

# Terraprobe

*Consulting Geotechnical & Environmental Engineering*

*Construction Materials Inspection & Testing*

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
UNNAMED CREEK CULVERT REPLACEMENT  
SITE 39E-0213/C0 - HIGHWAY 101  
ASSIGNMENT No. 5016-E-0023 - G.W.P. 5165-12-00  
MINISTRY OF TRANSPORTATION, ONTARIO  
NORTHEASTERN REGION  
GEOCRES NO. 42A-142**

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## **PART A – FOUNDATION INVESTIGATION REPORT**

**UNNAMED CREEK CULVERT REPLACEMENT, SITE 39E-0213/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 replacement of Unnamed Creek culvert.

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 Unnamed Creek culvert Site 39E-0213/C0 on Highway 101, Township of Taylor, District of Cochrane, Ontario.

## 2.0 SITE DESCRIPTION

The site (Latitude 48.538°; Longitude -80.590°) is located on Highway 101, approximately 3 km west of the highway’s intersection with Trans-Canada 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 culvert is 17.0 m long and its cross-section is 4.9 m wide and 1.2 m high. The Highway 101 embankment is approximately 2 m high at the culvert site with a pavement centre line elevation of 273.4 ±m.

The culvert conveys creek flows from north to south below Highway 101 and, the land adjacent to the site is relatively flat with meadow marsh vegetation on both sides of the roadway.

## 3.0 INVESTIGATION PROCEDURES

The field work for this project was carried out in two stages. Three boreholes numbered Boreholes 1, 2 and 3 were drilled and sampled to depths ranging from 18.2 m to 19.9 m below ground surface between July 17 and August 11, 2014. Under this current assignment, two additional boreholes numbered Boreholes 4 and 5 were drilled and sampled to depths of 18.7 m and 19.8 m below ground surface between April 24 and May 02, 2018, to supplement the preliminary investigation that was carried out in Year 2014. 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 Control Point HCP 101 with a geodetic elevation of 272.77 m. The data from this control point was used by Terraprobe’s staff to determine the ground surface elevations and coordinates of the boreholes. This data is summarized in the following table.

Borehole No.	MTM NAD 83 Coordinates (Zone 12)		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH1	5 377 783.6	335 079.4	271.1	18.5
BH2	5 377 773.3	335 064.7	273.2	18.2
BH3	5 377 753.2	335 065.4	271.4	19.9
BH4	5 377 767.1	335 091.3	273.1	19.8
BH5	5 377 760.0	335 079.2	271.9	18.7

The boreholes were drilled with truck-mounted and track-mounted drill rigs supplied and operated by specialist drilling contractors. 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 1586<sup>1</sup>. 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 3, a boulder was encountered within the till matrix and NQ-size diamond coring techniques was used to extend the borehole below the boulder. Dynamic Cone Penetration testing was also performed in Borehole 4.

Ground water conditions in the open boreholes were observed during the drilling operations. After drilling operations were complete a standpipe piezometer was also installed in Borehole 3 with the slotted screen located within the silty sand till strata. However, two days after the piezometer installation water was observed flowing out of the standpipe piezometer with an artesian head of 0.9 m above ground surface (ground water elevation of 272.3 m). The standpipe was therefore decommissioned and the borehole was sealed in accordance with current MTO procedures and Ontario Regulation 903 (as amended). A standpipe piezometer was also installed in Borehole 1 with the screen founded in the silty clay to clay strata, to permit longer term ground water level monitoring of this deposit.

The recovered soil samples were subjected to Visual Identification (VI) and select samples were also subjected to a laboratory testing programme consisting of natural moisture content, grain size distribution analyses, Atterberg limits determinations and one-dimensional consolidation testing in accordance with MTO and/or ASTM Standards as appropriate. One soil sample was 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 Till<sup>2</sup>. 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

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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.

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.

## 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 section 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 site is generally underlain by fill material consisting of dense to very dense gravelly sand, very loose to dense gravelly sand to sand, and soft to very stiff silty clay. The native overburden deposits consist of topsoil and amorphous peat, very soft to stiff varved silty clay to clay and very loose to very dense silty sand till. A more detailed description of the subsurface conditions is provided in the following sections.

### 4.2.1 Topsoil

Topsoil layers ranging from 50 mm to 300 mm in thickness were encountered at this site. Topsoil thickness may vary between and beyond the boreholes.

### 4.2.2 Flexible Pavement

Boreholes 2 and 4 were drilled through the Highway 101 pavement platform. The flexible pavement consists of 220 mm thick asphaltic concrete underlain by gravelly sand fill material. The locations, thicknesses and base elevations of the granular pavement fill are summarized in the following table.

Borehole No.	Fill Thickness (mm)	Fill Base Elevation (m)
BH2	700	272.5
BH4	570	272.3

Standard Penetration tests carried out in the gravelly sand fill measured SPT N-values of 34 and more than 100 blows for 0.3 m of penetration suggesting a dense to very dense relative density. The natural water content of a sample of the granular fill is 4% by weight.

The grain size distribution curve of a sample of the gravelly sand fill is shown on Figure B1 in Appendix B. The results show a grain size distribution consisting of 29% gravel, 63% sand and 8% silt size particles.



#### 4.2.3 Fill – Gravelly Sand to Sand

Gravelly sand to sand fill was encountered at this site. The locations, thicknesses and base elevations of the gravelly sand to sand fill are summarized in the following table.

