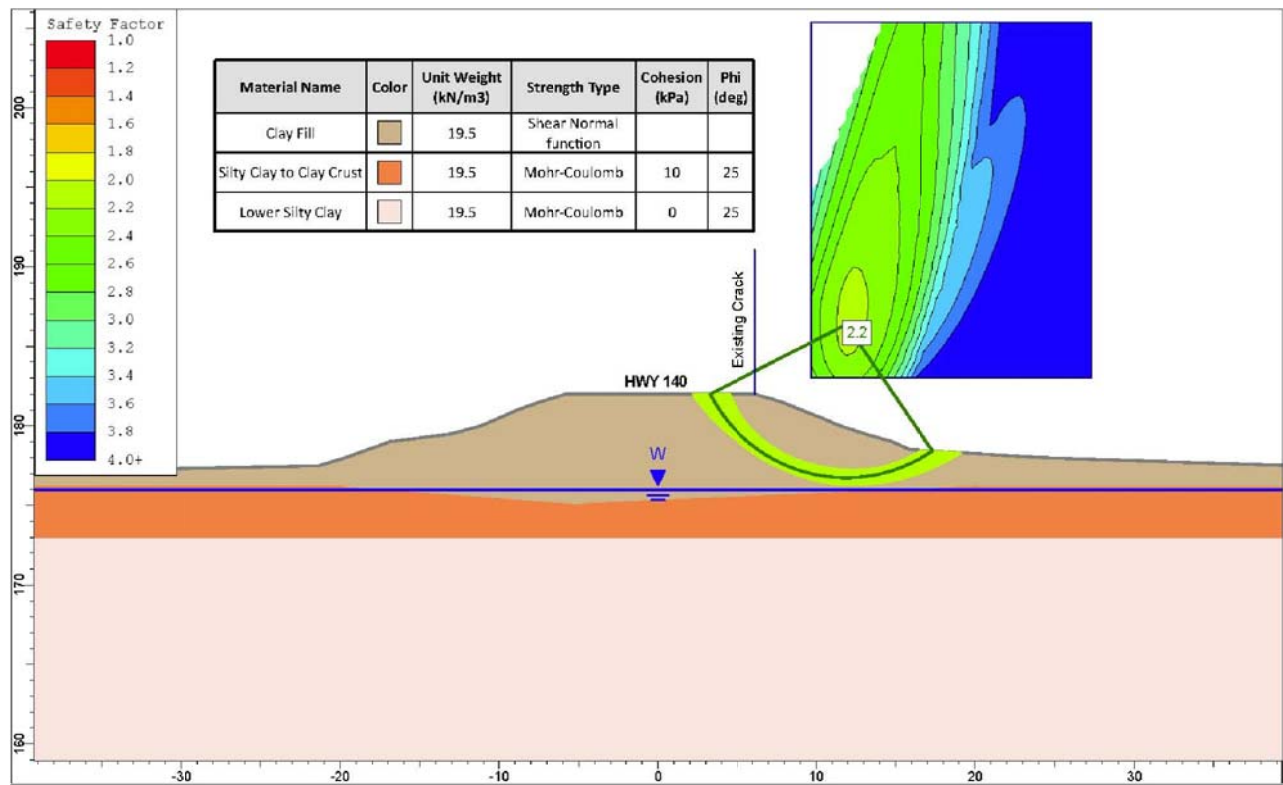
Date : August, 2014



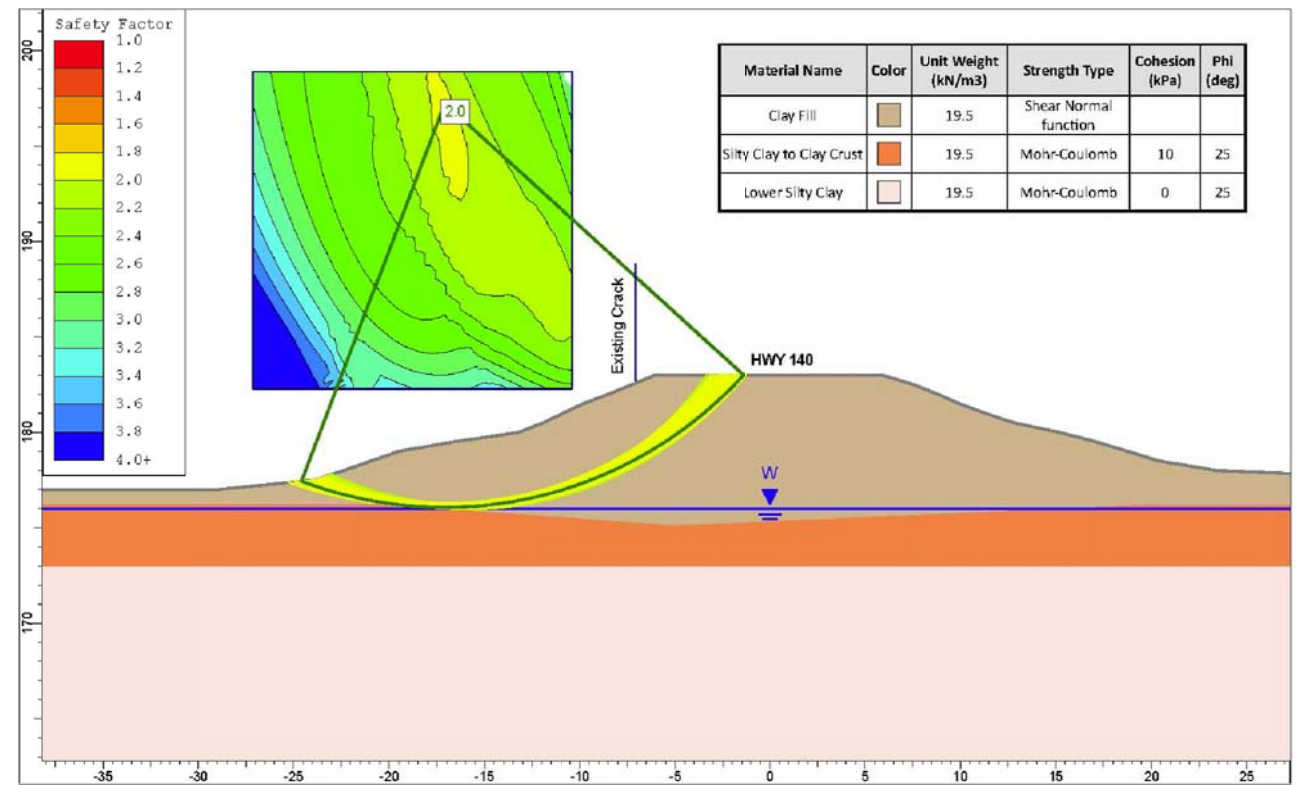
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Prepared by : AA

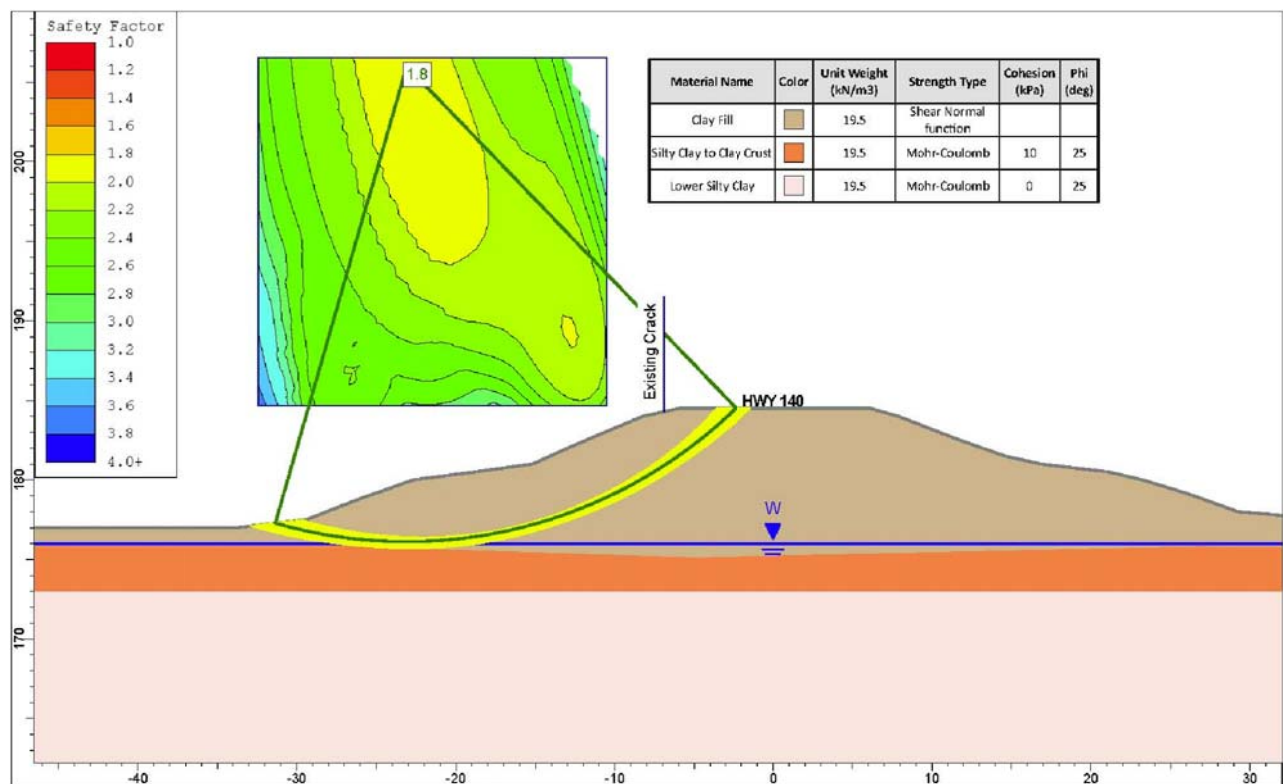
Checked by : RA



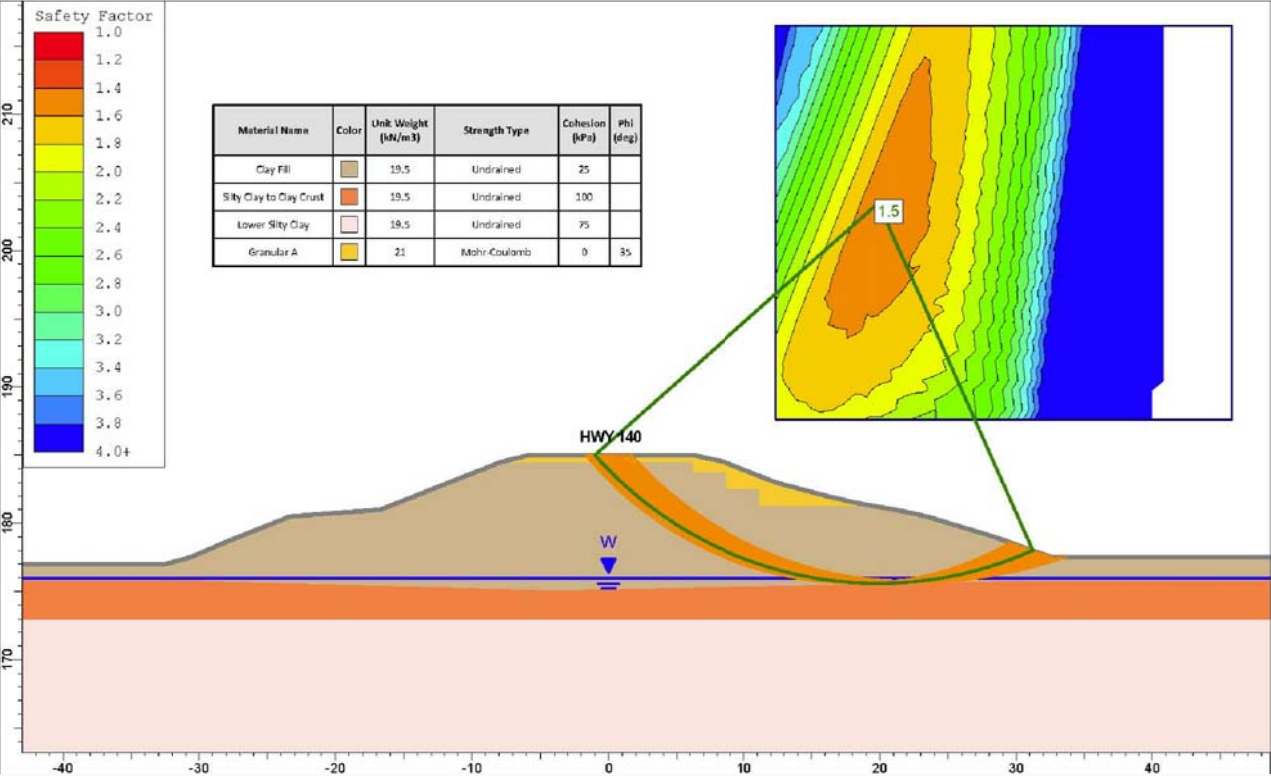
Sta. 15+375 - Effective Stress - Global Stability (Circular) - Year 2014



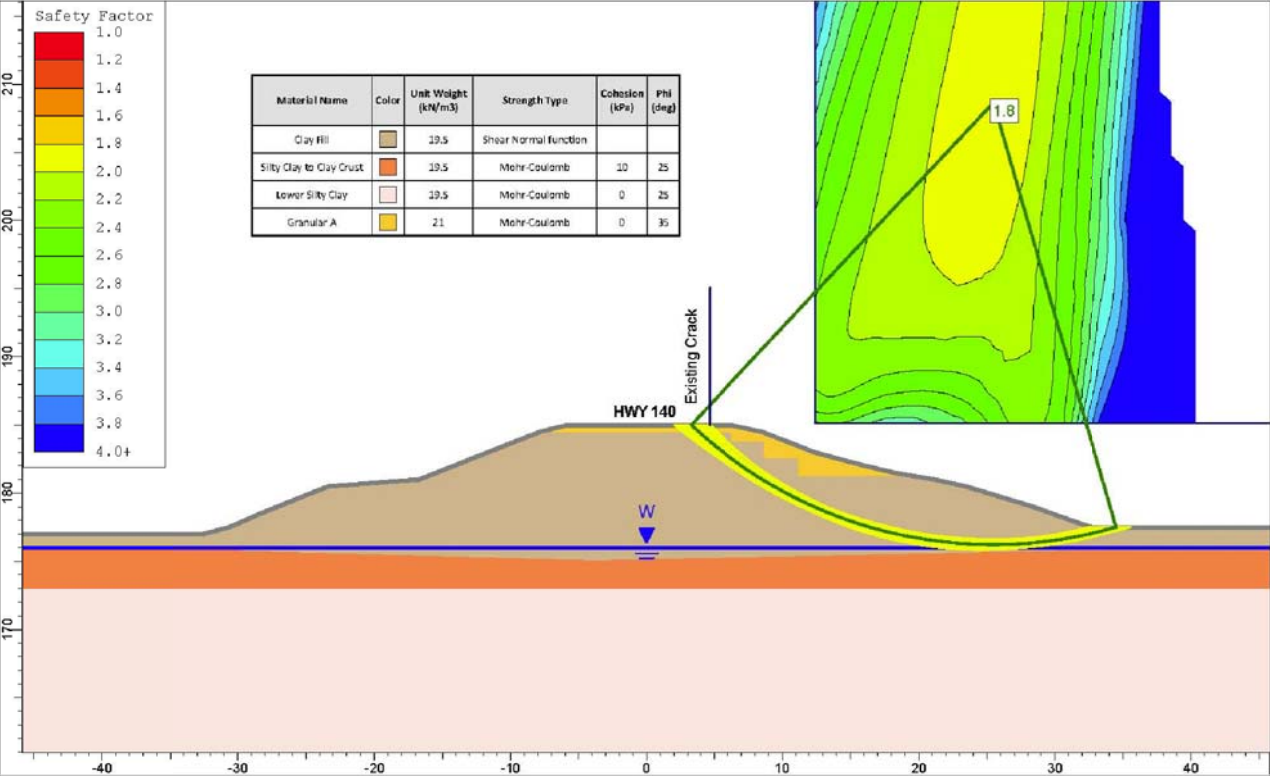
Sta. 15+400 - Effective Stress - Global Stability (Circular) - Year 2014



