

Terraprobe

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

**FOUNDATION INVESTIGATION AND DESIGN REPORT
ANTHONY CREEK CULVERT REPLACEMENT
SITE 39E-0212/C0 - HIGHWAY 101
ASSIGNMENT No. 5016-E-0023 - G.W.P. 5165-12-00
MINISTRY OF TRANSPORTATION, ONTARIO
NORTHEASTERN REGION
GEOCRES NO. 42A-140**

PREPARED FOR: WSP Canada Inc.
610 Chartwell Road, Suite 300
Oakville, Ontario
L6J 4A5

File No. 1-17-0822
October 21, 2021
Terraprobe Inc.

Distribution:

- 1 Copy - MTO Project Manager (Northeastern Region)
- 1 Copy - MTO Structural Section (Northeastern Region)
- 1 Copy - MTO Pavements and Foundations Section (Downsview)
- 1 Copy - WSP Canada Inc., Oakville
- 1 Copy - Terraprobe Inc., Brampton

Terraprobe Inc.

Greater Toronto

11 Indell Lane
Brampton, Ontario L6T 3Y3
(905) 796-2650 Fax: 796-2250
brampton@terraprobe.ca

Hamilton – Niagara

903 Barton Street, Unit 22
Stoney Creek, Ontario L8E 5P5
(905) 643-7560 Fax: 643-7559
stoneycreek@terraprobe.ca

Central Ontario

220 Bayview Drive, Unit 25
Barrie, Ontario L4N 4Y8
(705) 739-8355 Fax: 739-8369
barrie@terraprobe.ca

Northern Ontario

1012 Kelly Lake Rd., Unit 1
Sudbury, Ontario P3E 5P4
(705) 670-0460 Fax: 670-0558
sudbury@terraprobe.ca

www.terraprobe.ca

TABLE OF CONTENTS

PART A – FOUNDATION INVESTIGATION REPORT	I
1.0 INTRODUCTION	1
2.0 SITE DESCRIPTION	1
3.0 INVESTIGATION PROCEDURES	1
4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS.....	2
4.1 Regional Geology.....	2
4.2 Subsurface Conditions	3
4.2.1 Topsoil.....	3
4.2.2 Flexible Pavement.....	3
4.2.3 Fill – Sand and Gravel to Gravelly Sand	3
4.2.4 Fill – Silty Sand to Sand	4
4.2.5 Fill – Silty Clay.....	4
4.2.6 Peat.....	5
4.2.7 Silty Clay to Clay	5
4.2.8 Clayey Silt	6
4.2.9 Silt	7
4.2.10 Silty Sand to Sandy Silt	7
4.3 Ground Water Levels	8
5.0 MISCELLANEOUS	8
PART B – FOUNDATION DESIGN REPORT	II
6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS	9
6.1 General	9
6.2 Foundation Alternatives.....	9
6.3 Consequence and Site Understanding Classification.....	9
6.4 Seismic Design.....	10
6.4.1 Importance Category & Seismic Site Classification.....	10
6.4.2 Spectral Response Values	10
6.5 Geotechnical Resistances (Geogrid Reinforced Engineered Fill Pad)	10
6.6 Horizontal Geotechnical Resistances.....	11
6.7 Lateral Earth Pressure	12
6.7.1 Static Conditions	12
6.7.2 Seismic Conditions.....	12
6.8 Design Frost Depth	13
6.9 Erosion Protection	13
6.10 Culvert Bedding and Backfill	14
6.11 Global Stability Assessments	14
6.11.1 General	14
6.11.2 Static Conditions (Permanent Embankments).....	15
6.11.3 Seismic Conditions (Permanent Embankments)	15
6.11.4 Stability of Temporary Excavations	16



6.12	Settlement	16
6.12.1	General	16
6.12.2	Culvert Settlement.....	17
6.13	Embankment Construction	18
6.14	Excavations	18
6.15	Temporary Protection Systems	19
6.16	Ground Water Control	20
6.17	Soil Corrosivity and Sulphate Test Results	20
7.0	CLOSURE	20

REFERENCES

LIST OF DRAWINGS

Drawing 1	Borehole Locations and Soil Strata
Drawing 2	Site Photographs

LIST OF APPENDICES

APPENDIX A Record of Borehole Sheets

List of Symbols and Abbreviations
Record of Borehole Sheets – BH1, BH2, BH3, BH4 and BH5.

APPENDIX B Field and Laboratory Test Results and Photographs

Figure B1	Grain Size Distribution – Fill – Sand and Gravel to Gravelly Sand
Figure B2	Grain Size Distribution – Fill - Silty Sand to Sand
Figure B3	Grain Size Distribution – Fill - Silty Clay
Figure B4	Plasticity Chart – Fill - Silty Clay
Figure B5	Photographs of Varved Silty Clay to Clay
Figure B6	Silty Clay to Clay/Clayey Silt – Plot of Undrained Shear Strength versus Elevation
Figures B7 to B9	Grain Size Distribution – Silty Clay to Clay
Figures B10 to B12	Plasticity Chart – Silty Clay to Clay
Figure B13	Silty Clay to Clay/Clayey Silt – Atterberg Limits and Water Contents versus Elevation
Figures B14 to B17	Silty Clay to Clay – One Dimensional Consolidation Test Results
Figures B18 to B21	Silty Clay to Clay – Consolidated Undrained Triaxial Compression Test Results
Figure B22	Grain Size Distribution – Clayey Silt
Figure B23	Plasticity Chart – Clayey Silt
Figure B24	Grain Size Distribution – Silt
SGS Final Reports	Laboratory Test Results – Soil Chemistry

APPENDIX C Soil Design Parameters

APPENDIX D Slope Stability Models and Results

APPENDIX E Operational Constraint and Special Provisions

Special Provision, Geogrid
Special Provision, Geotechnical Assessment
Operational Constraint

APPENDIX F Preload Details



PART A – FOUNDATION INVESTIGATION REPORT

**ANTHONY CREEK CULVERT REPLACEMENT, SITE 39E-0212/C0
HIGHWAY 101
TOWNSHIP OF TAYLOR, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5016-E-0023, G.W.P. 5165-12-00**



1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by WSP Canada Inc. (WSP) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of detailed designs for the Anthony Creek culvert replacement.

This project is based on the Ministry of Transportation, Ontario (MTO) Request for Proposal titled “*Detail Design for the Replacement of Anthony Creek culvert (Site 39E-212), Unknown Creek culvert (Site 39E-213) and Shillington Creek culvert (Site 39E-214), Highway 101, Agreement 5016E-0023*”. The terms of reference and scope of work for the foundation engineering services are outlined in MTO’s Request for Proposal.

This report presents the factual data on the subsurface conditions at the Anthony Creek culvert Site 39E-212 on Highway 101, Township of Taylor, District of Cochrane, Ontario.

2.0 SITE DESCRIPTION

The site (Latitude 48.538°; Longitude -80.572°) is located on Highway 101, approximately 1.5 km west of the highway’s north intersection with Highway 11 in the Township of Taylor, Ontario. The key plan on the Borehole Locations and Soil Strata Drawing, (Drawing 1) provides an overview of the site location and site photos are provided in Drawing 2.

The existing concrete box culvert located at Station 11+620 is 30.5 ±m long and its cross-section is 3.0 m wide and 1.8 m high. The Highway 101 embankment is approximately 6.4 m high at the culvert crossing with a highway centre line elevation of 266.8 ±m.

The terrain is generally rolling to gently undulating with vegetation consisting mainly of grass and shrubs and occasional deciduous and coniferous trees. The culvert conveys the Anthony Creek flow from south to north below Highway 101 and a grave is located at the north west corner of the site.

3.0 INVESTIGATION PROCEDURES

The field work for this project was carried out between April 10 and May 01, 2018 and consisted of drilling and sampling five boreholes to depths ranging from 24.8 m to 31.7 m below ground surface. The approximate borehole locations are shown on Drawing 1.

Terraprobe’s staff staked out the borehole locations in the field relative to on-site features and, WSP surveyors established Horizontal Control Point 111 with a geodetic elevation of 266.380 m. The data from this horizontal control point was used by Terraprobe’s staff to determine the ground surface elevations and coordinates of Boreholes 1 to 5. The borehole data is summarized in the following table.

Borehole No.	MTM NAD 83 Coordinates (Zone 12)		Ground Surface Elevation (m)	Depth (m)
	Northing (m)	Easting (m)		
BH1	5 377 792.9	336 427.4	261.8	24.8
BH2	5 377 779.8	336 417.2	266.6	29.6
BH3	5 377 756.5	336 419.5	262.1	24.8
BH4	5 377 778.4	336 445.9	266.6	30.2
BH5	5 377 773.3	336 394.1	266.9	31.7



The boreholes were drilled with truck-mounted and track-mounted drill rigs supplied and operated by a specialist drilling contractor. Terraprobe's staff observed and recorded the drilling, sampling and in situ testing operations and logged the boreholes.

Samples of the overburden soils were generally obtained at intervals of 0.75 m and 1.5 m depth using a 50 mm outer diameter (O.D.) split-spoon sampler in conjunction with the Standard Penetration Testing (SPT) procedures as specified in ASTM Method D 1586¹. Relatively undisturbed samples of the clay soils were also collected with thin-walled Shelby Tube samplers. In the clay deposits an MTO 'N' vane was used to perform in-situ field vane tests, in order to determine the undrained shear strength of the soil. In Borehole 1, cobbles were encountered within the embankment fill and NQ-size diamond coring techniques were used to extend the borehole through the cobbles. Dynamic Cone Penetration tests were also performed in Boreholes 4 and 5.

Ground water conditions in the open boreholes were observed throughout the drilling operations. Artesian conditions were encountered in Boreholes 1 and 3 and the boreholes were backfilled immediately and sealed in accordance with current MTO procedures and Ontario Regulation 903 (as amended). A standpipe piezometer was installed 1.8 m east of Borehole 1 to permit longer term ground water level monitoring.

The recovered soil samples were subjected to Visual Identification (VI) and select soil samples were also subjected to a laboratory testing programme consisting of natural moisture content, grain size distribution analyses, Atterberg limits determinations, one-dimensional consolidation testing, and consolidated undrained triaxial testing in accordance with MTO and/or ASTM Standards as appropriate. Soil samples were also submitted to SGS Canada Inc. (SGS) for chemical testing.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The surficial geology of the study area generally consists of glaciolacustrine sediments of the Barlow-Ojibway Formation and Matheson Till². Fine-grained glaciolacustrine sediment, consisting of clay and silt, blankets most of the area and is on average 10 m to 15 m thick. Coarse-grained glaciolacustrine sediments, consisting of sand and minor gravel, are found mainly along the flanks of the major esker complexes and on bedrock uplands. The Matheson Till (deposited during the Late Wisconsinan period) is a silty sand till that varies in thickness from thin bands a few centimetres thick up to a maximum of 30 m. Most commonly, the Matheson Till is found beneath a thick cover of glaciolacustrine deposits.

The study area lies within the Abitibi Greenstone Belt of the Superior structural province of the Canadian Shield. The Abitibi Greenstone Belt consists of both volcanic and sedimentary rocks though typically dominated by mafic metavolcanic rocks. Several felsic and alkaline intrusions occur throughout the area.

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

2 McClenaghan, M.B. 1990. Summary of results from the Black River – Matheson (BRiM) reconnaissance surface till sampling program; Ontario Geological Survey, Open File Report 5749, p. 197.



4.2 Subsurface Conditions

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

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

In summary, the highway pavement is generally underlain by fill material consisting of compact to very dense sand and gravel to gravelly sand, compact to dense silty sand to sand and, very soft to hard silty clay. The native overburden deposits consist of amorphous peat, soft to stiff varved silty clay to clay, soft to very stiff clayey silt, loose to dense silt and, compact silty sand to sandy silt. A more detailed description of the subsurface conditions is provided in the following sections.

4.2.1 Topsoil

A 150 mm thick topsoil layer was encountered at Borehole 3. Topsoil thickness may vary between and beyond the boreholes.

4.2.2 Flexible Pavement

Boreholes 2, 4 and 5 were drilled through the Highway 101 lanes and paved shoulder. These boreholes encountered a flexible pavement consisting of 60 mm to 150 mm thick asphaltic concrete underlain by sand and gravel to gravelly sand granular fill.

4.2.3 Fill – Sand and Gravel to Gravelly Sand

Sand and gravel to gravelly sand fill was encountered at this site underlying the highway pavement and also as embankment fill. The locations, thicknesses, depths and base elevations of the sand and gravel to gravelly sand fill are summarized in the following table.

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH2	1.3	1.4	265.2
BH4	2.3	2.5	264.1
BH5	1.7	1.8	265.1
	1.6	3.9	263.0

Standard Penetration tests performed in the sand and gravel to gravelly sand fill measured SPT N-values that range from 29 to more than 100 blows for 0.3 m of penetration indicating a compact to very dense relative density. The natural water content of samples of the sand and gravel to gravelly sand fill range from 1% to 14% by weight.



Two samples of the sand and gravel to gravelly sand fill were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figure B1 in Appendix B. The results show a grain size distribution consisting of 18% gravel, 37% and 44% sand, 17% and 23% silt, and 15% and 28% clay size particles.

4.2.4 Fill – Silty Sand to Sand

Silty sand to sand fill was encountered in Borehole 2 extending to a depth of 3.7 m (elevation 262.9 m) below ground surface.

Standard Penetration tests performed in the silty sand to sand fill measured SPT N-values of 28 and 41 blows for 0.3 m of penetration indicating a compact to dense relative density. Cobbles were encountered within the silty sand to sand fill and NQ-size diamond coring techniques were used to extend the borehole through the cobbles. The moisture content of a sample of the silty sand to sand fill is 3% by weight.

The grain size distribution curve of a sample of silty sand to sand fill is presented on Figure B2 in Appendix B. The results show a grain size distribution consisting of 17% gravel, 71% sand, 8% silt and, 4% clay size particles.

4.2.5 Fill – Silty Clay

Silty clay fill was encountered at this site, and the locations, thicknesses, depths and base elevations of the silty clay fill are summarized in the following table.

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH1	2.1	2.1	259.7
BH2	0.7	2.1	264.5
	2.2	5.9	260.7
BH3	1.8	2.0	260.1
BH4	1.9	4.4	262.2
BH5	0.5	2.3	264.6

Standard Penetration tests in the silty clay fill measured SPT N-values ranging from 0 blows (weight of hammer) to 54 blows for 0.3 m of penetration indicating a very soft to hard consistency. The natural water content (by weight) of samples of the silty clay fill range from 23% to 35% by weight.

The grain size distribution curves of two samples of the silty clay fill are depicted on Figure B3 in Appendix B. These results show a grain size distribution consisting of 4% and 14% gravel, 6% and 17% sand, 31% and 58% silt and, 32% and 38% clay size particles.

Atterberg limits tests were also carried out on two samples of the silty clay fill and the results are presented in Figure B4 in Appendix B. These values indicate that the fill is a cohesive soil with intermediate plasticity (CI). The results from the Atterberg limits tests are summarized below:

Liquid Limit:	37% and 38%
Plastic Limit:	21% and 24%
Plasticity Index:	13% and 17%
Natural Moisture Content:	30% and 32%



4.2.6 Peat

The fill material at Boreholes 2 and 3 are underlain by 0.2 m and 0.5 m thick layers of amorphous peat that extend to depths of 6.1 m (elevation 260.5 m) and 2.5 m (elevation 259.6 m) respectively.

4.2.7 Silty Clay to Clay

The site is underlain by a varved silty clay to clay deposit. The deposit's structure consists of fine grained clay soils interlayered with silt ranging from 2 mm to 30 mm in thickness. Photographs illustrating the varved clay matrix are provided in Figure B5 in Appendix B. The locations, thicknesses, depths and base elevations of the silty clay to clay deposit are summarized in the following table.

Borehole No.	Thickness (m)	Depth (m)	Base Elevation (m)
BH1	17.2	19.3	242.5
BH2	16.6	22.7	243.9
BH3	16.8	19.3	242.8
BH4	14.2	18.6*	248.0
BH5	14.7	18.6*	248.3

* Dynamic Cone Penetration Test carried out. No soil sampling below this depth.

The N-values of Standard Penetration tests carried out in the silty clay to clay deposit range from 0 blows (weight of hammer) to 9 blows per 0.3 m of penetration. Field vane tests measured in-situ undrained shear strengths that range from 20 kPa to 80 kPa as illustrated on Figure B6 in Appendix B. Based on the undrained shear strength data, the consistency of the silty clay to clay is described as generally soft to stiff. The sensitivity of the silty clay to clay ranges from about 1.2 to 5.5, indicating a low sensitivity to sensitive soil class (April 01, 2018 errata to Canadian Foundation Engineering Manual [CFEM], 2006).

The variation of undrained shear strength with elevation plot depicted in Figure B6 illustrates lower undrained shear strength values ranging from 20 kPa to 38 kPa between elevation 262.0 m and 256.0 m. Below elevation 256.0 m, there is a trend of increasing undrained shear strength values with depth, with shear strength values ranging from about 24 kPa to 80 kPa.

Samples of the silty clay to clay soils were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figures B7 to B9 in Appendix B. The test results show a grain size distribution consisting of 0% gravel, 0% to 1% sand, 4% to 51% silt and, 49% to 96% clay sized particles.

Samples of the silty clay to clay soils were also subjected to Atterberg limits tests and the results are plotted on the plasticity charts, Figures B10 to B12 in Appendix B. The results indicate a cohesive deposit of generally intermediate to high plasticity (CI to CH) with layers of low plasticity clay (CL). The Atterberg limits test results are summarized below.

Liquid Limit:	31% to 66%
Plastic Limit:	18% to 27%
Plasticity Index:	13% to 39%
Natural Moisture Content:	39% to 77%

The Atterberg Limits tests results of the silty clay to clay deposit are also plotted against elevation in Figure B13. These results illustrate that the natural moisture contents of the tested samples are typically



higher than the liquid limits. The moisture content of samples of the silty clay to clay varies between 30% and 77% and the unit weight of two tested samples are 16.2 kN/m³ and 16.8 kN/m³.

A one-dimensional consolidation test was performed on a sample of the silty clay to clay and the results are presented in Figures B14 to B17 in Appendix B. The results of the one-dimensional consolidation test are summarized in the following table.

Borehole/Sample No.	Sample Depth/Elevation (m)	σ'_{vo} (kPa)	σ'_p (kPa)	C_c	C_r	e_o
BH2, Sample 12	9.4/257.2	120.0	120.0	0.742	0.074	1.51

Where: σ'_{vo} = effective overburden pressure (based on a ground water elevation of 261.3 m)
 σ'_p = Preconsolidation pressure;
 C_c = Compression index;
 C_r = Recompression index; and
 e_o = Initial void ratio.

The preconsolidation pressure derived from the consolidation test data is equal to the effective overburden pressure suggesting that the silty clay to clay deposit is normally consolidated.

Consolidated undrained triaxial compression tests were performed on samples of the silty clay to clay and the results are presented in Figures B18 to B21 in Appendix B. The results of the consolidated undrained triaxial compression tests which are only valid over the range of stress conditions in the test, are summarized in the following table.

Borehole/Sample No.	Sample Depth (m)	Sample Elevation (m)	ϕ' (degree)	c' (kPa)
BH3, Sample 6	4.6 – 5.0	257.5 - 257.1	24	3

Where: ϕ' = effective angle of internal friction; and
 c' = effective cohesion intercept.

4.2.8 Clayey Silt

Underlying the varved silty clay to clay deposit is a clayey silt deposit. The locations, thicknesses, depths and base elevations of the clayey silt deposit are summarized in the following table.

Borehole No.	Thickness (m)	Depth (m)	Base Elevation (m)
BH1	2.0	21.3	240.5
BH2	5.3	28.0	238.6
BH3	3.4	22.7	239.4

The N-values of Standard Penetration tests carried out in the clayey silt deposit range from 3 to 18 blows per 0.3 m of penetration. Field vane tests measured in-situ undrained shear strengths that range from 56 kPa to 72 kPa as illustrated on Figure B6 in Appendix B. Based on these results the consistency of the clayey silt is described as generally soft to very stiff. The sensitivity of the clayey silt ranges from about 2.1 to 3.1, indicating a medium sensitivity soil class (April 01, 2018 errata to Canadian Foundation Engineering Manual [CFEM], 2006).



Three samples of the clayey silt deposit were subjected to grain size distribution tests and the grain size distribution curves are illustrated on Figure B22 in Appendix B. The test results show a grain size distribution consisting of 0% gravel, 0% to 3% sand, 70% to 79% silt and 19% to 28% clay sized particles.

Three samples of the clayey silt deposit were also subjected to Atterberg limits tests and the results are plotted on the plasticity chart, Figure B23 in Appendix B. The results indicate a cohesive deposit of low plasticity (CL-ML to CL). The Atterberg limits test results are summarized below.

Liquid Limit:	21% to 22%
Plastic Limit:	15% to 16%
Plasticity Index:	5% to 7%
Natural Moisture Content:	22% to 28%

The Atterberg Limits tests results of the clayey silt deposit are also plotted against elevation in Figure B13. These results illustrate that the natural moisture contents of the tested samples are typically higher than the liquid limits. The moisture content of samples of this deposit range from 22% to 40% by weight.

4.2.9 Silt

The clayey silt deposit is further underlain by a silt deposit. The locations, thicknesses, depths and base elevations of the silt deposit are summarized in the following table.

Borehole No.	Thickness (m)	Depth (m)	Base Elevation (m)
BH1	3.5	24.8*	237.0
BH2	1.4	29.4	237.2
BH3	1.5	24.2	237.9

* Borehole termination depth.

Standard Penetration tests carried out in the silt deposit gave N-values that range from 9 to 36 blows per 0.3 m of penetration indicating a loose to dense relative density. The moisture content of samples from this stratum range from 17% to 22% by weight.

Grain size distribution tests were carried out on two samples from this deposit and the results are illustrated in Figure B24, Appendix B. The test results show a grain size distribution consisting of 0% and 1% gravel, 5% and 15% sand, 76% and 82% silt and, 9% and 12% clay sized particles.

4.2.10 Silty Sand to Sandy Silt

A silty sand to sandy silt deposit was encountered at Boreholes 2 and 3. The locations, thicknesses, depths and base elevations of the silty sand to sandy silt deposit are summarized in the following table.

Borehole No.	Thickness (m)	Depth (m)	Base Elevation (m)
BH2	0.2	29.6*	237.0
BH3	0.6	24.8*	237.3

* Borehole termination depth.

Standard Penetration tests carried out in the silty sand to sandy silt deposit gave N-values of 20 and 27 blows per 0.3 m of penetration indicating a compact relative density. The moisture content of a sample from this stratum is 17% by weight.



4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling. Artesian conditions were encountered in Boreholes 1 and 3 in the cohesionless silt deposit. The recorded artesian heads are 1.5 m and 1.8 m above ground surface corresponding to a ground water elevation of 263.6 m. A standpipe piezometer was installed 1.8 m east of Borehole 1 in the varved silty clay deposit and the water level readings measured on separate visits made after the completion of drilling are presented below.

Borehole No.	Date	Water Levels	
		Depth (m)	Elevation (m)
BH1	May 19, 2018	0.5	261.3
	November 21, 2018	0.4	261.4

The ground water level in the silty clay to clay deposit is at elevation 261.4± m. The ground water level can be expected to fluctuate seasonally and after severe weather events and will also be controlled by the free water level in the creek.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by Landcore Drilling of Chelmsford, Ontario. The field operations were organized by Ms. Fatemeh Yazdandoust, MSc. The routine laboratory testing, one-dimensional consolidation testing and consolidated undrained triaxial compression testing were carried out at Terraprobe's Brampton laboratory.

This report was prepared by Ms. Sepideh D-Monfared, P.Eng., a geotechnical engineer with Terraprobe and reviewed by Mr. Rehman Abdul, P.Eng., Terraprobe's Designated MTO Contact.

Terraprobe Inc.

Sepideh D-Monfared

Sepideh D-Monfared, P.Eng.
Geotechnical Engineer



Rehman Abdul

Rehman Abdul, P.Eng.
Designated MTO Foundations Contact



PART B – FOUNDATION DESIGN REPORT

**ANTHONY CREEK CULVERT REPLACEMENT, SITE 39E-0212/C0
HIGHWAY 101
TOWNSHIP OF TAYLOR, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5016-E-0023, G.W.P. 5165-12-00**



6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to carry out designs for the Anthony Creek Culvert replacement. The discussion and recommendations presented in this report are based on our understanding of the project and our interpretation of the factual data obtained from the subsurface investigations. These geotechnical recommendations are intended for use by the Ministry of Transportation and their design consultants and shall not be used or relied upon for any other purposes or by any parties including contractors.

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.

The existing 30.5 ±m long concrete box culvert with an opening that is 3.0 m wide and 1.8 m high, will be replaced with a 39.5 ±m long concrete box culvert with a cross-section measuring 3.0 m in width and 2.4 m in height. The upstream and downstream streambed elevations of the new culvert are 260.9 m.

The construction will be carried out by open cut excavation techniques under a full closure of Highway 101 and it is understood that the new culvert will be installed in the footprint area of the existing culvert. A temporary diversion channel will be constructed on the east side of the existing culvert to divert the creek's flow during construction.

6.2 Foundation Alternatives

Although a weak silty clay to clay deposit exists at this site, the proposed box culvert will be a relatively light structure that can be supported on an engineered fill pad founded on the silty clay to clay deposit. Therefore, expensive foundation options such as driven piles or ground improvement techniques were ruled out from further consideration. An open footing culvert is also not recommended because the underlying soils are settlement sensitive and the geotechnical resistances are low. A pre-cast box culvert replacement is recommended because pre-cast box culvert segments are more tolerant of total and differential settlement.

6.3 Consequence and Site Understanding Classification

The proposed structure carries Highway 101 traffic with the potential to impact this transportation corridor as well as alternative transportation corridors or structures. Therefore, a "typical consequence level" is considered appropriate as outlined in Section 6.5 of the *Canadian Highway Bridge Design Code (CHBDC) S6-19*.

A "typical degree of site and prediction model understanding" has been utilized given the scope of the foundation investigation and laboratory testing programme.

The consequence factor (ψ) and geotechnical resistance factors (ϕ_{gu} & ϕ_{gs}) used for designs and stipulated in Clause 6.5.2 and Clause 6.9 of the CHBDC S6-19, are based on a “typical consequence level” and a “typical degree of site and prediction model understanding”.

6.4 Seismic Design

6.4.1 Importance Category & Seismic Site Classification

Ground conditions for seismic site characterization were established based on the field investigation and laboratory test data. The average undrained shear strength in the upper 30 m of soil below founding level was used to define the seismic site classification in accordance with Table 4.1 of the CHBDC. Based on this methodology and the data, the structure shall be designed based on Site Class E.

6.4.2 Spectral Response Values

The CHBDC requires that the seismic hazard values associated with the design earthquake be established based on the National Building Code of Canada (NBCC). These values, Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Spectral Acceleration (S_a) can be obtained from the Geological Survey of Canada (GSC) “2015 National Building Code of Canada Seismic Hazard Calculator” and are for a reference ground condition of Site Class C.

In accordance with Section 4.4.3.3 of the CHBDC, the NBCC values were adjusted to reflect local site conditions i.e., Site Class E. As per Section 4.4.3.3 of the CHBDC, the value of PGA_{ref} for use with Tables 4.2 to 4.9 was taken as 80% of the PGA since the $S_a(0.2)/PGA$ ratio is less than 2.0. A PGA_{ref} value of 0.082 for the 2,475-year return was used. The NBCC spectral response values and the site-specific design values are tabulated below.