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH2	1.4	2.1	271.1
BH4	0.6	1.4	271.7
BH5	2.0	2.1	269.8

Standard Penetration tests carried out in the gravelly sand to sand fill measured SPT N-values ranging from 1 to 38 blows for 0.3 m of penetration indicating a very loose to dense relative density. The moisture content of samples of the gravelly sand to sand fill range from 3% to 20% by weight.

The grain size distribution curve of a sample of the sand fill is presented on Figure B2 in Appendix B. The results show a grain size distribution consisting of 4% gravel, 85% sand and 11% silt size particles.

#### 4.2.4 Fill – Silty Clay

Silty clay fill was encountered below the sand fill in Boreholes 2 and 4. The locations, thicknesses 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)
BH2	1.6	3.7	269.5
BH4	0.5	1.9	271.2

Standard Penetration tests in the silty clay fill gave N-values ranging from 3 to 24 blows for 0.3 m of penetration indicating a soft to very stiff consistency. The natural water content of two samples of the silty clay fill are 35% and 71% by weight.

The grain size distribution curve of a sample of the silty clay fill is depicted on Figure B3 in Appendix B. These results show a grain size distribution consisting of 0% gravel, 9% sand, 47% silt and 44% clay size particles.

An Atterberg Limits test was also carried out on a sample of the silty clay fill and the results are plotted on the plasticity chart shown on Figure B4 in Appendix B. These results indicate that the fill is a cohesive soil with intermediate plasticity (CI). The results of the Atterberg Limits test are summarized below:

Liquid Limit:	45%
Plastic Limit:	23%
Plasticity Index:	22%
Natural Water Content:	35%

#### 4.2.5 Peat

The fill material at Borehole 4 is underlain by a 1.4 m thick layer of amorphous peat that extends to a depth of 3.3 m below ground surface (elevation 269.8 m).

#### 4.2.6 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 1 mm to 20 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	14.4	14.7	256.4
BH2	12.9	16.6	256.6
BH3	13.2	13.5	257.9
BH4	14.4	17.7	255.4
BH5	14.1	16.2	255.7

The N-values of Standard Penetration tests carried out in the silty clay to clay deposit range from 0 blows (weight of hammer) to 7 blows per 0.3 m of penetration and, field vane tests measure in-situ undrained shear strengths ranging from 8 kPa to 84 kPa as illustrated on Figure B6 in Appendix B. Based on the undrained shear strength values and SPT N-values, the consistency of the silty clay to clay is described as very soft to stiff. The sensitivity of the silty clay to clay ranges from approximately 1.0 to 6.5, 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 generally illustrates relatively low undrained shear strength values ranging from 8 kPa to 28 kPa up to an elevation of 261.0 m. Below elevation 261.0 m, there is a trend of increasing undrained shear strength values with depth, with shear strength values ranging from about 16 kPa to 84 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 and B8 in Appendix B. The test results show a grain size distribution consisting of 0% gravel, 0% to 2% sand, 12% to 56% silt and 26% to 88% clay sized particles.

A grain size distribution test was carried out on a sample of the silt layer embedded within the silty clay to clay matrix and the grain size distribution curve is shown on Figure B9, Appendix B. The silt layer has a grain size distribution consisting of 0% gravel, 1% sand, 89% silt and 10% clay sized particles.

Atterberg limits tests were also carried out on samples of the silty clay to clay and the results are plotted on the plasticity chart, Figures B10 and B11 in Appendix B. The results indicate a cohesive deposit of generally high plasticity (CH) clay with occasional low to intermediate plasticity clay (CL to CI). The Atterberg limits test results are summarized as follows:

Liquid Limit:	22% to 61%
Plastic Limit:	15% to 25%
Plasticity Index:	7% to 36%
Natural Moisture Content:	28% to 72%

The Atterberg Limits tests results are also plotted against elevation in Figure B12. These results illustrate that the natural moisture contents of the tested samples are typically higher than their liquid limits.

The moisture content of samples of the silty clay to clay varies between 21% and 74% and the unit weight of a tested sample is  $16.0 \pm \text{kN/m}^3$ .

A one-dimensional consolidation test was performed on a sample of the silty clay to clay and the results are presented in Figures B13 to B16 in Appendix B. The results of the one-dimensional consolidation test are summarized below.

Borehole/Sample No.	Sample Depth/Elevation (m)	$\sigma'_{vo}$ (kPa)	$\sigma'_p$ (kPa)	$C_c$	$C_r$	$e_o$
BH1, Sample 7	6.3/264.8	43.6	44	0.524	0.056	1.81

Where:  $\sigma'_{vo}$  = effective overburden pressure  
 $\sigma'_p$  = 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 approximately equal to the effective overburden pressure suggesting that the silty clay to clay deposit is normally consolidated.

#### 4.2.7 Silty Sand Till

In all of the boreholes a silty sand till deposit was encountered below the silty clay to clay deposit. Summarized below are the explored depths and base elevations of the silty sand till.

Borehole No.	Thickness (m)	Depth (m)	Base Elevation (m)
BH1	3.8	18.5*	252.6
BH2	1.6	18.2*	255.0
BH3	6.4	19.9*	251.5
BH4	2.1	19.8*	253.3
BH5	2.5	18.7*	253.2

\* Borehole termination depth.