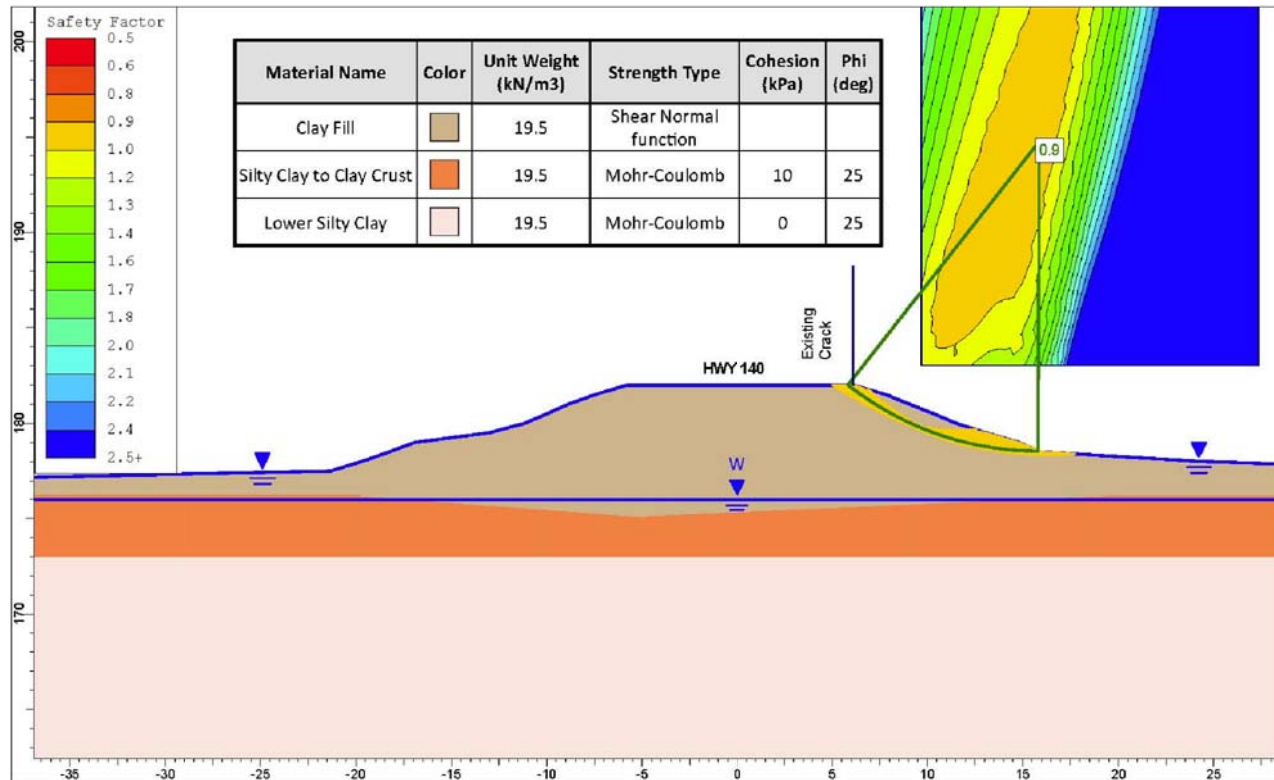
Sta. 15+450 - Effective Stress - Global Stability (Circular) - Year 2014



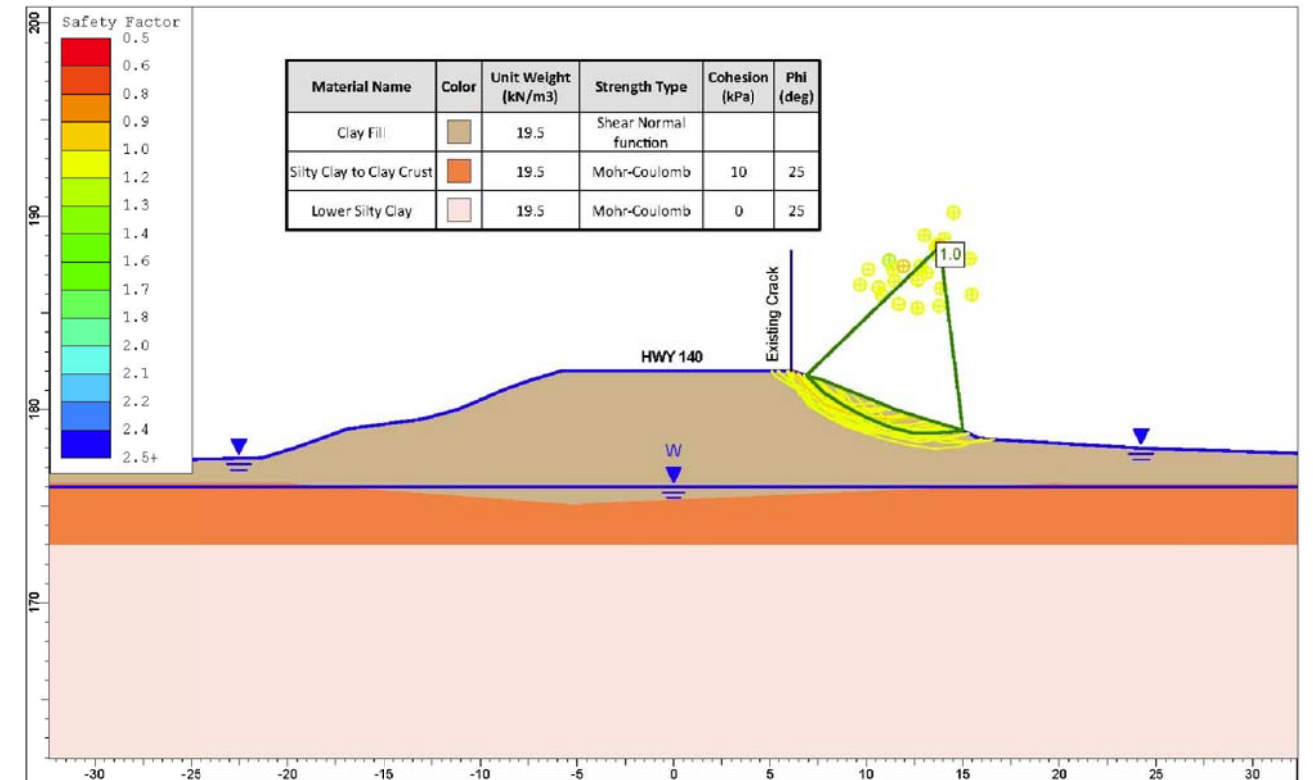
Sta. 15+475 - Total Stress - Global Stability (Circular) - Year 1998



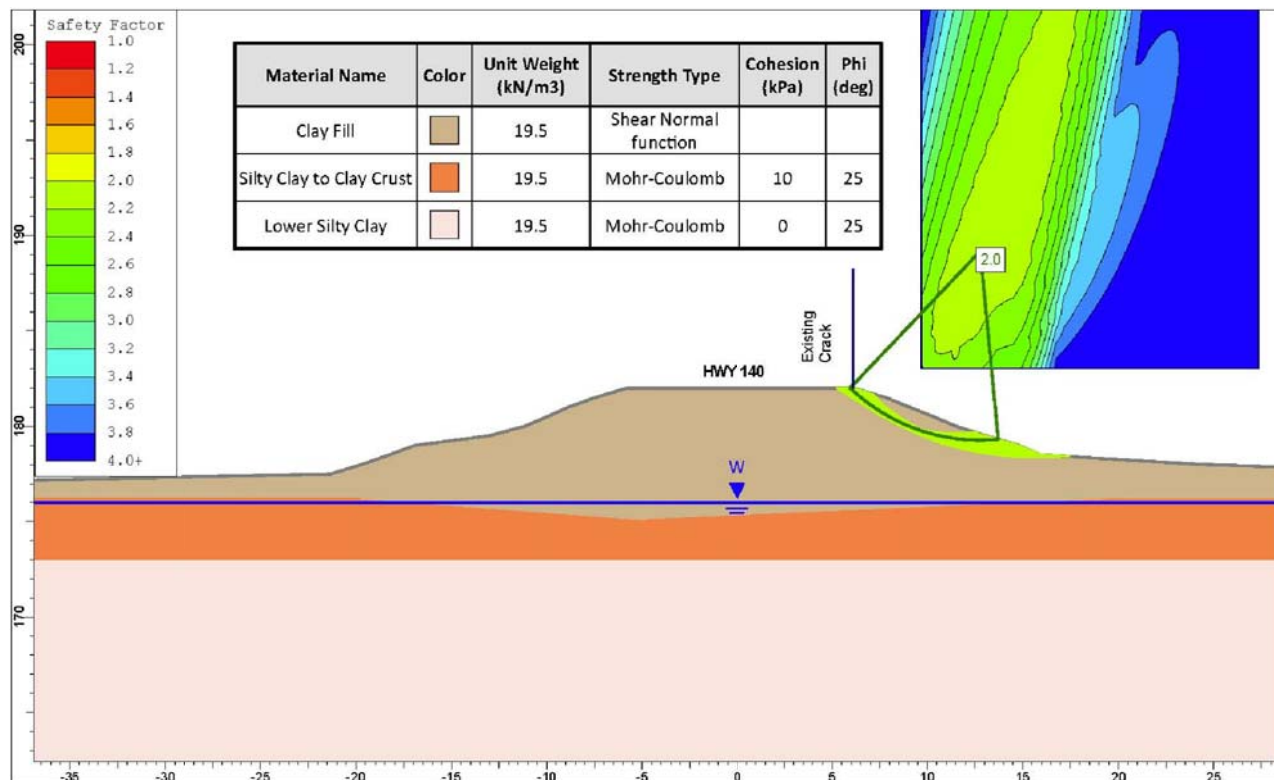
Sta. 15+475 - Effective Stress - Global Stability (Circular) - Year 2014



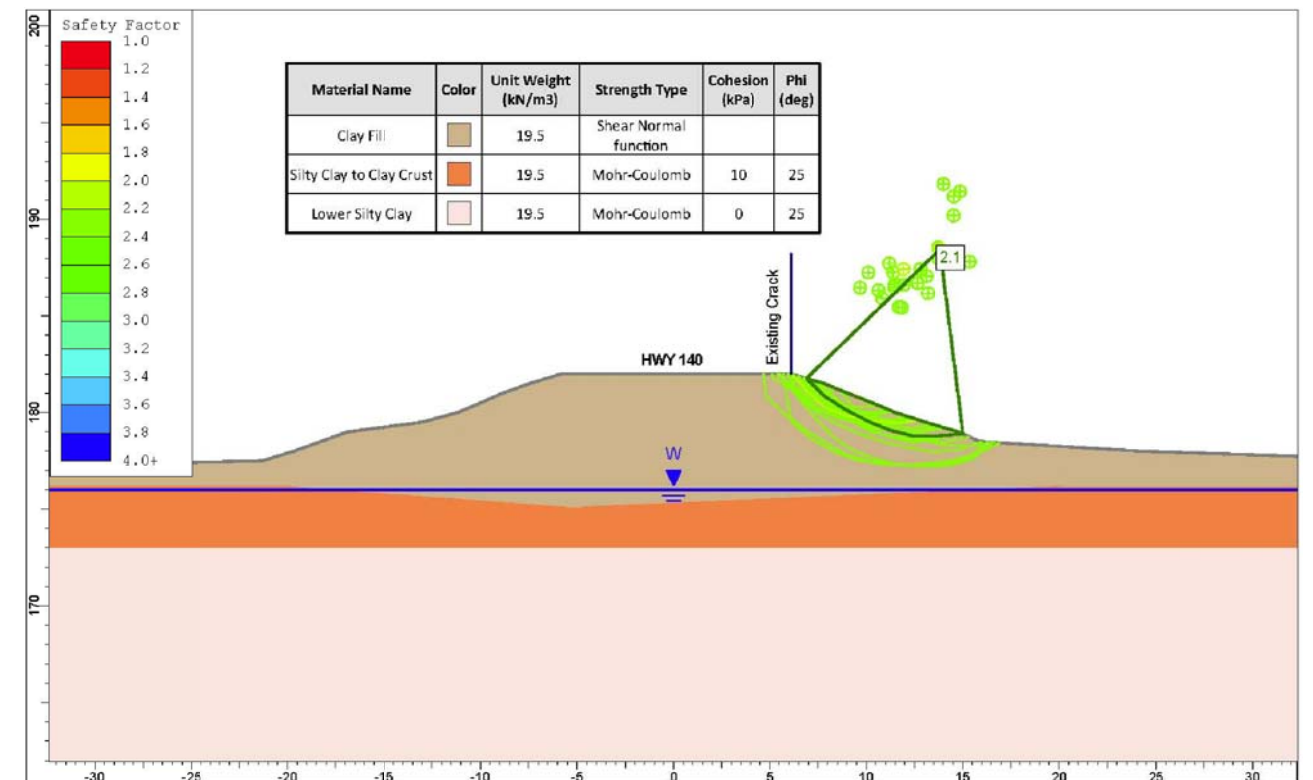
Sta. 15+375 - Effective Stress - Surficial Stability - Wet Conditions (Circular) - Year 2014



Sta. 15+375 - Effective Stress - Surficial Stability - Wet Conditions (Non-circular) - Year 2014



Sta. 15+375 - Effective Stress - Surficial Stability - Dry Conditions (Circular) - Year 2014



Sta. 15+375 - Effective Stress - Surficial Stability - Dry Conditions (Non-Circular) - Year 2014



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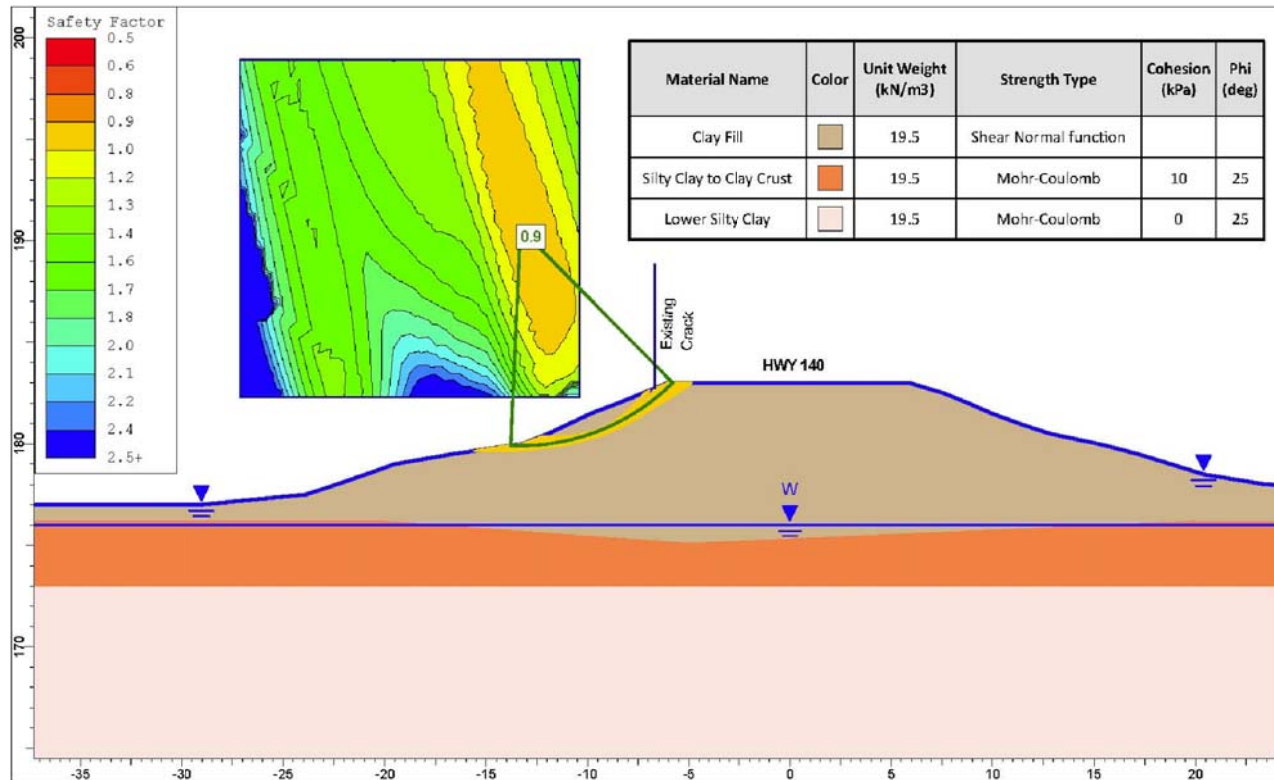
HWY 140 CNR OVERPASS, SOUTH EMBANKMENT
SLOPE INSTABILITIES