NBCC Seismic Hazard Values							
2% Exceedance in 50 years (2,475 Year Return Period)							
PGA (g)	PGV (m/s)	S_a (0.2) (g)	S_a (0.5) (g)	S_a (1.0) (g)	S_a (2.0) (g)	S_a (5.0) (g)	S_a (10.0) (g)
0.103	0.073	0.162	0.091	0.050	0.025	0.006	0.003
Site Specific Design Seismic Hazard Values Site Class E							
2% Exceedance in 50 years (2,475 Year Return Period)							
0.186	0.180	0.266	0.225	0.141	0.073	0.018	0.008

6.5 Geotechnical Resistances (Geogrid Reinforced Engineered Fill Pad)

It is not practical to support a new culvert directly on the soft to stiff silty clay to clay because of the deposit's low bearing capacity. The borehole data also shows that there is buried peat that extends to elevation 259.6 m± and this peat needs to be removed from below the footprint area of the new culvert. Therefore, a 0.7 m± thick OPSS Granular A engineered fill pad founded at a base elevation of 259.6 m is required to achieve a bottom of culvert design elevation of 260.3 m±.



The engineered fill pad shall be compacted to 95% Standard Proctor maximum dry density (SPMDD) at $\pm 2\%$ of optimum moisture content. At elevation 260.3 m \pm (bottom of culvert elevation) the engineered fill pad should extend at least 1 m horizontally beyond the culvert sides and should extend down to elevation 259.6 m at a 1 Horizontal to 1 Vertical (1H:1V) side slope.

A box culvert supported on a 0.7 m thick OPSS Granular A engineered fill pad that is founded at elevation of 259.6 m on the undisturbed silty clay to clay deposit, can be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 150 kPa. A geotechnical reaction at the Serviceability Limit State (SLS) is not provided since the settlement analysis that was carried out (based on the general arrangement drawings), indicate that settlement will be greater than 25 mm at the inlet and outlet areas of the new culvert.

The geotechnical resistance value of 150 kPa at ULS is for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in, Clause 6.10 of the CHBDC S6-19.

Since the silty clay to clay soils will be susceptible to loosening/softening and degradation on exposure to water and construction traffic; the engineered fill pad shall be placed expeditiously to avoid disturbance of the silty clay to clay deposit. A non-woven geotextile fabric with a Filtration Opening Size (FOS) of 100 microns shall be installed at the silty clay to clay/engineered fill pad interface to prevent soil migration. The engineered fill pad should also be reinforced at the base with a biaxial geogrid to provide support and stability during its placement and compaction. A Special Provision for the geogrid is provided in Appendix E. We also recommend that no backfill or precast units shall be placed until the depth of the excavation and the character of the foundation have been approved by a foundation engineer.

6.6 Horizontal Geotechnical Resistances

The ultimate geotechnical horizontal resistance should be evaluated in accordance with Clause 6.10.4 of the CHBDC S6-19. In accordance with Clause 6.10.4 of the CHBDC S6-19, the ultimate geotechnical horizontal resistance within the ground, close to the ground-structure interface (R_{ug}) and; the ultimate geotechnical horizontal shear resistance at the interface between the footing and the ground (R_{ui}), shall be derived based on the following effective angle of internal friction values (ϕ').

- OPSS Granular A bedding – internal friction angle $\phi' = 35^\circ$; and
- Silty Clay to Clay – internal friction angle $\phi' = 24^\circ$.

Along the interface between a shallow foundation and ground, an effective friction angle (δ'_i) equivalent to 2/3 of the soil's effective angle of internal friction (ϕ') shall be used.



6.7 Lateral Earth Pressure

6.7.1 Static Conditions

Earth pressures are generally calculated using the following expression:

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

P_h = horizontal pressure on the wall (kPa)

K = lateral earth pressure coefficient

γ = unit weight of retained soil (kN/m³)

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

q = value of any surcharge (kPa)

Earth pressures acting on the structure should be computed in accordance with Clause 6.12 of the CHBDC S6-19 and according to Clause 6.12.3 of the CHBDC S6-19; a compaction surcharge should also be added. For soils with an angle of internal friction ranging from 30° to 35° the magnitude should be 12 kPa at the top of the fill decreasing linearly to 0 kPa at a depth of 1.7 m; or decreasing linearly to 0 kPa at a depth of 2.0 m for soils with an angle of internal friction that exceeds 35°. Compaction equipment including hand operated vibratory equipment should comply with OPSS.PROV 501.

The lateral earth pressure coefficients are dependent on the material used as backfill and typical values are provided in the following table.

Wall Condition	Lateral Earth Pressure Coefficient (K)			
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$; $\gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ$; $\gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.27	0.38*	0.30	0.46*
At rest (Restrained Wall)	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-

* For wing walls.

The lateral earth pressure coefficients provided in the table above are “ultimate” values that require certain structural movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.27 in the Commentary to the CHBDC S6.1-19.

6.7.2 Seismic Conditions

In accordance with Section 4.6 of the CHBDC, seismic loads shall be considered in the design. The designs shall take into consideration:

- The wall should be designed to withstand the combined static lateral loads plus the earthquake induced loads;

- The horizontal seismic coefficient (k_h) used to calculate the seismic active pressure coefficient is taken as 1.0 times the PGA for structures that do not permit lateral yielding and 0.5 times PGA for structures that permit lateral yielding; and
- Where sloping backfill exists above the top of the wall, the weight of the backfill above the top of the wall should be treated as a surcharge when calculating the lateral earth pressure under seismic conditions.

The Mononobe-Okabe (M-O) method was used to calculate the active earth pressure coefficients for yielding and non-yielding walls assuming that the angle of friction between the wall and backfill material is 0.5ϕ . The seismic active earth pressure coefficients provided in the following table may be used for designs.

Wall Condition	Seismic Active Earth Pressure Coefficients (K)	
	OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ$; $\delta = 17.5^\circ$ $\gamma = 22.8 \text{ kN/m}^3$	OPSS Granular B Type I $\phi = 32^\circ$; $\delta = 16.0^\circ$ $\gamma = 21.2 \text{ kN/m}^3$
	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall
K_{AE} (Yielding Wall)	0.30	0.34
K_{AE} (Non-Yielding Wall)	0.37	0.41

6.8 Design Frost Depth

The frost penetration depth is 2.4 m of earth cover below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSS 3090.100. For frost protection purposes it is not necessary to found a box culvert at or below the frost depth, as the box structure is tolerant of small magnitudes of movement related to freeze-thaw cycles, should these occur. However, frost treatment for a box culvert should conform to OPSS 803.010.

Concrete wing walls/retaining walls founded on strip footings below the frost penetration depth were considered but these structures are not feasible at this site. The geotechnical reaction at the Serviceability Limit State is low and foundation settlements will be greater than 25 mm.

6.9 Erosion Protection

Erosion protection should be provided at the culvert inlets and outlets (including the slopes and sides). At the inlet area a clay seal shall be provided such that water is channelled through the culvert and does not seep through the backfill around and underneath the structure. The clay seal shall be placed in accordance with Section 422.07.13 of OPSS 422, should extend to cover all the granular backfill materials, should be a continuous layer around the culvert, should have a minimum compacted thickness of 0.6 m, and should extend at least 1 m above the high water level. The clay seal should also be protected by a layer of rip-rap. Material used for the clay seal should conform to the requirements stipulated in OPSS.PROV 1205. Concrete cut-off and head walls can also be used as an alternative to a clay seal to protect the granular fill around the culvert from erosion.



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

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

6.10 Culvert Bedding and Backfill

The recommended backfill and cover geometry is illustrated in OPSD 803.010 (concrete culvert). Prior to placing bedding and backfill material, all organic material and any fill soils found within the footprint of the new box culvert should be removed. Bedding material should consist of OPSS Granular "A" material placed and compacted to 95% of the materials SPMDD in accordance with OPSS 422 and SP 422S01. As specified in OPSS 422, a levelling course is also required. In addition to the non-woven geotextile fabric and biaxial geogrid specified in Section 6.5, additional bedding requirements that may be imposed by the supplier must also be followed.

Equal heights of backfill should be maintained on both sides of the structure during all stages of backfill placement, and backfilling operations should be carried out in accordance with OPSS.PROV 902 and OPSS.PROV 422. Heavy compaction equipment should not be used adjacent to the walls and roof of the culvert. Compaction equipment should be restricted in accordance with OPSS.PROV 501.

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

6.11 Global Stability Assessments

6.11.1 General

The global, internal and surficial stability of the embankment will depend on the embankment height, the slope geometry, the subsurface soils, and material used to construct the embankment. For the purpose of embankment stability analyses, the commercially available slope stability program Slide 2018 developed by Rocscience Inc. was used. The Morgenstern-Price and Spencer methods for stability analysis were employed and the target factors of safety for temporary and permanent conditions were derived based on the site consequence factor (ψ) and the geotechnical resistance factors (Φ_{gu}) provided in Table 6.2 of the CHBDC. Accordingly, minimum target factors of safety of 1.3 and 1.5 were established for temporary (short term) and permanent (long term) conditions respectively.



6.11.2 Static Conditions (Permanent Embankments)

The soil parameters used for the slope stability analyses and the factors of safety that were obtained are provided in the following table.

Material Type	γ (kN/m ³)	Total Stress Analysis		Effective Stress	
		ϕ (degrees)	c (kPa)	ϕ' (degrees)	c' (kPa)
New Embankment Fill	21.2	32	0	32	0
Existing Embankment Fill	20	32	0	32	0
Silty Clay Fill	18	28	0	28	0
Upper Silty Clay to Clay	16.5	0	25	24	3
Lower Silty Clay to Clay	16.5	0	25+1.6H	24	3
Clayey Silt	19	0	45	28	0
Silt	19	30	0	30	0
Design Factor of Safety	-	1.3		1.5	

The analyses indicate that for a 5.5± m high embankment, the factors of safety will be equal to the minimum target factors of safety, provided that the embankment is constructed at a minimum side slope geometry of 2.25 Horizontal to 1 Vertical (2.25H:1V). The results of the stability analyses are presented in Appendix D, Figure D1.

6.11.3 Seismic Conditions (Permanent Embankments)

Embankment stability can be assessed using conventional pseudo-static methods of slope stability analysis under the earthquake-induced peak ground acceleration. A calculated factor of safety of 1.1 to 1.3 indicates that the slope is considered to be generally stable and meets the seismic design requirements. A calculated factor of unity or less does not necessarily indicate full-scale slope failure because the soil mass is subjected to the peak load in a given direction for only a fraction of a second.

Soil slopes are not rigid and the peak acceleration during an earthquake lasts for only a very short period of time. Therefore, seismic coefficients used in practice generally correspond to acceleration values well below the predicted peak accelerations.

For a 2% probability of exceedance in 50 years, the derived site-specific peak ground acceleration (PGA) is 0.186g consistent with Site Class E. The horizontal and vertical seismic coefficients shall not be less than one-half of the corresponding peak ground acceleration resulting in a design seismic coefficient value of 0.093 i.e., 50% of the site-specific PGA.

The pore water pressure in the subsurface soils will increase under earthquake conditions. In granular, cohesionless deposits the pore water pressures are expected to dissipate very quickly due to the soils relatively high permeability, and the effective stress parameters of these soils were used for the pseudo-static analyses. For silty clay to clay soils however, total stress parameters were used for the pseudo-static analysis to account for excess pore water pressures that develop during earthquake conditions.

Pseudo-static seismic slope stability analyses carried out on embankments with a 2.25H:1V side slope geometry indicate that the embankments will have a factor of safety of 0.9. The results of the seismic stability analyses are presented in Appendix D, Figure D1.

Since the pseudo-static limit equilibrium analyses indicates a factor of safety less than unity, the CHBDC requires an assessment of the permanent slope deformation. The permanent slope deformation was assessed using the Newmark sliding block analysis, and an estimated deformation value of 75 mm was obtained. Shallow sloughing and toe failure could occur during seismic events. This sloughing and toe failure is expected to be limited, would not impair the use of the highway, and would mainly be a maintenance issue. The potential for sloughing following seismic events can be reduced by providing well-vegetated side slopes.

6.11.4 Stability of Temporary Excavations

It is understood that Highway 101 will be fully closed to replace the culvert. Based on the General Arrangement drawings, global stability analyses of the open cut excavations were carried out based on the following design concepts:

- On the west side of the existing culvert a temporary cut perpendicular to the highway and extending down to elevation 259.6 m (i.e., 7.2 m± high cut) is required; and
- On the east side of the existing culvert a temporary diversion channel with a base elevation of 260.9 m will be constructed. The excavation on the east side of the temporary diversion channel will extend from the highway platform down to elevation 260.9 m (i.e., 5.9 m± high cut).

The global stability of the 7.2 m± high and 5.9 m± high forward slopes of the excavation were assessed using the total stress analysis soil properties provided in Section 6.11.2 of this report. The analyses indicate that a 7.2 m± high cut will require a forward slope geometry of 3.5H:1V or flatter to achieve a target factor of safety of 1.3. For a 5.9 m± high cut on the east side of the diversion channel, a forward slope geometry of 3.0H:1V or flatter is required to achieve the target factor of safety of 1.3. The results of the stability analyses are presented in Appendix D, Figure D2.

The effect of construction loads on the global stability of excavations shall be further assessed by the Contractor's geotechnical consultant. A Special Provision for this aspect of the work is provided in Appendix E.

6.12 Settlement

6.12.1 General

To predict the magnitude and time rate of settlement of the underlying silty clay to clay soils the commercially available program Settle 3D developed by Rocscience Inc. was used. The deformation parameters used for the analyses were established using data obtained from a consolidation test, empirical correlations of undrained shear strengths, laboratory index tests, soil moisture contents as well as information on New Liskeard Clay research publications referenced in this report. The preconsolidation pressure derived from the consolidation test data is approximately equal to the effective overburden pressure suggesting that the silty clay to clay deposit is normally consolidated.

The deformation parameters used for the analysis are summarized in the following table.



Parameter	Silty Clay to Clay
Preconsolidation Pressure – σ'_p (kPa)	120
Overconsolidation Ratio	1.0
Compression Index - C_c	0.65, 0.65 – 0.042H*, and 0.25
Recompression Index - C_r	0.065, 0.065 – 0.0047H*, and 0.020
Initial Void Ratio - e_o	1.5, 1.5 – 0.074H*, and 0.8
Coefficient of Consolidation - C_v (cm ² /s)	1.0×10^{-3}

* H is the incremental change in elevation

6.12.2 Culvert Settlement

The settlement analysis is based on a reconstructed embankment side slope geometry of 2.25H:1V and takes into consideration the embankment side slope geometry beyond the Highway 101 pavement platform. The model is based a new culvert that is 3.0 m wide and 2.4 m high (internal dimensions); founded at elevation 259.6 m on a 0.7 m± thick Granular A engineered fill pad with Granular B Type I backfill.

Tabulated below are the estimated total long term settlements along the culvert centre line for a 39.3 m long culvert, constructed above the footprint area of the existing culvert.

Culvert constructed above the footprint area of the existing culvert						
Offset Distance (m)*	0	±5	±10	±15	±18	±20
Estimated Settlement (mm)	5	10	10	135	350	145

* Hwy. 101 Centre Line is at 0 m and distances are measured relative to the highway centre line

The settlement data tabulated above indicates that within the highway platform the settlement will be less than 25 mm. However, the estimated settlement at offset distances of 15 m to 20 m from the highway centre line is large because this area has not experienced the benefit of any preloading from the current embankment. Therefore, it is imperative that the culvert joints be designed to accommodate this differential settlement. Alternatively, it may be preferable to preload the inlet and outlet areas with creek flow being maintained through temporary culvert extensions (such as CSPs) as illustrated on Figure F1 in Appendix F, and described below:

- A preload equivalent to 40 kPa shall be placed partially on the existing embankment side slope and beyond the existing embankment toe of slope extending horizontally to 1 m beyond the new culvert inlet and outlet;
- The preload area shall extend 5 m horizontally beyond the existing culvert sides. At the north-west corner the preload area shall be adjusted by tapering from 5 m reducing to 2 m at the culvert outlet to avoid encroaching the existing grave; and
- The preload shall be placed at least six months prior to installing the new culvert.

A recommended Operational Constraint for preloading is provided in Appendix E. Tabulated below are the estimated total settlements along the culvert centre line provided that the new culvert is installed after a 6 months preload period. A settlement monitoring programme is not warranted because there is a high confidence level that the predicted settlement will be complete under the advance contract.



Culvert constructed above the footprint area of the existing culvert						
Offset Distance (m)*	0	±5	±10	±15	±18	±20
Estimated Settlement (mm)	5	10	10	0	0	0

* Hwy. 101 Centre Line is at 0 m and distances are measured relative to the highway centre line

Embankments constructed with non-cohesive earth fill will also settle during construction (fill compression) and, the magnitude of this settlement is expected to be about 1% of the fill height. This settlement should be immediate in nature and will be essentially complete shortly after construction is complete.

6.13 Embankment Construction

Embankment construction should be carried out in accordance with OPSS.PROV 206 and OPSS.PROV 501. Granular materials used for embankment construction should be placed in lifts not exceeding 300 mm (before compaction), and each lift should be uniformly compacted to at least 95 % of the material's SPMDD.

Topsoil, any deleterious material and soft/loose and other unsuitable soils shall be removed below the footprint area of the embankment and, at the toe of the proposed embankment within an envelope given by an imaginary slope not steeper than 1H:1V from the toe of the proposed embankment. Bonding between existing fill and new fill should be carried out by benching in accordance with OPSD 208.010.

Proper erosion control measures should be implemented both during construction and permanently. Temporary erosion and sediment control must be provided in accordance with OPSS 805. Embankment slopes shall be reinstated with permanent erosion protection in accordance with OPSS 803 and OPSS.PROV 804.

6.14 Excavations

All excavations must be carried out in accordance with the guidelines outlined in the *Occupational Health and Safety Act (OHSA) and Regulations for Construction Projects*. Where workers must enter excavations extending deeper than 1.2 m, the trench walls must be suitably sloped and/or braced in accordance with the OHSA. Within the envisaged depths of temporary excavations (i.e., up to elevation 259.6± m), the OHSA soil classifications are:

- Embankment fill – Type 3 soils above ground water and Type 4 soils below ground water; and
- Silty Clay to Clay – Type 4 soils.

Based on the global stability analyses that were carried out for temporary excavations, a side slope geometry of 3.5H:1V or flatter is required for temporary excavations on the west side of the new culvert and a side slope geometry of 3H:1V or flatter is required for temporary excavations on the east side of the new culvert and adjacent to the diversion channel. Excavations should be undertaken in accordance with OPSS.PROV 902.



6.15 Temporary Protection Systems

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary shoring should be designed in accordance with OPSS.PROV 539 and the designs should be carried out by a licensed Professional Engineer experienced in shoring design.

The shape of the soil pressure distribution diagram behind a shoring system depends upon the type of soil to be encountered and the amount of movement that can be permitted. The shoring system can be restrained, fixed or flexible and the sequence of work will alter the shape of the pressure diagram during the various construction phases.

Earth pressure computations must also take into account the ground water level. Above the ground water level, earth pressure is computed using the bulk unit weight of the retained soil. Below the ground water level, the earth pressures are computed using the submerged unit weight of the soil. A hydrostatic pressure is also applied if the retained soil is not fully drained.

Flexible shoring should be designed on the basis of the active earth pressure coefficient (K_a). In this case, the performance level should be Level 2 – Angular Distortion 1:200 but shall not be more than 25 mm. Where limited shoring movement (Performance Level 1A or 1B) is required the design should be based on the at rest earth pressure coefficient (K_o). For “kick out” design the lateral resistance should be computed on the basis of the passive earth pressure coefficient (K_p). The lateral earth pressure coefficients chosen for design require certain movements for the active and passive conditions to be mobilized. The values to use in design can be estimated from Figure C6.27 in the Commentary to the CHBDC, S6-19.

The appropriate lateral earth pressure parameters for use in the design of structures subject to unbalanced earth pressures are provided in the following table. The active earth pressure coefficients are based on the assumption that the ground surface behind the temporary excavation support system is horizontal. Where the retained ground is sloping, the lateral earth pressure coefficients must be adjusted to account for the slope and, these earth pressure coefficients can be estimated from the equations provided on Figures C6.28 and C6.29 of the Commentary to the CHBDC S6-19.

Stratigraphic Unit	Friction Angle ϕ (degrees)	Unit Weight γ (kN/m ³)	Active Earth Pressure Coefficient	At - Rest Earth Pressure Coefficient	Passive Earth Pressure Coefficient
			K_a	K_o	K_p
Existing Embankment Fill	32	20	0.31	0.47	3.25
Silty Clay Fill	28	18	0.36	0.53	2.77
Silty Clay to Clay	24	16.5	0.42	0.59	2.37
Clayey Silt	28	19	0.36	0.53	2.77
Silt	30	19	0.33	0.50	3.00

The lateral earth pressure coefficients given above are “ultimate values” and require specific wall movements for the active and passive conditions to be mobilized. The values to use in design can be estimated from Figure C6.27 in the Commentary to the CHBDC, S6-19.

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



Where interlocking sheet piles are used to construct a temporary protection system, these units shall be cut off at a depth of 1.0 m below ground surface after they are no longer required.

6.16 Ground Water Control

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

The design of the unwatering system is the Contractor's responsibility. The excavation will extend through the existing embankment fill terminating in the cohesive silty clay to clay deposit. A suitable unwatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

6.17 Soil Corrosivity and Sulphate Test Results

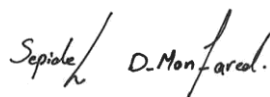
Two soil samples were submitted for soil chemical tests consisting of pH, water soluble sulphate, sulphide, chloride, resistivity and electrical conductivity analyses and the laboratory report is provided in Appendix B. This suite of tested parameters is used for assessing the corrosivity potential of the native soil to ductile iron pipe in accordance with the 10-point soil evaluation procedure described in ANSI/AWWA C105/A21.5 Standard³. Based on this soil corrosivity scale, a total of 10 points or more indicates that the soil is corrosive to as-manufactured ductile iron pipe (DIP), and additional corrosion protection measures are recommended. The Corrosivity Indices of the tested samples are 4.

The water-soluble sulphate concentrations of the tested samples were compared to Table 3 of the Canadian Standards Association A23-1-09. The results indicate that the degree of exposure to water soluble sulphate in soil is low i.e., less than the moderate range.

7.0 CLOSURE

This report was prepared by Ms. Sepideh D-Monfared, P.Eng., a geotechnical engineer with Terraprobe and reviewed by Mr. Rehman Abdul, P.Eng., Terraprobe's Designated MTO Contact.

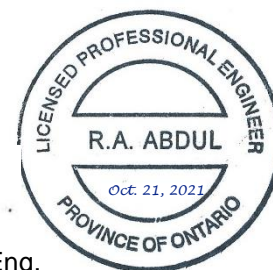
Terraprobe Inc.



Sepideh D-Monfared, P.Eng.
Geotechnical Engineer



Rehman Abdul, P.Eng.
Designated MTO Foundations Contact



³ American Water Works Association (AWWA) C-105 (2005) Standard, "Polyethylene Encasement for Ductile-Iron Pipe Systems" Catalog No. 43105. AWWA Denver, CO.



REFERENCES

- Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils, Second Edition*. McGraw Hill Book Company, New York.
- Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.
- Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-19. 2019*. Canadian Standard Association.
- Chan, H. T., and Kenny, T. C., 1973, *Laboratory Investigation of Permeability Ratio of New Liskeard Varved Soil* Canadian Geotechnical Journal, Vol. 10 pp. 453-472.
- Chandler, R. J. 1988. *The In Situ Measurement of the Undrained Shear Strength of Clays Using the Field Vane, STP 1014, Vane Shear Strength Testing in Soils: Field and Laboratory Studies*, American Society for Testing and Materials.
- Hough, B.K., 1957. *Basic Soils Engineering*. The Ronald Press Company, New York.
- Kulhawy, F. H., and Mayne, P. W. 1990. *Manual on Estimating Soil Properties for Foundation Design, Report EL-6800*, Electric Power Research Institute Inc.
- Lacasse, Suzanne M., Ladd, Charles C., and Barsvary, Alex K., 1977, *Undrained behaviour of embankments on New Liskeard varved clay*, Canadian Geotechnical Journal, Vol 14, pp 367-388.
- Lo, K. Y., and Stermac, A. G., 1965, *Failure of an Embankment Founded on Varved Clay*, Canadian Geotechnical Journal, Vol. II, no. 3, pp. 234-253.
- McClenaghan, M.B. 1990. *Summary of results from the Black River – Matheson (BRiM) reconnaissance surface till sampling program*. Ontario Geological Survey, Open File Report 5749, p. 197.
- Nagaraj, T. S., and Murthy, B. R. S. 1985. *Prediction of the Preconsolidation Pressure and Recompression Index of Soils*. Geotechnical Testing Journal, American Society for Testing and Materials, Vol. 8, No. 4.
- Newmark, N. (1965)., *“Effects of Earthquakes on Dams and Embankments”*, Geotechnique, Vol. 15, No. 2, pp. 139-160.
- Quigley, R. M. and Ogunbadejo, T. A., 1972, *Clay Layer Fabric and Oedometer Consolidation of a Soft Varved Clay* Canadian Geotechnical Journal, Vol. 9, pp. 165-175.
- Rendon-Herrero, O. 1983. *Universal Compression Index*, Journal of Geotechnical Engineering, Vol. 109, No. 10
- Stermac, A. G., Lo, K. G., and Barsvary, A. K., 1967, *The Performance of an Embankment on a Deep Deposit of Varved Clay*, Canadian Geotechnical Journal, Vol. IV, no. 1, pp 45-61.
- Terzaghi, K., and Peck, R. B. (1967). *Soil Mechanics in Engineering Practice*. Wiley, New York.

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification For Grading.
OPSS 422	Construction Specification For Precast Reinforced Concrete Box Culverts in Open Cut.
OPSS.PROV 501	Construction Specification For Compacting.
OPSS.PROV 510	Construction Specification For Removal.
OPSS.PROV 539	Construction Specification For Temporary Protection Systems.
OPSS 803	Construction Specification For Sodding.
OPSS.PROV 804	Construction Specification For Seed and Cover.
OPSS 805	Construction Specification For Temporary Erosion And Sediment Control Measures.
OPSS.PROV 902	Construction Specification For Excavating and Backfilling – Structures.