Standard Penetration tests carried out in this deposit measured N-values that range from 0 blows (weight of hammer) to more than 100 blows per 0.3 m of penetration indicating a very loose to very dense relative density. The Standard Penetration tests carried out in the silty sand till deposit immediately below the silty clay to clay deposit measured N-values that range from 0 blows (weight of hammer) to 10 blows per 0.3 m of penetration. These low SPT N-values are likely due to zones within the silty sand till that have experienced hydrostatic uplift because of the artesian conditions present. The natural water content of samples from this stratum ranged from 9% to 22%.

Grain size distribution tests were carried out on two samples from this deposit and the results illustrated in Figure B17, Appendix B show a grain size distribution consisting of 4% and 10% gravel, 71% and 59% sand, 21% and 26% silt and, 4% and 5% clay sized particles. The field investigations also indicate that the matrix of this silty sand till contains random cobble and boulder inclusions.

### 4.3 Ground Water Levels

Ground water conditions in the open boreholes were observed during the drilling operations and a standpipe piezometer was installed in Borehole 3 with the slotted screen located within the silty sand till stratum. Two days after the standpipe installation in Borehole 3 water was observed flowing out of the piezometer with a recorded artesian head of 0.9 m above ground surface i.e. a ground water elevation of 272.3 m. Subsequently, the standpipe piezometer in Borehole 3 was decommissioned and sealed in accordance with current MTO procedures and Ontario Regulation 903 (as amended). A standpipe piezometer was also installed in Borehole 1 with the screen founded in the silty clay to clay strata, to permit longer term ground water level monitoring. The ground water levels measured in the piezometers are summarized in the following table:

Borehole No.	Date	Ground Water Levels	
		Depth (m)	Elevation (m)
BH1	September 16, 2014	0.5	270.6
	October 28, 2014	0.5	270.6
	November 24, 2018	0.2	270.9
BH3	August 13, 2014	0.9*	272.3

♦ Water level above ground surface.

The ground water level at this site is estimated to be at an approximate elevation of 271 m based on the soil moisture conditions, measured ground water levels and creek water levels. The ground water levels will be controlled by the creek water level, will fluctuate seasonally, and are expected to rise during wet periods. The ground water level observations made in Borehole 3 indicates that excess hydrostatic pressure exists in the water bearing and relatively permeable silty sand till unit.

### 5.0 MISCELLANEOUS

The investigation was carried out using drilling equipment supplied and operated by Tattr Environmental Drilling of Timmins, Ontario and Landcore Drilling of Chelmsford, Ontario. The field operations were supervised by Mr. Satyajit Manani, C.E.T. and Ms. Fatemeh Yazdandoust, MSc. and the routine laboratory and one-dimensional consolidation testing was 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.

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## **PART B – FOUNDATION DESIGN REPORT**

**UNNAMED CREEK CULVERT REPLACEMENT, SITE 39E-0213/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 Unnamed 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 17.0 ±m long concrete culvert with an internal opening that is 4.9 m wide and 1.2 m high, will be replaced with a 29.0 ±m long concrete box culvert with a cross-section measuring 4.8 m in width and 2.1 m in height. The upstream and downstream streambed elevations of the new culvert are 270.2 ±m and 269.9 ±m respectively.

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 at an offset distance 8.2 ±m east of the existing culvert.

### **6.2 Foundation Alternatives**

Although a weak silty clay to clay deposit exists at this site, the proposed 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. The current work is based on a preliminary design study that was completed by WSP and Terraprobe and, a Preliminary FIDR dated September 16, 2015 was accepted by MTO. The preliminary study explored more than one foundation alternatives and the Pre-Cast Box Culvert was selected as the preferred alternative for detail design. From a foundation perspective a pre-cast box culvert replacement is preferred over a cast-in-place open footing culvert because:

- Pre-cast box culvert segments can be installed more expeditiously than cast-in-place open footing culverts, resulting in a shorter construction period and hence shorter closure of the highway; and
- 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 and/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 ( $\psi$ ) and geotechnical resistance factors ( $\phi_{gu}$  &  $\phi_{gs}$ ) 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 (Sa) 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  $Sa(0.2)/PGA$  ratio is less than 2.0. A  $PGA_{ref}$  value of 0.083 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)	Sa (0.2) (g)	Sa (0.5) (g)	Sa (1.0) (g)	Sa (2.0) (g)	Sa (5.0) (g)	Sa (10.0) (g)
0.104	0.073	0.163	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)							
0.188	0.180	0.267	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. However, a box culvert founded at an invert elevation of  $269.3 \pm m$ , can be supported on a 0.5 m thick OPSS Granular A engineered fill pad founded at a base elevation of  $268.8 \pm m$ .

The engineered fill pad shall be compacted to 95% Standard Proctor maximum dry density (SPMDD) at  $\pm 2\%$  of optimum moisture content. At elevation  $269.3 \pm m$  (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 268.8 m at a 1 Horizontal to 1 Vertical (1H:1V) side slope.

A box culvert supported on a 0.5 m thick OPSS Granular A engineered fill pad that is founded at elevation of 269.3 m on the undisturbed silty clay to clay deposit, can be designed for a factored geotechnical resistance at Ultimate Limit State (ULS) of 85 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 85 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 ( $\phi'$ ).