SLOPE STABILITY RESULTS

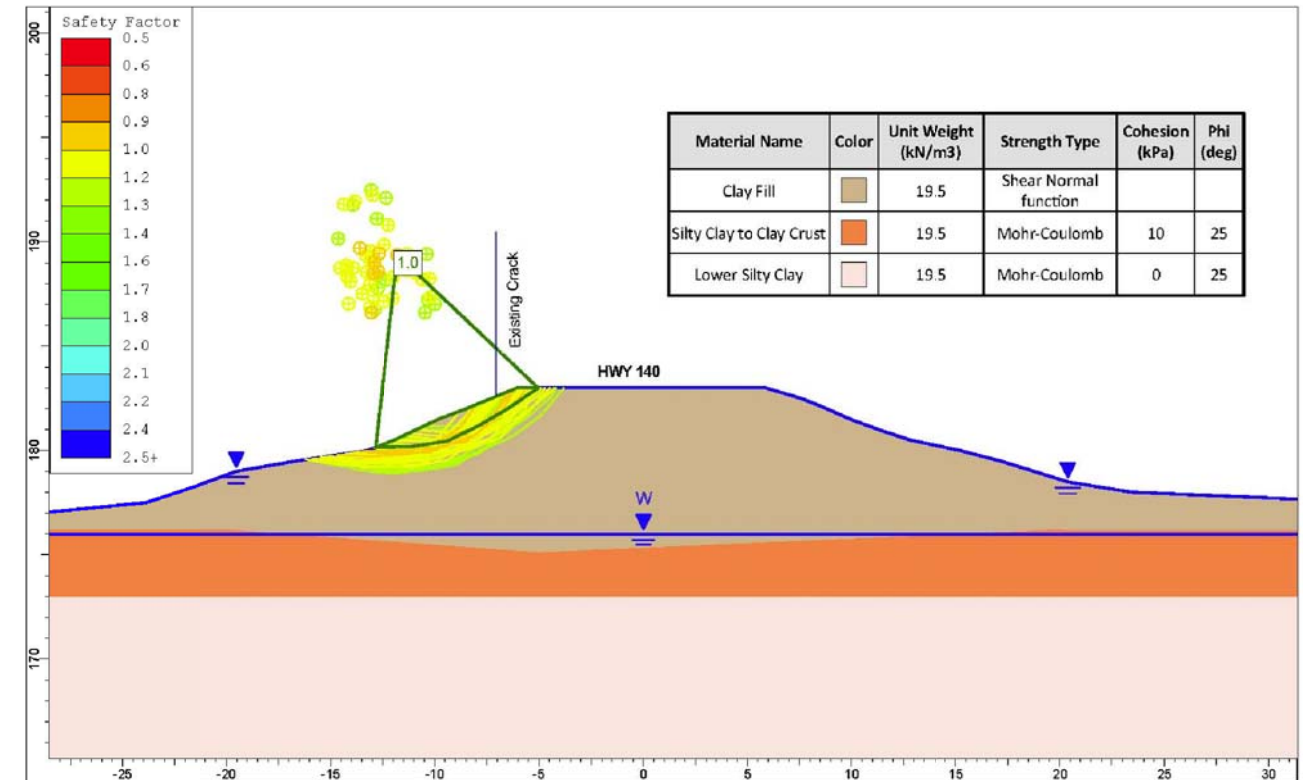
G.W.P. 2044-13-00 DATE: August 2014

SUBM'D. HA CHKD. RA APPD: MT

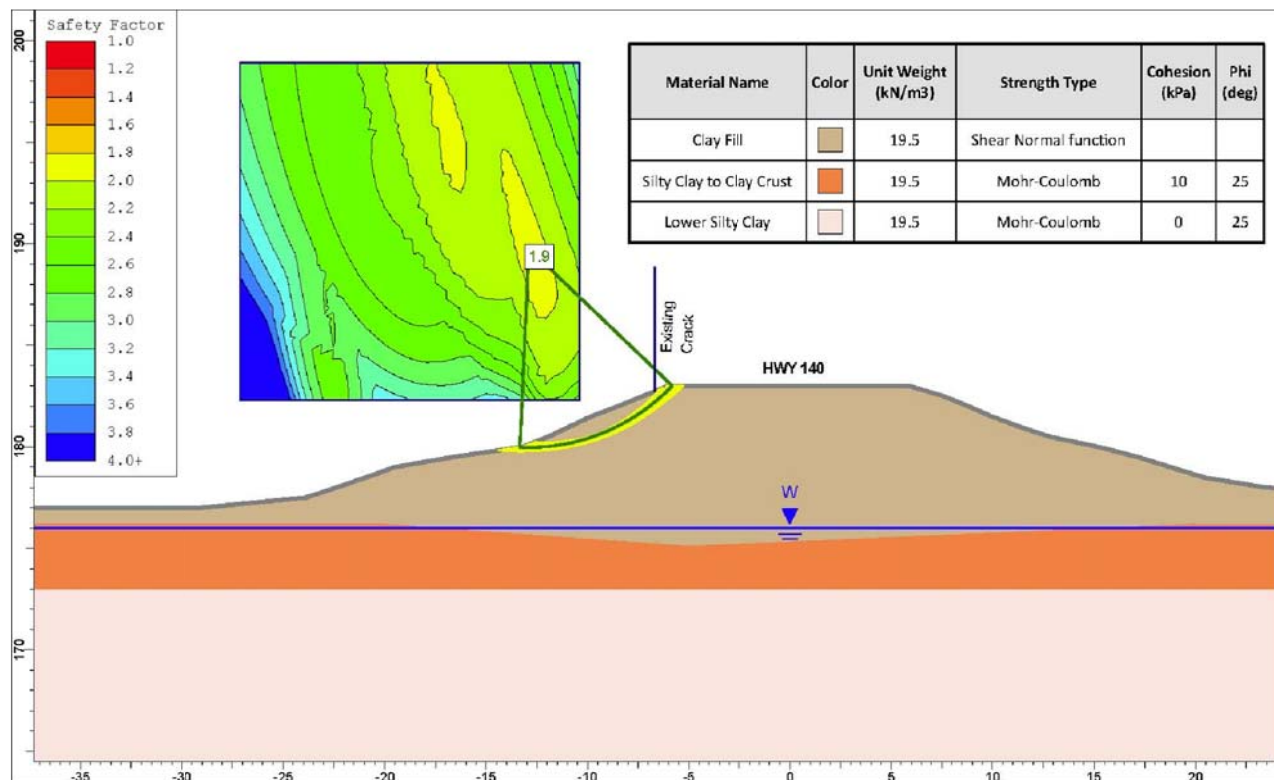
Project No: 11-14-4076 Figure D11



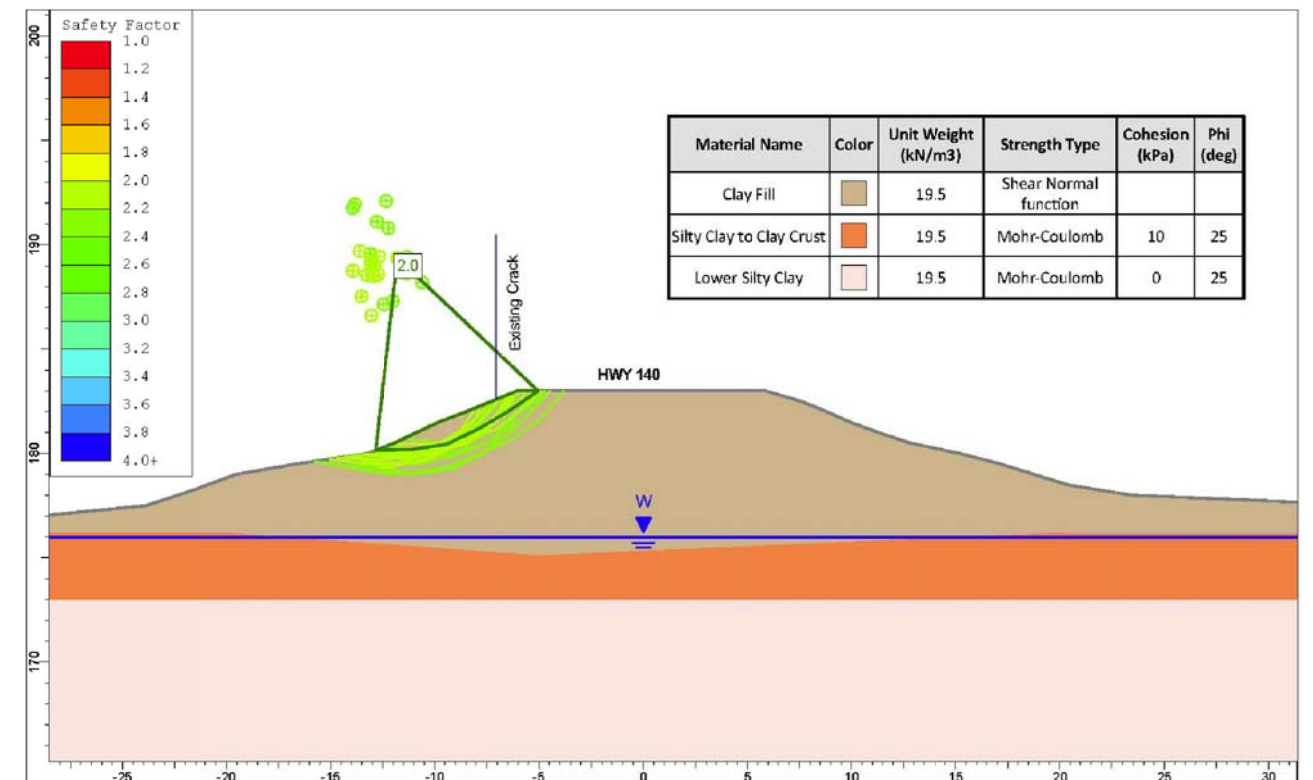
Sta. 15+400 - Effective Stress - Surficial Stability - Wet Conditions (Circular - Year 2014)



Sta. 15+400 - Effective Stress - Surficial Stability - Wet Conditions (Non-circular) - Year 2014



Sta. 15+400 - Effective Stress - Surficial Stability - Dry Conditions (Circular) - Year 2014



Sta. 15+400 - Effective Stress - Surficial Stability - Dry Conditions (Non-Circular) - Year 2014



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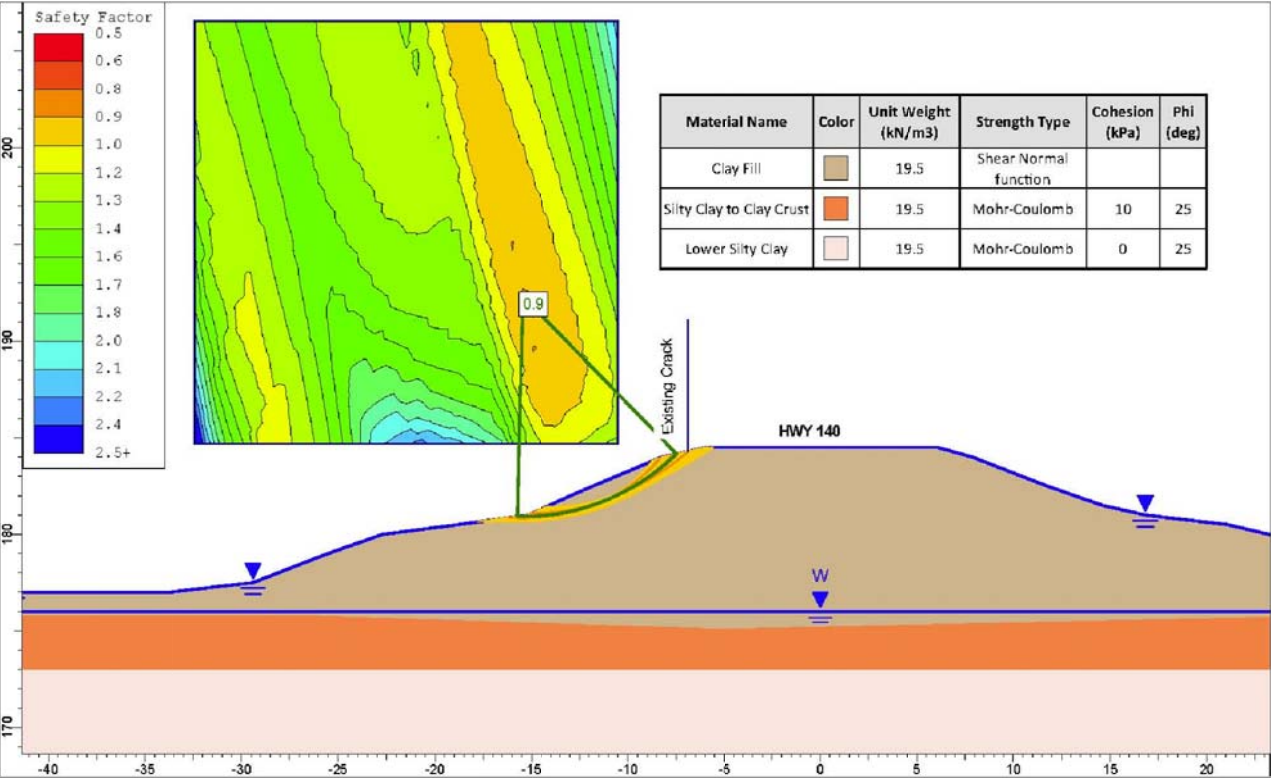
HWY 140 CNR OVERPASS, SOUTH EMBANKMENT
SLOPE INSTABILITIES