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

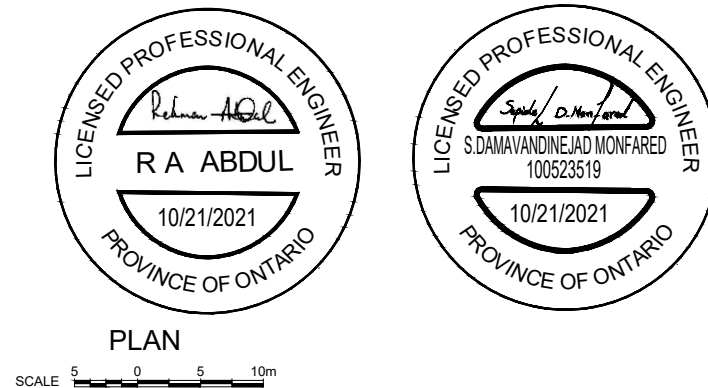
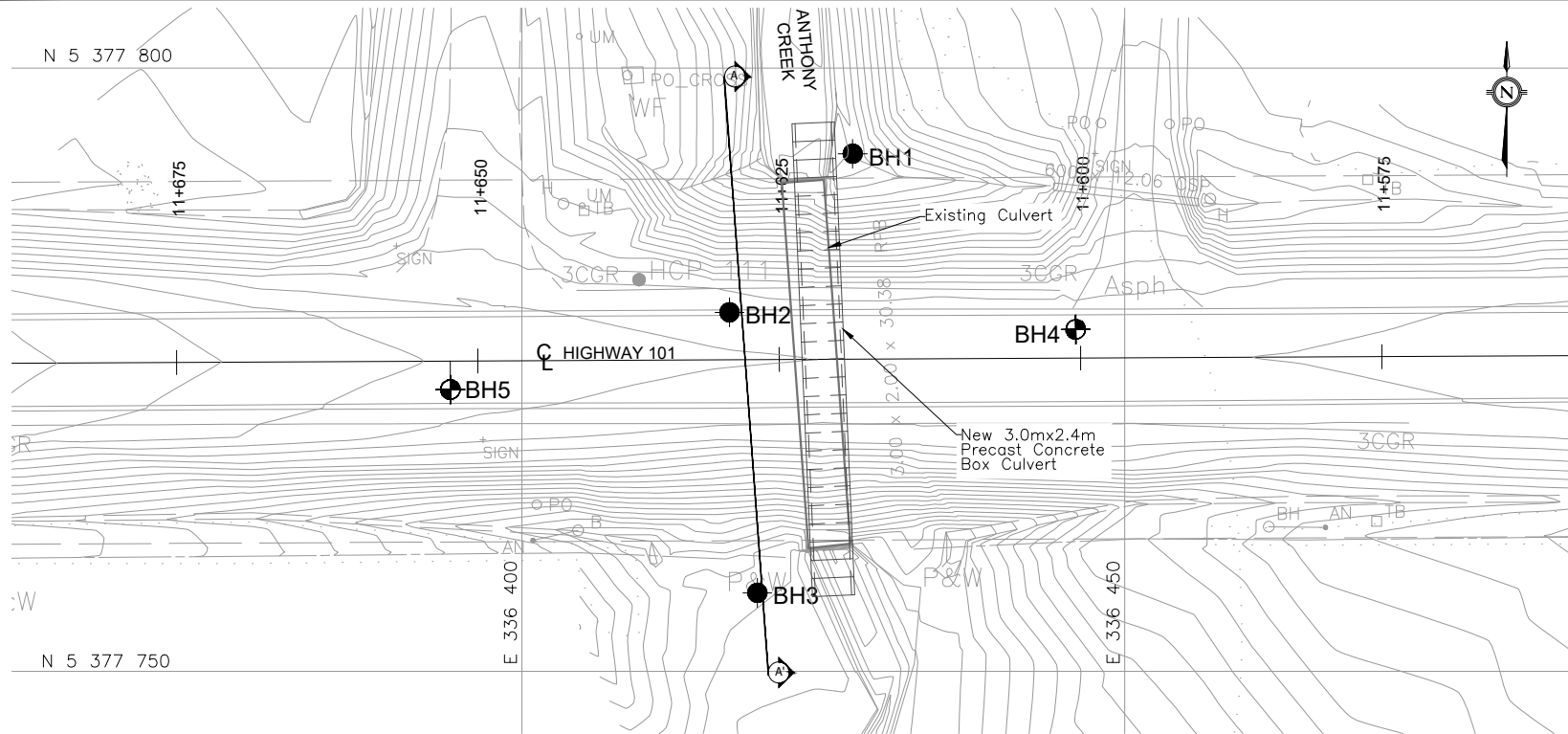
Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching Of Earth Slopes.
OPSD 803.010	Backfill And Cover For Concrete Culverts With Spans Less Than Or Equal To 3.0 m
OPSD 3090.100	Foundation, Frost Penetration Depths For Northern Ontario



DRAWINGS & SITE PHOTOGRAPHS

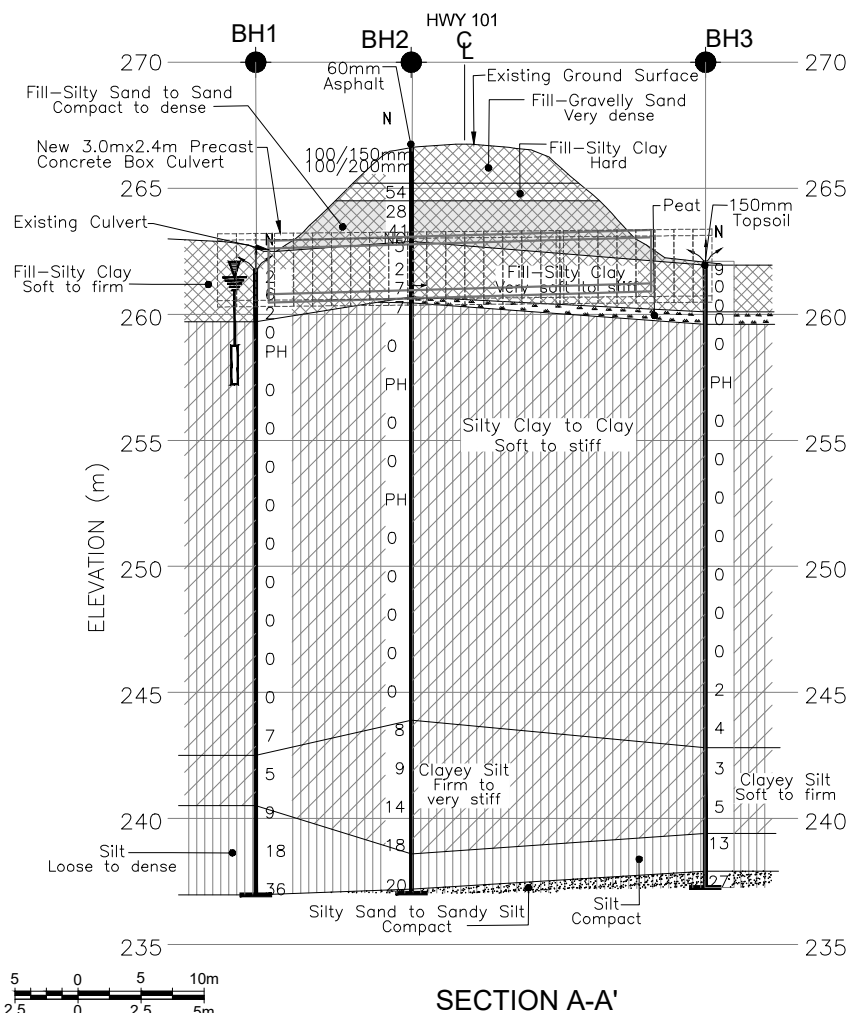
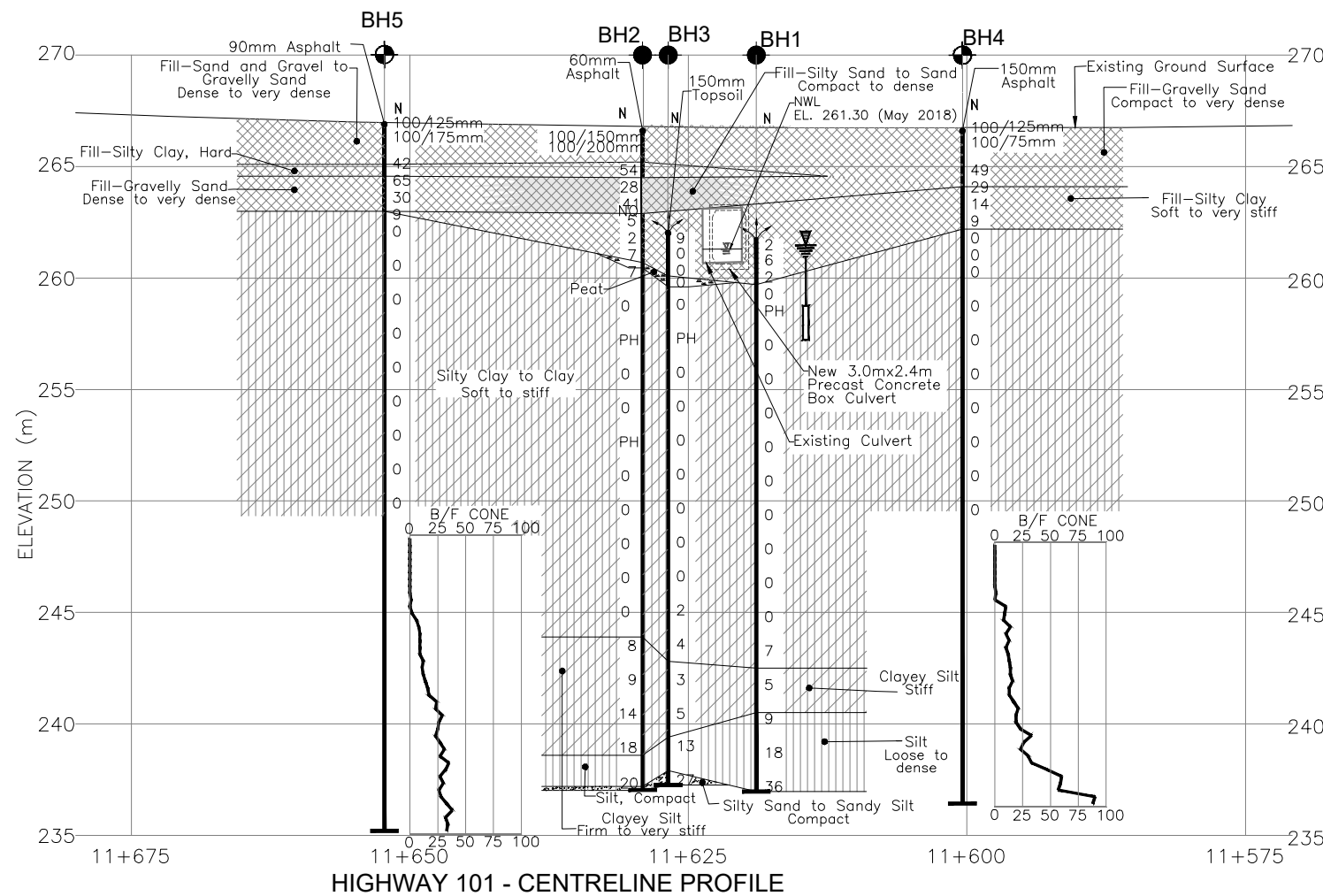









METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETERS
UNLESS OTHERWISE SHOWN

CONT. No. WP No. 5290-14-01	
ANTHONY CREEK CULVERT REPLACEMENT	SHEET
BOREHOLE LOCATION AND SOIL STRATA	40
	METRIC

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		
		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 (May. 2018)		
		Piezometer		
A/R		Auger Refusal		
BH No.		ELEV. (m)	COORDINATES (MTM, ZONE 12)	
			NORTHING (m)	EASTING (m)
1		261.8	5 377 792.9	336 427.4
2		266.6	5 377 779.8	336 417.2
3		262.1	5 377 756.5	336 419.5
4		266.6	5 377 778.4	336 445.9
5		266.9	5 377 773.3	336 394.1
NOTE				
<p>This drawing is for subsurface information only. The proposed structure details/works if shown are for illustration purposes only and may not be consistent with final design configuration as shown elsewhere in the contract documents.</p> <p>The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.</p> <p>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</p>				
REFERENCE				
Drawing provided in digital format by WSP, drawing file Hwy 101-Hwy 577 Base Mapping.dwg received Feb 27, 2018 via email and contract drawings emailed June 11, 2021.				
REVISIONS				
	DATE	BY	DESCRIPTION	
HWY. 101		PROJECT No. 1-17-0822		GEOCREC No. 42A-140
SUBM'D. SD		CHKD. SD	DATE: OCT. 2021	SITE: 39E-0212/CO
DRAWN: KC		CHKD. SD	APPD: RA	DWG. 2



Hwy. 101 at Sta. 11+600 - Looking West - July 15, 2019




Anthony Creek Culvert - Looking North - July 15, 2019



Hwy. 101 at Sta. 11+700 - Looking East - July 15, 2019



Anthony Creek Culvert - Looking South - July 15, 2019

 Terraprobe Inc. <small>Consulting Geotechnical & Environmental Engineering Construction Materials, Inspection & Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 706-2650</small>	HWY 101 ANTHONY CREEK CULVERT REPLACEMENT, SITE 39E-0/C0212	
	G.W.P: 5165-12-00	DATE: October 2021
	SUBM'D: SD	CHKD: RA
	Project No: 1-17-0822	Drawing: 2

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						

METRIC[illegible]

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

METRIC

SOIL PROFILE						SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV. DEPTH (m)	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	SPT 'N' VALUE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)						
						20 40 60 80 100	w _p w w _L						
	(continued)												
	SILTY CLAY to CLAY (varved) firm to 14.1m, stiff below, grey, wet		13	SS	0*								
			14	SS	0*								
			15	SS	7								
242.5 19.3	CLAYEY SILT , trace sand, stiff, grey, wet		16	SS	5								
240.5 21.3	SILT , trace to some sand, trace to some clay, trace gravel, loose to 22.3 m, compact to dense below, grey, wet		17	SS	9								
			18	SS	18								
237.0			19	SS	36								

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

METRIC[illegible]

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

RECORD OF BOREHOLE No 2

2 of H

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336417.2 N:5377779.8 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-4-10 - 2018-4-11 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				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 X LAB VANE										
(continued)							20 40 60 80 100				10 20 30			GR SA SI CL			
	SILTY CLAY to CLAY (varved) firm to 20.8m, stiff below, grey, wet		16	SS	0*		251							54 LL=45	0 0 19 81		
							250										
							249										
							248								45	0 0 26 74	
							247										
							246										
							245										
							244										
							243										
							242										
243.9 22.7	CLAYEY SILT, trace sand, firm to very stiff, grey, wet		21	SS	8		241								0 2 70 28		
					240												
					239												
					238												
			22	SS	9												
			23	SS	14												
			24	SS	18												
238.6 28.0	SILT, trace to some sand, trace clay, compact, grey, wet																
237.2 237.0 29.6	SILTY SAND, trace clay, trace gravel, compact, grey, wet		25	SS	20												
END OF BOREHOLE																	

END OF BOREHOLE

+³, X³: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE


file: anthony creek culvert.gpj

RECORD OF BOREHOLE No 2

H of H

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336417.2 N:5377779.8 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-4-10 - 2018-4-11 CHECKED BY RA

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE								
								SHEAR STRENGTH (kPa) 20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE					
								20 40 60 80 100					

Borehole filled with drill water upon completion of drilling.

*Sampler sinking under weight of hammer and/or rods.

Consolidation test performed on TW12.

RECORD OF BOREHOLE No 3

1 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336419.5 N:5377756.5 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-5-1 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)						
								20	40	60	80	100		
262.1	GROUND SURFACE													
261.9	150mm TOPSOIL		1	SS	9		262							
0.2	FILL, silty clay, trace to some sand, trace gravel, trace organics, very soft to stiff, brown, wet		2	SS	0*		261							
			3	SS	0*		260							
260.1	PEAT, trace to some clay, amorphous, black		4	SS	0*									
2.0														
259.6	SILTY CLAY to CLAY (varved) soft to firm to 14.9m, firm to stiff below, grey, wet		5	SS	0*		259							
2.5														
							258							
			6	TW	PH		257							
			7	SS	0*		256							
			8	SS	0*		255							
							254							
			9	SS	0*		253							
							252							
			10	SS	0*		251							
							250							
			11	SS	0*									
							249							
			12	SS	0*		248							

Continued Next Page

+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 3

2 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336419.5 N:5377756.5 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-5-1 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 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 X LAB VANE							WATER CONTENT (%)		
							20 40 60 80 100					10 20 30					
	(continued)																
	SILTY CLAY to CLAY (varved) soft to firm to 14.9m, firm to stiff below, grey, wet		13	SS	0*		247							0 0 47 53			
							246										
			14	SS	2		245										
							244										
			15	SS	4		243										
242.8 19.3	CLAYEY SILT, trace sand, soft to stiff, grey, wet		16	SS	3		242							0 0 79 21			
							241										
			17	SS	5		240										
239.4 22.7	SILT, trace to some sand, trace clay, compact, grey, wet		18	SS	13		239										
							238										
237.9 24.2 237.3	SILTY SAND to SANDY SILT, trace gravel, trace to some clay, compact, grey, wet		19	SS	27												

END OF BOREHOLE

Borehole filled with drill water upon
completion of drilling.

*Sampler sinking under weight of
hammer and/or rods.

Insufficient sample available for grain
size distribution test on SS19.

Artesian conditions encountered at
22.7 m and head of water recorded at
1.5 m above ground surface upon
completion of drilling. Borehole was
sealed/grouted with bentonite slurry
mixture.

file: anthony creek culvert.gpj

RECORD OF BOREHOLE No 4

1 of 3

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336445.9 N:5377778.4 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-4-11 - 2018-4-23 CHECKED BY RA

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 (%)					
								20 40 60 80 100							W _p W W _L					
266.6	GROUND SURFACE																			
266.4	150mm ASPHALTIC CONCRETE		1	SS	100 / 125mm															
0.2	FILL, gravelly sand, containing silty clay pockets, compact to very dense, brown, dry to 1.3 m, wet below		2	SS	100 / 75mm															
			3	SS	49															
264.1			4	SS	29															
2.5	FILL, silty clay, some sand to sandy, trace gravel, stiff to very stiff, brown, wet		5	SS	14															
			6	SS	9															
262.2			7	SS	0*															
4.4	SILTY CLAY to CLAY (varved) firm, brown to 5.9m, grey below, wet		8	SS	0*															
			9	SS	0*															
			10	SS	0*															
			11	SS	0*															
			12	SS	0*															
			13	SS	0*															
			14	SS	0*															

RECORD OF BOREHOLE No 4

2 of 3

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336445.9 N:5377778.4 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-4-11 - 2018-4-23 CHECKED BY RA

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								
	(continued)												
	SILTY CLAY to CLAY (varved) firm, brown to 5.9m, grey below, wet		15	SS	0*		251	2.8				56	0 0 26 74
			16	SS	0*		250	2.3 4.2				45	
							249	3.0 2.7					
248.0	Dynamic Cone Penetration test (DCPT) performed from 18.6 m to 30.2 m.						248						
18.6							247						
							246						
							245						
							244						
							243						
							242						
							241						
							240						
							239						
							238						
							237						

Continued Next Page


+ 3, X 3: Numbers refer to Sensitivity O 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

3 of 3

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336445.9 N:5377778.4 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-4-11 - 2018-4-23 CHECKED BY RA

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 (%)				
						20										40	60	80	100
	(continued)																		
236.4																			

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

*Sampler sinking under weight of hammer and/or rods.

METRIC[illegible]

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

RECORD OF BOREHOLE No 5

2 of 3

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336394.1 N:5377773.3 ORIGINATED BY FY
DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
DATUM GEODETIC DATE 2018-4-23 - 2018-4-24 CHECKED BY RA

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 (%)					
	(continued)													
	SILTY CLAY to CLAY (varved) soft to firm, grey, wet		14	SS	0*		251	4.0						
			15	SS	0*		250	3.1						April 23, 2018
							249	3.1						April 24, 2018
248.3							248	3.6						
18.6	Dynamic Cone Penetration test (DCPT) performed from 18.6 m to 31.7 m.						247							
							246							
							245							
							244							
							243							
							242							
							241							
							240							
							239							
							238							
							237							

Continued Next Page

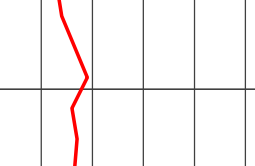
+ 3, X 3: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 5

3 of 3

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:336394.1 N:5377773.3 ORIGINATED BY FY
 DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
 DATUM GEODETIC DATE 2018-4-23 - 2018-4-24 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
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 (%)						
	(continued)							<div><div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div><div><div>+ FIELD VANE</div><div>× LAB VANE</div></div></div>						
	Dynamic Cone Penetration test (DCPT) performed from 18.6 m to 31.7 m.						236							
235.2														
31.7														

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

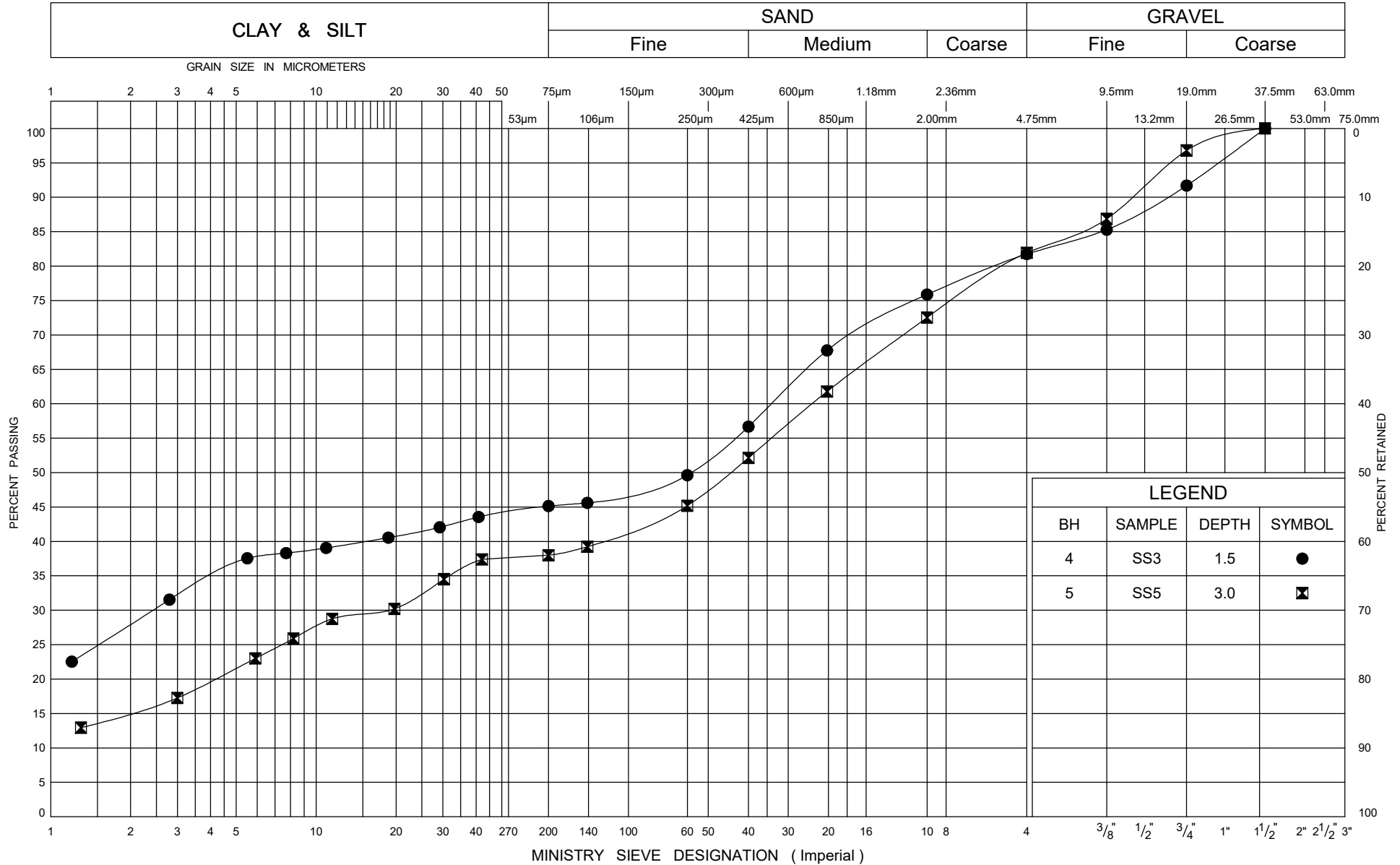
*Sampler sinking under weight of hammer and/or rods.

APPENDIX B

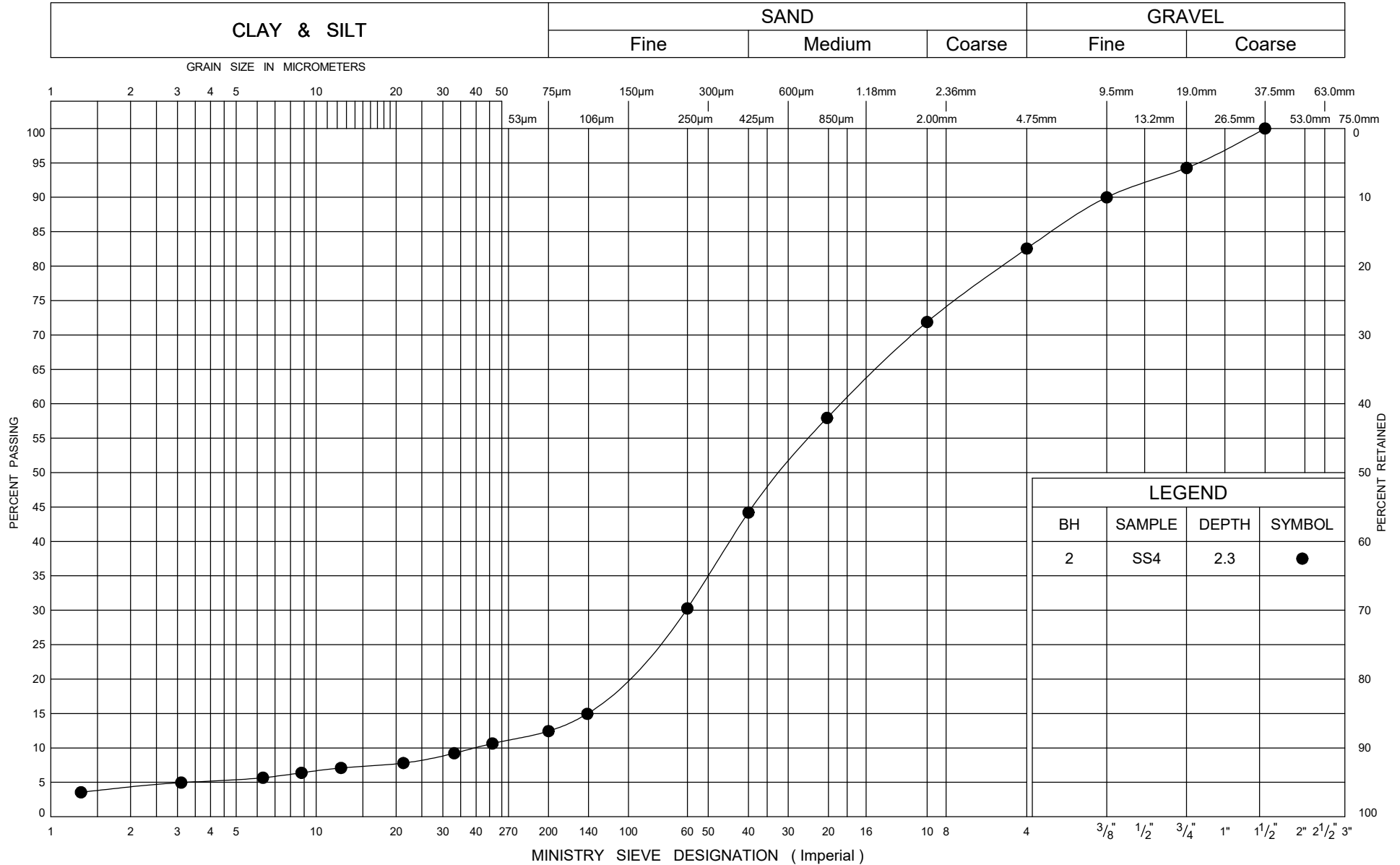
Field & Laboratory Test Results & Photographs