- 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 ( $\delta'_i$ ) equivalent to 2/3 of the soil's effective angle of internal friction ( $\phi'$ ) 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

$\gamma$  = unit weight of retained soil (kN/m<sup>3</sup>)

$h$  = 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  $30^\circ$  to  $35^\circ$  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  $35^\circ$ . 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  $0.5 \phi$ . 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 <sub>AE</sub> (Yielding Wall)	0.30	0.34
K <sub>AE</sub> (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 OPSD 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 OPSD 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.PROV 912 and OPSS.PROV 501. As specified in OPSS.PROV 912, 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 ( $\psi$ ) and the geotechnical resistance factors ( $\Phi_{gu}$ ) 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	$\gamma$ (kN/m <sup>3</sup> )	Total Stress Analysis		Effective Stress	
		$\phi$ (degrees)	c (kPa)	$\phi'$ (degrees)	c' (kPa)
New Embankment Fill	21.2	32	0	32	0
Existing Embankment Fill	18.5	29	0	29	0
Peat	11.0	28	0	28	0
Upper Silty Clay to Clay	16.5	0	15	24	3
Lower Silty Clay to Clay	16.5	0	15+3.13H	24	3
Silty Sand Till	21	32	0	32	0
Design Factor of Safety	-	1.4		1.6	

The analyses indicate that the factors of safety will be equal to or greater than the minimum target factors of safety provided that the embankment is constructed at a minimum side slope geometry of 2.0 Horizontal to 1.0 Vertical (2H: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 0.188g 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 0.094 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 2H:1V side slope geometry indicate that the embankments will have a factor of safety of 0.8. 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 analysis was carried out for a temporary cut on the east side of the existing culvert and perpendicular to the highway extending from the highway platform down to elevation 268.8 m (i.e., 4.6 m± high cut) where the new culvert will be installed.

The global stability of the 4.6 m± high forward slope of the excavation was assessed using the total stress analysis soil properties provided in Section 6.11.2 of this report. The analyses indicate that a 4.6 m± high

cut will require a forward slope geometry of 4H:1V or flatter to achieve a target factor of safety of 1.3. The results of the stability analyses are presented in Appendix D, Figure D1.

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 – $\sigma'_p$ (kPa)	44
Overconsolidation Ratio	1.0
Compression Index - $C_c$	$0.2 + 0.11H^*$ , 0.42, and $0.42 - 0.04H^*$
Recompression Index - $C_r$	$0.03 + 0.015H^*$ , 0.06, and $0.06 - 0.0053H^*$
Initial Void Ratio - $e_o$	$0.7 + 0.4H^*$ , 1.5, and $1.5 - 0.133H^*$
Coefficient of Consolidation - $C_v$ (cm <sup>2</sup> /s)	$1.0 \times 10^{-3}$

\* H is the incremental change in elevation

### 6.12.2 Culvert Settlement

The settlement analysis is based on the understanding that the new culvert will be installed at an offset distance east of the existing culvert, the embankment will be reconstructed at a side slope geometry of 2H:1V and the assumption that the embankment height will remain unchanged i.e., no grade raises of Highway 101. The model is based a new culvert that is 4.8 m wide and 2.1 m high (internal dimensions); founded at elevation 269.2 m on a 0.5 m $\pm$  thick Granular A engineered fill pad with Granular B Type I backfill.

Tabulated below are the estimated total settlements along the culvert centre line for a 29.0 $\pm$  m long culvert, constructed at an offset distance from the existing culvert.

Culvert constructed at an offset distance from the existing culvert					
Offset Distance (m)*	0	$\pm 5$	$\pm 10$	$\pm 12$	$\pm 15$
Estimated Settlement (mm)	5	5	5	70	185

\* 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 12 m to 15 m from the highway centre line is large because this area has not experienced the benefit of preloading from the full height of 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 as illustrated on Figure F1 in Appendix F, and described below:

- A preload equivalent to 30 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 (See Figure F1);
- The preload area shall extend 5 m horizontally beyond the new culvert sides, except for the south-west corner of the new culvert where the preload area shall be reduced to 4 m; 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.

<b>Culvert constructed at an offset distance from the existing culvert</b>					
Offset Distance (m)*	0	±5	±10	±12	±15
Estimated Settlement (mm)	5	5	5	0	0

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

It is understood that the existing culvert's top slab will be removed and we recommend filling the void with Cematrix to mitigate settlement. A total settlement of  $10 \pm \text{mm}$  is estimated to occur under the road platform over a 20-year period. The Cematrix shall be placed 0.5 m above the existing top of slab elevation and shall be extended laterally 1 m beyond the existing culvert sidewalls.

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 essentially be 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 268.8± 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 analysis that was carried out for the 4.6 m deep temporary excavation, a side slope geometry of 4H:1V or flatter is required. Shallower excavations up to 1.2 m deep can be carried out at 1H:1V in Type 3 soils and 3H:1V in Type 4 soils. 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 $\phi$ (degrees)	Unit Weight $\gamma$ (kN/m <sup>3</sup> )	Active Earth Pressure Coefficient	At - Rest Earth Pressure Coefficient	Passive Earth Pressure Coefficient
			$K_a$	$K_o$	$K_p$
Existing Embankment Fill	29	18.5	0.35	0.52	2.88
Silty Clay to Clay	24	16.5	0.42	0.59	2.37
Silty Sand Till	32	21	0.31	0.47	3.25

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.