SLOPE STABILITY RESULTS

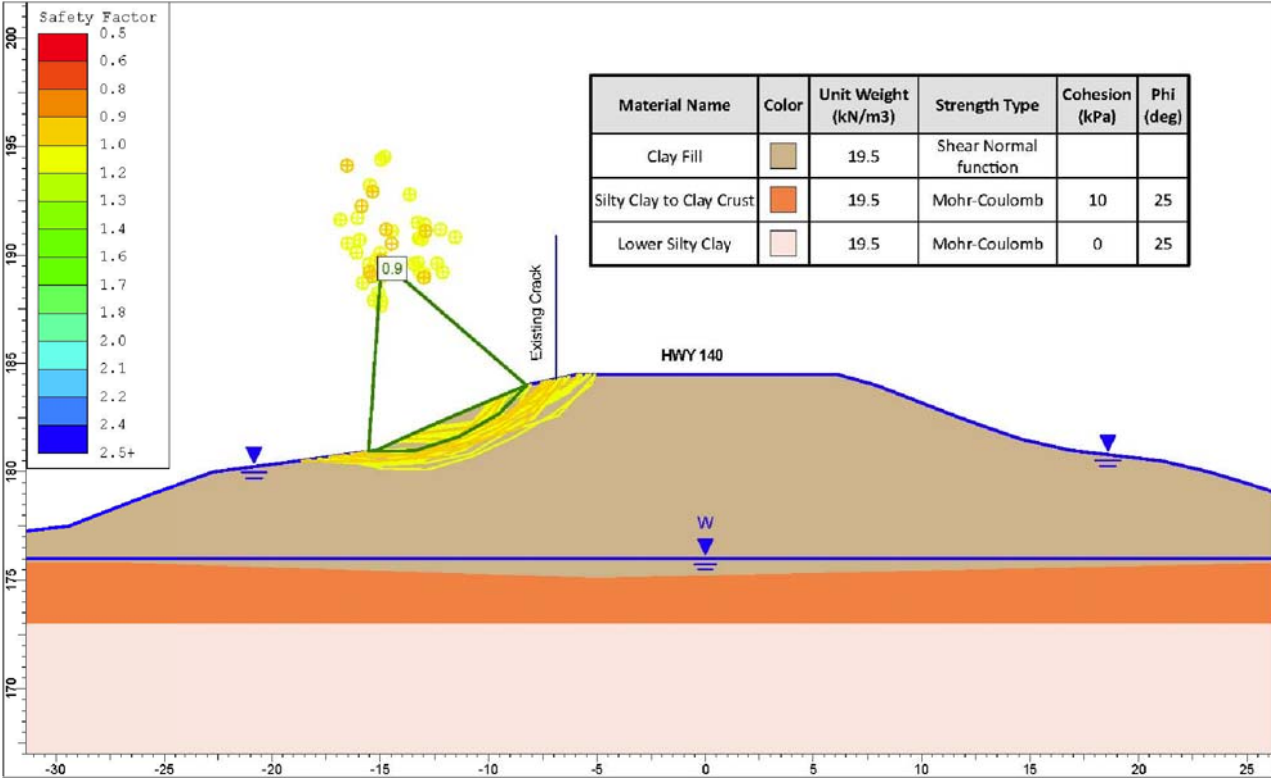
G.W.P. 2044-13-00 DATE: August 2014

SUBM'D. HA CHKD. RA APPD: MT

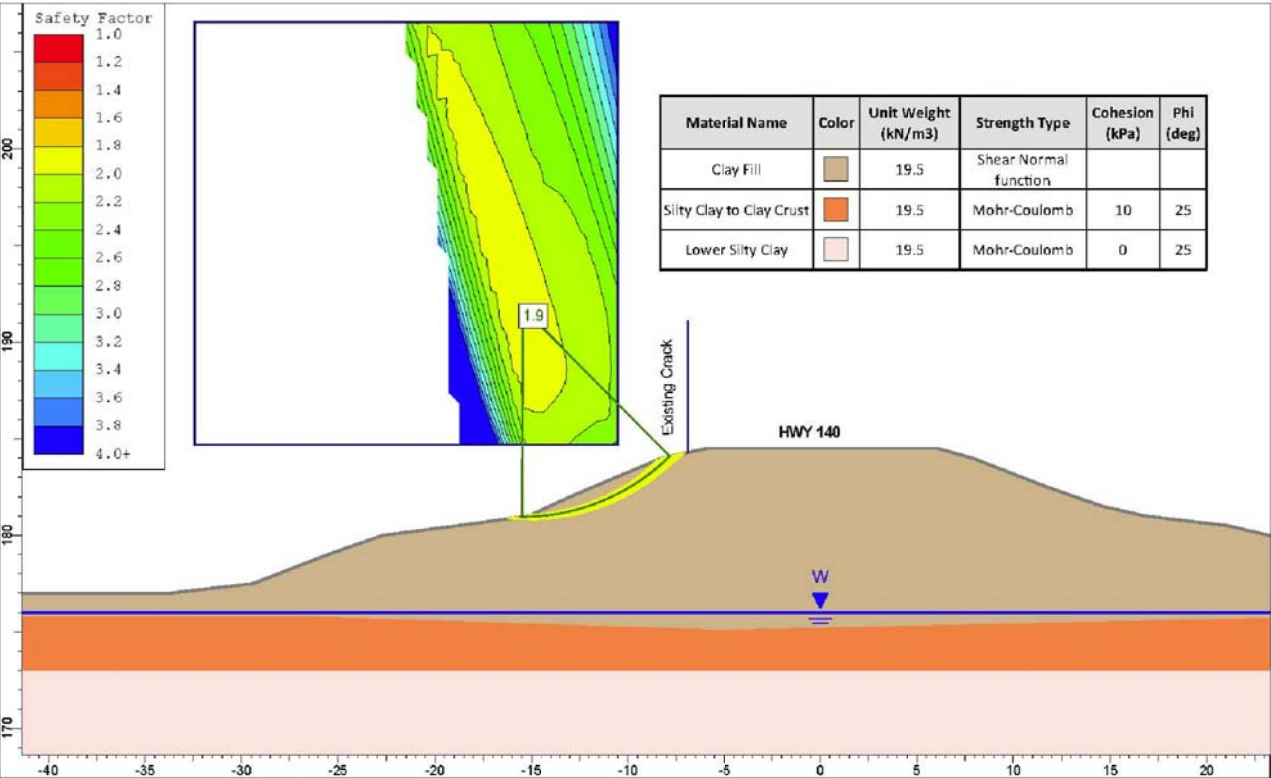
Project No: 11-14-4076 Figure D12



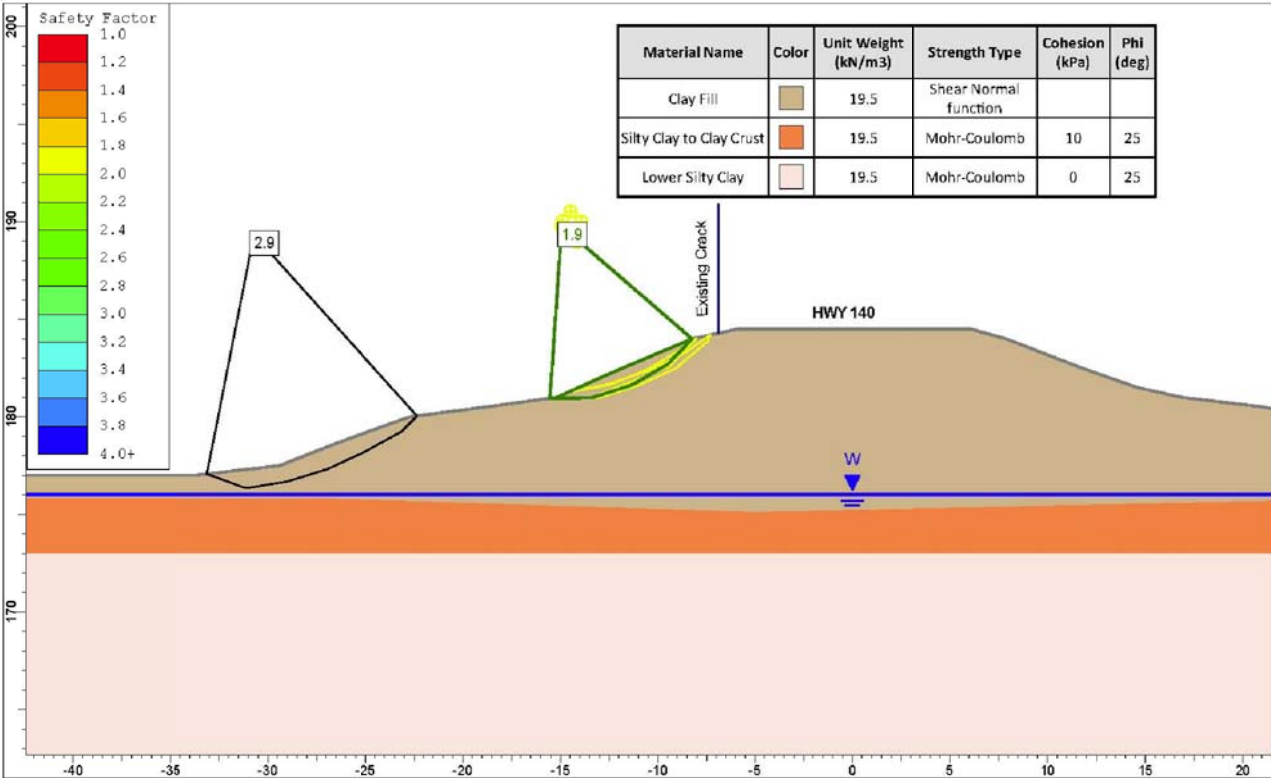
Sta. 15+450 - Effective Stress - Surficial Stability - Wet Conditions (Circular) - Year 2014



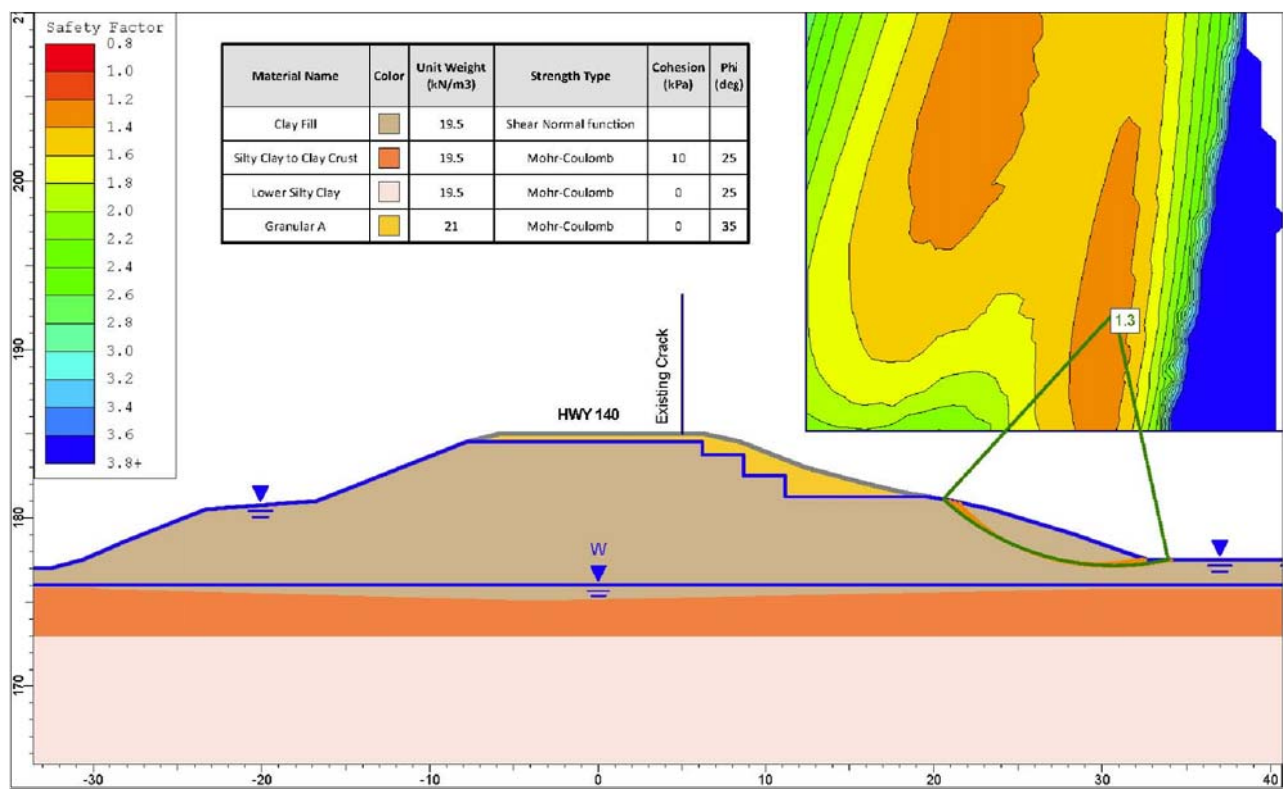
Sta. 15+450 - Effective Stress - Surficial Stability - Wet Conditions (Non-circular) - Year 2014



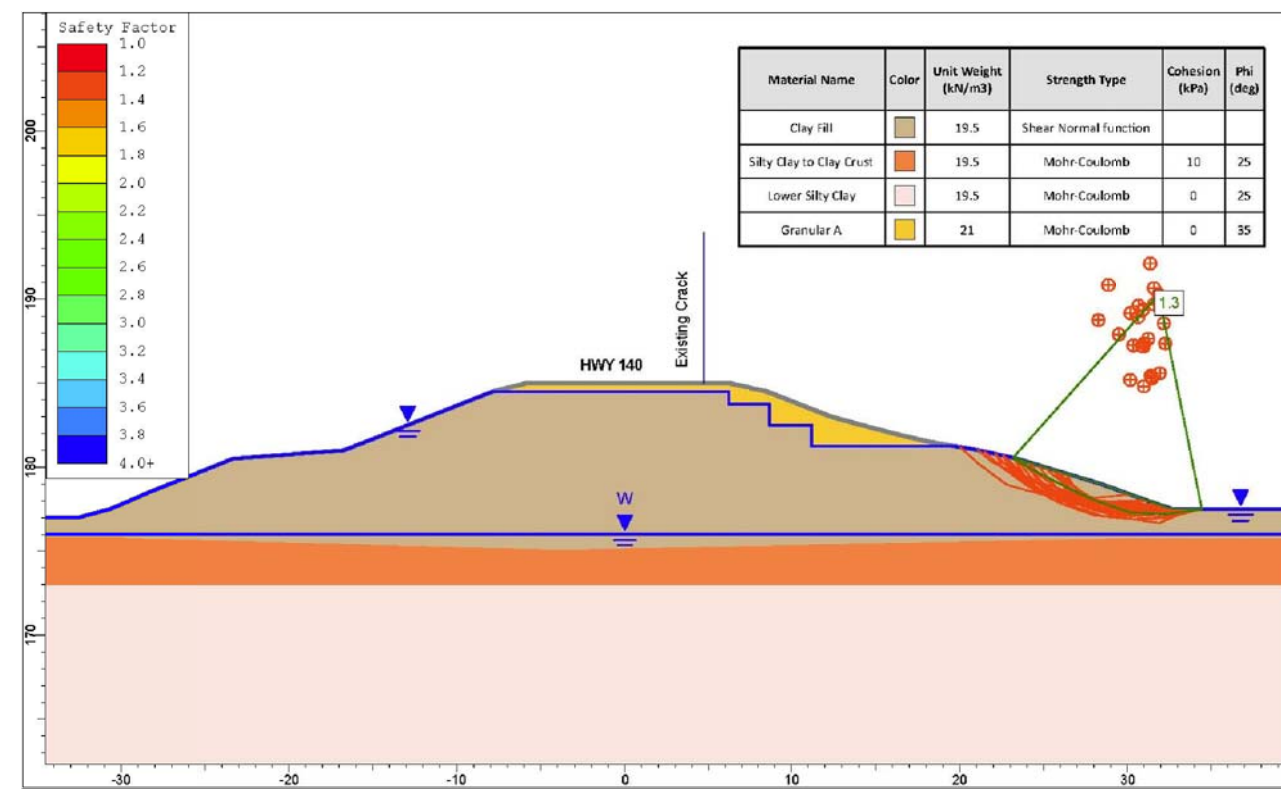
Sta. 15+450 - Effective Stress - Surficial Stability - Dry Conditions (Circular) - Year 2014



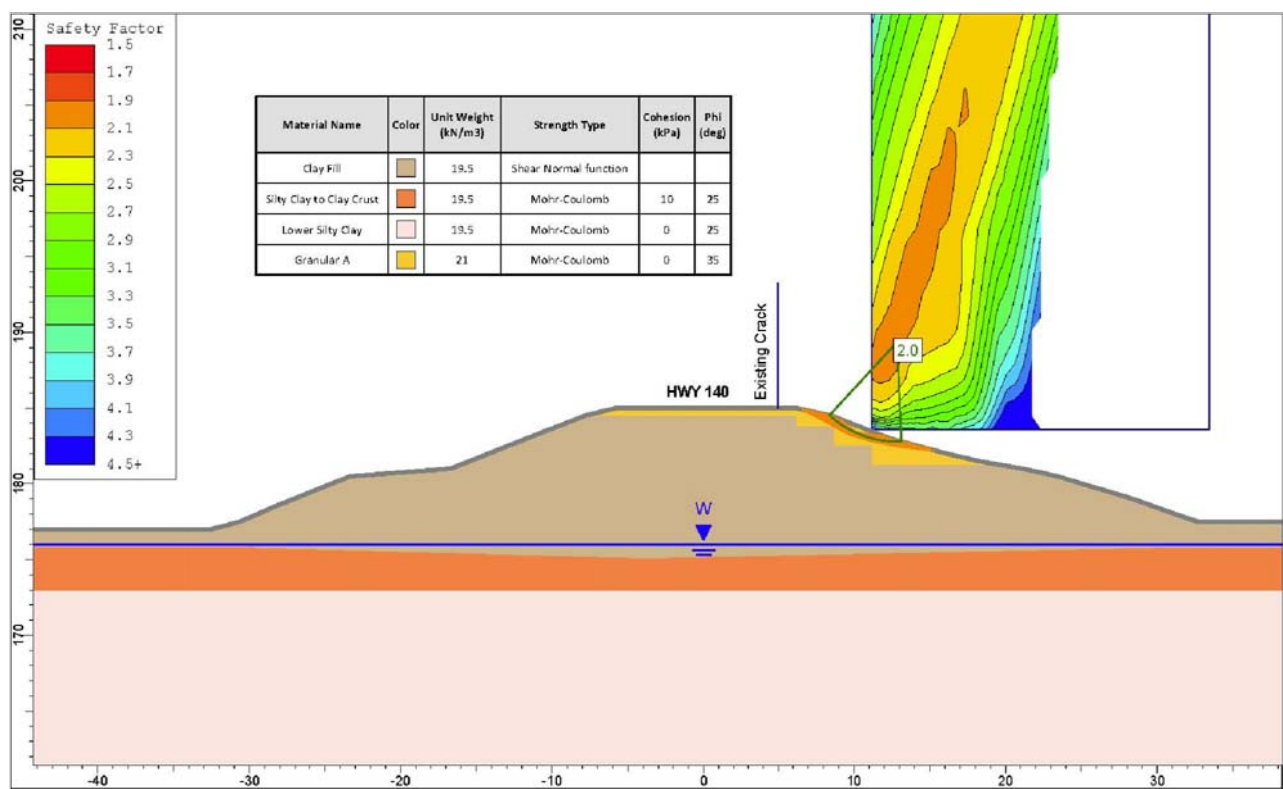
Sta. 15+450 - Effective Stress - Surficial Stability - Dry Conditions (Non-Circular) - Year 2014



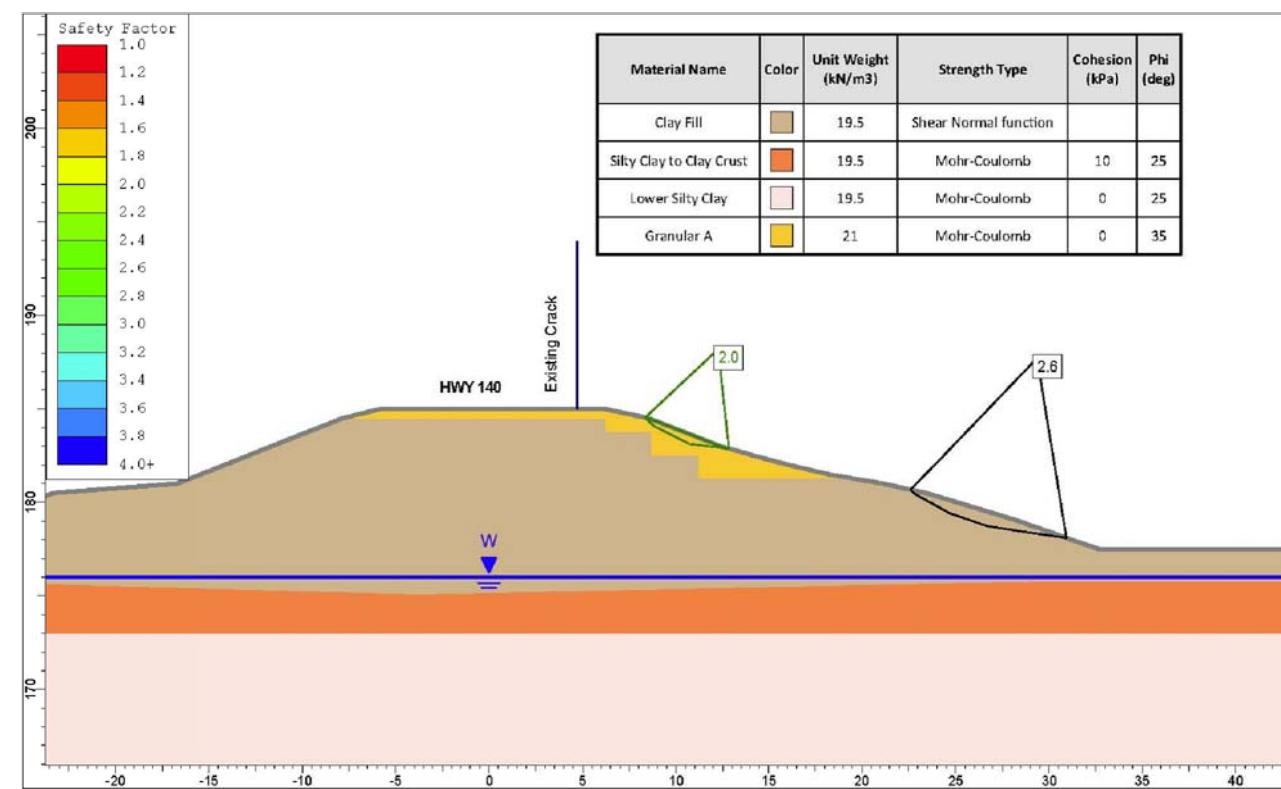
Sta. 15+475 - Effective Stress - Surficial Stability - Wet Conditions (Circular - Year 2014)



Sta. 15+475 - Effective Stress - Surficial Stability - Wet Conditions (Non-circular) - Year 2014



Sta. 15+475 - Effective Stress - Surficial Stability - Dry Conditions (Circular) - Year 2014



Sta. 15+475 - Effective Stress - Surficial Stability - Dry Conditions (Non-Circular) - Year 2014