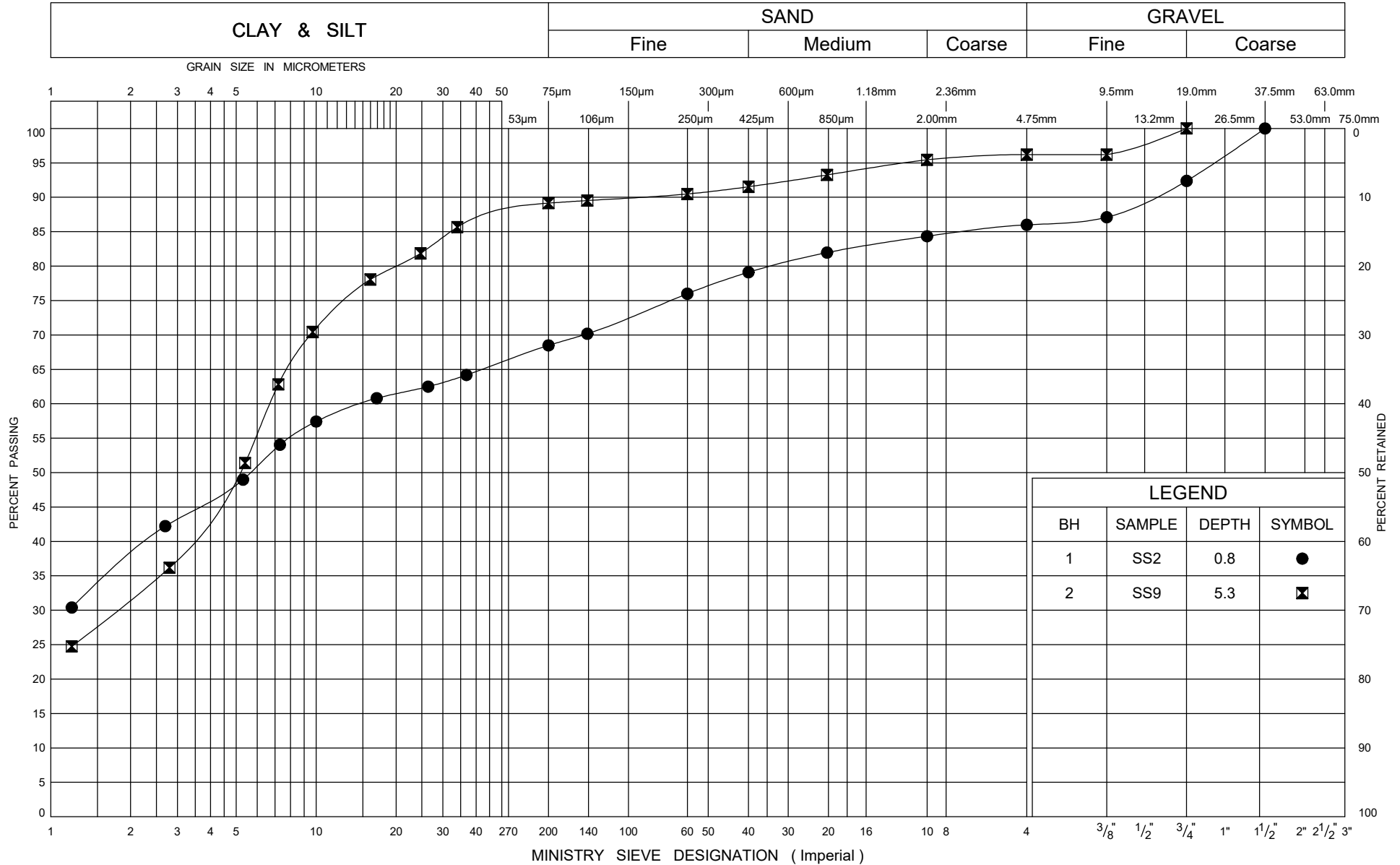
UNIFIED SOIL CLASSIFICATION SYSTEM

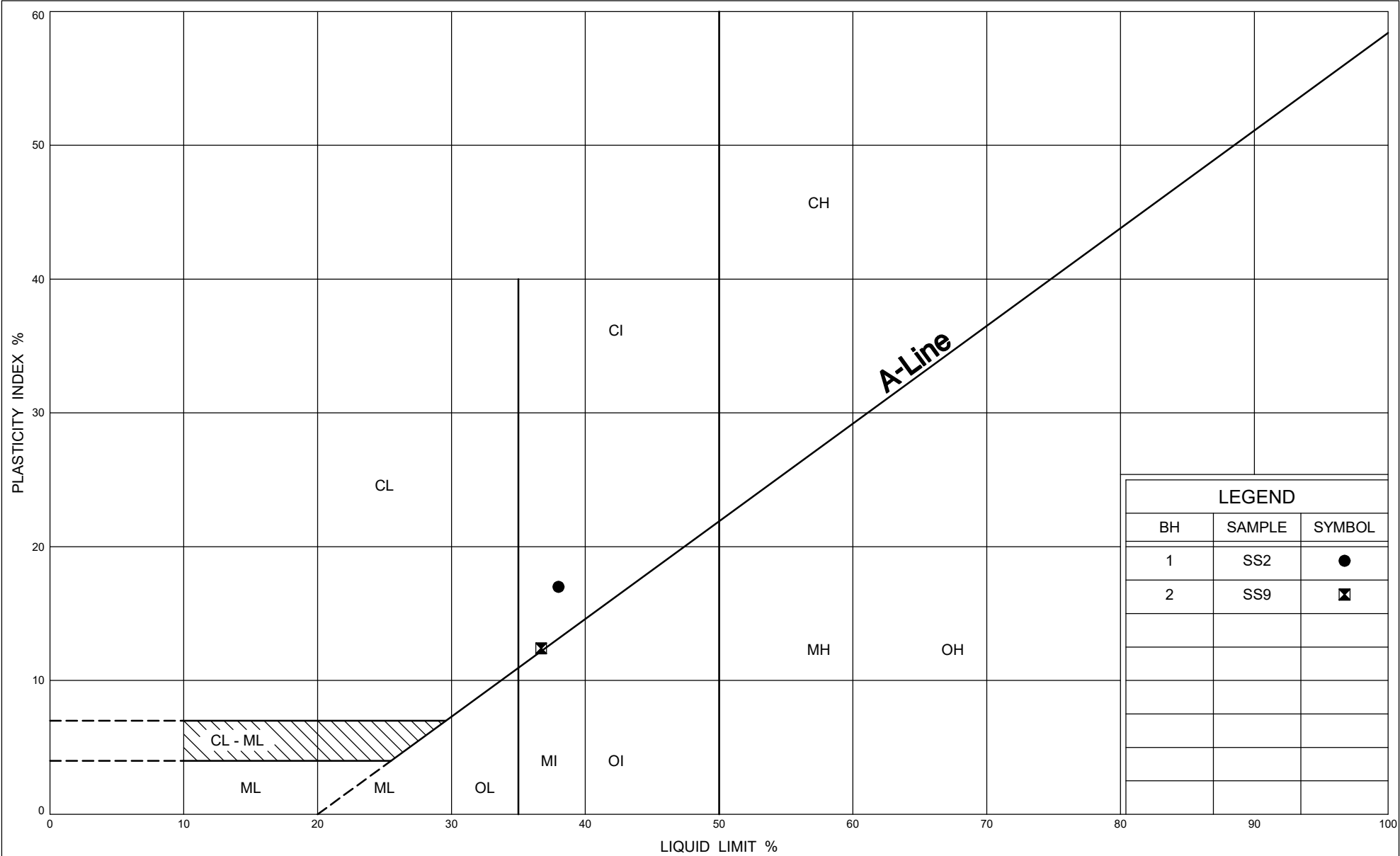


UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM





file: anthony creek culvert.gpj



Ministry of
Transportation

PLASTICITY CHART
FILL - SILTY CLAY

FIG No B4

G W P 5165-12-00

Anthony Creek Culvert, Site 39E-212

VARVED SILTY CLAY TO CLAY SAMPLES

FIGURE B5

Anthony Creek Culvert (Site 39E-212)
Silty Clay to Clay



BH2 - SS17



BH3 - SS10

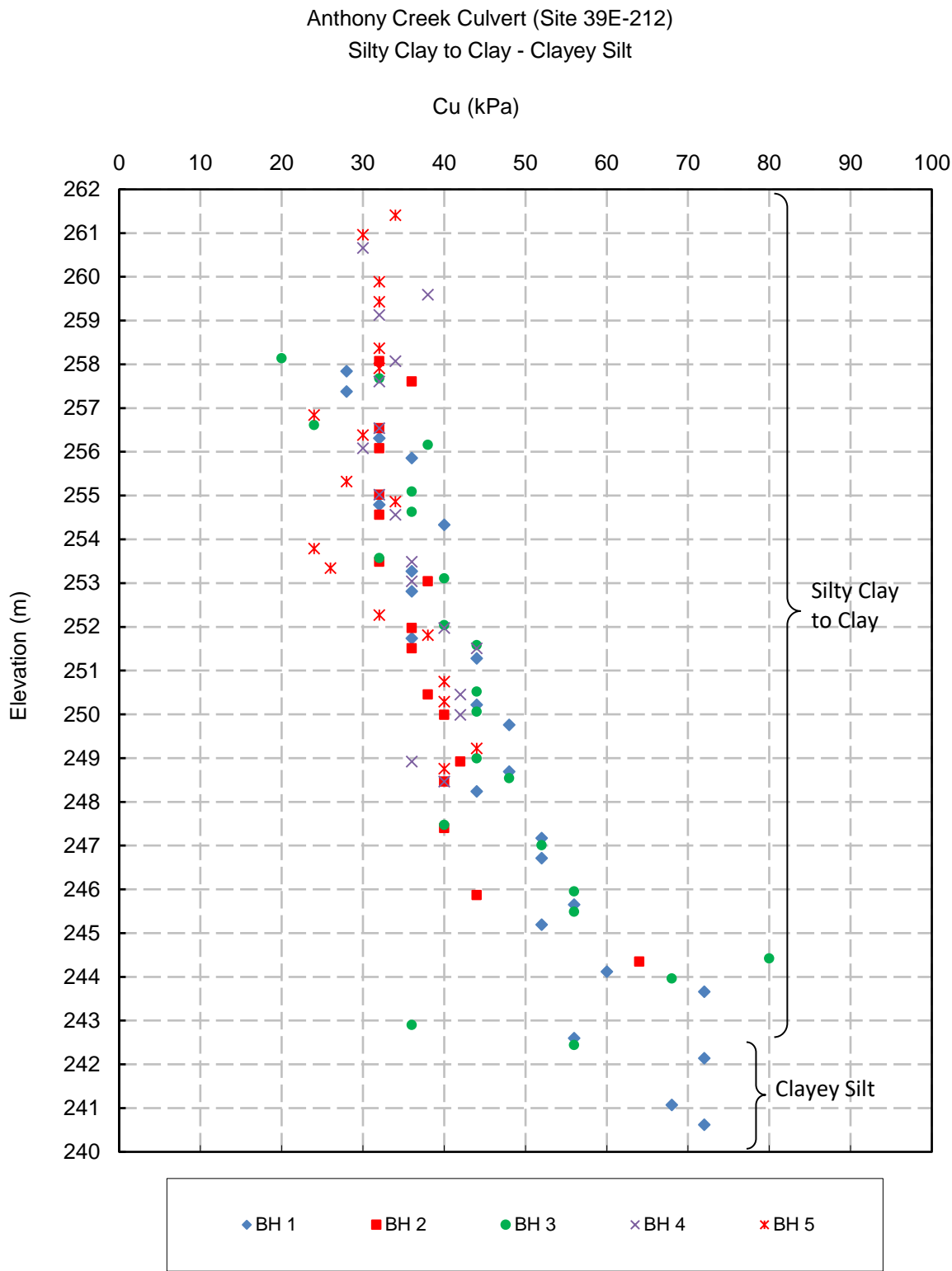
Project No. : 1-17-0822
Date : May, 2018



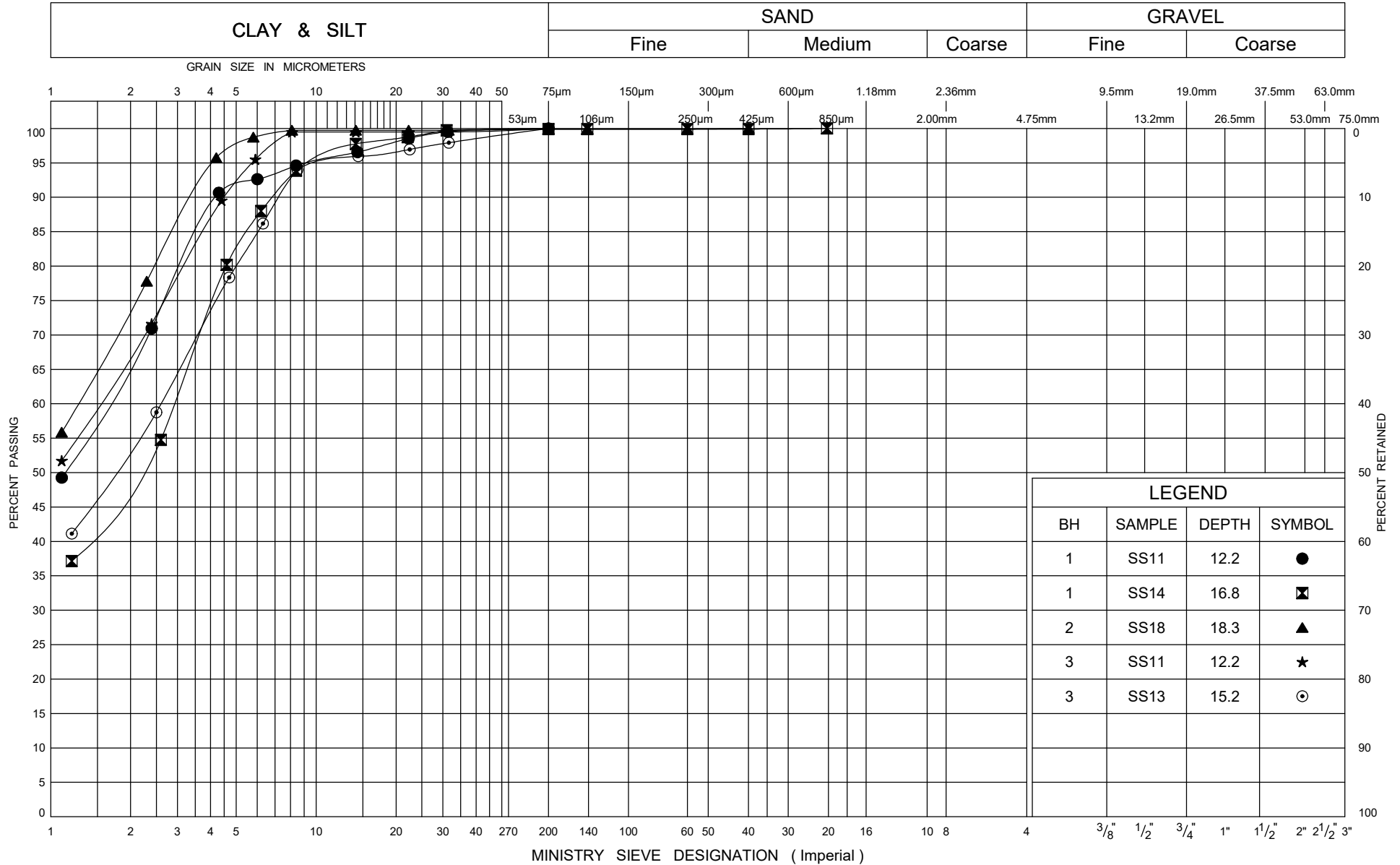
Prepared by : SD
Checked by : RA

UNDRAINED SHEAR STRENGTH

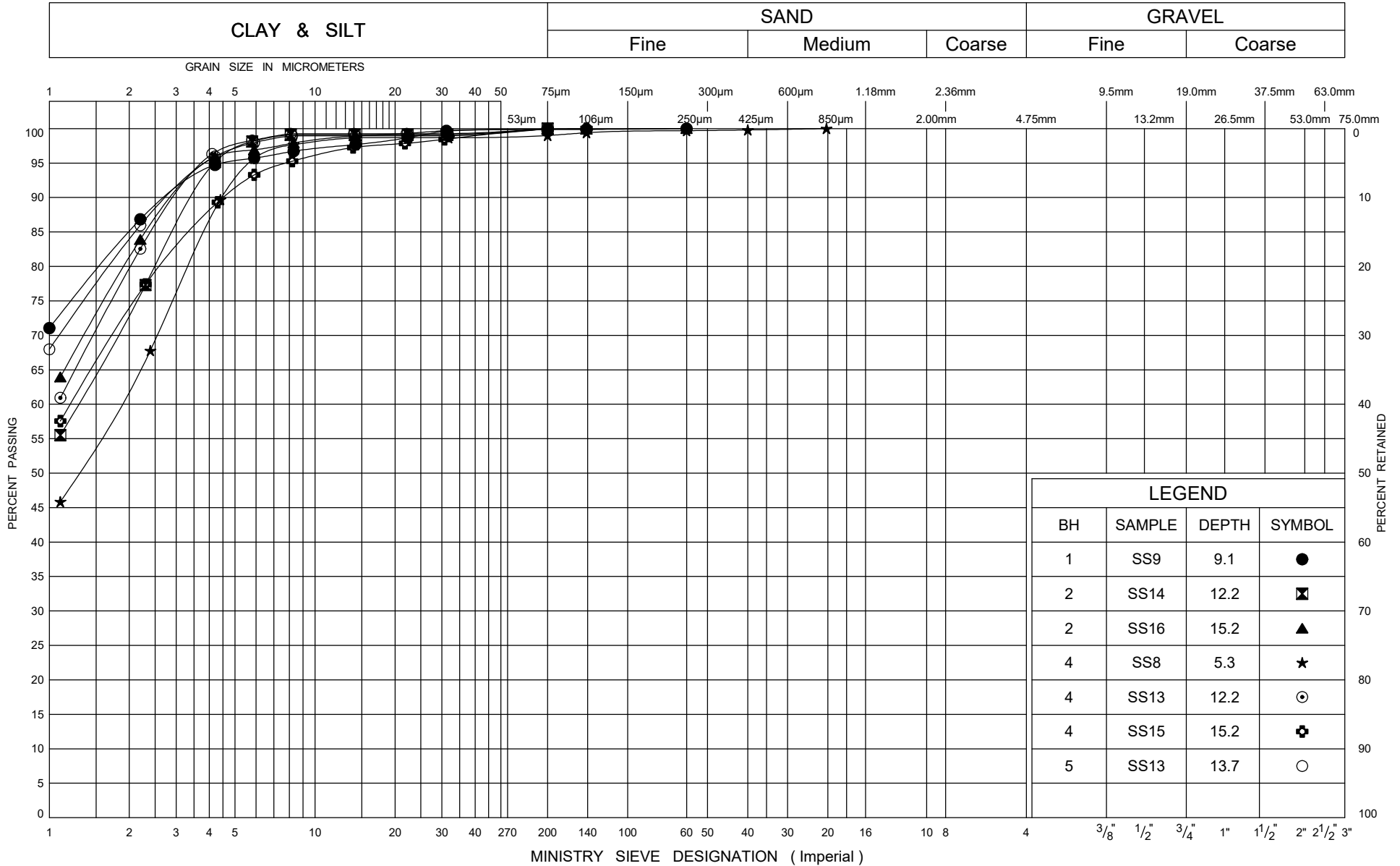
FIGURE B6



UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
Transportation

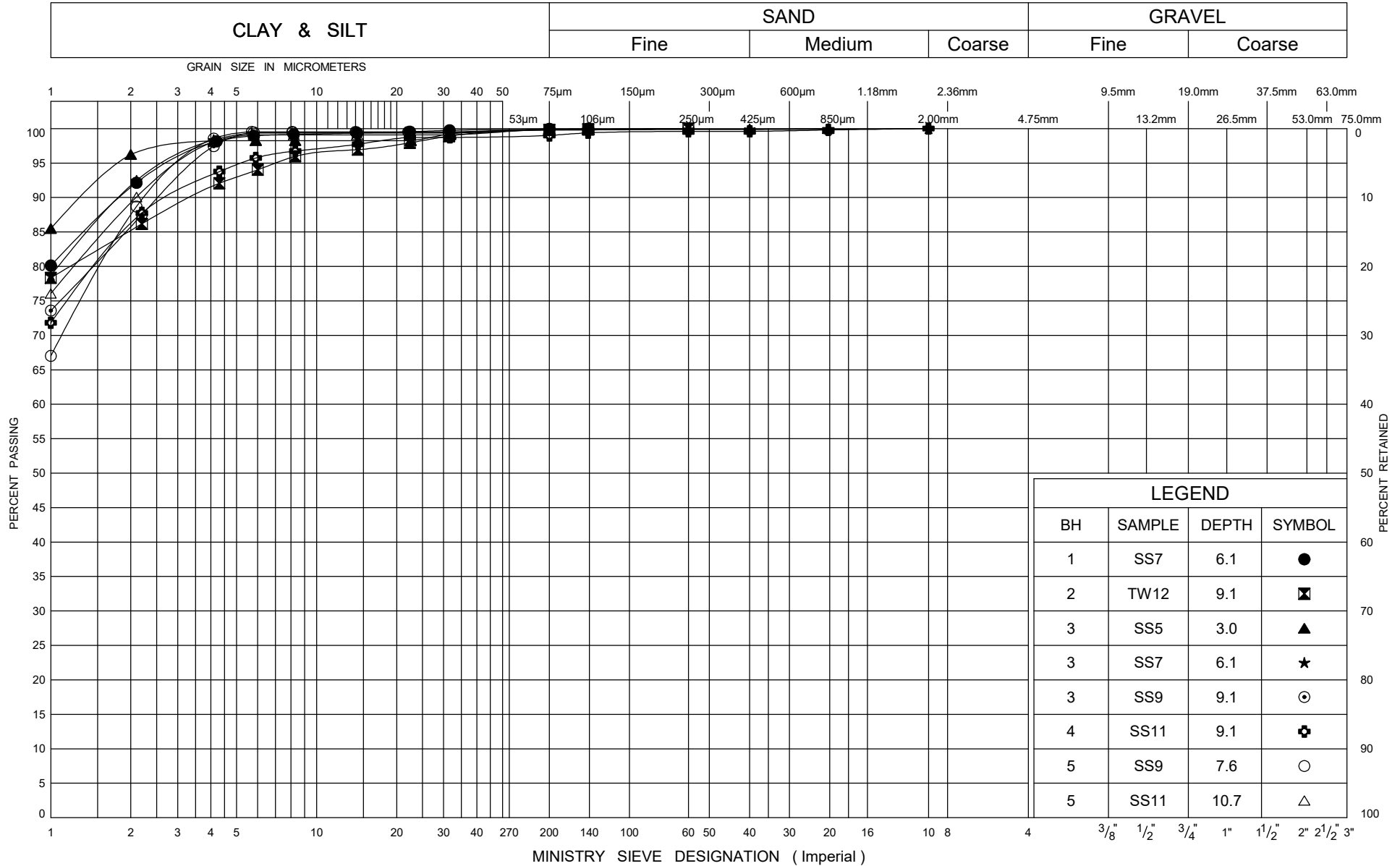
GRAIN SIZE DISTRIBUTION SILTY CLAY TO CLAY