## 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

One soil sample was 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 Standard<sup>3</sup>. 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 Index of the tested sample is 8.5.

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.

<sup>3</sup> American Water Works Association (AWWA) C-105 (2005) Standard, “Polyethylene Encasement for Ductile-Iron Pipe Systems” Catalog No. 43105. AWWA Denver, CO.



## 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.

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## 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

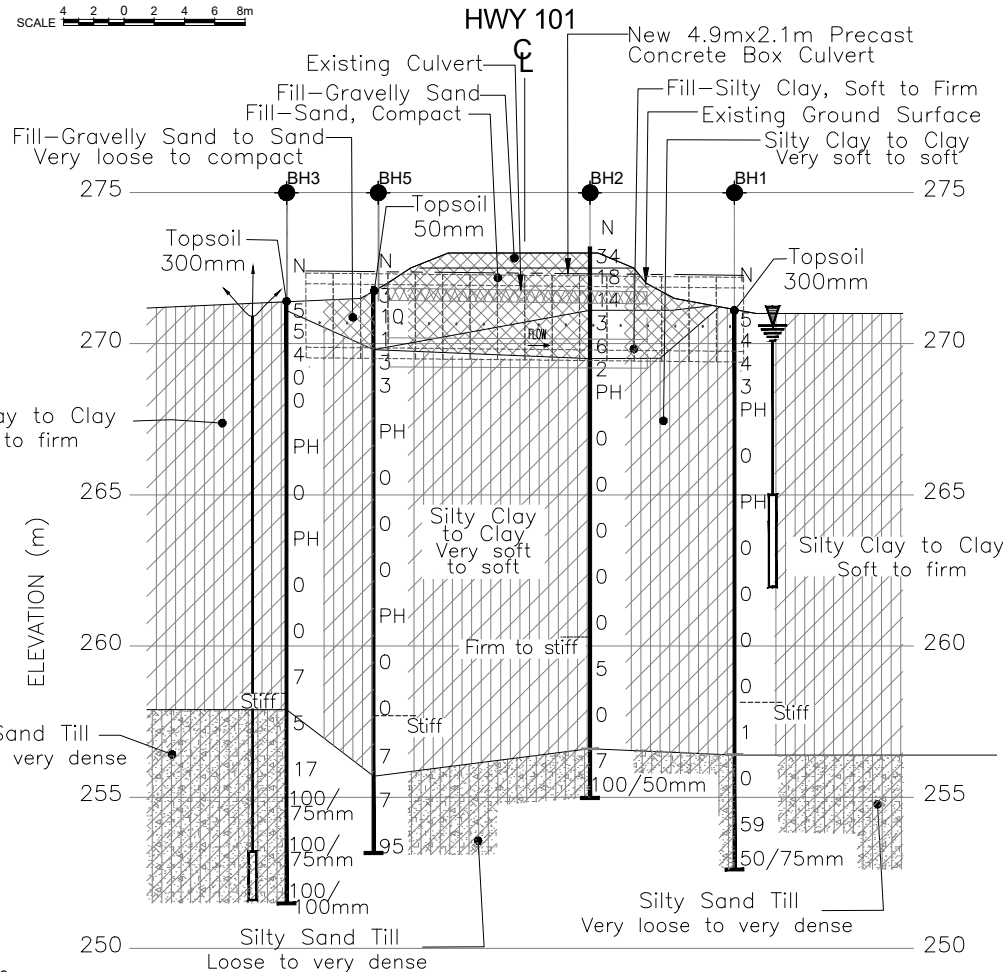
**Ontario Provincial Standard Drawings (OPSD)**

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

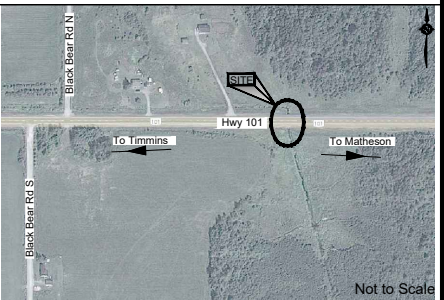








# **DRAWINGS & SITE PHOTOGRAPHS**





 **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 (November 2018)
	Piezometer
A/R	Auger Refusal
	Artesian Condition

BH No.	ELEV. (m)	COORDINATES	
		(MTM, NAD27, Zone 12)	
		NORTHING (m)	EASTING (m)
BH1	271.1	5 377 783.6	335 079.4
BH2	273.2	5 377 773.3	335 064.7
BH3	271.4	5 377 753.2	335 065.4
BH4	273.1	5 377 767.1	335 091.3
BH5	271.9	5 377 760.0	335 079.2

NOTE				
<p>This drawing is for subsurface information only. The proposed structure details/work is shown as 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. 1-17-0B22 GEOCRES No. 42A-142			
SUBM'D. SD	CHKD. SD	DATE: OCT. 2021	SITE: 39E-0213/CO
DRAWN: KC	CHKD. SD	APPD: RA	DWG. 2

\\192.168.1.16\tss1-Project Files\2017\1-17-0822 - Hwy 101, Anthony\1- Culvert Replacements, MTOA, Dwg. Logs\AutoCad\Unknown Creek\Unknown Creek 2021-10-07.dwg.  
DWG To PDF.pc3, Kamal, Kamal