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HWY 140 CNR OVERPASS, SOUTH EMBANKMENT
SLOPE INSTABILITIES

SLOPE STABILITY RESULTS

G.W.P. 2044-13-00 DATE: August 2014

SUBM'D. HA CHKD. RA APPD: MT

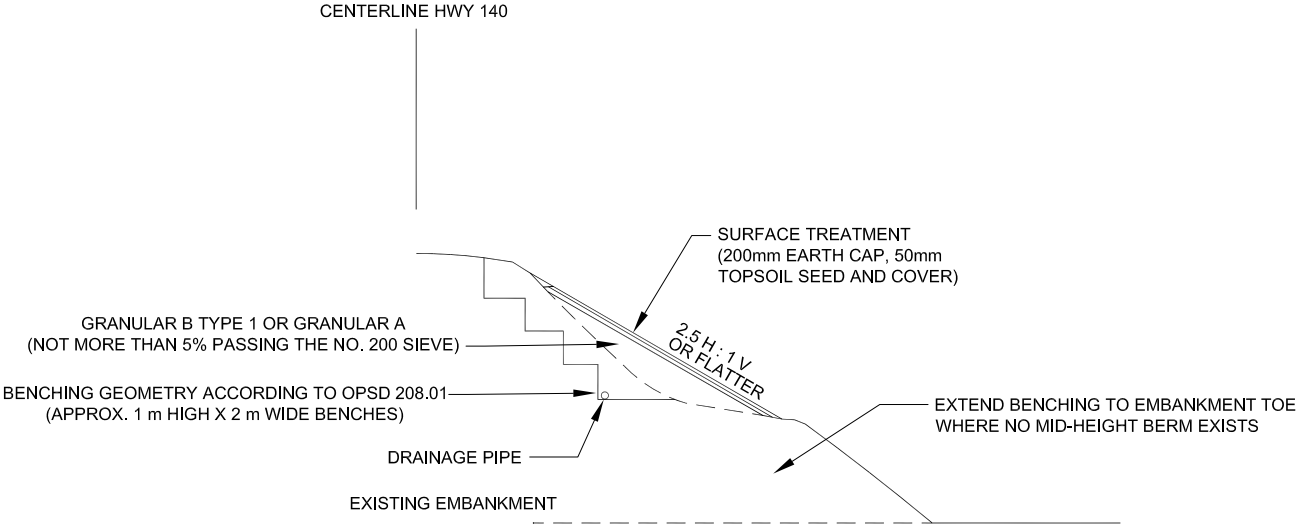
Project No: 11-14-4076 Figure D14

APPENDIX E

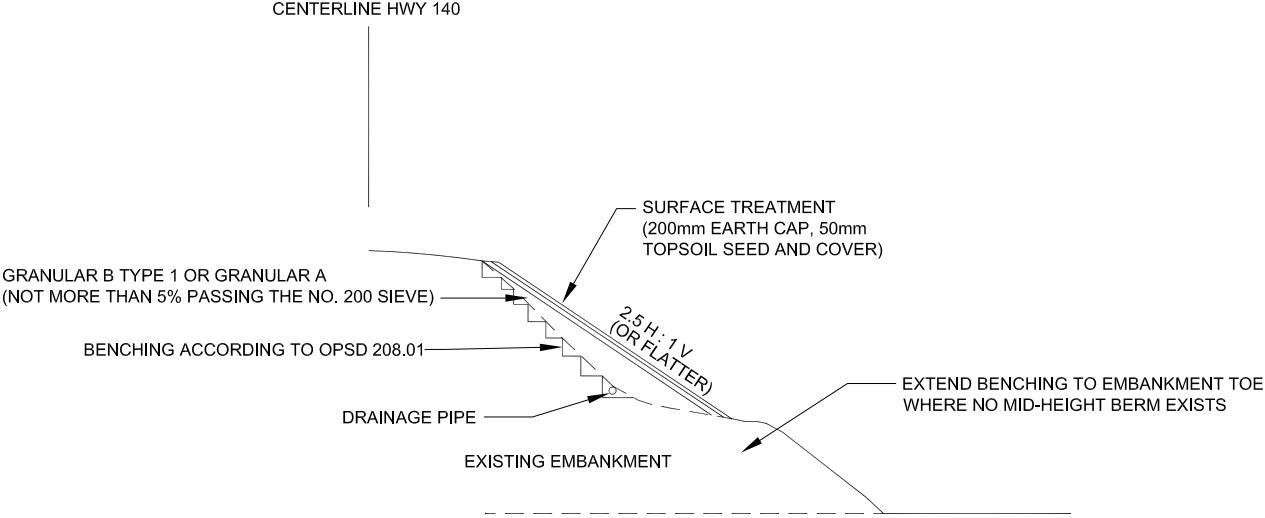
Remediation Options



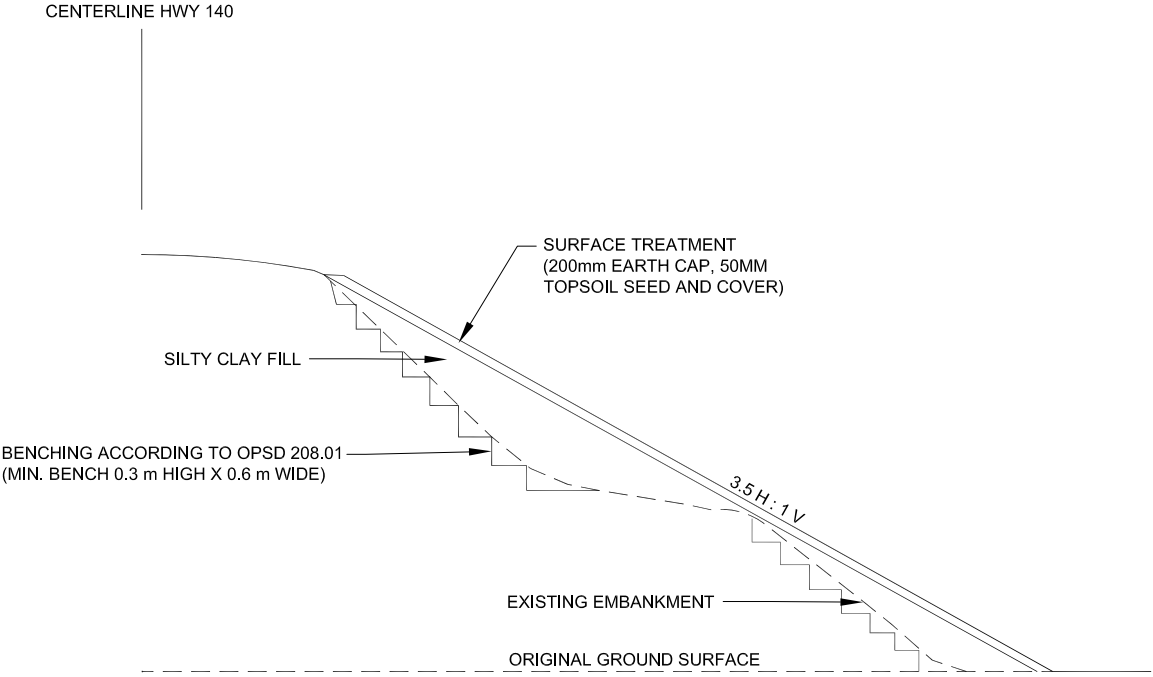
\\PDS\Server1\Project Files\11-Geo\2014\11-14-4076 Hwy 140 Port Colborne\A. Dwy. Laps\AutoCAD\11-14-4076 Slope Remediation Dwg\11-14-4076 Slope Remediation Figure ----- dwg, kenal



DEEP BENCHING WITH GRANULAR MATERIAL
2.5H:1V SIDE SLOPE OR FLATTER (OPTION 1)



STANDARD BENCHING WITH GRANULAR MATERIAL
2.5H:1V SIDE SLOPE OR FLATTER (OPTION 2)



SLOPE FLATTENING WITH SILTY CLAY (OPTION 3)

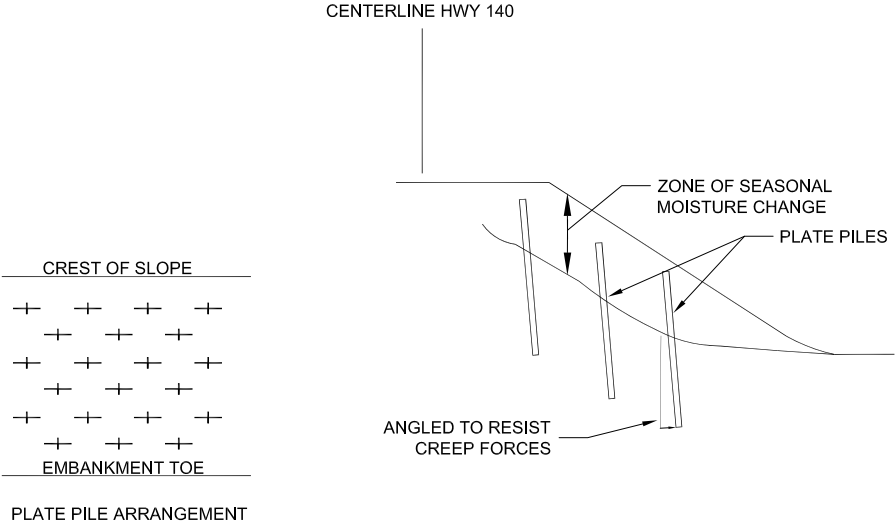

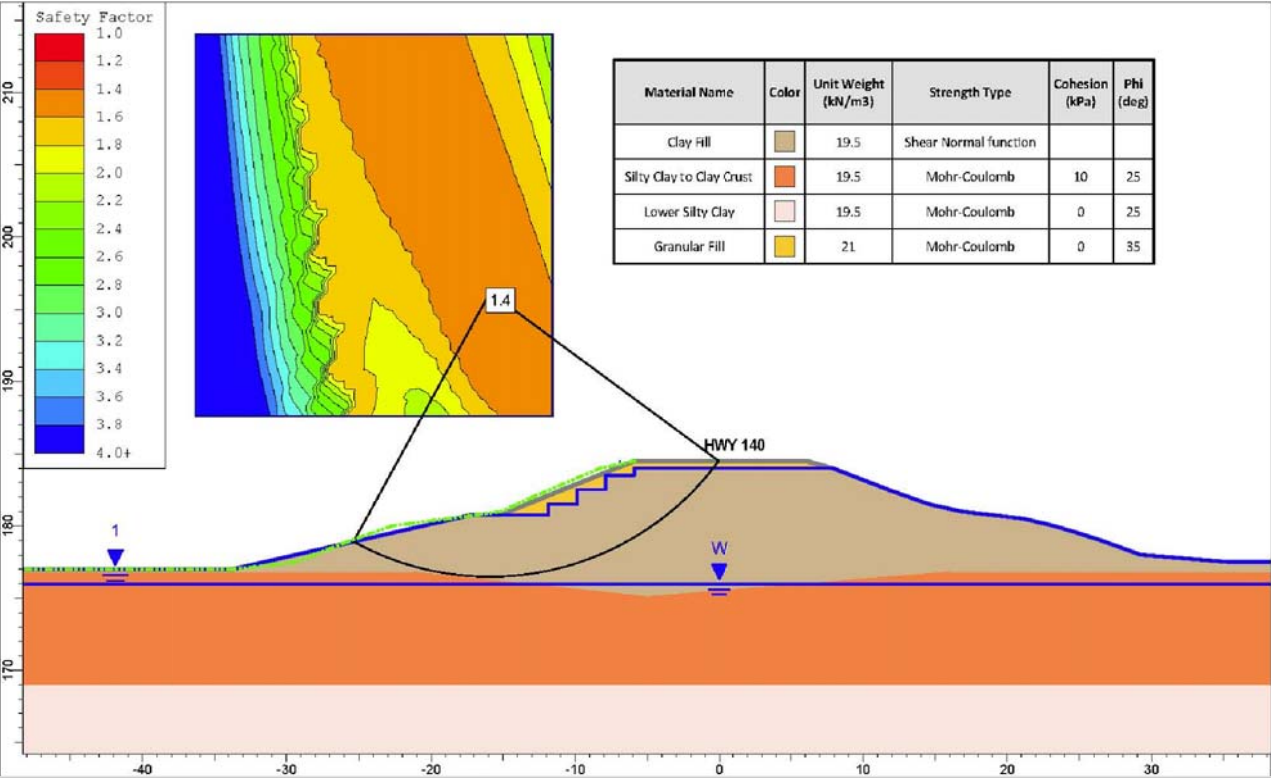


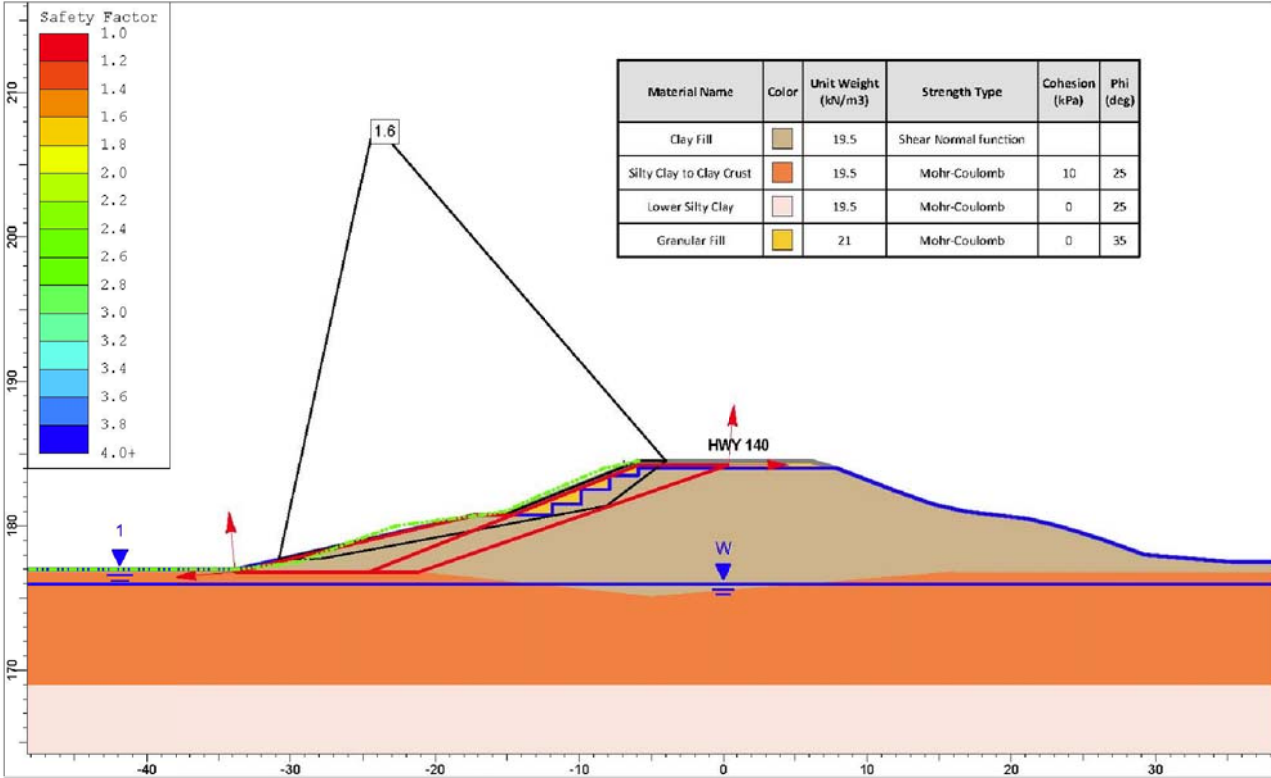
PLATE PILE SLOPE STABILIZATION (OPTION 4)

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	SLOPE REMEDIATION OPTIONS		
	G.W.P. 2044-13-00	DATE:	August 2014
	SUBM'D. HA	CHKD. RA	APPD: MT
	Project No: 11-14-4076	Figure	E1