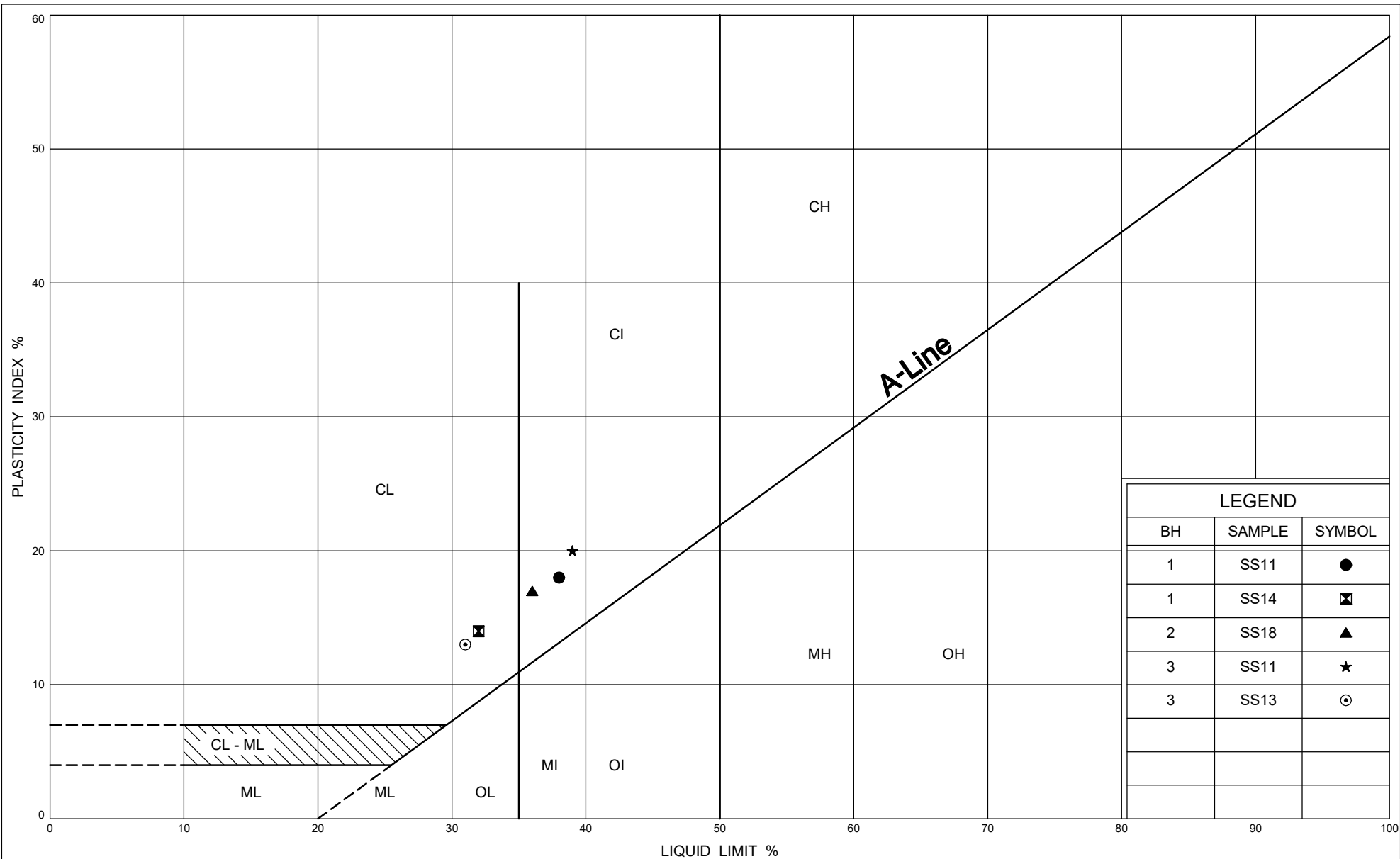
FIG No B8

G W P 5165-12-00

Anthony Creek Culvert, Site 39E-212

UNIFIED SOIL CLASSIFICATION SYSTEM





LEGEND		
BH	SAMPLE	SYMBOL
1	SS11	●
1	SS14	⊠
2	SS18	▲
3	SS11	★
3	SS13	⊙



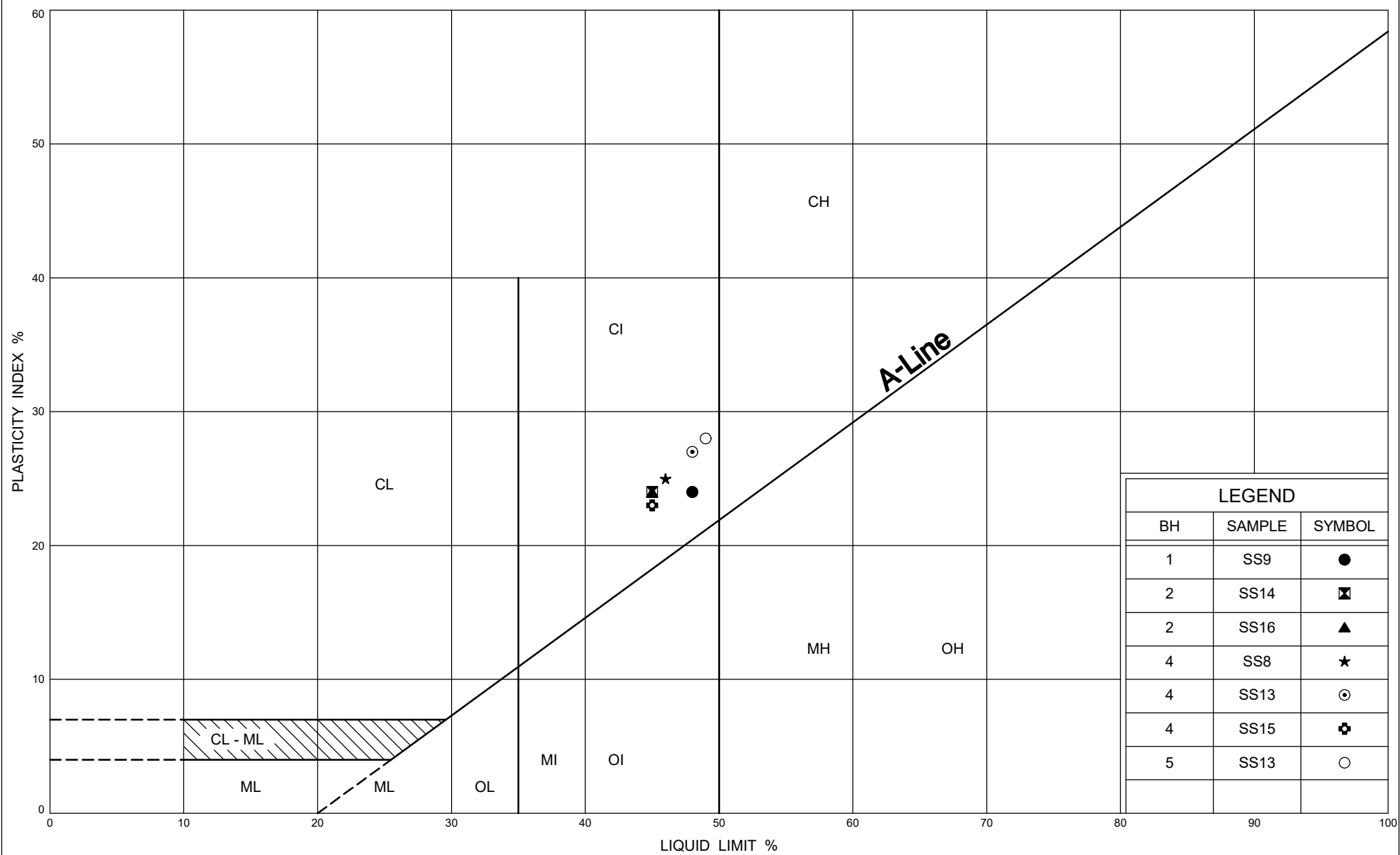
Ministry of
Transportation

PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B10

G W P 5165-12-00

Anthony Creek Culvert, Site 39E-212



file: anthony creek culvert.jpg



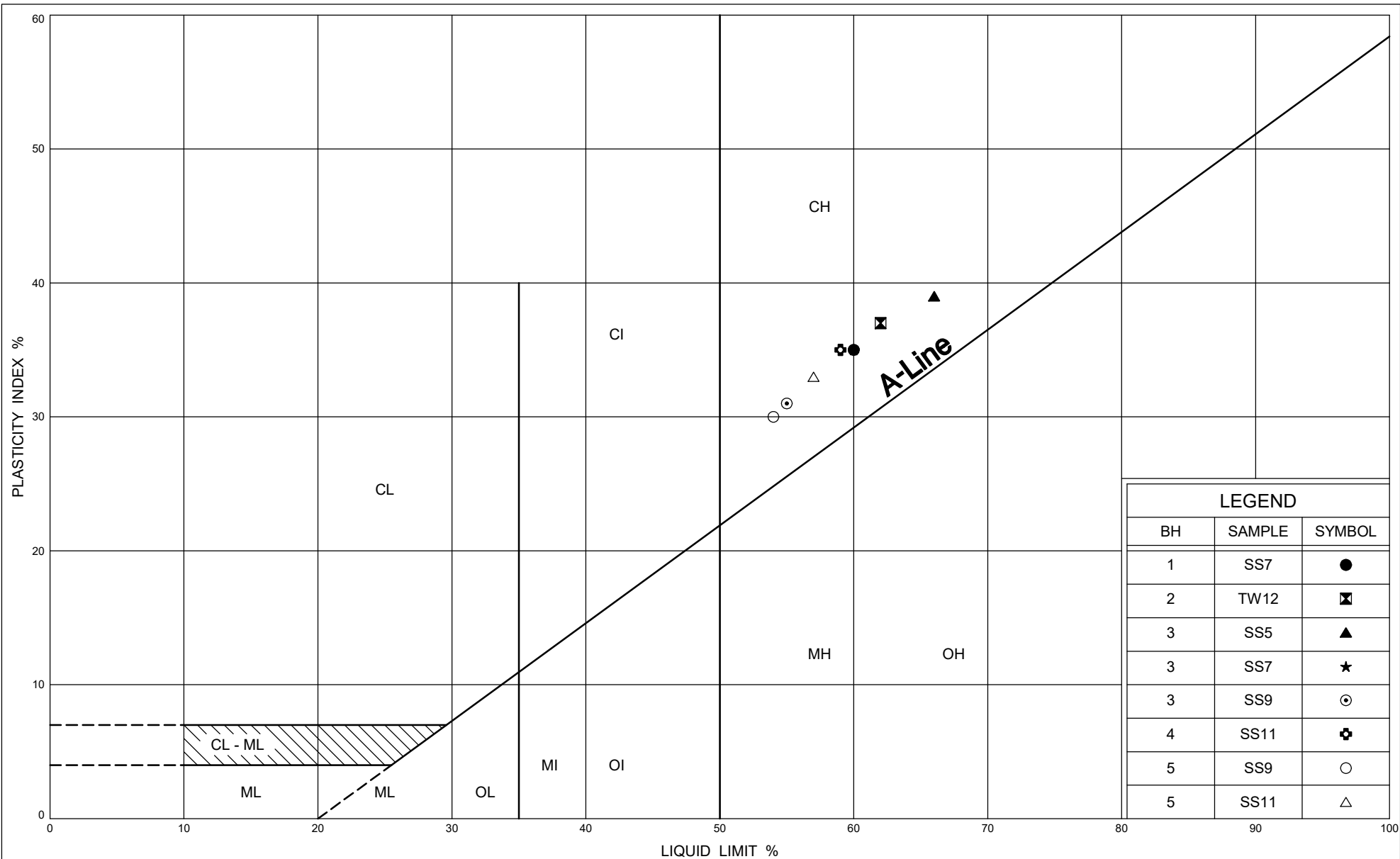
Ministry of
Transportation

PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B11

G W P 5165-12-00

Anthony Creek Culvert, Site 39E-212



Ministry of
Transportation

PLASTICITY CHART SILTY CLAY TO CLAY

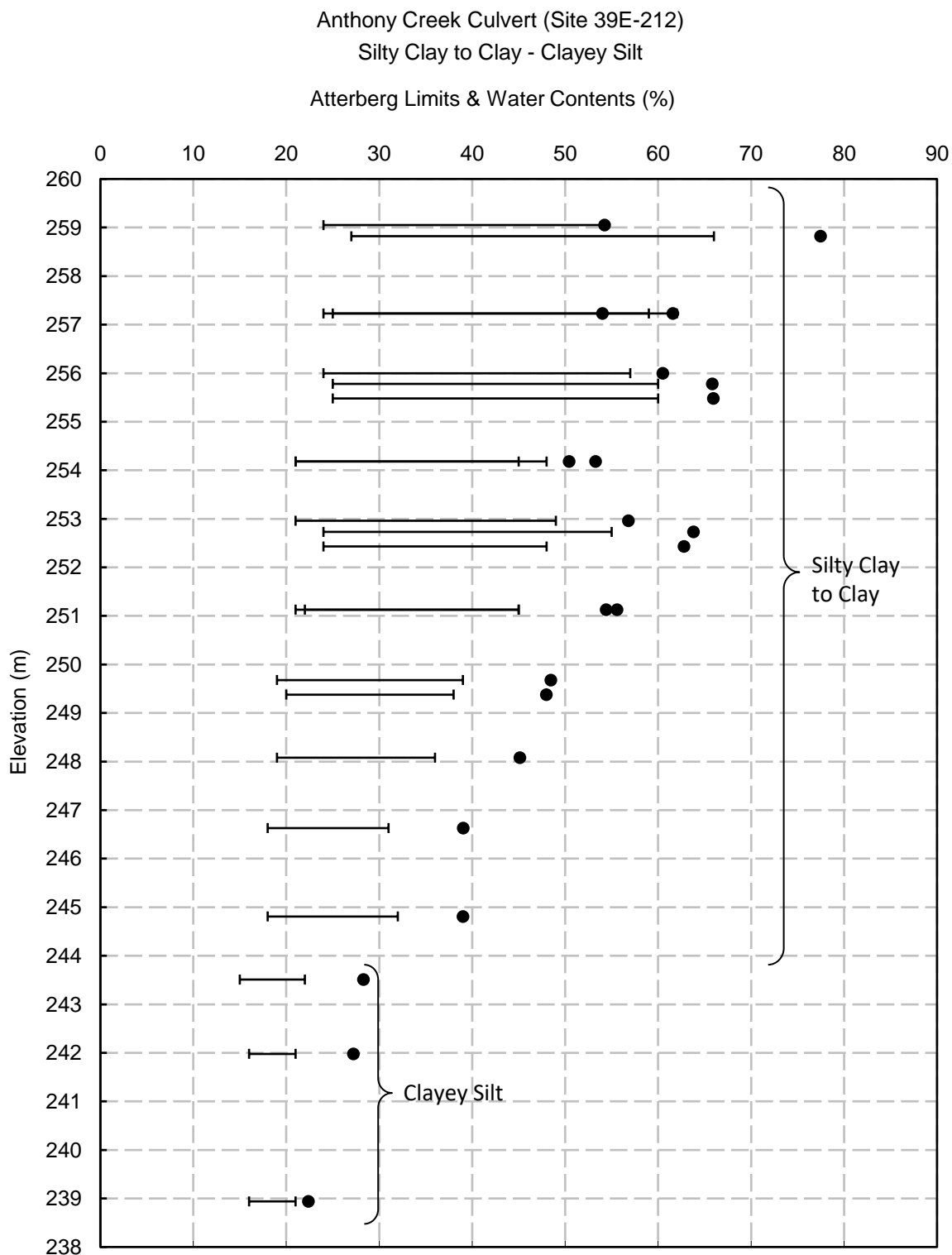
FIG No B12

G W P 5165-12-00

Anthony Creek Culvert, Site 39E-212

ATTERBERG LIMITS AND WATER CONTENTS

FIGURE B13




Project No. : 1-17-0822

Date : May, 2018



Prepared by : SD

Checked by : RA

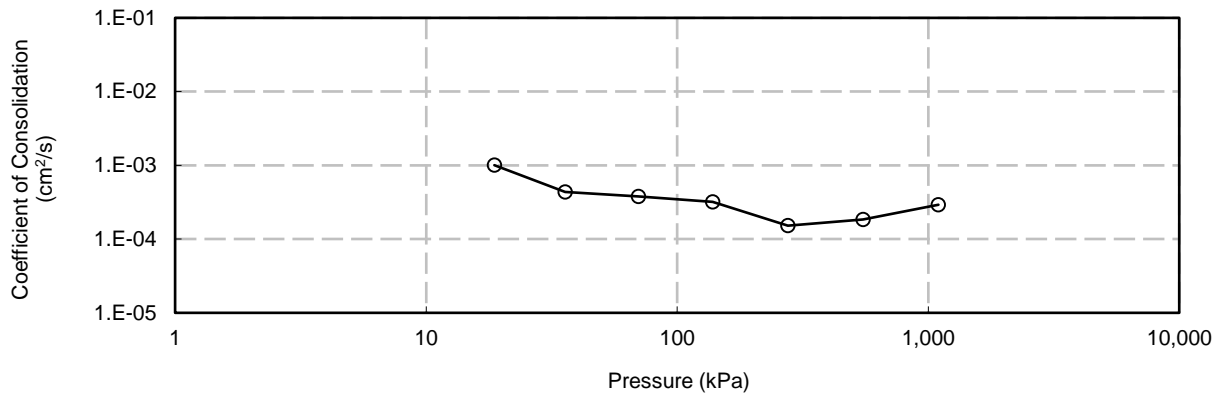
CONSOLIDATION TEST SUMMARY				FIGURE B14				
SAMPLE IDENTIFICATION								
Borehole No. :	2	Sample No. :	TW12					
		Sample Depth (m) :	9.1- 9.6					
TEST CONDITIONS								
Test Type :	Laboratory Standard	Date Started :	17-Apr-18					
Load Duration (hr) :	24	Date Completed :	27-Apr-18					
SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL								
Sample Height (mm) :	19.04	Unit Weight (kN/m ³) :	16.31					
Sample Diameter (mm) :	63.44	Dry Unit Weight (kN/m ³) :	10.61					
Area (cm ²) :	31.61	Specific Gravity :	2.71					
Volume (cm ³) :	60.18	Solid Height (mm) :	7.6					
Water Content (%) :	53.70	Volume of Solids (cm ³) :	24.01					
Wet Mass (g) :	100.12	Volume of Voids (cm ³) :	36.17					
Dry Mass (g) :	65.10	Degree of Saturation (%) :	96.82					
TEST COMPUTATIONS								
Stress (kPa)	Initial Height (mm)	Final Height (mm)	Void Ratio	t ₉₀ (min)	C _v (cm ² /s)	m _v (m ² /kN)	k (cm/s)	
1.567	19.04	19.04	1.51					
18.7	19.04	18.63	1.45	12.25	1.01E-03	1.26E-03	1.24E-07	
35.83	18.63	18.39	1.42	27.56	4.35E-04	7.65E-04	3.26E-08	
70.09	18.39	17.95	1.36	30.25	3.78E-04	6.92E-04	2.57E-08	
138.6	17.95	17.21	1.26	33.06	3.19E-04	6.03E-04	1.89E-08	
275.7	17.21	15.96	1.10	60.06	1.52E-04	5.31E-04	7.92E-09	
549.8	15.96	14.69	0.93	42.25	1.84E-04	2.89E-04	5.21E-09	
1098.0	14.69	13.57	0.79	22.56	2.93E-04	1.39E-04	3.99E-09	
275.7	13.57	13.74	0.81					
70.09	13.74	14.24	0.87					
35.83	14.24	14.74	0.94					
SAMPLE DIMENSIONS AND PROPERTIES _ FINAL								
Sample Height (mm) :	14.74	Unit Weight (kN/m ³) :	18.66					
Sample Diameter (mm) :	63.44	Dry Unit Weight (kN/m ³) :	13.22					
Area (cm ²) :	31.61	Specific Gravity :	2.71					
Volume (cm ³) :	46.59	Solid Height (mm) :	7.6					
Water Content (%) :	41.20	Volume of Solids (cm ³) :	23.17					
Wet Mass (g) :	88.67	Volume of Voids (cm ³) :	23.42					
Dry Mass (g) :	62.81							
Project No. : 1-17-0822 Date : May, 2018				 Terraprobe Inc.				Prepared By : SD Checked By : RA

CONSOLIDATION TEST**FIGURE B15**

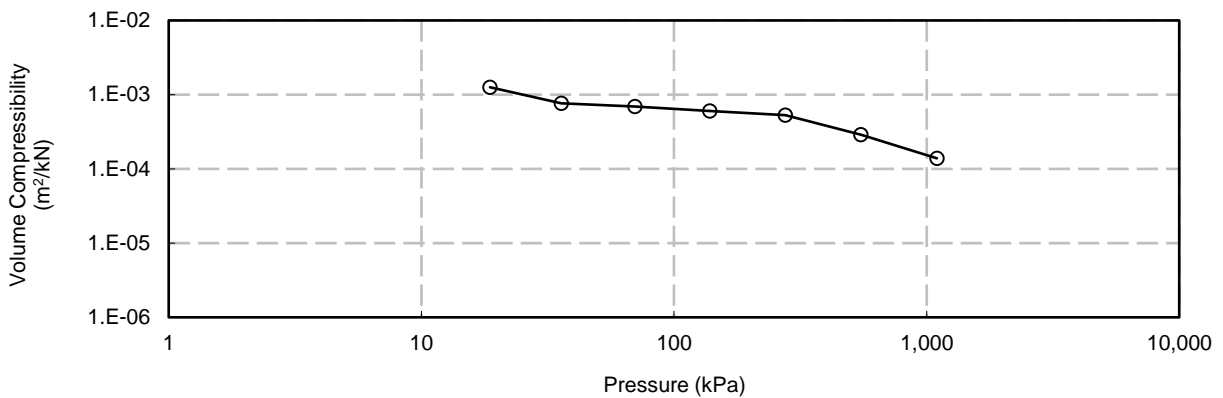
Anthony Creek Culvert (Site 39E-212)

BH 2, TW 12

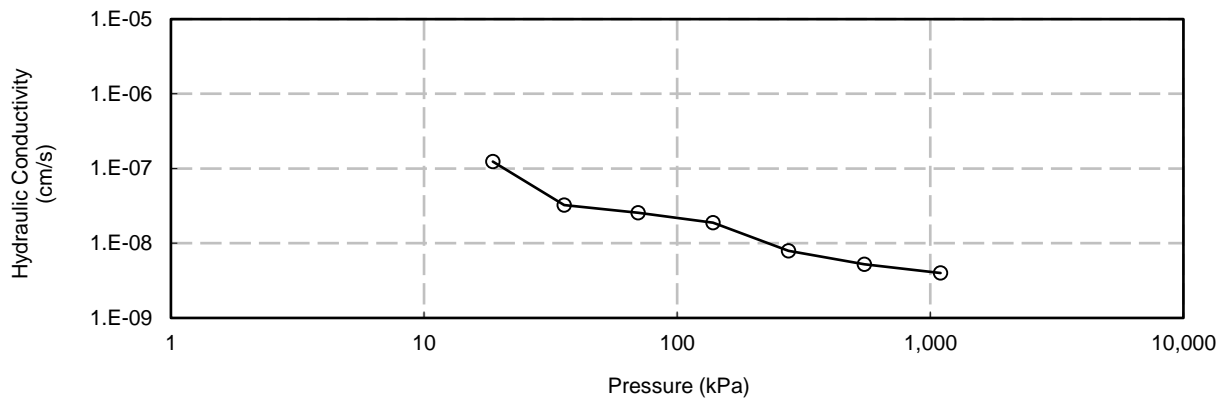
Cv vs Pressure



mv vs Pressure



k vs Pressure



Project No. : 1-17-0822
Date : May, 2018

 **Terraprobe Inc.**

Prepared By : SD
Checked By : RA

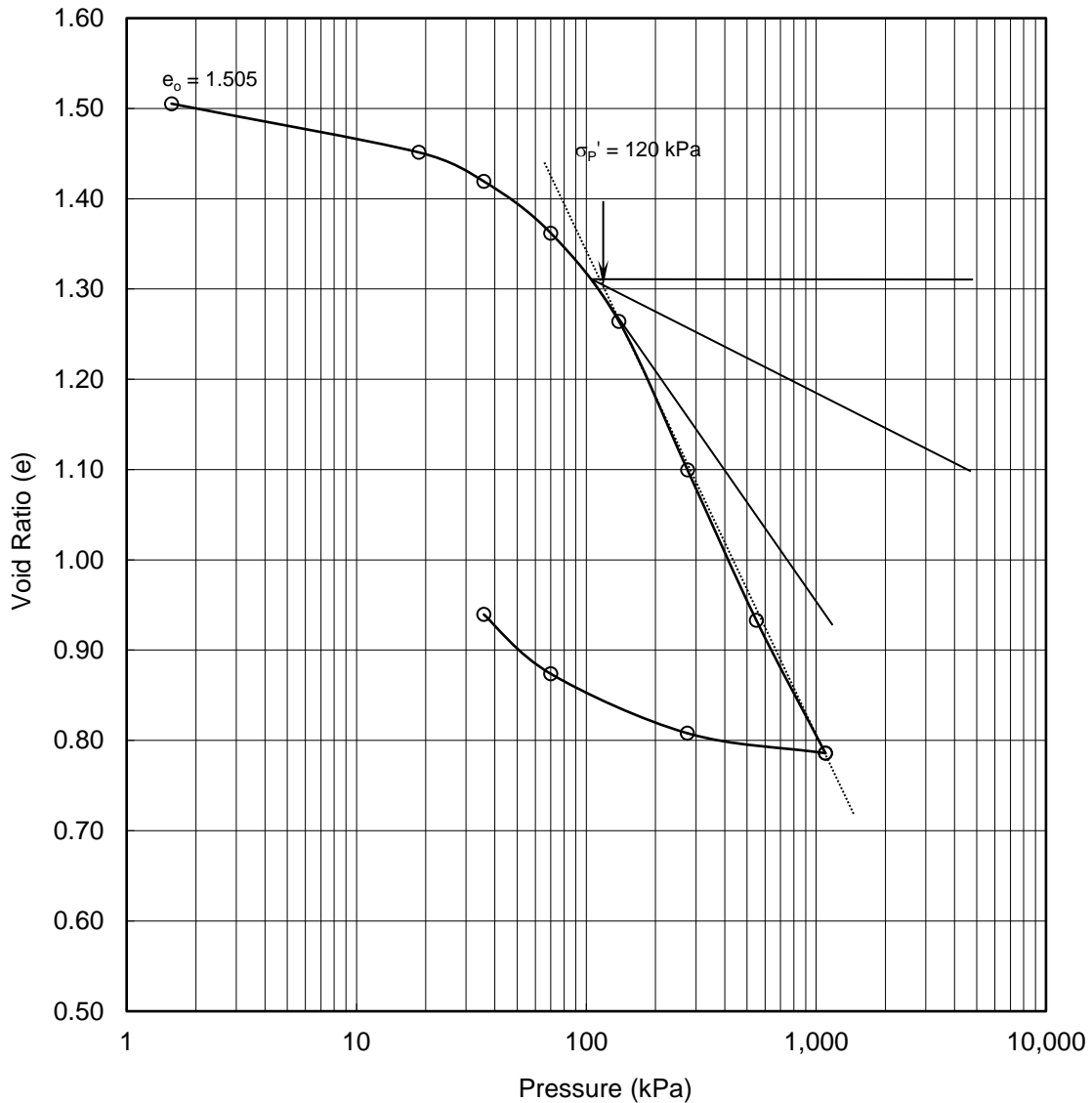
CONSOLIDATION TEST

FIGURE B16

Anthony Creek Culvert (Site 39E-212)

BH 2, TW 12

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

$e_o =$	1.51	$\omega_L =$	62%	$\sigma_{v0}' =$	120.0± kPa
$\omega =$	54%	$\omega_P =$	25%	$\sigma_P' =$	120.0 kPa
$\gamma =$	16.3 kN/m ³	PI =	37%		
Gs =	2.71				

Project No. : 1-17-0822
Date : May, 2018



Prepared By : SD
Checked By : RA

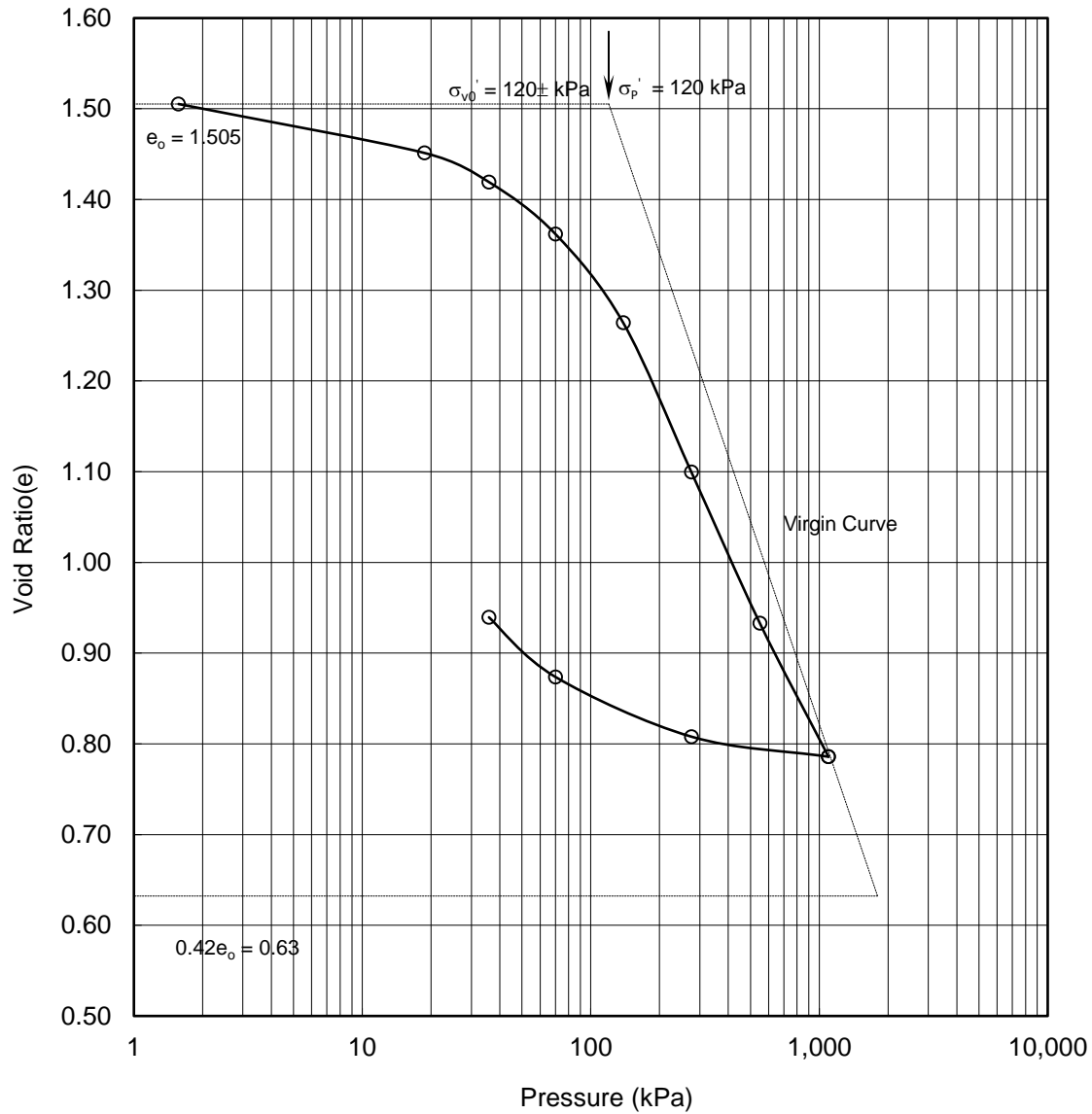
CONSOLIDATION TEST

FIGURE B17

Anthony Creek Culvert (Site 39E-212)

BH 2, TW 12

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

$e_o =$	1.51	$\omega_L =$	62%	$\sigma_{v0}' =$	120.0± kPa
$\omega =$	54%	$\omega_P =$	25%	$\sigma_P' =$	120.0 kPa
$\gamma =$	16.3 kN/m ³	PI =	37%	$C_c =$	0.742
Gs =	2.71			$C_r =$	0.074

Project No. : 1-17-0822
Date : May, 2018



Prepared By : SD
Checked By : RA

**CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
(ASTM D4767-11)**

Figure B18

SPECIMEN IDENTIFICATION

Borehole/Sample No.:	BH3 - TW6	Sample Type:	Intact
Sample Depth (m):	4.6 - 5.0	Soil Classification:	Silty Clay
Liquid Limit:	-	Specific Gravity:	2.72 (assumed)
Plastic limit:	-		