North Side of Hwy. 101 - Looking at Unnamed Creek Culvert Inlet - July, 2019




Hwy. 101 at Sta. 12+985 - South Side of Hwy. 101 - Looking East - July, 2019



South Side of Hwy. 101 - Looking at Unnamed Creek Culvert Outlet - April, 2018



Hwy. 101 at Sta. 12+985 - South Side of Hwy. 101 - Looking East - April, 2018

 <p><b>Terraprobe Inc.</b> Consulting Geotechnical &amp; Environmental Engineering Construction Materials, Inspection &amp; Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 706-2650</p>	HWY 101 UNNAMED CREEK CULVERT REPLACEMENT, SITE 39E-0213/C0	
	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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

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

## PHYSICAL PROPERTIES OF SOIL

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

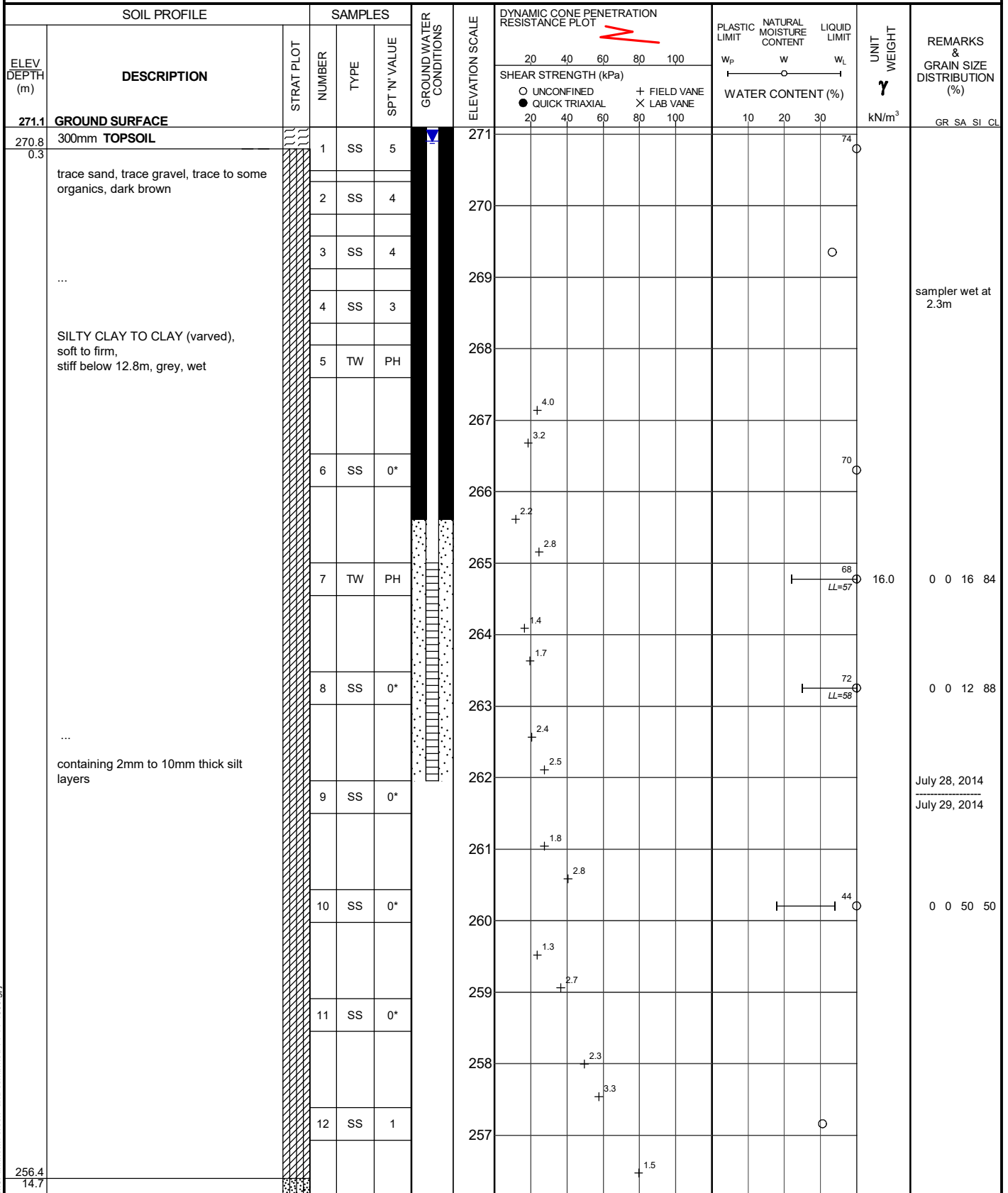


# RECORD OF BOREHOLE No 1

1 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:335079.4 N:5377783.6 ORIGINATED BY S.M  
DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY H.A  
DATUM GEODETIC DATE 2014-7-28 - 2014-7-29 CHECKED BY R.A



Continued Next Page

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

# RECORD OF BOREHOLE No 1

2 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:335079.4 N:5377783.6 ORIGINATED BY S.M  
DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY H.A  
DATUM GEODETIC DATE 2014-7-28 - 2014-7-29 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUNDWATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																									
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT N' VALUE			20	40	60	80	100						SHEAR STRENGTH (kPa)					WATER CONTENT (%)																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																																			
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## END OF BOREHOLE

Wet cave at 4.6m upon completion.