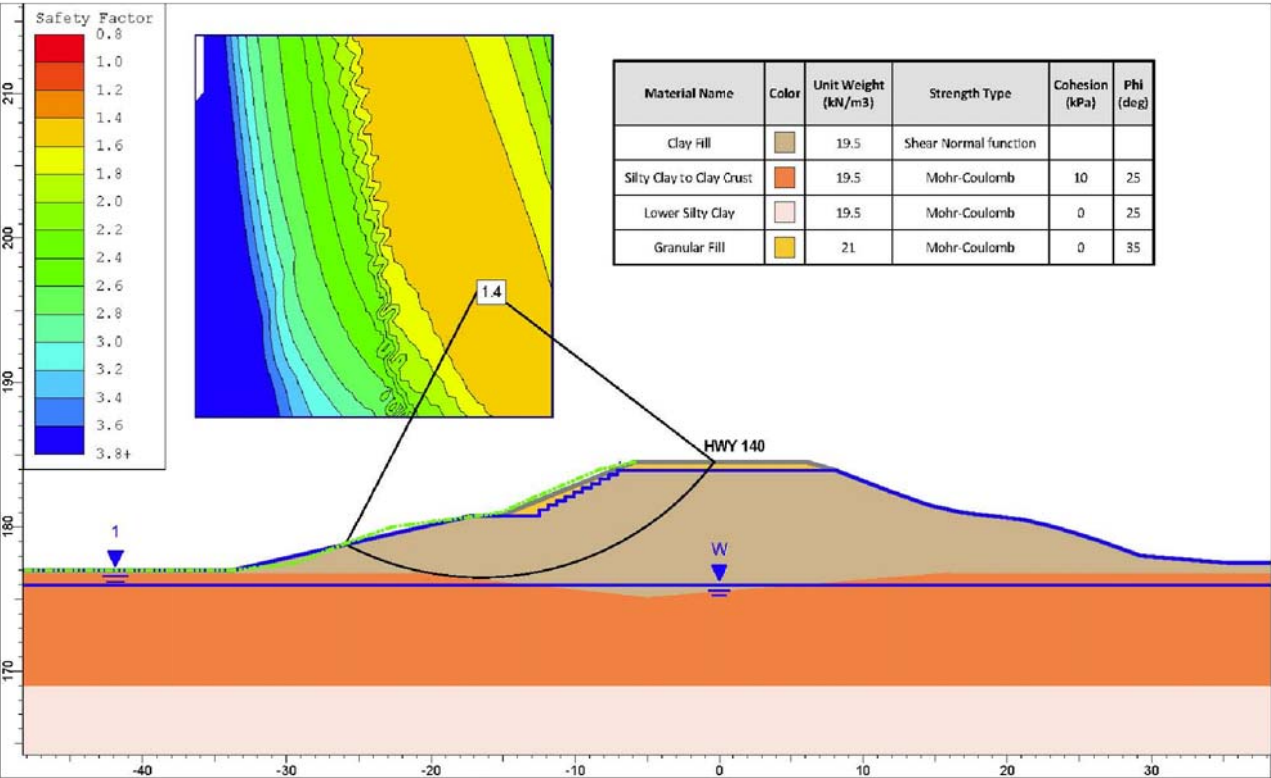




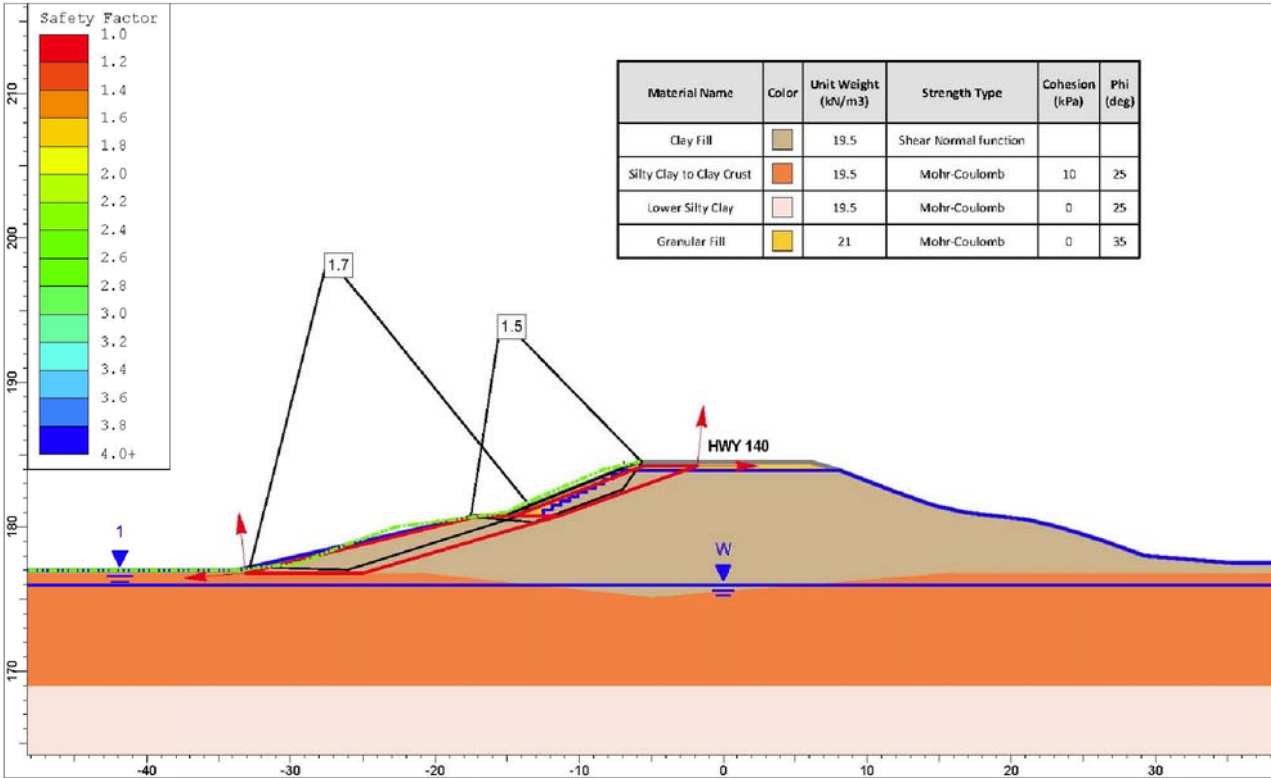
Sta. 15+450 - Deep Benching with Granular Material - Global Stability




Sta. 15+450 - Deep Benching with Granular Material - Surficial Stability



Sta. 15+450 - Standard Benching with Granular Material - Global Stability

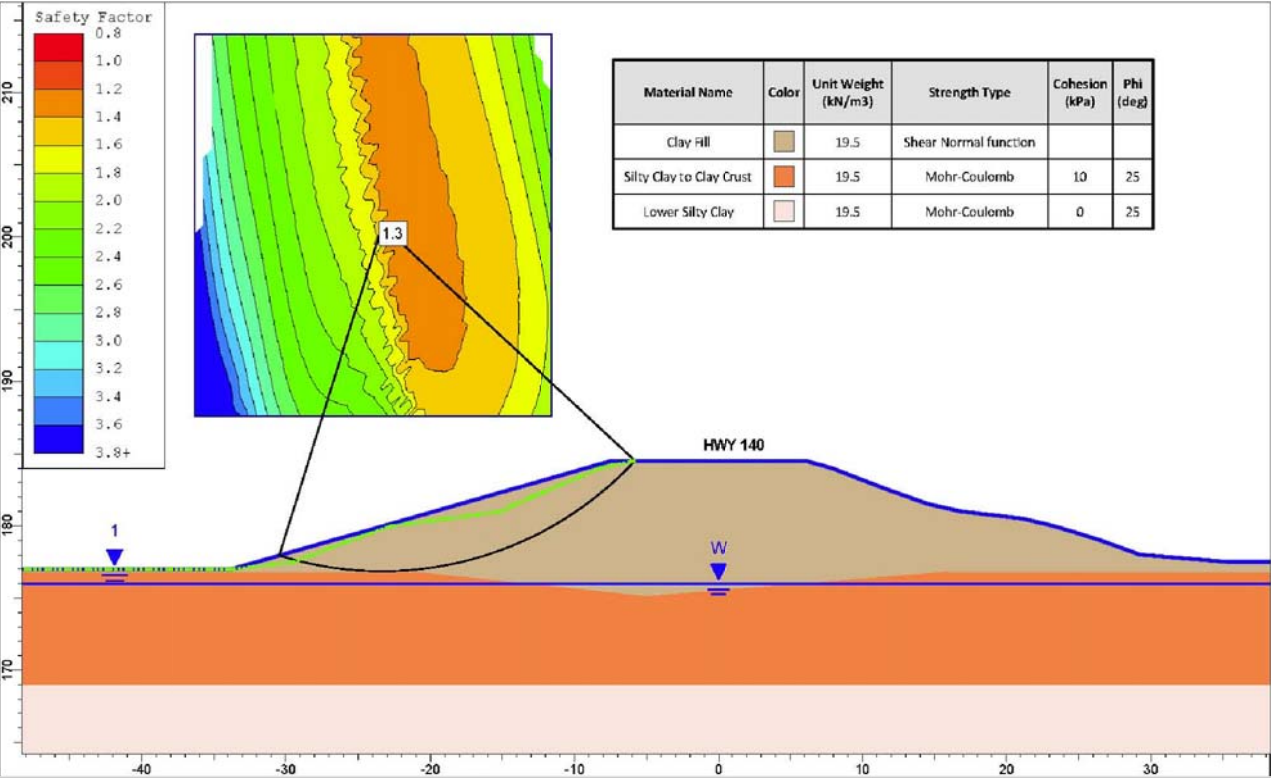


Sta. 15+450 - Standard Benching with Granular Material - Surficial Stability

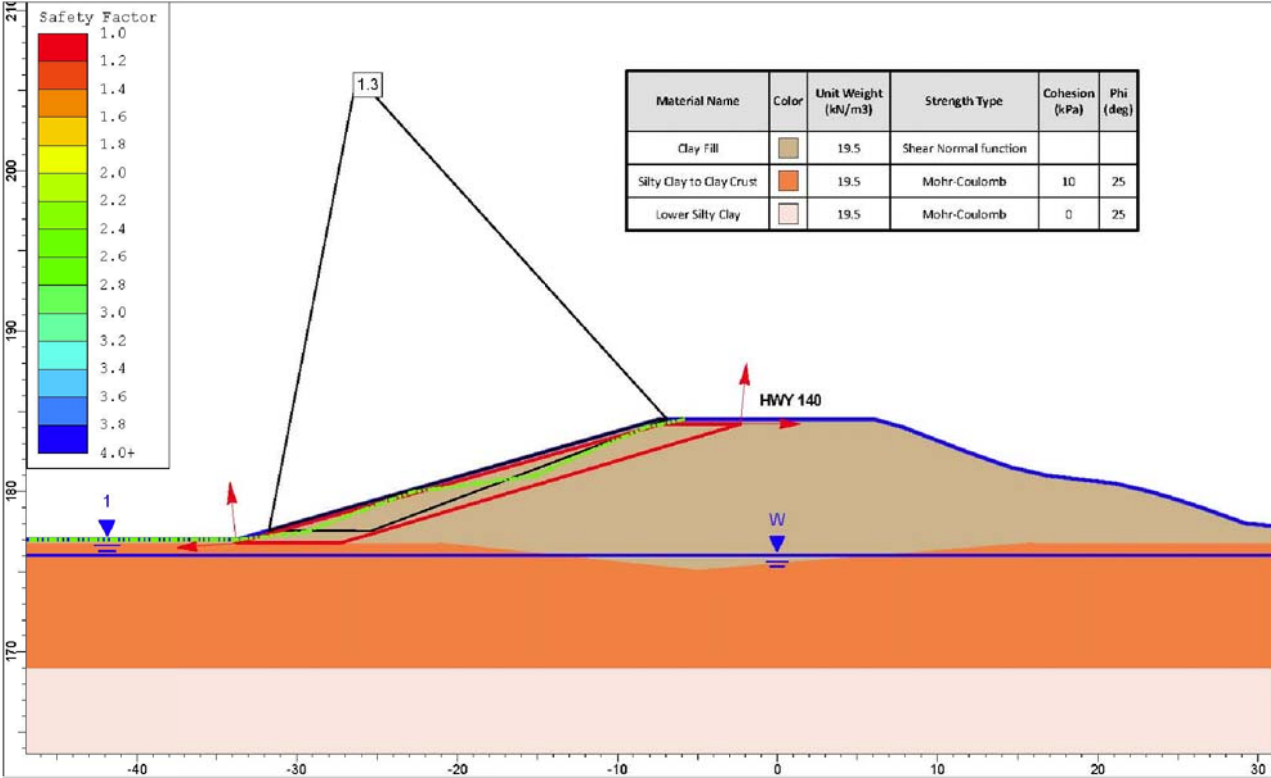


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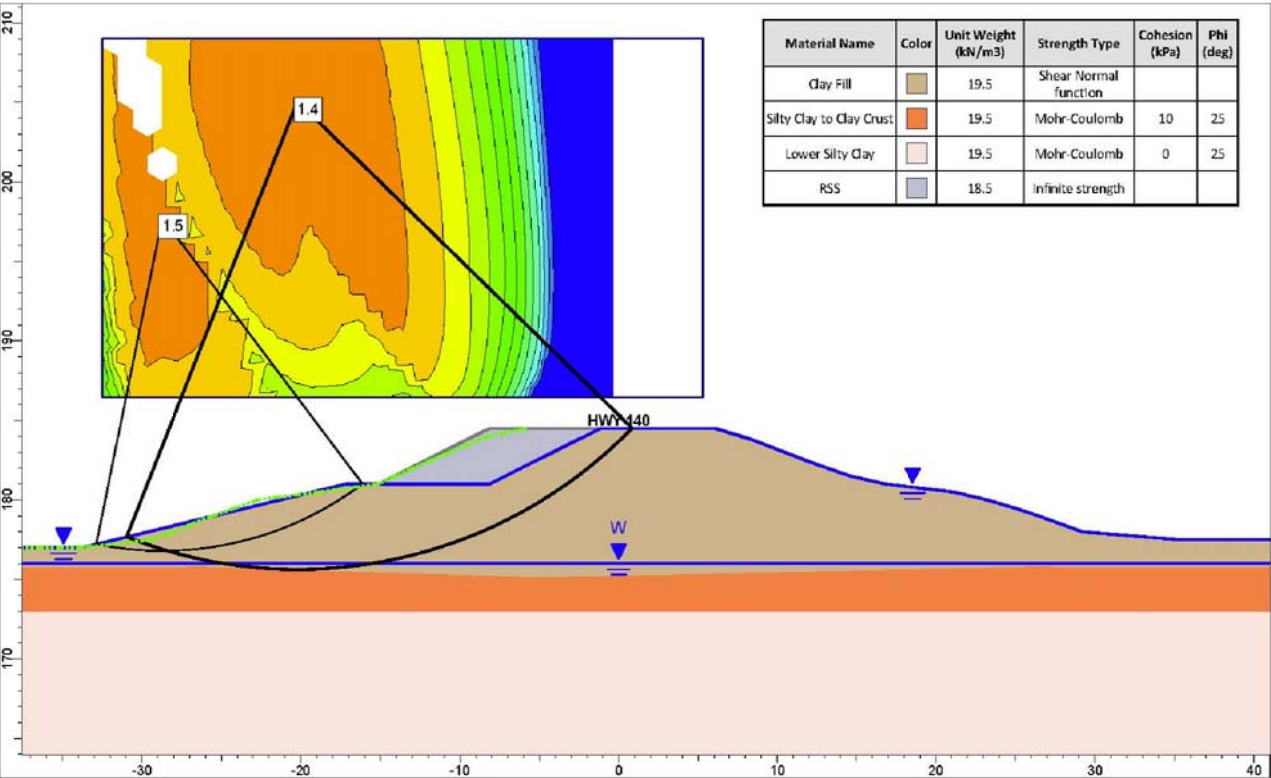
HWY 140 CNR OVERPASS, SOUTH EMBANKMENT SLOPE INSTABILITIES			
REMEDIATION OPTIONS SLOPE STABILITY RESULTS			
G.W.P	2044-13-00	DATE:	August 2014
SUBM'D.	HA	CHKD.	RA
Project No:	11-14-4076	Figure	E3




Sta. 15+450 - Slope Flattening with Silty Clay - Global Stability



Sta. 15+450 - Slope Flattening with Silty Clay - Surficial Stability



Sta. 15+450 - Geogrid Side Slope - Global Stability



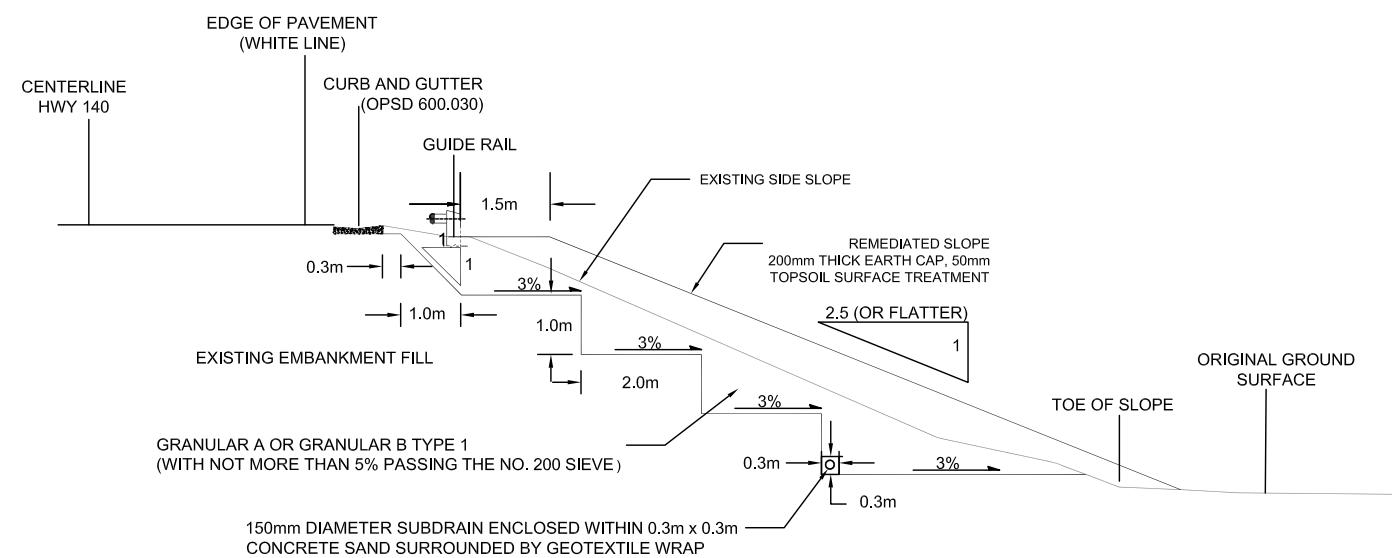
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Construction Materials, Inspection & Testing
11 Indell Lane • Brampton Ontario L6T 3Y3 (905) 796-2650

HWY 140 CNR OVERPASS, SOUTH EMBANKMENT SLOPE INSTABILITIES			
REMEDATION OPTIONS SLOPE STABILITY RESULTS			
G.W.P	2044-13-00	DATE:	August 2014
SUBM'D.	HA	CHKD.	RA
Project No:	11-14-4076	Figure	E4