INITIAL SPECIMEN DIMENSIONS AND PROPERTIES

Test No.:	1	2	3
Specimen Height, (mm)	139.2	139.6	137.8
Specimen Diameter, (mm)	68.8	68.6	68.2
Natural Water Content (Cuttings/Specimen), (%)	60.6%	60.6%	53.6%
Void Ratio	1.734	1.717	1.483
Degree Of Saturation, (%)	95.0%	96.0%	98.3%
Dry Unit Weight (kN/m ³)	9.76	9.82	10.74

CONSOLIDATION

Cell Pressure, σ_3 , (kPa)	555	590	380
Back Pressure, (kPa)	520	520	240
Pore Pressure Parameter "B"	0.96	0.96	0.98
Effective Consolidation Stress, σ_c , (kPa)	35	70	140
Water Content After Consolidation, (%)	58.0%	53.2%	48.5%
Void Ratio After Consolidation	1.578	1.446	1.319
Degree Of Saturation After Consolidation, (%)	36.8%	36.8%	36.8%
Dry Unit Weight After Consolidation, (kN/m ³)	10.3	10.9	11.5

SHEARING/FAILURE

Max. Deviator Stress, $(\sigma_1 - \sigma_3)$, (kPa)	30	45	78
Effective Major Principal Stress At Failure, σ'_1 , (kPa)	43	73	132
Effective Minor Principal Stress At Failure, σ'_3 , (kPa)	13	28	54
Axial Strain At Maximum $(\sigma_1 - \sigma_3)$, (%)	3.7%	3.3%	5.2%
Average Rate of Strain, (%/hr)	0.50	0.50	0.50
Time to Failure, (hr)	7.4	6.6	10.4

Test Notes:

Specimen Saturation Method	Dry	Dry	Dry
Failure Criterion	Max. $\sigma_1 - \sigma_3$	Max. $\sigma_1 - \sigma_3$	Max. $\sigma_1 - \sigma_3$
Membrane Thickness Correction Applied, Y/N	N	N	N

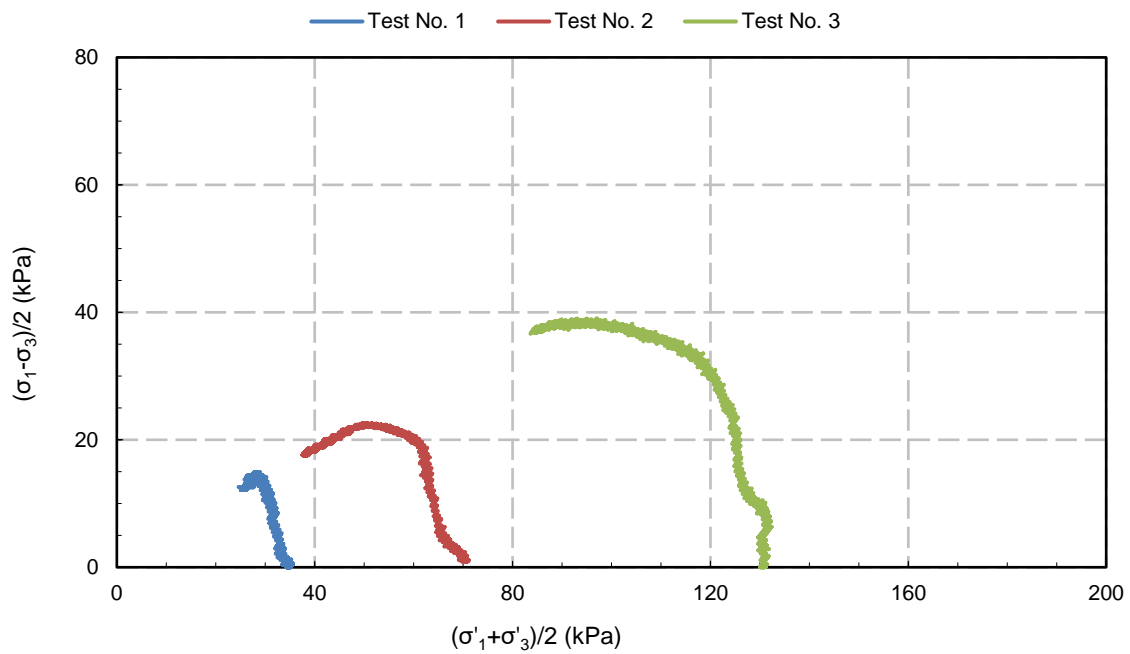
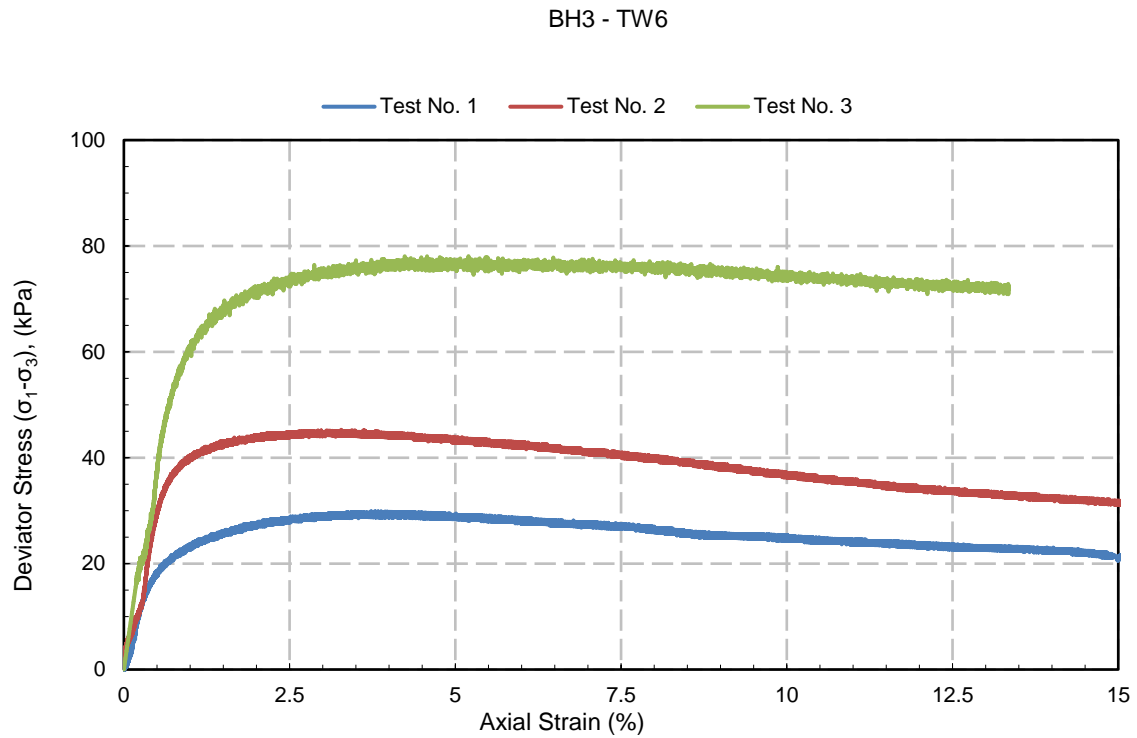
Project No.: 1-17-0822
Date: October, 2019



Prepared By : SD
Checked By : RA

**CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
(ASTM D4767-11)**

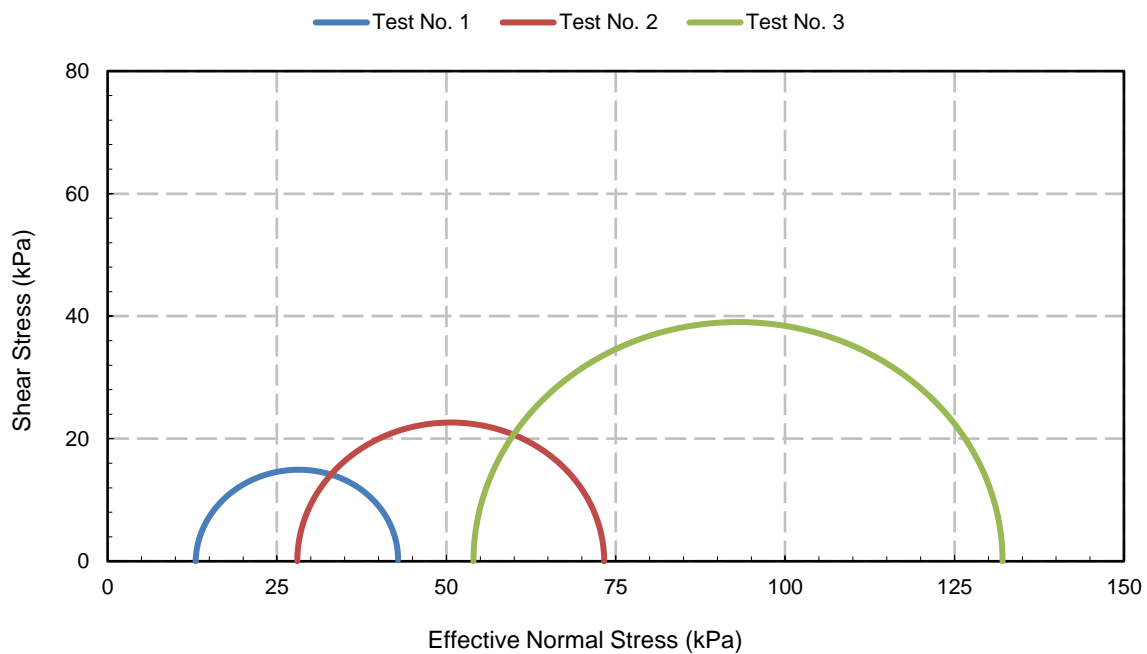
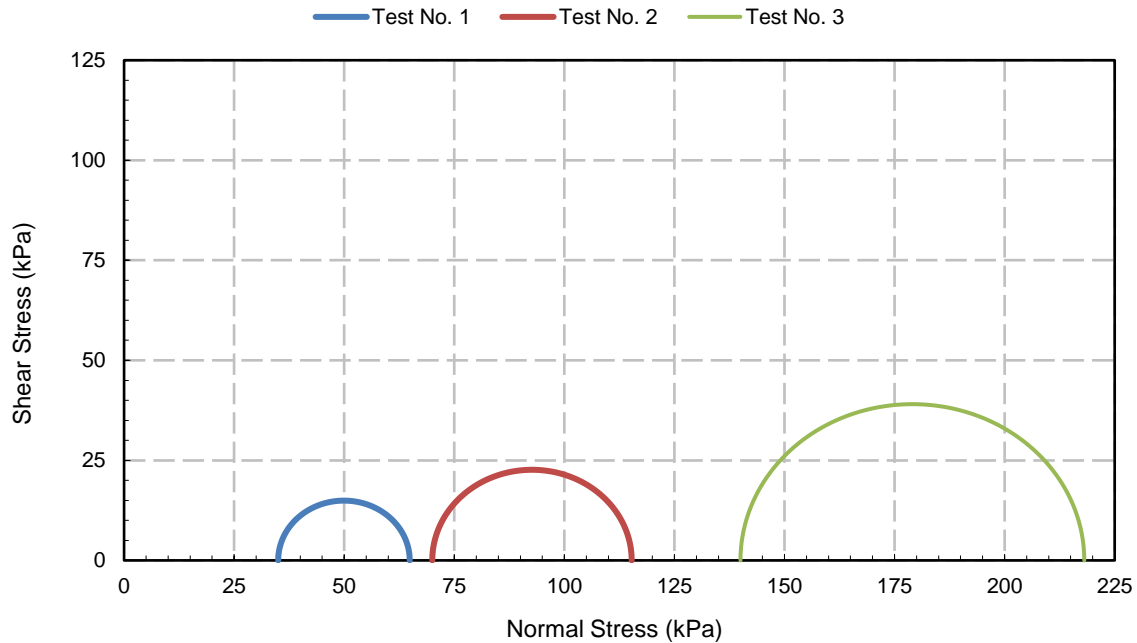
Figure B19



**CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
(ASTM D4767-11)**

Figure B20

BH3 - TW6



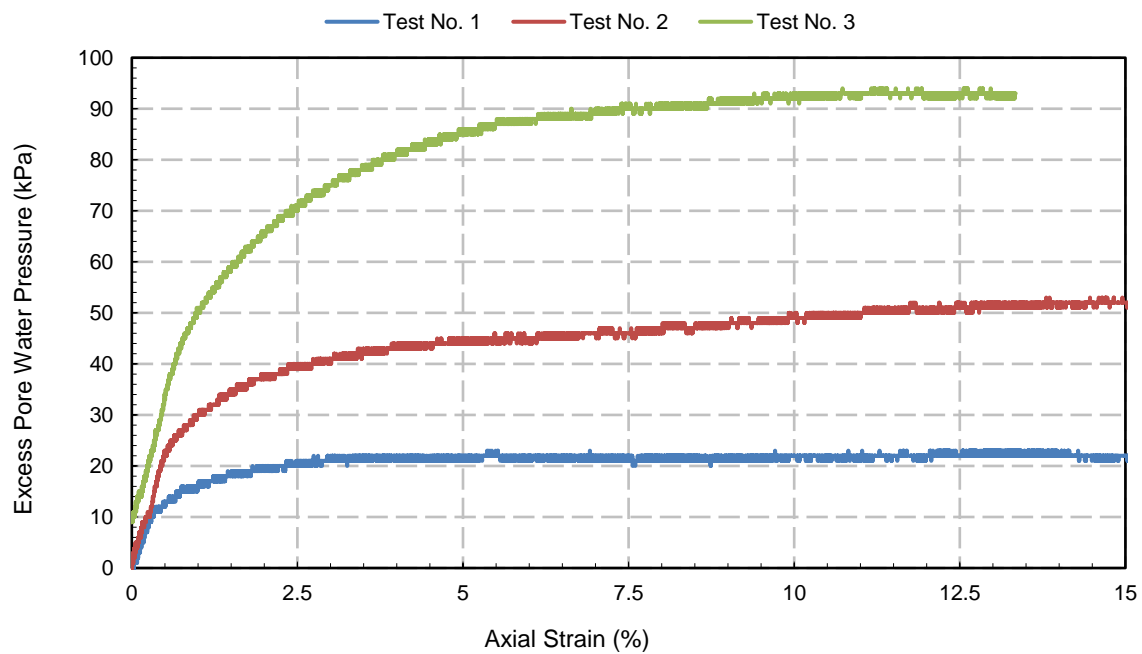
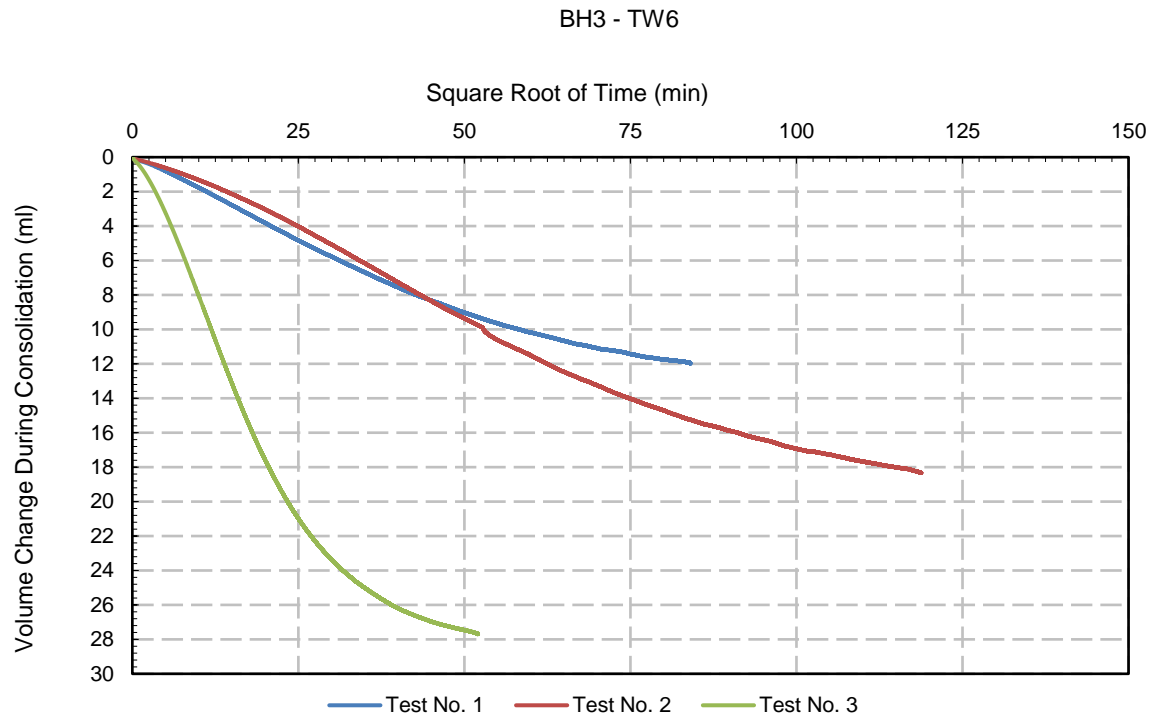
Project No.: 1-17-0822
Date: October, 2019



Prepared By : SD
Checked By : RA

**CONSOLIDATED UNDRAINED TRIAXIAL COMPRESSION TEST
(ASTM D4767-11)**

Figure B21

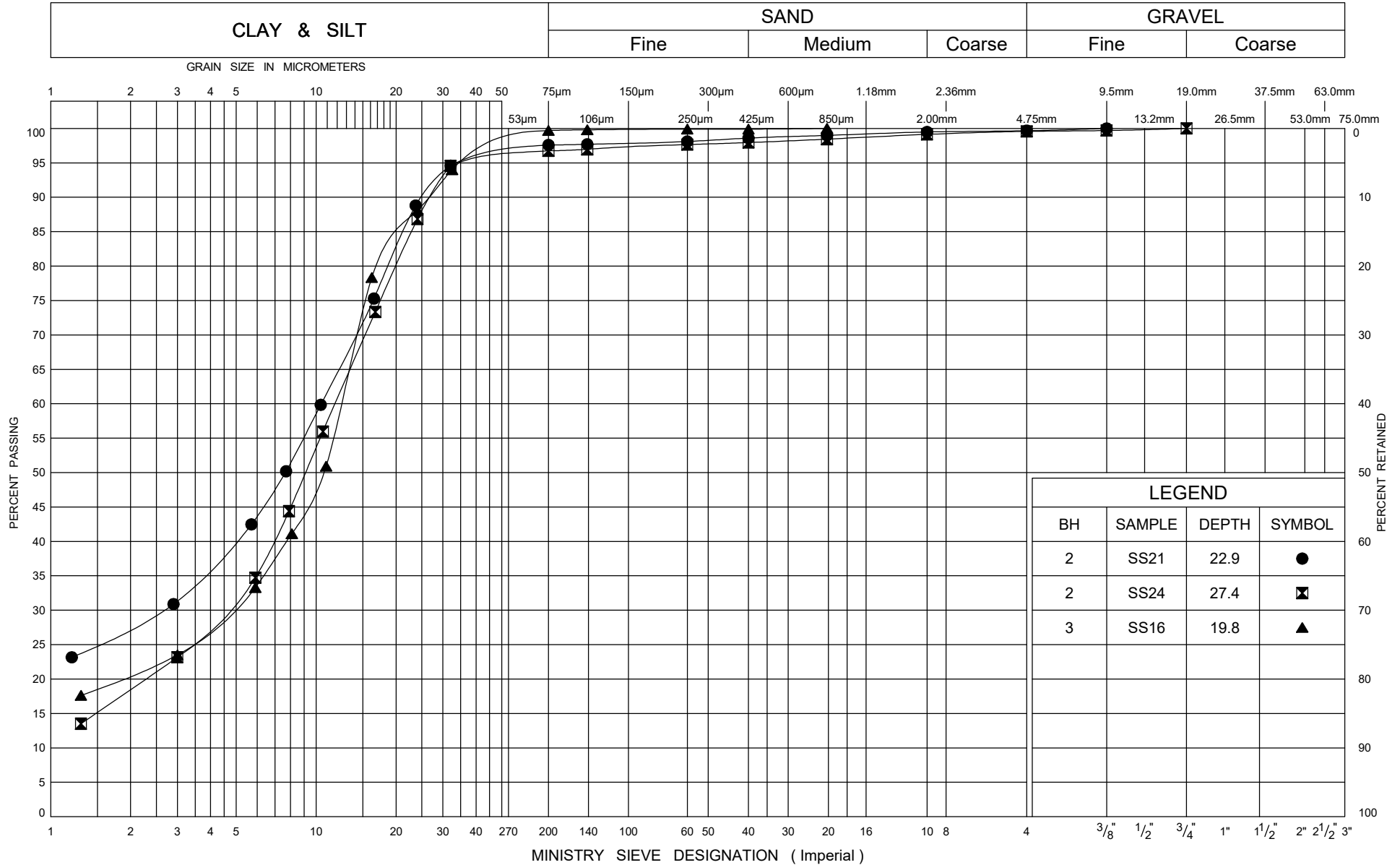


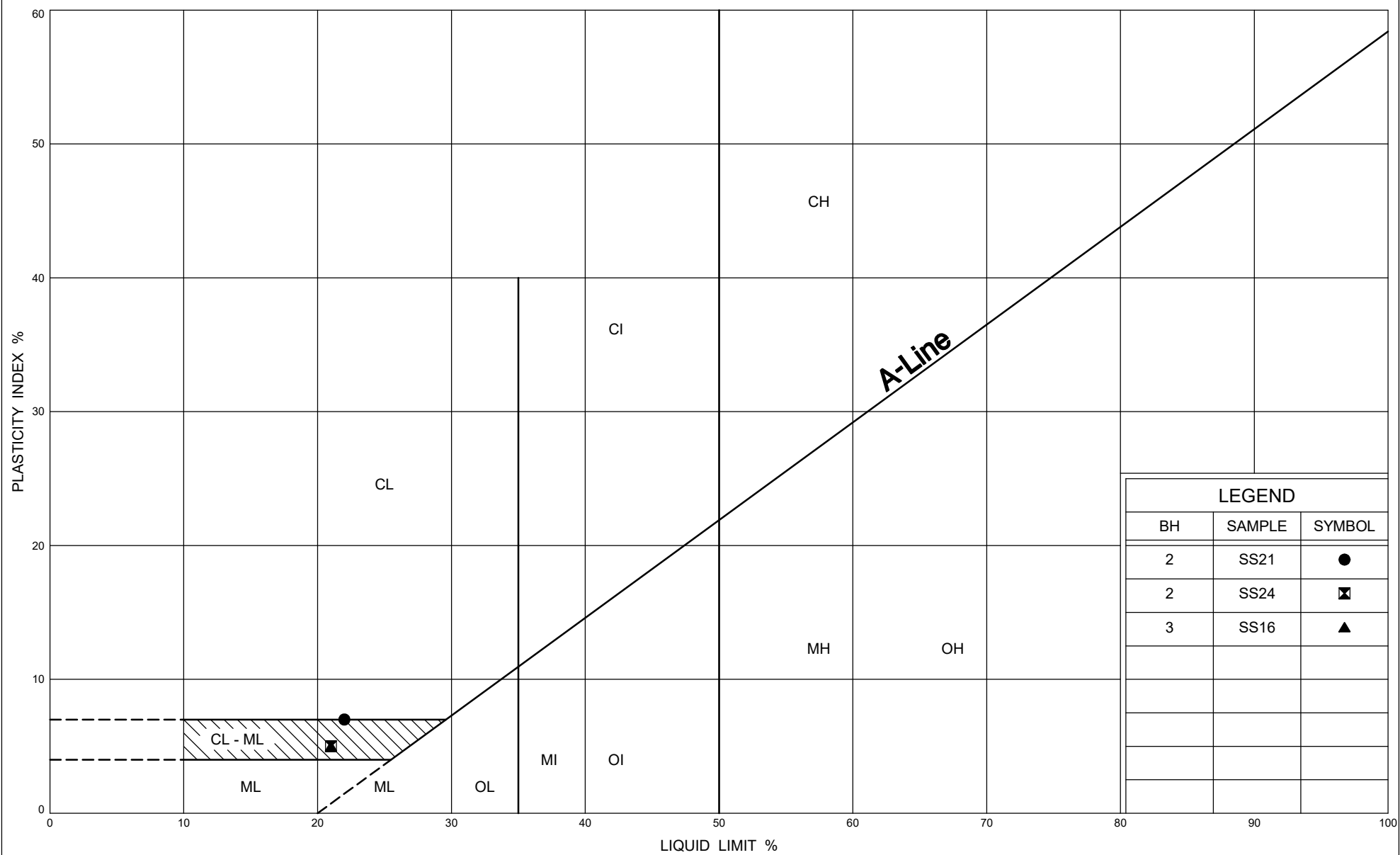
Project No.: 1-17-0822
Date: October, 2019



Prepared By : SD
Checked By : RA

UNIFIED SOIL CLASSIFICATION SYSTEM





file: anthony creek culvert.gpj



Ministry of
Transportation

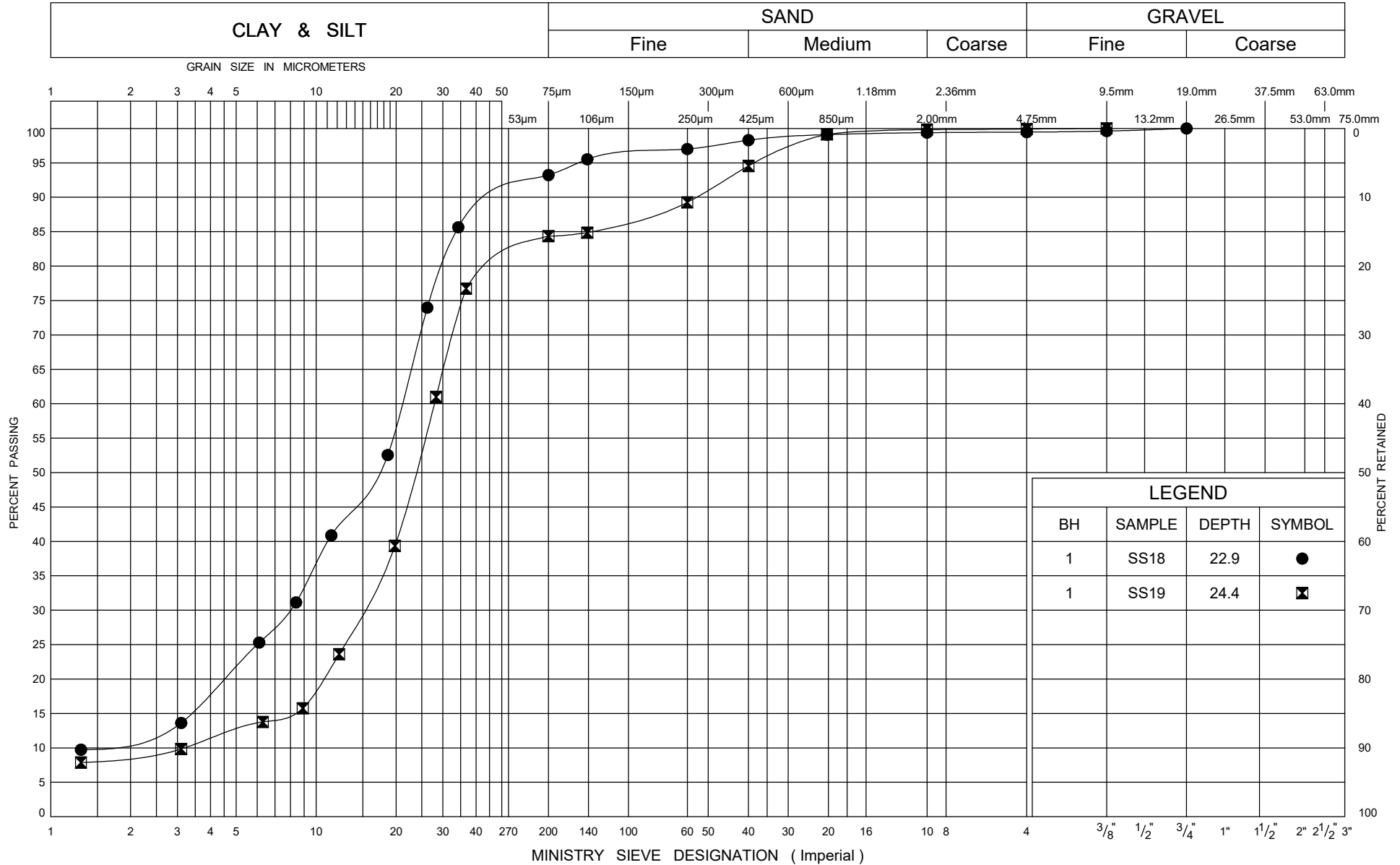
PLASTICITY CHART
CLAYEY SILT

FIG No B23

G W P 5165-12-00

Anthony Creek Culvert, Site 39E-212

UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION SILT

FIG No B24

G W P 5165-12-00

Anthony Creek Culvert, Site 39E-212



Ministry of
Transportation



FINAL REPORT

CA14434-MAY18 R1

1-17-0822 Anthony Creek

Prepared for

Terraprobe Inc

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Terraprobe Inc	Project Specialist	Brad Moore Hon. B.Sc
Address	11 Indell Lane Brampton, ON L6T 3Y3, Canada	Laboratory	SGS Canada Inc.
Contact	Sepideh D. Monfared	Address	185 Concession St., Lakefield ON, K0L 2H0
Telephone	(905) 796-2650	Telephone	705-652-2143
Facsimile	(905) 796-2250	Facsimile	705-652-6365
Email	smonfared@terraprobe.ca	Email	brad.moore@sgs.com
Project	1-17-0822 Anthony Creek	SGS Reference	CA14434-MAY18
Order Number		Received	05/15/2018
Samples	Soil (2)	Approved	05/22/2018
		Report Number	CA14434-MAY18 R1
		Date Reported	09/18/2019