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

Unable to push 'N' vane below 14.7m.

Soil heaved from 15.2m to 13.7m in  
augers.

Consolidation test performed on TW7.

Piezometer installation consists of a  
50mm diameter, Schedule 40 PVC pipe  
with a 3.0m slotted screen.

## WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 16, 2014	0.5	270.6
Oct 28, 2014	0.5	270.6
Nov 24, 2018	0.2	270.9



# RECORD OF BOREHOLE No 2

2 of 2

**METRIC**

G.W.P. 5165-12-00 LOCATION Coords: E:335064.7 N:5377773.3 ORIGINATED BY S.M  
 DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS COMPILED BY H.A  
 DATUM GEODETIC DATE 2014-7-17 - 2014-7-28 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100					
								SHEAR STRENGTH (kPa) ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE					
	(continued)						20 40 60 80 100	WATER CONTENT (%) 10 20 30				KN/m <sup>3</sup>	GR SA SI CL
	SILTY CLAY to CLAY, (varved), soft to stiff, grey, wet		14	SS	0*		258						July 17, 2014
							257						July 28, 2014
256.6 16.6	SILTY SAND, trace clay, trace gravel, loose to 17.2m, very dense below, grey, wet (GLACIAL TILL)		15	SS	7		256						
255.0 18.2			16	SS	100/ 50mm		255						

## END OF BOREHOLE

Wet cave at 3.0m upon completion of  
drilling.

'N' Vane sinking under weight of rods  
at 5.6m, 8.7m, 10.1m and 11.6m.

\*Sampler sinking under weight of  
hammer and rods.

# RECORD OF BOREHOLE No 3

1 of 2

**METRIC**

G.W.P. 5165-12-00 LOCATION Coords: E:335065.4 N:5377753.G ORIGINATED BY S.M  
DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / NQ CORING COMPILED BY H.A  
DATUM GEODETIC DATE 2014-7-30 - 2014-8-11 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100	20 40 60 80 100	W <sub>p</sub> W W <sub>L</sub>	WATER CONTENT (%)			
271.4	GROUND SURFACE													
271.1 0.3	300mm TOPSOIL		1	SS	5		271							
	trace sand, trace gravel, trace to some organics, dark brown		2	SS	5									
	...		3	SS	4		270							
	SILTY CLAY TO CLAY (varved), c <sub>u</sub> : soft to firm, stiff below 13.3m, grey, wet		4	SS	0*		269							sampler wet at 2.3m 0 0 29 71
			5	SS	0*		268							
			6	TW	PH		267	2.2	3.9					
			7	SS	0*		266	3.1	3.2					
			8	TW	PH		265	2.4	4.0					0 0 18 82
			9	SS	0*		264	1.5	1.8					sampler wet at 9.1m July 30, 2014 July 31, 2014
			10	SS	0*		263	1.9	1.7					
	containing 20mm thick silt layers		11	SS	7		262	1.5	1.2					July 31, 2014 Aug.11, 2014 0 1 89 10
257.9 13.5	SILTY SAND, trace clay, trace gravel, loose to compact, grey, wet (GLACIAL TILL)		12	SS	5		261	1.0	2.8					
							260							
							259							
							258							
							257							

Continued Next Page

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

# RECORD OF BOREHOLE No 3

2 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:335065.4 N:5377753.G ORIGINATED BY S.M  
DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / NQ CORING COMPILED BY H.A  
DATUM GEODETIC DATE 2014-7-30 - 2014-8-11 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)		WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE	W <sub>p</sub>	W	W <sub>L</sub>		
							20 40 60 80 100	20 40 60 80 100			10 20 30			
	(continued)		13	SS	17		256							
	...containing cobbles and boulders below 16.8m, very dense		14	SS	100 / 75mm		255							
			15	SS	100 / 75mm		254							
			16	SS	100 / 100mm		253							
251.5 19.9							252							

## END OF BOREHOLE

Water level at 3.7m (not stabilized) and hole open to 12.2m on July 31, 2014.

Unable to push 'N' vane below 13.5m.

NQ coring through boulder from 16.5m to 16.8m.

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

\*(ag)-above ground (water flowing from top of piezometer)

Piezometer was decommissioned and borehole sealed/grouted with bentonite slurry mixture.

Piezometer installation consists of a 19mm diameter schedule 40 PVC pipe with a 1.52m slotted screen.

## WATER LEVEL READINGS

Date Aug 13, 2014 Water Depth (m) 0.9 (ag)\* Elevation (m) 272.3

# RECORD OF BOREHOLE No 4

1 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:335091.3 N:5377767.1 ORIGINATED BY FY  
DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / CASING AND WASH BORING COMPILED BY SD  
DATUM GEODETIC DATE 2018-4-24 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT $\gamma$  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)										WATER CONTENT (%)		
								20	40	60	80	100						20	40	60

273.1	GROUND SURFACE						273										
272.9	220mm ASPHALTIC CONCRETE		1	SS	100 / 125mm												
0.2	570mm FILL, gravelly sand, trace to some silt, very dense, brown, dry																
272.3																	
0.8	FILL, sand, trace to some gravel, some silt, dense, brown, dry		2	SS	38		272										
271.7																	
1.4	FILL, silty clay, trace to some sand, very stiff, brown, dry to moist		3	SS	24												
271.2																	
1.9	PEAT, amorphous, black		4	SS	11		271										
269.8			5	SS	2		270										
3.3	SILTY CLAY to CLAY, grey, wet		6	SS	3		269										
			7	TW	PH		268										

Continued Next Page

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

file: unknown creek.gpj

**METRIC**[illegible]

Dynamic Cone Penetration test (DCPT) performed from 18.3 m to 19.8 m.