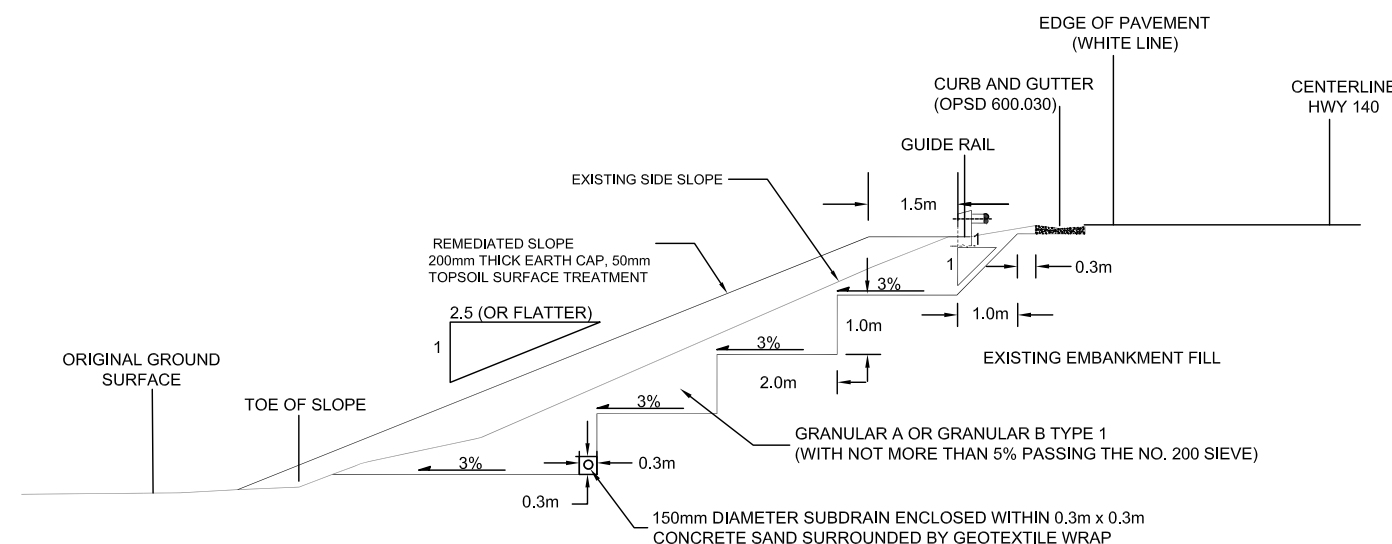
APPENDIX F

Preferred Remediation Details

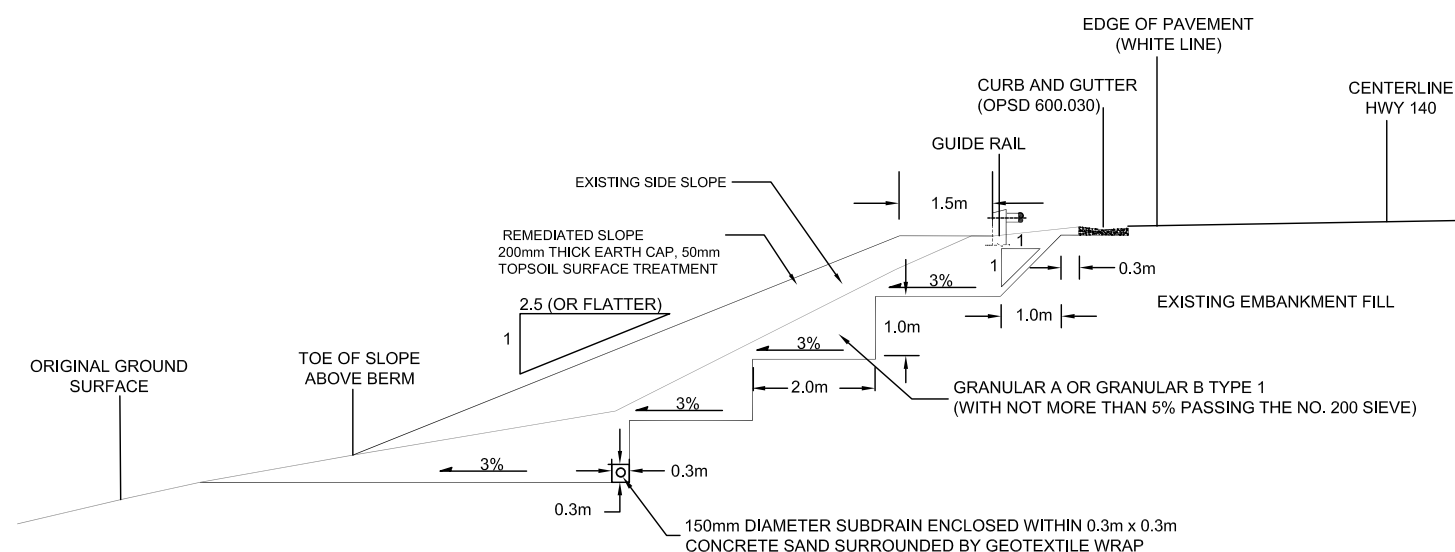




EAST SIDE SLOPE - STA. 15+340 TO STA 15+410



WEST SIDE SLOPE - STA. 15+340 TO STA 15+360



WEST SIDE SLOPE - STA. 15+360 TO STA 15+500



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HWY 140 CNR OVERPASS, SOUTH EMBANKMENT
SLOPE INSTABILITIES

PREFERRED REMEDIATION DETAILS

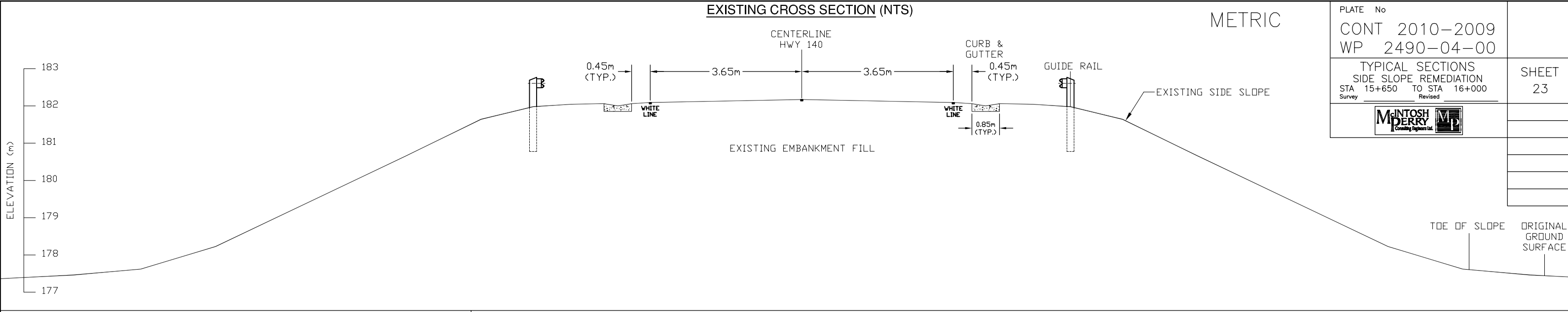
G.W.P. 2044-13-00 DATE: August 2014

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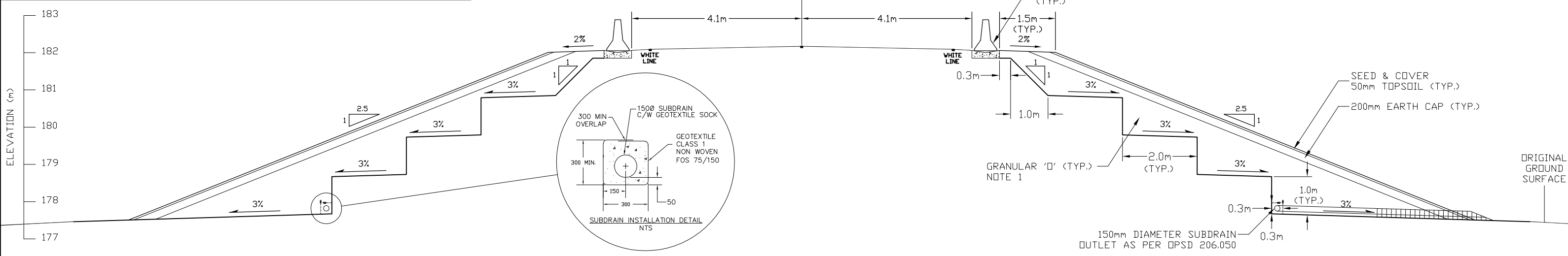
Project No: 11-14-4076 Figure F1

**Contract No. 2010-2009
Relevant Drawings and Specifications
North Embankment Repair Contract**

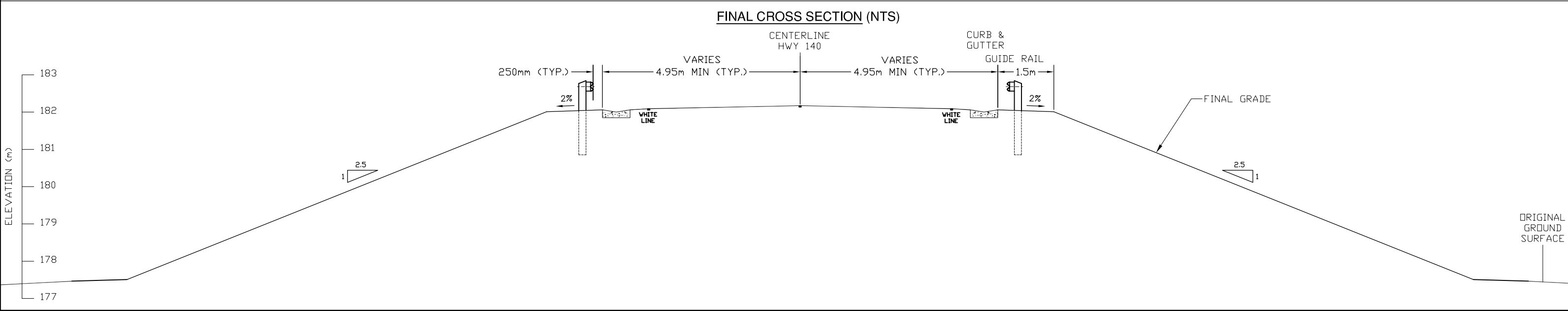


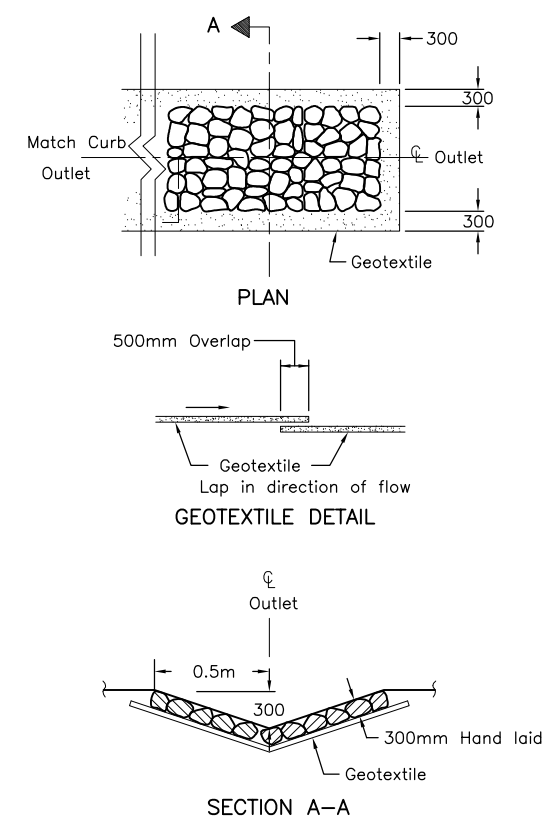
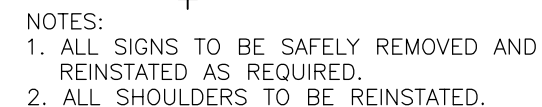


- NOTES: SEQUENCE OF CONSTRUCTION
1. PLACE T.C.B. WITH SINGLE LANE CLOSURES AND CONNECT T.C.B. TO PARAPET WALL WITH STEEL BEAM CONNECTOR.
 2. CONSTRUCT TEMPORARY ACCESS/EGRESS POINTS ±16+000.
 3. CONSTRUCT REMEDIATED SIDE SLOPES.
 4. INSTALL STEEL BEAM GUIDE RAIL WITH CHANNEL, CONNECT TO PARAPET WALL, REMOVE T.C.B. WITH SINGLE LANE CLOSURES.
- RESURFACING OF HWY 140 FROM STATION 15+000 TO NORTH LIMITS OF PAVING SHALL NOT BE CARRIED OUT WHILE T.C.B. IS IN PLACE.

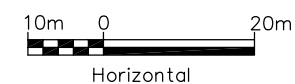


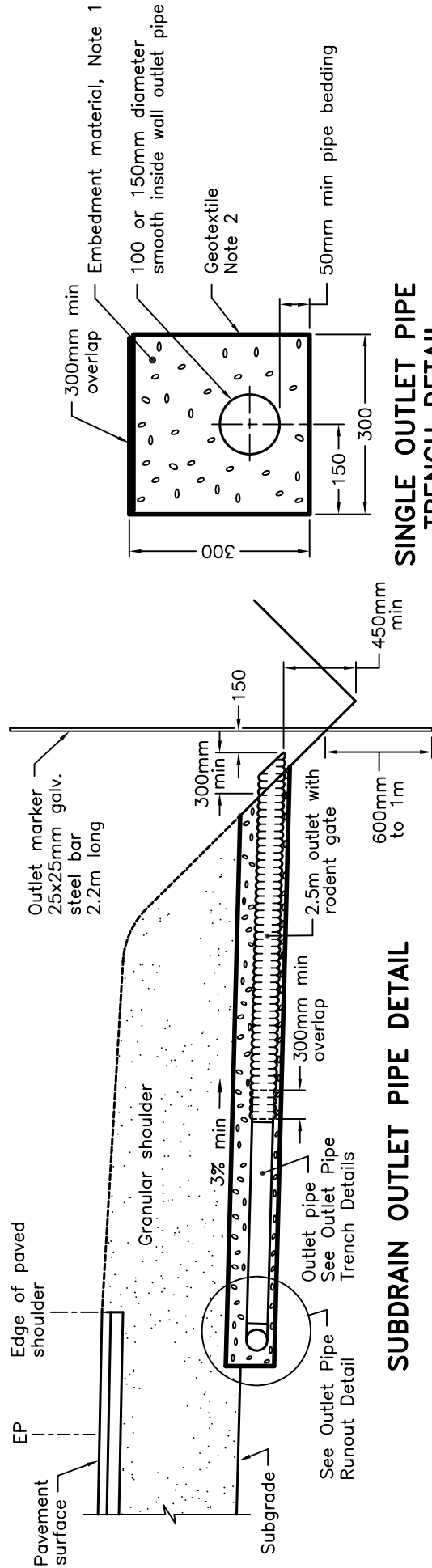
- NOTES: SIDE SLOPE REMEDIATION
1. GRANULAR 'A' NOT TO HAVE MORE THAN 5% PASSING NO. 200 SIEVE.
 2. GRANULAR FILL TO BE PLACED IN LIFTS NOT EXCEEDING 300mm LOOSE THICKNESS.
 3. SUBDRAINS TO HAVE OUTLETS AT 25m CENTERS.
 4. STRIP TOPSOIL – 150mm



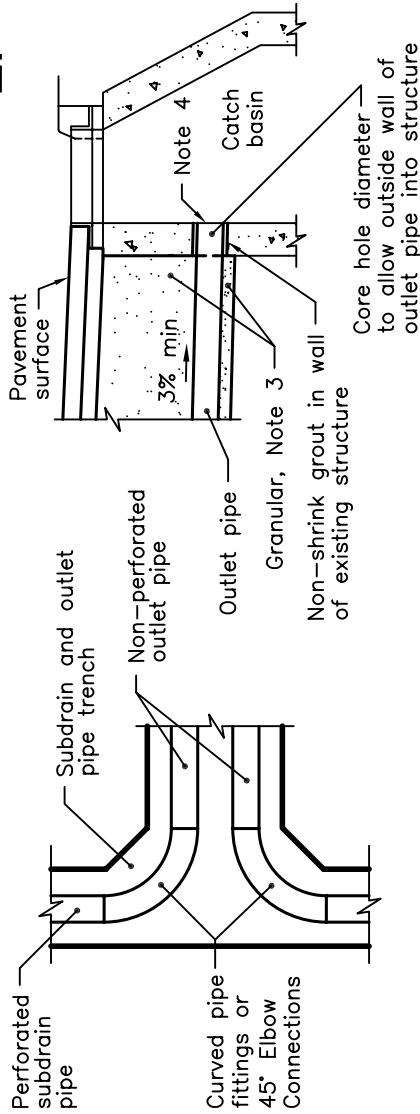


RIP-RAP TREATMENT
NTS





SUBDRAIN OUTLET PIPE DETAIL



OUTLET PIPE RUNOUT DETAIL

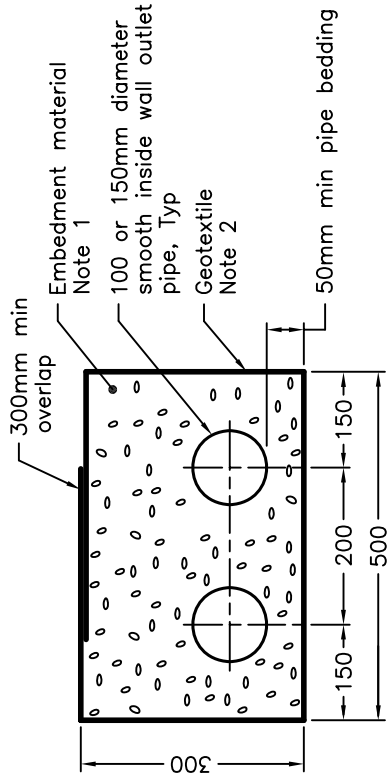
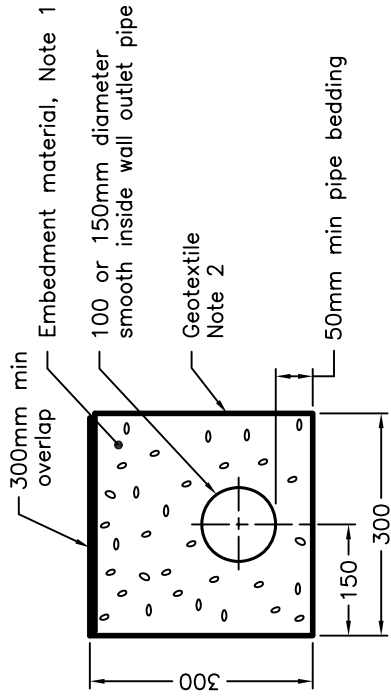
OUTLET PIPE CONNECTION TO CATCH BASIN DETAIL

NOTES:

- 1 Embedment material shall be 19mm clear stone or granular as specified.
- 2 Outlet pipe and subdrain trench embedment material shall be wrapped with geotextile.
- 3 For catch basin connections, outlet pipe trenches backfilled with granular do not require geotextile wrap.
- 4 Install outlet pipe flush with inside face of catch basin.