COMMENTS

Temperature of Sample upon Receipt: 7 degrees C
Cooling Agent Present: No
Custody Seal Present: No

SIGNATORIES

Brad Moore Hon. B.Sc

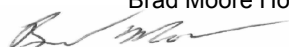




TABLE OF CONTENTS

First Page..... 1

Index..... 2

Results..... 3-4

QC Summary..... 5-6

Legend..... 7

Annexes..... 8-9



FINAL REPORT

CA14434-MAY18 R1

Client: Terraprobe Inc

Project: 1-17-0822 Anthony Creek

Project Manager: Sepideh D_Monfared

Samplers: Sepideh D_Manfared

PACKAGE: - Corrosivity Index (SOIL)

Sample Number	5	6
Sample Name	Anthony Creek, BH1, SS4, 71/2-9	Anthony Creek, BH3, SS4B (8'4"-81/2)
Sample Matrix	Soil	Soil
Sample Date	30/04/2018	01/05/2018

Parameter	Units	RL		Result	Result
Corrosivity Index					
Soil Redox Potential	mV	-		132	246
Sulphide	%	0.02		< 0.02	< 0.02
pH	no unit	0.05		8.42	8.43
Resistivity (calculated)	ohms.cm	-9999		6200	4900

PACKAGE: - General Chemistry (SOIL)

Sample Number	5	6
Sample Name	Anthony Creek, BH1, SS4, 71/2-9	Anthony Creek, BH3, SS4B (8'4"-81/2)
Sample Matrix	Soil	Soil
Sample Date	30/04/2018	01/05/2018

Parameter	Units	RL		Result	Result
General Chemistry					
Conductivity	uS/cm	2		161	204

PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6
Sample Name	Anthony Creek, BH1, SS4, 71/2-9	Anthony Creek, BH3, SS4B (8'4"-81/2)
Sample Matrix	Soil	Soil
Sample Date	30/04/2018	01/05/2018

Parameter	Units	RL		Result	Result
Metals and Inorganics					



FINAL REPORT

CA14434-MAY18 R1

Client: Terraprobe Inc
Project: 1-17-0822 Anthony Creek
Project Manager: Sepideh D_Monfared
Samplers: Sepideh D_Manfared

PACKAGE: - Metals and Inorganics (SOIL)

Sample Number	5	6
Sample Name	Anthony Creek, BH1, SS4, 71/2-9	Anthony Creek, BH3, SS4B (8'4"-81/2)
Sample Matrix	Soil	Soil
Sample Date	30/04/2018	01/05/2018

Parameter	Units	RL		Result	Result
Metals and Inorganics (continued)					
Moisture Content	%	0.1		33.3	23.2
Sulphate	µg/g	0.4		14	8.2

PACKAGE: - Other (ORP) (SOIL)

Sample Number	5	6
Sample Name	Anthony Creek, BH1, SS4, 71/2-9	Anthony Creek, BH3, SS4B (8'4"-81/2)
Sample Matrix	Soil	Soil
Sample Date	30/04/2018	01/05/2018

Parameter	Units	RL		Result	Result
Other (ORP)					
Chloride	µg/g	0.4		45	94



FINAL REPORT

CA14434-MAY18 R1

QC SUMMARY

Anions by IC
Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-IENVIIC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Chloride	DIO0325-MAY18	µg/g	0.4	<0.4	7	20	93	80	120	121	75	125
Sulphate	DIO0325-MAY18	µg/g	0.4	<0.4	3	20	98	80	120	107	75	125

Carbon/Sulphur
Method: ASTM E1915-07A | Internal ref.: ME-CA-IENVIARD-LAK-AN-020

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Sulphide	ECS0030-MAY18	%	0.02	<0.02	ND	20	108	80	120			

Conductivity
Method: SM 2510 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
Conductivity	EWL0273-MAY18	uS/cm	2	< 0.002	ND	10	99	90	110	NA		



QC SUMMARY

pH
Method: SM 4500 | Internal ref.: ME-CA-IENVIEWL-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%)		Spike Recovery (%)	Recovery Limits (%)	
								Low	High		Low	High
pH	EWL0273-MAY18	no unit	0.05	NA	0		100			NA		

Method Blank: a blank matrix that is carried through the entire analytical procedure. Used to assess laboratory contamination.

Duplicate: Paired analysis of a separate portion of the same sample that is carried through the entire analytical procedure. Used to evaluate measurement precision.

LCS/Spike Blank: Laboratory control sample or spike blank refer to a blank matrix to which a known amount of analyte has been added. Used to evaluate analyte recovery and laboratory accuracy without sample matrix effects.

Matrix Spike: A sample to which a known amount of the analyte of interest has been added. Used to evaluate laboratory accuracy with sample matrix effects.

Reference Material: a material or substance matrix matched to the samples that contains a known amount of the analyte of interest. A reference material may be used in place of a matrix spike.

RL: Reporting limit

RPD: Relative percent difference

AC: Acceptance criteria

Multielement Scan Qualifier: as the number of analytes in a scan increases, so does the chance of a limit exceedance by random chance as opposed to a real method problem. Thus, in multielement scans, for the LCS and matrix spike, up to 10% of the analytes may exceed the quoted limits by up to 10% absolute and the spike is considered acceptable.

Duplicate Qualifier: for duplicates as the measured result approaches the RL, the uncertainty associated with the value increases dramatically, thus duplicate acceptance limits apply only where the average of the two duplicates is greater than five times the RL.

Matrix Spike Qualifier: for matrix spikes, as the concentration of the native analyte increases, the uncertainty of the matrix spike recovery increases. Thus, the matrix spike acceptance limits apply only when the concentration of the matrix spike is greater than or equal to the concentration of the native analyte.

LEGEND

FOOTNOTES

NSS Insufficient sample for analysis.

RL Reporting Limit.

↑ Reporting limit raised.

↓ Reporting limit lowered.

NA The sample was not analysed for this analyte

ND Non Detect

Samples analysed as received. Solid samples expressed on a dry weight basis. "Temperature Upon Receipt" is representative of the whole shipment and may not reflect the temperature of individual samples.

Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act" published by the Ministry and dated March 9, 2004 as amended.

SGS provides criteria information (such as regulatory or guideline limits and summary of limit exceedances) as a service. Every attempt is made to ensure the criteria information in this report is accurate and current, however, it is not guaranteed. Comparison to the most current criteria is the responsibility of the client and SGS assumes no responsibility for the accuracy of the criteria levels indicated. This document is issued, on the Client's behalf, by the Company under its General Conditions of Service available on request and accessible at http://www.sgs.com/terms_and_conditions.htm. The Client's attention is drawn to the limitation of liability, indemnification and jurisdiction issues defined therein. Any other holder of this document is advised that information contained hereon reflects the Company's findings at the time of its intervention only and within the limits of Client's instructions, if any. The Company's sole responsibility is to its Client and this document does not exonerate parties to a transaction from exercising all their rights and obligations under the transaction documents.

This report must not be reproduced, except in full. This report supersedes all previous versions.

-- End of Analytical Report --

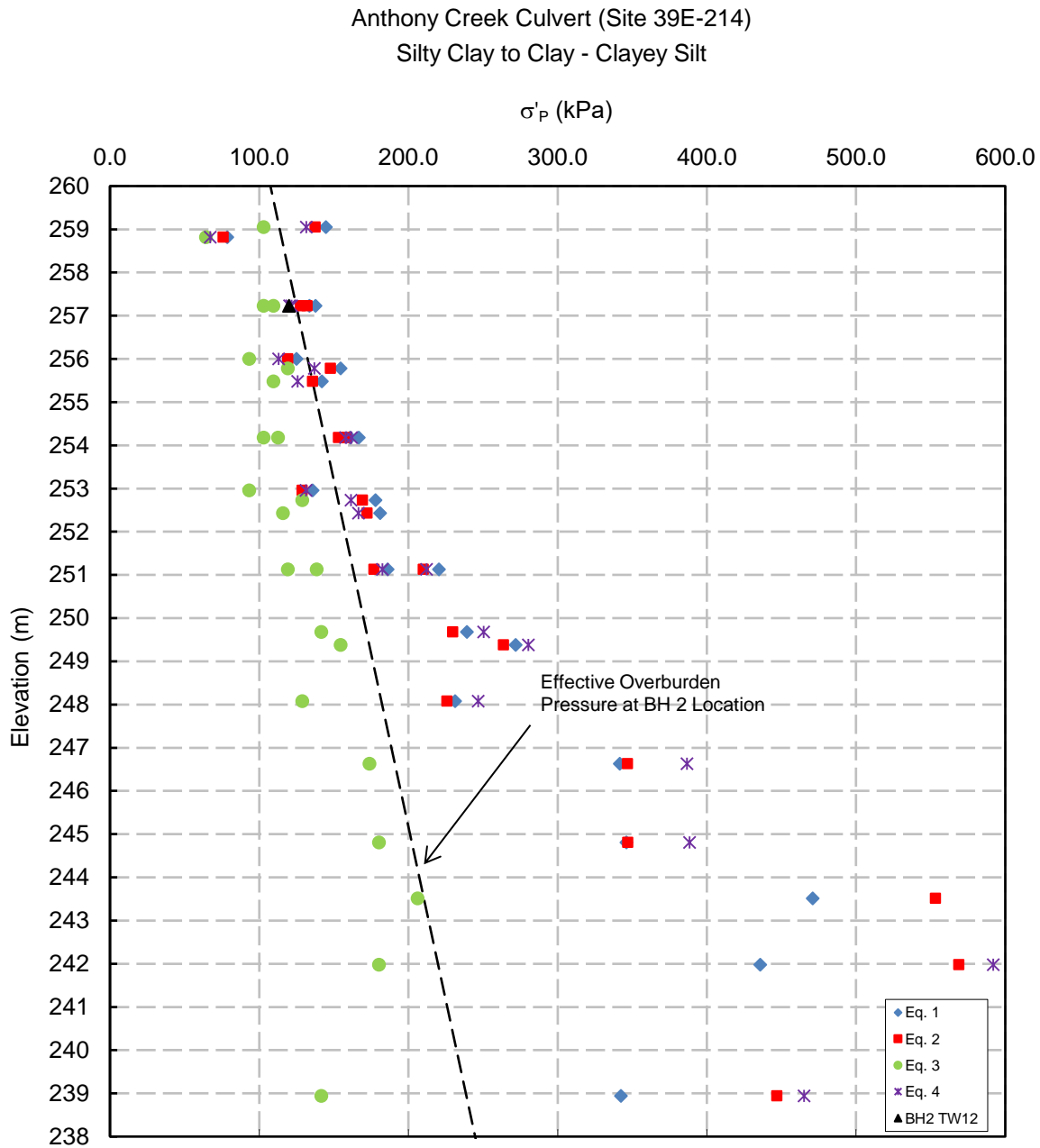
APPENDIX C

Soil Design Parameters



PREDICTED AND MEASURED PRECONSOLIDATION STRESSES

FIGURE C1



Eq. 1 $\sigma'_p = Cu / (0.11 + 0.0037 * I_p)$

Chandler (1988)

Eq. 2 $\sigma'_p = 22 * (I_p^{-0.48}) * Cu$

Mayne and Mitchell (1988)

Eq. 3 derived from $OCR = 3.22 * Cu / \sigma'_{v0}$

Mayne and Mitchell (1988)

Eq. 4 $\sigma'_p = 222 / LL * Cu$

Hansbo (1957)

Project No. : 1-17-0822

Date : June, 2018



Terraprobe Inc.

Prepared by : SD

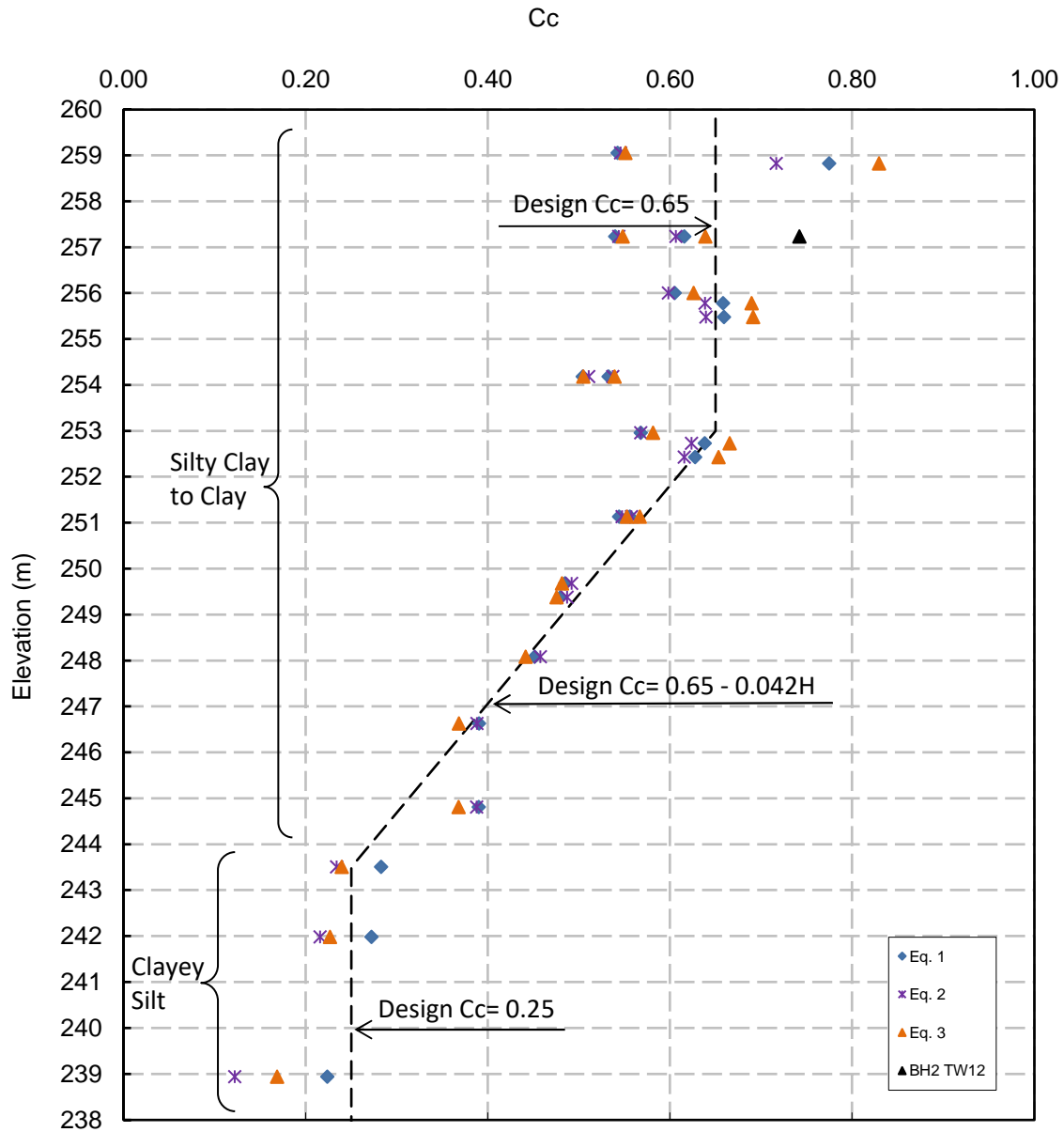
Checked by : RA

PREDICTED AND MEASURED COMPRESSION INDICES

FIGURE C2

Anthony Creek Culvert (Site 39E-212)

Silty Clay to Clay - Clayey Silt



Eq. 1 $C_c = 0.01 * \omega$

Eq. 2 $C_c = 0.479 * \ln(\omega) - 1.367$

Eq. 3 $C_c = 0.012 * \omega - 0.1$

Osterberg (1972)

Lav & Ansal (2001)

Lav & Ansal (2001)

Project No. : 1-17-0822

Date : June, 2018



Prepared by : SD

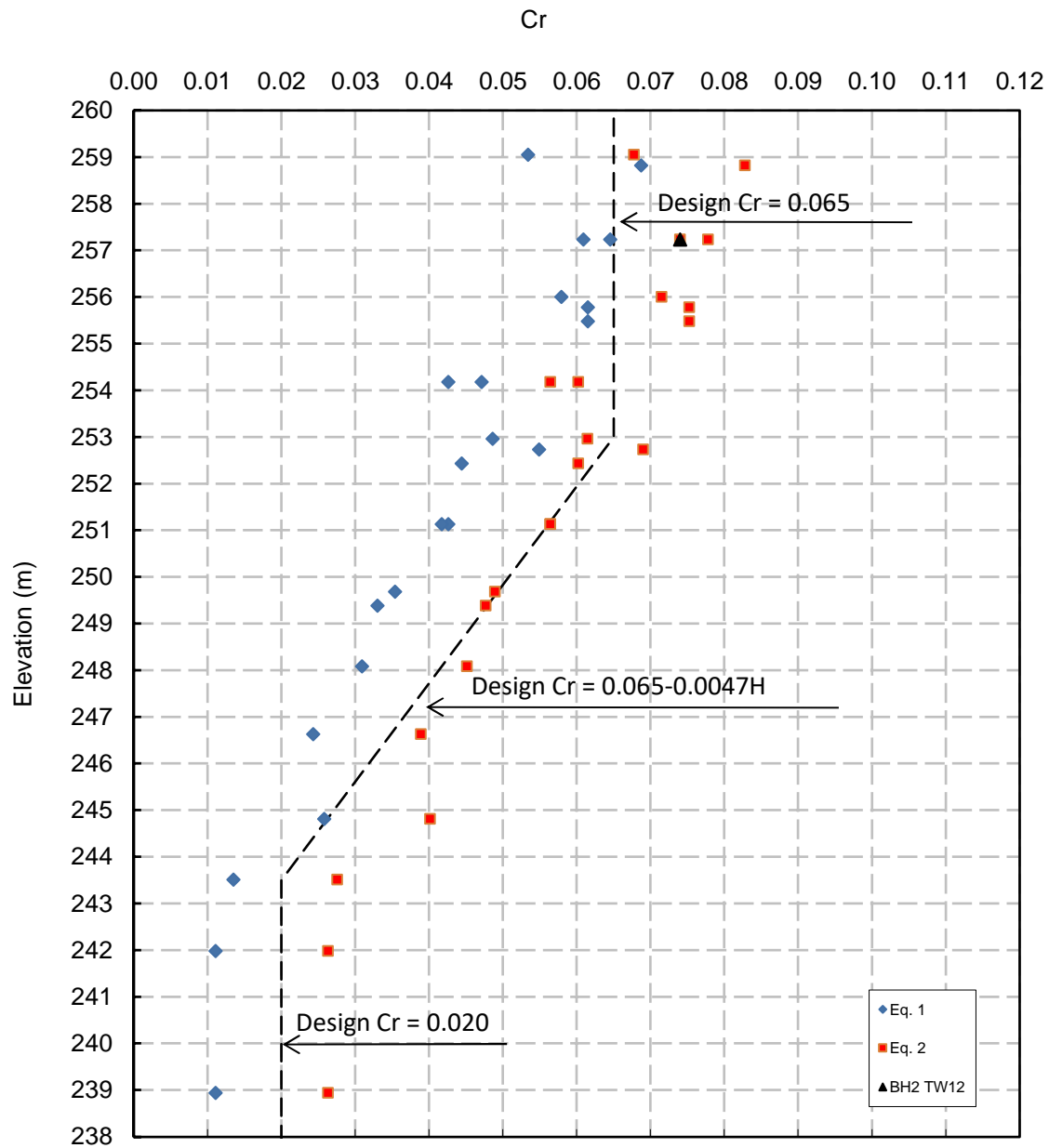
Checked by : RA

PREDICTED AND MEASURED RECOMPRESSION INDICES

FIGURE C3

Anthony Creek Culvert (Site 39E-212)

Silty Clay to Clay - Clayey Silt



Eq. 1 $Cr = Cc / 5 \sim Cc / 10$

Das (1993)

Eq. 2 $Cr = 0.000463 * LL * Gs$

Nagaraj & Murty (1985)

Project No. : 1-17-0822

Date : June, 2018



Prepared by : SD

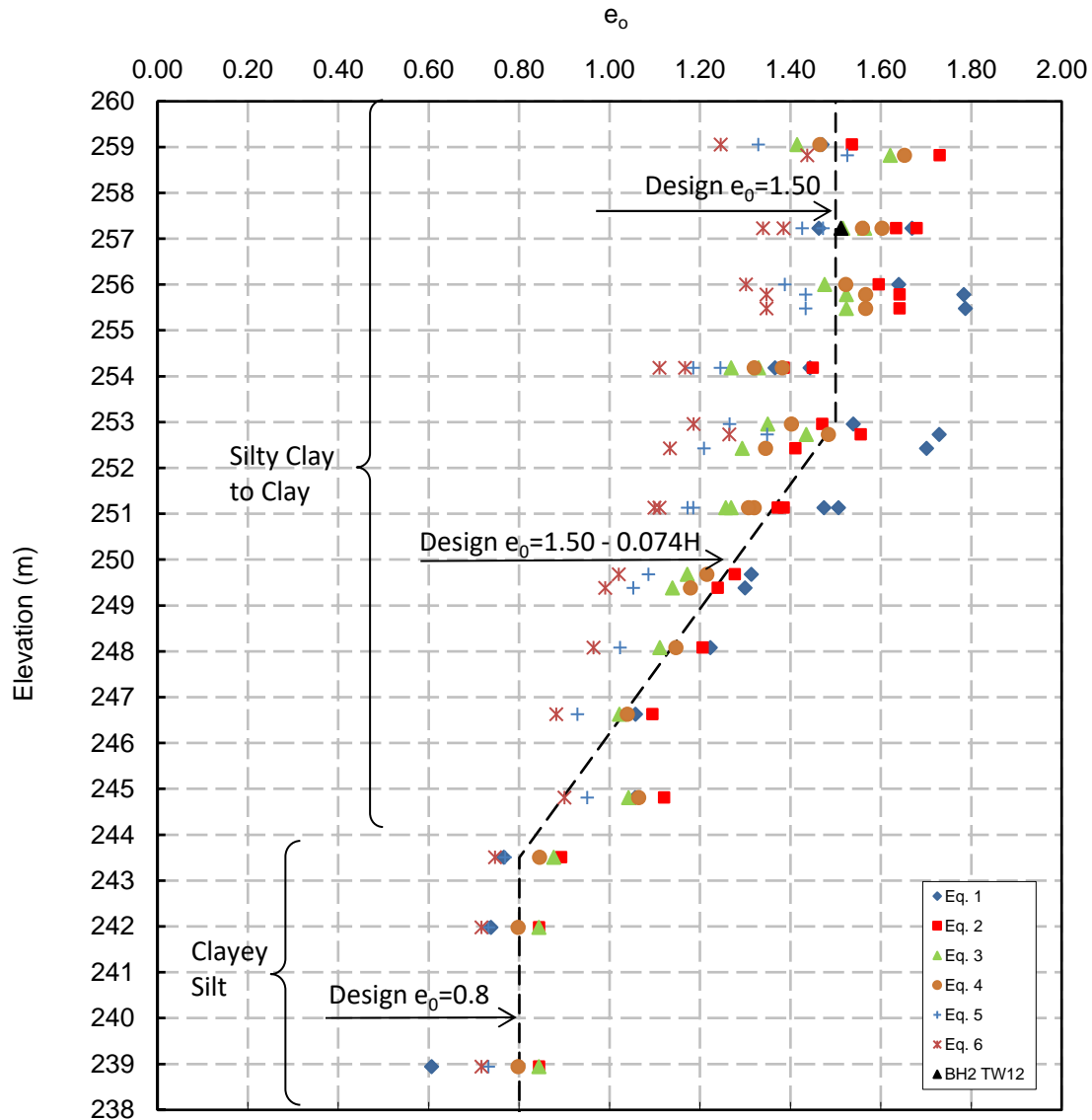
Checked by : RA

PREDICTED AND MEASURED VOID RATIOS

FIGURE C4

Anthony Creek Culvert (Site 39E-212)

Silty Clay to Clay - Clayey Silt



Eq. 1 $e_o = w * G_s$

Eq. 2 $e_o = (C_c / 0.141)^{0.4202} * G_s^{0.4958} - 1$

Eq. 3 $e_o = C_c / 0.37 - 0.003 * LL - 0.0004 * \omega + 0.34$

Eq. 4 $e_o = G_s * (2C_c)^{0.4167} - 1$

Eq. 5 $e_o = e^{(\ln C_c + 1.282) / 1.272}$

Eq. 6 $e_o = (C_c + 0.10) / 0.40$

when saturated

derived from Rendon - Herrero (1983)

derived from Azzouz et al. (1976)

derived from Rendon - Herrero (1980)

derived from Lav & Ansal (2001)

derived from Lav & Ansal (2001)

Project No. : 1-17-0822

Date : June, 2018

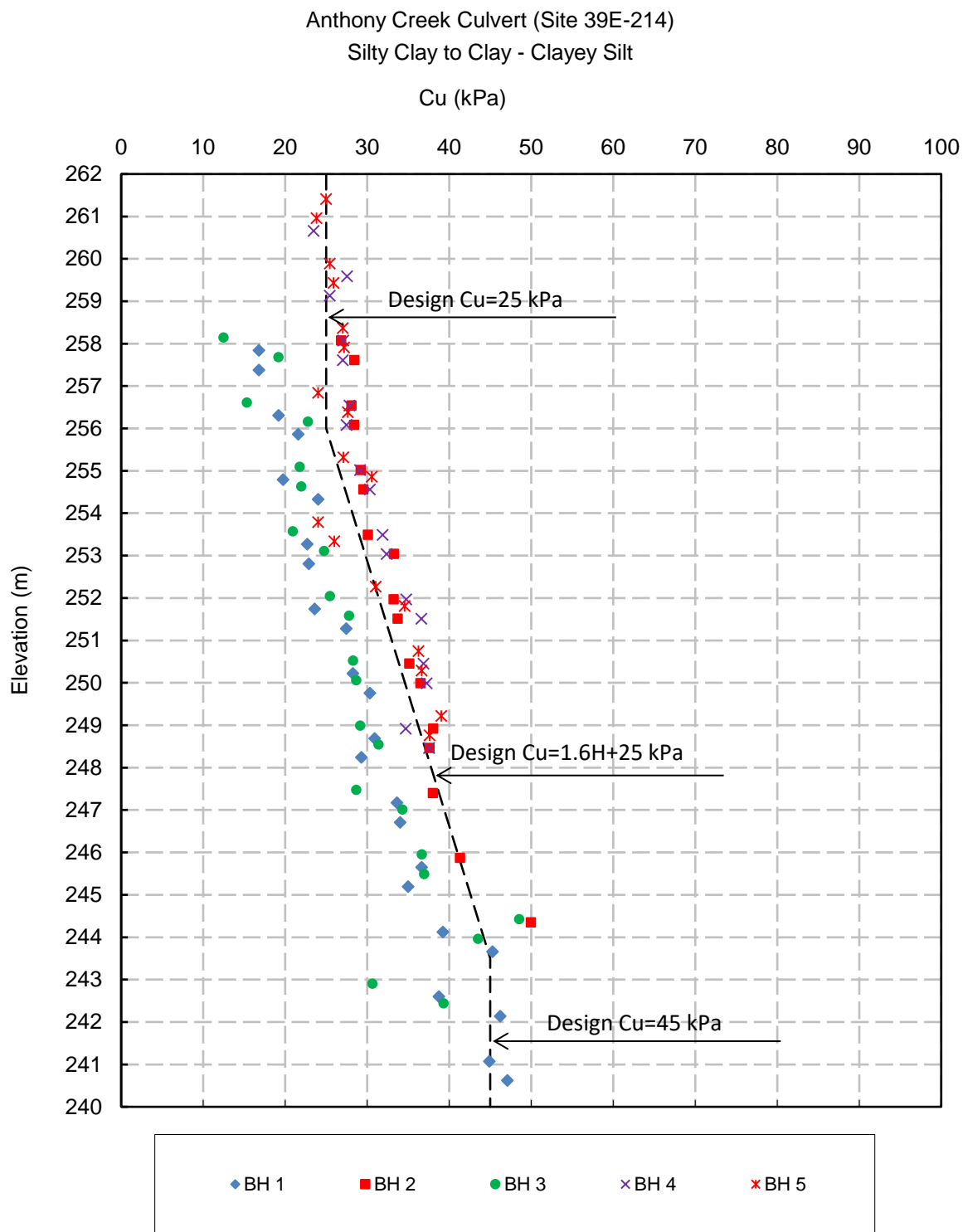


Prepared by : SD

Checked by : RA

UNDRAINED SHEAR STRENGTH

FIGURE C5



Field vane shear strengths were corrected based on Aas, et al. (1986)

Project No. : 1-17-0822

Date : June, 2018



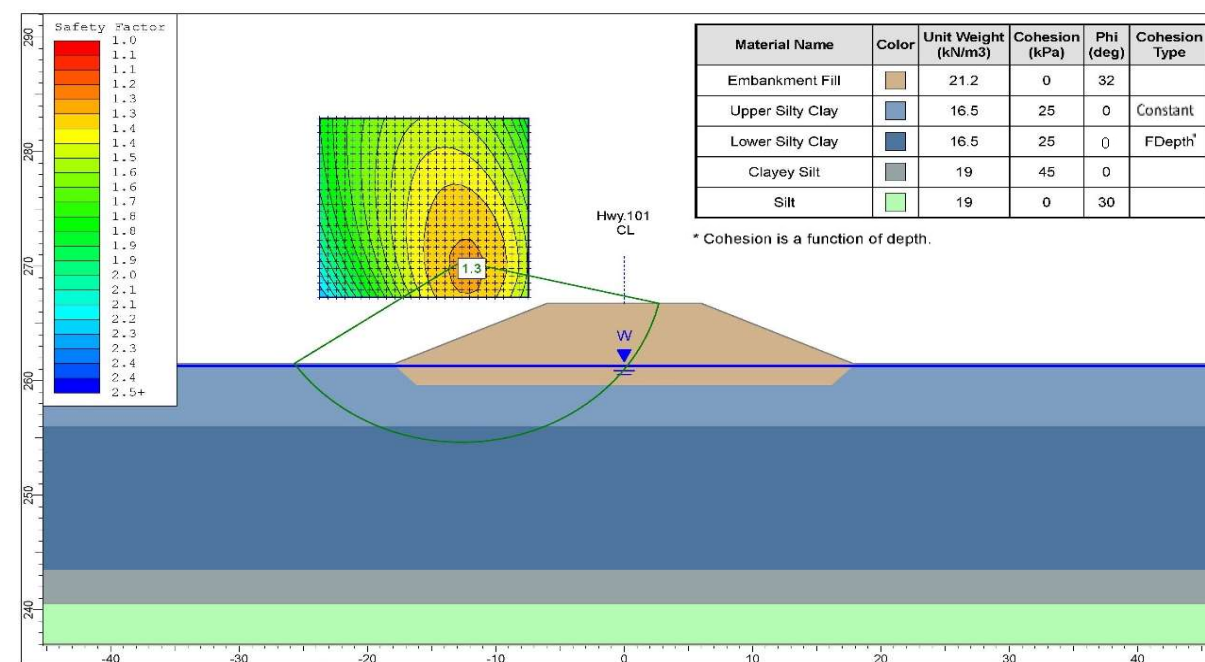
Prepared by : SD

Checked by : RA

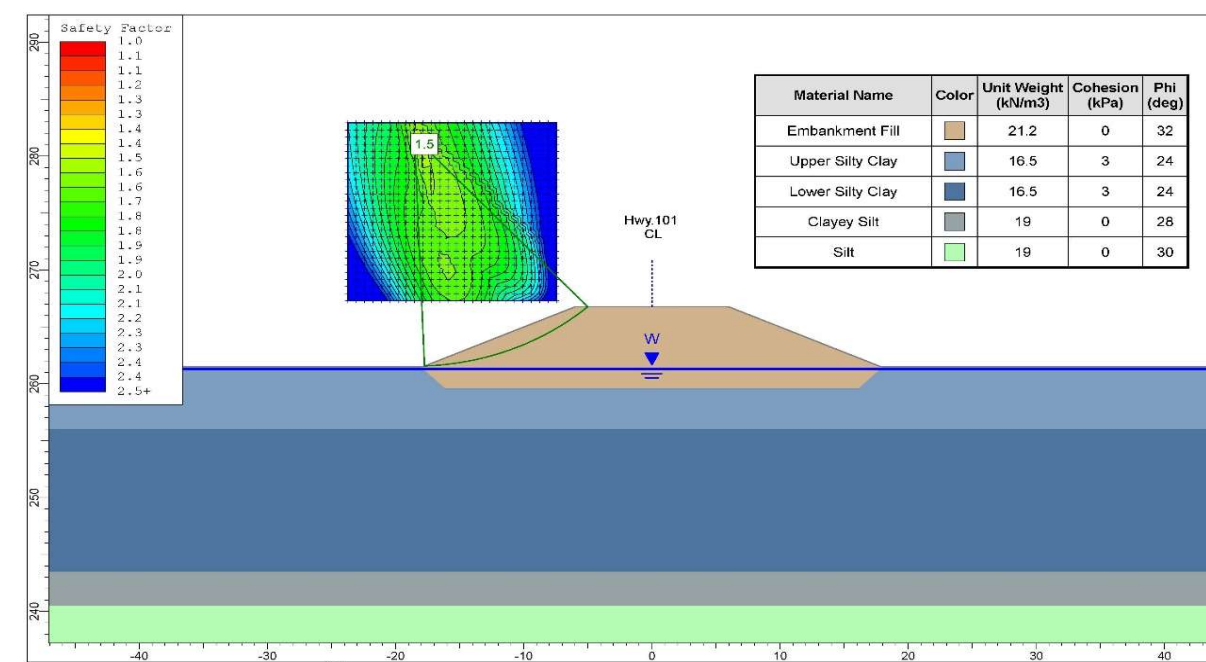
APPENDIX D

Slope Stability Models & Results

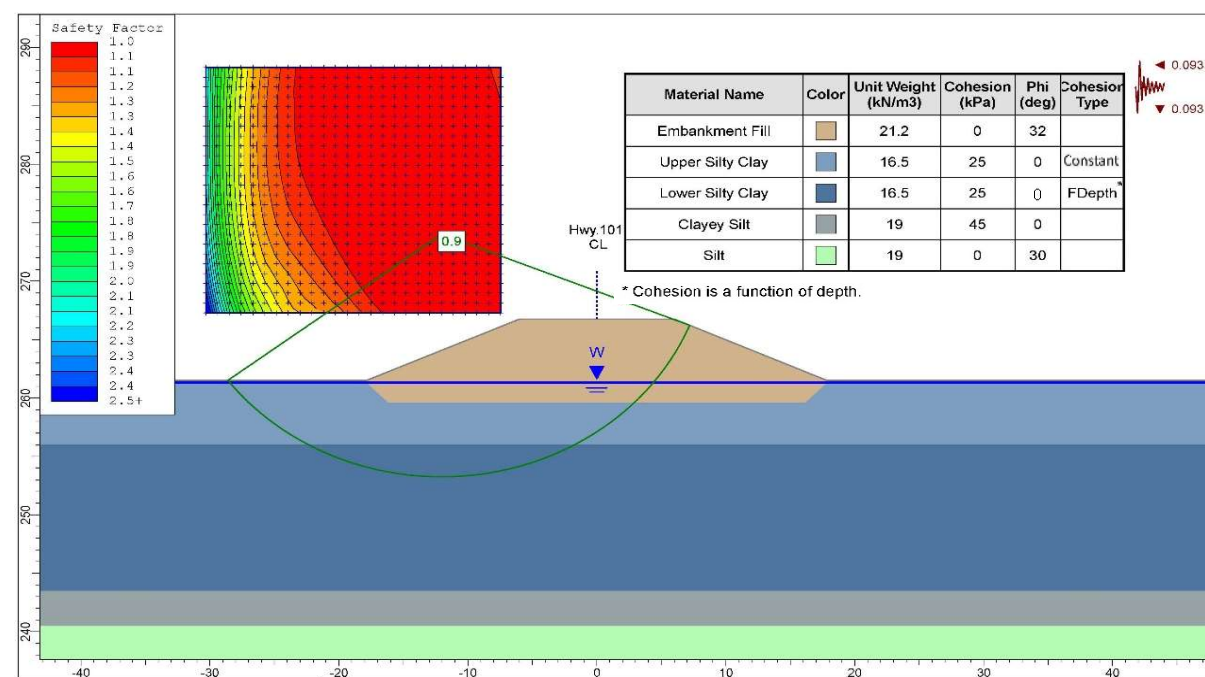





Hwy. 101 Permanent Side Slopes - Static Analysis
(Total Stress Analysis)

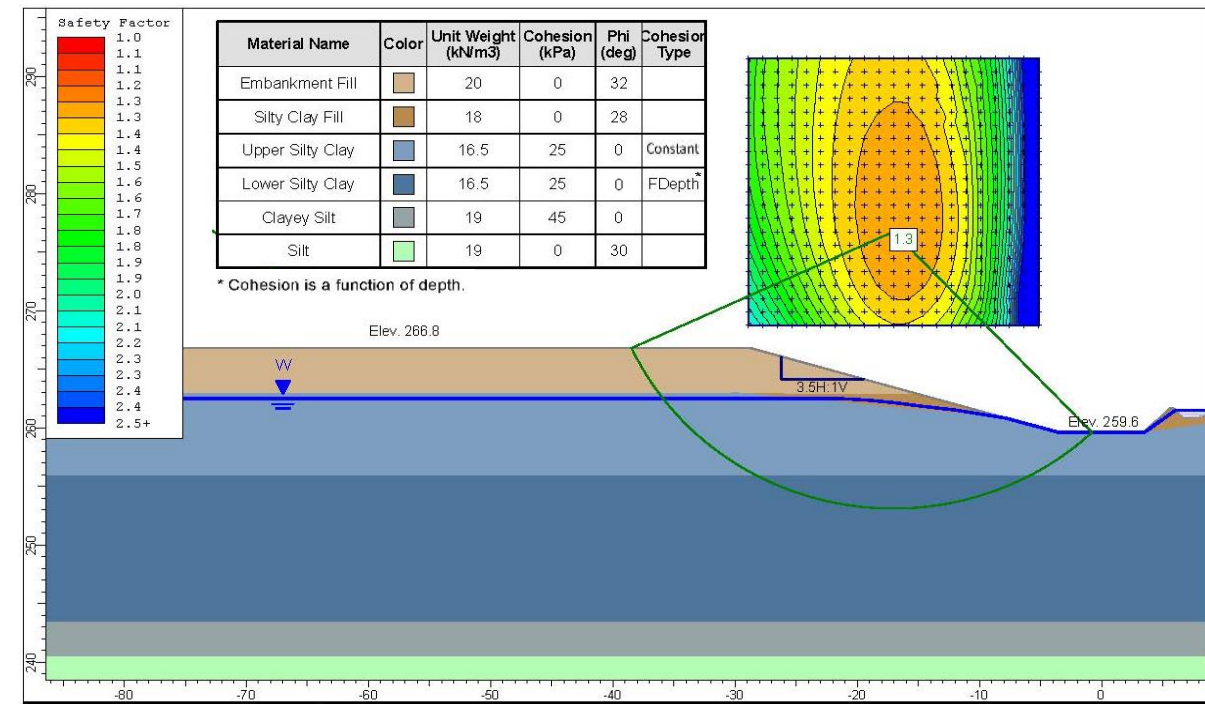


Hwy. 101 Permanent Side Slopes - Static Analysis
(Effective Stress Analysis)

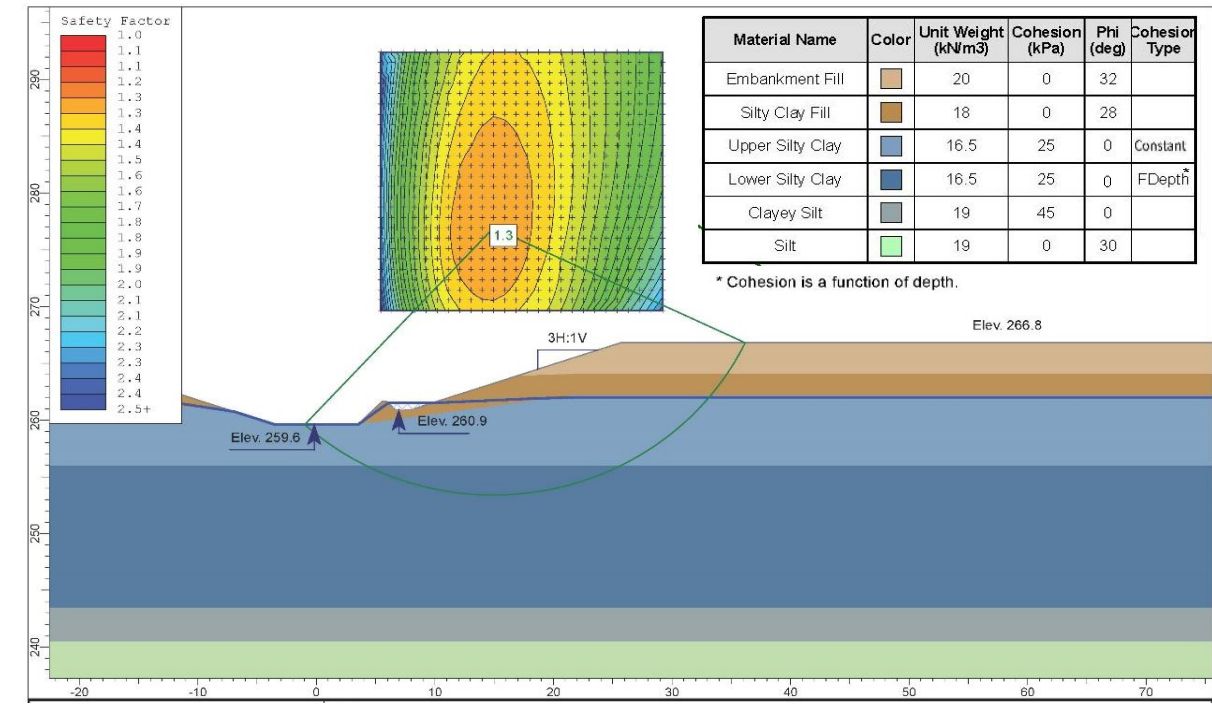


Hwy. 101 Permanent Side Slopes - Seismic Analysis

 Terraprobe Inc. Consulting Geotechnical & Environmental Engineering Construction Materials, Inspection & Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	HWY 101		
	ANTHONY CREEK CULVERT REPLACEMENT, SITE 39E-212		
	G.W.P: 5165-12-00		DATE: January 2021
	SUBM'D: SD	CHKD: RA	APPD: RA
Project No: 1-17-0822		Figure: D1	



Temporary Excavation Side Slopes - West Side of Anthony Creek Culvert



Temporary Excavation Side Slopes - East Side of Anthony Creek Culvert

APPENDIX E

Special Provisions and Operational Constraint



GEOGRID - Item No.

Special Provision

1.0 SCOPE

This special provision covers the requirements for the supply and placement of a bi-axial geogrid (geogrid) on soft/weak soils at the locations specified in the Contract. The geogrid is intended to provide support for the new culvert and the preload material.

2.0 REFERENCES

This special provision references the following standards, specifications or publications where applicable:

ASTM International

D4355 - 07	Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus
D4759 - 02(2007)	Standard Practice for Determining the Specification Conformance of Geosynthetics
D5818 - 11	Standard Practice for Exposure and Retrieval of Samples to Evaluate Installation Damage of Geosynthetics
D6637 - 10	Standard Test Method for Determining Tensile Properties of Geogrids by the Single or Multi-Rib Tensile Method

Ontario Provincial Standard Specifications, Construction

OPSS.PROV 206	Construction Specification for Grading;
OPSS.PROV 209	Construction Specification for Embankments Over Swamps And Compressible Soils;
OPSS.PROV 501	Construction Specification for Compacting;
OPSS.PROV 510	Construction Specification for Removal; and
OPSS.PROV 1860	Material Specification for Geotextiles.

3.0 DEFINITIONS

For the purposes of this specification the following definitions apply:

Aperture means an opening, such as a hole, gap, or slit.

Geogrid means a sheet-like woven or non-woven geosynthetic having a regular network of apertures that function as reinforcement by allowing interlocking of soils, rock or similar material. Used to assist in the engineered creation of monolithic structures to resist forces and loads.

Geosynthetic means a synthetic material used in geotechnical engineering applications. Geosynthetics may include such items as geotextiles, geomembranes, geocells, geogrids, geonets, and geocomposites.

MD means machine direction.

Quality Assurance (QA) means a system or series of activities carried out by the Owner to ensure that materials received from the Contractor meet the specified requirements.

Quality Control (QC) means a system or series of activities carried out by the Contractor, Subcontractor, supplier, and manufacturer to ensure that materials supplied to the Owner meet the specified requirements.

XMD means cross-machine direction.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

4.01.01 Foundation Investigation Report

The subsurface conditions at the site are described in the Foundation Investigation Report for this Contract.

The Owner warrants the data in the Foundation Investigation Report, except that interpretations of the data and opinions expressed in the Foundation Investigation Report are not warranted.

4.01.02 Submission Requirements

At least three (3) weeks before commencement of work, the Contractor shall submit to the Contract Administrator six copies with information on the geogrid installation including:

- a) Equipment to be used and methodology to deal with obstructions that might be encountered;
- b) Methodology of laying geogrid on the geotextile, backfill placement and compaction;
- c) Designs verifying that the selected geogrid will support the weight of construction equipment;
- d) Drawings illustrating equipment layout;
- e) Geogrid material specifications, data sheets; and
- f) Installation schedule.

The Contract Administrator shall be notified a minimum of 10 working days in advance of commencing the installation.

The Contractor is advised that the undrained shear strength of the silty clay soils at the design geogrid elevation may vary from 15 kPa to 25 kPa.

5.0 MATERIALS, DELIVERY AND STORAGE

Non-woven geotextile fabric has been specified elsewhere in the contract. The Contractor shall supply and install a geogrid designed to support the proposed construction equipment intended to be used for placing the preload material and construction of the engineered fill pad that will support the new culvert.

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

6.0 CONSTRUCTION

6.01 General

The geogrid shall be installed as specified in the Contract Documents and as per the manufacturer's specifications. No changes to the layout, shall be made without the prior written consent of the Contract Administrator.

The area shall be cleared of sharp objects that may damage the geogrid.

The geogrid shall be placed with a minimum overlap of 0.3 m longitudinally, and a minimum overlap of 0.3 m transversely between rolls, or greater as specified by the manufacturer. As part of the installation, the geogrid shall be pulled taut to remove any slack prior to placement of granular fill. The geogrid shall be temporarily secured in place with staples, pins, sand bags or backfill material to prevent movement during backfill placement.

The Contractor shall be responsible for any damage to the geogrid during construction, including on-site storage and installation. If the geogrid is damaged it shall be replaced at no additional cost to the Owner.

Should a discrepancy exist between the Contract Documents and the manufacturer's specifications regarding the installation procedure, then the manufacturer's specifications shall take precedence.

6.02 Operational Constraints

Vehicular and/or construction equipment shall not be allowed to operate directly on the geogrid. A minimum 150 mm thick layer of granular material shall be placed on top of the geogrid prior to allowing any vehicular and/or construction equipment traffic over the area. Sudden braking and sharp turns shall be avoided.

The Contractor is advised that the undrained shear strength of the silty clay soils at the design geogrid elevation may vary from 15 kPa to 25 kPa.

6.03 Management of Excess Materials

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

7.0 MEASUREMENT FOR PAYMENT

7.01 Geogrid

Measurement will be in square metres with no allowance made for overlap.

8.0 BASIS OF PAYMENT

8.01 Geogrid - Item

Payment at the Contract price for the above item shall be full compensation for all Labour, Equipment, and Material to do the work.

GEOTECHNICAL ASSESSMENT - Item No.

Special Provision

1.0 SCOPE

The use of heavy construction equipment, including but not limited to heavy lift cranes etc. will be required to decommission the existing culverts and install the new culverts. The bearing, global stability and settlement impacts of the heavy equipment loads on the existing embankment, soft soils underlying the embankment, and the forward slopes of open cut excavations must be taken into consideration when selecting the methodology and equipment employed for construction.

2.0 REFERENCES

The subsurface conditions at the sites are described in the following reports:

- a) Foundation Investigation Report titled *Anthony Creek Culvert Replacement, Site 39E-0212/C0, Highway 101, Assignment No. 5016-E-0023, G.W.P. No. 5165-12-00, Ministry of Transportation, Ontario, Northeastern Region.*
- b) Foundation Investigation Report titled *Shillington Creek Culvert Replacement, Site 39E-0214/C0, Highway 101, Assignment No. 5016-E-0023, G.W.P. No. 5165-12-00, Ministry of Transportation, Ontario, Northeastern Region.*
- c) Foundation Investigation Report titled *Unnamed Creek Culvert Replacement, Site 39E-0213/C0, Highway 101, Assignment No. 5016-E-0023, G.W.P. No. 5165-12-00, Ministry of Transportation, Ontario, Northeastern Region.*

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

All Foundation Engineering services required for this project shall be performed by consultant(s) listed as accepted under the MTO's RAQS for providing services under the specialty of Geotechnical (Structures and Embankments) – Medium Complexity.

Minimum safe crane pad and heavy construction equipment setback distances from the crest of open cuts must be assessed and established by the Contractor's Geotechnical Consultant to avoid any adverse stability and settlement effects on the existing highway platform during construction.

The Contractor shall retain a Geotechnical Consultant to assess the impacts of the proposed equipment loads and construction methodology on the bearing capacity, stability and settlement of the subsurface soils. The Geotechnical Consultant shall determine the requirements and/or restrictions necessary to safely support the loads without a foundation or slope failure.

The geotechnical assessment shall include, but not be limited to the following:

- a) Review of available geotechnical information and supplementing with additional subsurface information in the equipment pads and temporary support areas, as required.
- b) Determining appropriate setbacks for heavy equipment from the crest of slopes (excavation forward slopes and embankment side slopes), and the existing culvert;
- c) Determining the permissible ground pressure that may be applied to the foundation soils by the equipment, such as through a combination of equipment/crane pad design and grade lowering;
- d) Providing recommendations for distributing equipment loads to limit disturbance of the existing culvert foundation soils, and also to maintain global stability of the temporary excavation forward slopes and embankment side slopes; and
- e) The Contractor shall consider a range of proposed equipment types and their load combinations and shall ensure that these equipment types and their load combinations meet the minimum target factor of safety of 1.3 with respect to global stability.

4.02 Submission Requirements

At least two weeks prior to the mobilization of heavy equipment, the Contractor shall submit to the Contract Administrator a geotechnical assessment report(s) for approval. The report shall contain details of the proposed construction equipment and methodology and details/findings of the geotechnical assessment.

The report shall be signed and sealed by two (2) Professional Engineers licensed by the Professional Engineers of Ontario, one (1) of whom shall be the RAQS Approved Key Personnel. The report shall include as a minimum:

- a) Appropriate set back distances for heavy equipment from existing/new structures and temporary excavation forward slopes as well as embankment side slopes;
- b) Permissible ground pressures that may be applied to the foundation soils by heavy equipment; and
- c) Recommendations for the distribution of equipment loads to limit disturbance of the existing culvert foundation soils and to maintain global stability of the temporary excavation forward slopes and embankment side slopes.

5.0 MATERIALS – Not Used

6.0 EQUIPMENT

The pressure applied by the proposed construction equipment that requires analysis, including but not limited to heavy lift cranes; shall not exceed 30 kPa per metre width applied over a length of 11.0 m. This applied pressure shall take into consideration equipment type/crane pads and grade lowering if warranted. The leading edge of this applied pressure shall not be closer than:

- a) 29.0 m to the centre line of Anthony Creek;
- b) 21.0 m to the new culvert centre line at Shillington Creek; and
- c) 26.0 m to the new culvert centre line at Unnamed Creek.

7.0 CONSTRUCTION

Any excavation and/or material stockpiles, including excavated soils, construction materials and/or demolition debris, shall not be permitted anywhere between the crest of open cut excavation and the edge of water on both sides of Anthony Creek, Shillington Creek and Unnamed Creek.

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

Payment at the Contract price for the above tender item shall be full compensation for all labour to do the work.

Payment for costs associated with heavy construction equipment necessary to complete the work, such as design and construction of temporary works, supply, mobilization/de-mobilization, and operation shall be made under the associated items.

OPERATIONAL CONSTRAINT (GRADING) - Preloading

Special Provision

The Contractor is advised that the timing for placement of the preload is critical to the overall schedule. The Contractor shall schedule his/her operations such that all preload is in place for the entire period as indicated in the table below.

The Contractor shall place preload material in the following areas;

Roadway/ Township	Location	Preload Period (months)
Hwy 101 / Township of Taylor	North of Hwy 101 From 10.5 m North of CL to 1 m Beyond New Culvert Outlet	6 months
	South of Hwy 101 From 10.5 m South of CL to 1 m Beyond New Culvert Inlet	6 months

The Contractor shall not proceed with removing the preload and construction of new culvert until approval has been given by the Contract Administrator.

APPENDIX F

Preload Details



PRELOAD DETAILS