- A Use compatible manufactured fittings for all connectors and couplings.
 B All dimensions are in millimetres unless otherwise shown

SINGLE OUTLET PIPE TRENCH DETAIL



DUAL OUTLET PIPE TRENCH DETAIL

ONTARIO PROVINCIAL STANDARD DRAWING

**SUBDRAIN PIPE
CONNECTION AND OUTLET**
RURAL

Nov 2008 Rev 1



OPSD 206.050

GRANULAR A – Item No.
GRANULAR A, STOCKPILED – Item No.
GRANULAR A, FROM STOCKPILE – Item No.
GRANULAR B TYPE I – Item No.
GRANULAR B TYPE I, STOCKPILED – Item No.
GRANULAR B TYPE I, FROM STOCKPILE – Item No.
GRANULAR B TYPE II – Item No.
GRANULAR B TYPE II, STOCKPILED – Item No.
GRANULAR B TYPE II, FROM STOCKPILE – Item No.
GRANULAR B TYPE III – Item No.
GRANULAR B TYPE III, STOCKPILED – Item No.
GRANULAR B TYPE III, FROM STOCKPILE – Item No.
GRANULAR M – Item No.
GRANULAR M, STOCKPILED – Item No.
GRANULAR M, FROM STOCKPILE – Item No.
GRANULAR O – Item No.
GRANULAR O, STOCKPILED – Item No.
GRANULAR O, FROM STOCKPILE – Item No.
SELECT SUBGRADE MATERIAL, COMPACTED – Item No.

Special Provision No. 314S03

August 2007

OPSS 314, Construction Specification for Untreated Granular Subbase, Base, Surface, Shoulder, and Stockpiling (December, 1993) is amended as follows:

OPSS 314 is amended by the addition of Section 314.03 as follows:

314.03 DEFINITIONS

For the purpose of this specification the following definitions shall apply:

Tolerance – Minus: a construction working tolerance only which:

- a) Means narrower than the contract standard pertaining to horizontal dimensions as measured from centreline; and
- b) Means lower in elevation than the contract standard pertaining to vertical dimensions.

Tolerance – Plus: a construction working tolerance only which:

- a) Means wider than the contract standard pertaining to horizontal dimensions as measured from centreline; and
- b) Means higher in elevation than the contract standard pertaining to vertical dimensions.

314.07 CONSTRUCTION

Subsection 314.07.04 of OPSS 314 is deleted and replaced with the following:

314.07.04 Shoulders

Granular shouldering material shall be placed and compacted in locations and to the line, grade and cross-section specified in the Contract Documents.

All debris and deleterious material shall be removed from the shoulder area before commencing shoulder construction.

Shouldering operations shall commence as soon as, but not before, the following pavement conditions will permit:

a) **Bituminous Pavements:**

Placement of granular material for shouldering operations shall not commence along any section of pavement until 24 hours have elapsed from the time of completion of the final bituminous pavement course in that section. The shouldering operations shall be completed within the next 24 hours on sections that are open to traffic. Where the pavement is not open to traffic, the shouldering shall be completed before traffic is permitted.

b) **Concrete Pavements, Concrete Base, and Lean Concrete Base:**

Shouldering operations shall commence in accordance with OPSS 350. Shouldering shall be completed before opening the concrete base or concrete pavement to traffic.

All shoulder construction material shall be conveyed from the transport vehicle onto the shoulder area. End dumping of shoulder construction material directly onto the adjacent pavement surface or directly onto the shoulder shall not be permitted. The material shall then be uniformly distributed within the specified shoulder width without segregation. Grading and shaping shall confine all material to within the specified shoulder limits without overspill. Any aggregate dragged onto the pavement surface shall be removed immediately and the pavement surface shall be thoroughly cleaned with the use of a power broom or other suitable means.

Operation of equipment shall not cause any damage to the pavement.

Subsection 314.07.06 of OPSS 314 is amended as follows:

The title, Surface Tolerances of subsection 314.07.06 is deleted and replaced with the following:

314.07.06 Tolerances

314.07.06.01 General

Clause 314.07.06.01 of OPSS 314 is amended by the addition of the following:

In the event of a conflict between meeting horizontal grading tolerances and meeting vertical grading tolerances, the vertical grading tolerances shall take precedence.

Clause 314.07.06.02 of OPSS 314 is deleted and replaced with the following:

314.07.06.02 Tolerances for Granular Courses

All granular grade surfaces shall, on completion, be shaped to the specified line, grade and cross section within the following tolerances, and the surface shall not deviate more than 15 mm at any place along a 3 m straightedge.

- a) Vertical grading tolerances of the finished granular base, subbase and shoulder:
 - + 30 mm
 - 30 mm

- b) Horizontal grading tolerances of the finished granular base, subbase and shoulder:
 - + 30 mm
 - 0 mm

Section 314.07 of OPSS 314 is amended by the addition of the following subsection:

314.07.08 Quality Control

314.07.08.01 Submission of Grade Checks

The Contract Administrator shall be notified within 12 hours when each granular course, including shoulders, has been completed, and prior to the next course being placed.

All Contractor grade checks relating to vertical and horizontal grading tolerances, including all non-compliances shall be submitted to the Contract Administrator within 2 business days following completion of each grade check.

314.07.08.02 Certification of Grade

The grade shall be certified at the stations and offsets shown on the Construction Grading Report. Minimum frequency requirements shall be those provided for layout in Tables 1 & 2, Section GC 7.02 of the Ministry of Transportation (Ontario) General Conditions.

The Contractor shall sign and certify on the grading template that the component(s) of the work indicated on the grading template have been correctly constructed as to the specified line and are within grading tolerances. If no grading template is available, the Contractor shall complete, sign and submit form PH-CC-820 "Certification of Grade Elevation/Crossfall" to the Contract Administrator.

Where the finished granular grade or cross section does not meet the acceptance criteria, the granular course shall be brought to grade within the specified vertical and horizontal tolerance.

OPSS 314 is amended by the addition of Section 314.08 as follows:

314.08 QUALITY ASSURANCE

The Owner may conduct random Quality Assurance grade checks (station, actual elevation and offset), to verify vertical and horizontal grading tolerances after notification by the Contractor that the granular course has been completed. During the grade check, the width of granular placement shall be checked and recorded, and when the horizontal tolerances are exceeded, the actual elevations and distances shall be recorded. When such checking is undertaken, the Contract Administrator shall notify the Contractor of any overbuilding.

The Contractor will be charged \$250 for each Quality Assurance check of the finished grade with results outside of specification limits. All grading carried out by the Contractor as a result of Quality Assurance grade checks shall be performed at no additional cost to the Owner.

314.09 MEASUREMENT FOR PAYMENT

The title, Granular A, B, Type I and II, M, Stockpiled and from Stockpile, and Select Subgrade Material of subsection 314.09.01 is deleted and replaced with the following:

314.09.01 Granular A, B Type I, B Type II, B Type III, M, O, Stockpiled and from Stockpile, and Select Subgrade Material

314.09.01.01.01 Tonne

Subsection 314.09.01.01.01 of OPSS 314 is amended in that the total measured mass of air-cooled iron blast furnace slag incorporated into the work shall be multiplied by a factor of 1.116.

314.10 BASIS OF PAYMENT

Subsection 314.10.01.01.01 of OPSS 314 is deleted in its entirety and replaced by the following:

**314.10.01 Granular A - Item
Granular A, Stockpiled – Item
Granular A, from Stockpile – Item
Granular B Type I – Item
Granular B Type I, Stockpiled – Item
Granular B Type I, from Stockpile – Item
Granular B Type II – Item
Granular B Type II, Stockpiled – Item
Granular B Type II, from Stockpile – Item
Granular B Type III – Item
Granular B Type III, Stockpiled - Item
Granular B Type III, from Stockpile – Item
Granular M – Item
Granular M, Stockpiled – Item
Granular M, from Stockpile – Item
Granular O – Item
Granular O, Stockpiled – Item
Granular O, from Stockpile – Item
Select Subgrade Material, Compacted – Item**

Payment at the contract price for the above item(s) shall be full compensation for all labour, equipment and materials required to do the work.

No payment will be made for overbuilding as identified in section 314.08. Where the finished granular course width exceeds the contract horizontal tolerance in subsection 314.07.06, the Owner will deduct from payment that quantity of material outside the tolerance. The conversion for deduction purposes shall be based on 2.0 t/m³ for all types of granular material.

Payment for grade checks, including provision for labour, equipment and materials to conduct Quality Control testing, shall be included in the contract price as part of the work of placing the material.

Subsection 314.10.02 of OPSS 314 is deleted in its entirety and replaced by the following:

314.10.02 From Stockpile

Compensation for clearing, grubbing, stripping, cleanup of the stockpile site, and for supplying and placing a pad upon which the materials are to be stockpiled shall be included as part of the granular item cost.

Compensation for the cleanup of the stockpile site on completion of the operation, when required, shall be included as part of the granular item cost.



CONSTRUCTION SPECIFICATION FOR RIP-RAP, ROCK PROTECTION, AND GRANULAR SHEETING

TABLE OF CONTENTS

511.01	SCOPE
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511.03	DEFINITIONS - Not Used
511.04	DESIGN AND SUBMISSION REQUIREMENTS - Not Used
511.05	MATERIALS
511.06	EQUIPMENT - Not Used
511.07	CONSTRUCTION
511.08	QUALITY ASSURANCE - Not Used
511.09	MEASUREMENT FOR PAYMENT
511.10	BASIS OF PAYMENT

APPENDICES

511-A	Commentary
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511.01 SCOPE

This specification covers the requirements for the installation of rip-rap, rock protection, granular sheeting, and geotextile.

511.01.01 Specification Significance and Use

This specification has been developed for use in provincial- and municipal-oriented Contracts. The administration, testing, and payment policies, procedures, and practices reflected in this specification correspond to those used by many municipalities and the Ontario Ministry of Transportation.

Use of this specification or any other specification shall be according to the Contract Documents.

511.01.02 Appendices Significance and Use

Appendices are not for use in provincial contracts as they are developed for municipal use, and then, only when invoked by the Owner.

Appendices are developed for the Owner's use only.

Inclusion of an appendix as part of the Contract Documents is solely at the discretion of the Owner. Appendices are not a mandatory part of this specification and only become part of the Contract Documents as the Owner invokes them.

Invoking a particular appendix does not obligate an Owner to use all available appendices. Only invoked appendices form part of the Contract Documents.

The decision to use any appendix is determined by an Owner after considering their contract requirements and their administrative, payment, and testing procedures, policies, and practices. Depending on these considerations, an Owner may not wish to invoke some or any of the available appendices.

511.02 REFERENCES

When the Contract Documents indicate that provincial-oriented specifications are to be used and there is a provincial-oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.PROV, unless use of a municipal-oriented specification is specified in the Contract Documents. When there is not a corresponding provincial-oriented specification, the references below shall be considered to be to the OPSS listed, unless use of a municipal-oriented specification is specified in the Contract Documents.

When the Contract Documents indicate that municipal-oriented specifications are to be used and there is a municipal-oriented specification of the same number as those listed below, references within this specification to an OPSS shall be deemed to mean OPSS.MUNI, unless use of a provincial-oriented specification is specified in the Contract Documents. When there is not a corresponding municipal-oriented specification, the references below shall be considered to be the OPSS listed, unless use of a provincial-oriented specification is specified in the Contract Documents.

This specification refers to the following standards, specifications, or publications:

Ontario Provincial Standard Specifications, Material

OPSS 1004	Aggregates - Miscellaneous
OPSS 1860	Geotextiles

511.05 MATERIALS

511.05.01 Rip-Rap, Rock Protection, and Granular Sheeting

Rip-rap shall be according to OPSS 1004 and as specified in the Contract Documents.

Rock protection and granular sheeting shall be according to OPSS 1004.

511.05.02 Geotextile

Geotextile shall be nonwoven, Class II according to OPSS 1860, with an FOS of 75-150 mm.

511.07 CONSTRUCTION

511.07.01 Excavation

Prior to placing any material, the area shall be excavated to the lines and dimensions specified in the Contract Documents and fine graded to a uniform even surface. Depressions shall be filled and compacted with acceptable material.

511.07.02 Placing Material

511.07.02.01 General

Material shall be placed to the lines and dimensions specified in the Contract Documents.

On slopes where rip-rap or rock protection is being placed, the rock shall commence at the toe of the slope and progress up the slope.

When geotextile is specified in the Contract Documents, rip-rap, rock protection, and granular sheeting shall be placed in a manner as not to tear or damage the geotextile.

511.07.02.02 Rip-Rap

Rip-rap shall be placed in a set and stable manner, flat on the slope with the largest dimension parallel to the slope contours. The larger pieces of rip-rap shall be placed in the bottom courses. The rip-rap shall be laid closely such that a reasonable semblance of courses is achieved. Smaller pieces of rip-rap shall be used to fill the voids.

511.07.02.03 Rock Protection

Rock protection shall be placed in a random but stable manner.

511.07.02.04 Granular Sheeting

Granular sheeting operations shall follow earth excavation operations as closely as practical and possible.

Compaction of granular sheeting material is not required.

511.07.02.05 Geotextile

Geotextile shall be free of folds, tears and wrinkles and as specified in the Contract Documents. The geotextile shall be joined so that the material laps a minimum of 500 mm and shall be pinned together. Alternatively, the geotextile shall be joined to conform to the seam requirements of OPSS 1860.

Geotextiles shall be fixed to prevent movement during installation. Geotextile shall be wrapped down into the ground a minimum 300 mm at termination points.

511.07.03 Management of Excess Material

Management of excess material shall be according to the Contract Documents.

511.09 MEASUREMENT FOR PAYMENT

511.09.01 Actual Measurement

511.09.01.01 Rip-Rap

Measurement of rip-rap shall be by area in square metres following the contour of the ground.

511.09.01.02 Rock Protection

Measurement of rock protection shall be by volume in cubic metres. The volume is a product of the area of the rock protection, measured following the contour of the ground, by its depth.

Truck box measurement shall be used when it is not possible to take an in place measurement.

511.09.01.03 Granular Sheeting

Measurement of granular sheeting shall be by area in square metres following the contour of the ground.

511.09.01.04 Geotextile

Measurement of geotextile shall be by area in square metres following the contour of the ground, with no allowance made for overlaps.

511.09.02 Plan Quantity Measurement

When measurement is by Plan Quantity, such measurement shall be based on the units shown in the clauses under Actual Measurement.

511.10 BASIS OF PAYMENT

**511.10.01 Rip-Rap - Item
Rock Protection - Item
Granular Sheeting - Item
Geotextile - Item**

Payment at the Contract price for the above tender items shall be full compensation for all labour, Equipment, and Materials to do the work.

When there is not a separate tender item for geotextile, payment for the geotextile shall be included in the tender item for rip rap, rock protection, or granular sheeting, as appropriate.

Appendix 511-A, November 2010
FOR USE WHILE DESIGNING MUNICIPAL CONTRACTS

Note: This is a non-mandatory Commentary Appendix intended to provide information to a designer, during the design stage of a contract, on the use of the OPS specification in a municipal contract. This appendix does not form part of the standard specification. Actions and considerations discussed in this appendix are for information purposes only and do not supersede an Owner's design decisions and methodology.

Designer Action/Considerations

The designer should specify the following in the Contract Documents:

- Gradation requirements of rip-rap as per Table 5 of OPSS 1004. (511.05.01)
- Lines and dimensions for excavation of rip-rap, rock protection, or granular sheeting areas. (511.07.01)
- Lines and dimension for placement of material. (511.07.02.01)
- Geotextile locations, if required. (511.07.02.01)
- Placement requirements for geotextile. (511.07.02.05)

The designer should ensure that the General Conditions of Contract and the 100 Series General Specifications are included in the Contract Documents.

Related Ontario Provincial Standard Drawings

OPSD 810.010	Rip-Rap Treatment, for Sewer and Culvert Outlets
OPSD 810.020	Rip-Rap Treatment, for Ditch Inlets