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



**METRIC**[illegible]



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

# RECORD OF BOREHOLE No 5

2 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:335079.2 N:5377760 ORIGINATED BY FY  
 DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / CASING AND WASH BORING COMPILED BY SD  
 DATUM GEODETIC DATE 2018-5-2 CHECKED BY RA

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT N' VALUE			20 40 60 80 100										
								SHEAR STRENGTH (kPa)								W <sub>p</sub> W W <sub>L</sub>		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE								WATER CONTENT (%)		
	(continued)					20 40 60 80 100	10 20 30	kN/m <sup>3</sup>	GR SA SI CL									
255.7	SILTY CLAY to CLAY (VARVED) stiff, grey, wet		13	SS	7													
16.2	SILTY SAND, trace to some gravel, trace clay, loose to 17.8m, very dense below, grey, wet (GLACIAL TILL)		14	SS	7													
253.2			15	SS	95													

## END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

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

Casing sank from 9.6 m to 10.6 m.

# **APPENDIX B**

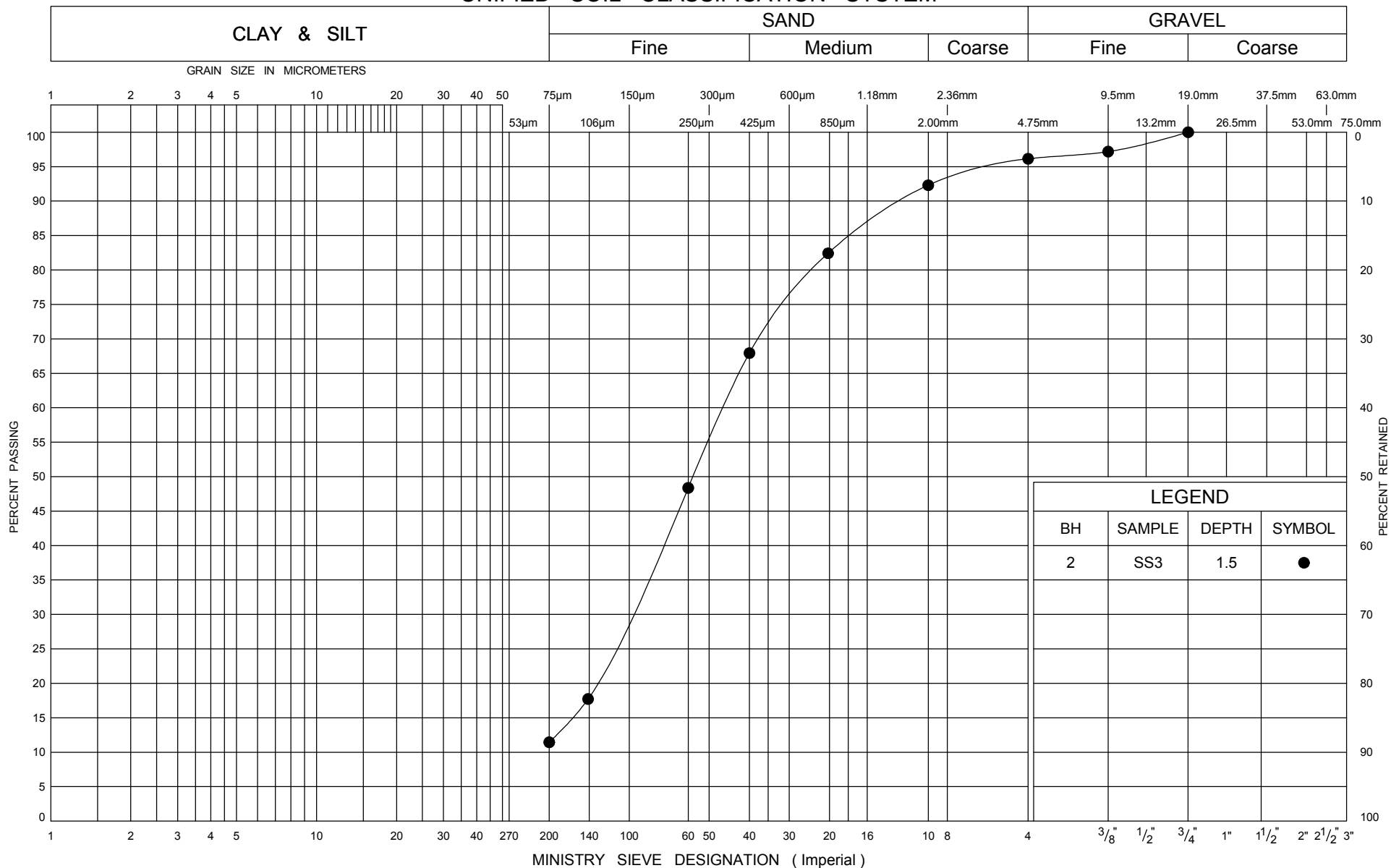
## **Field & Laboratory Test Results & Photographs**



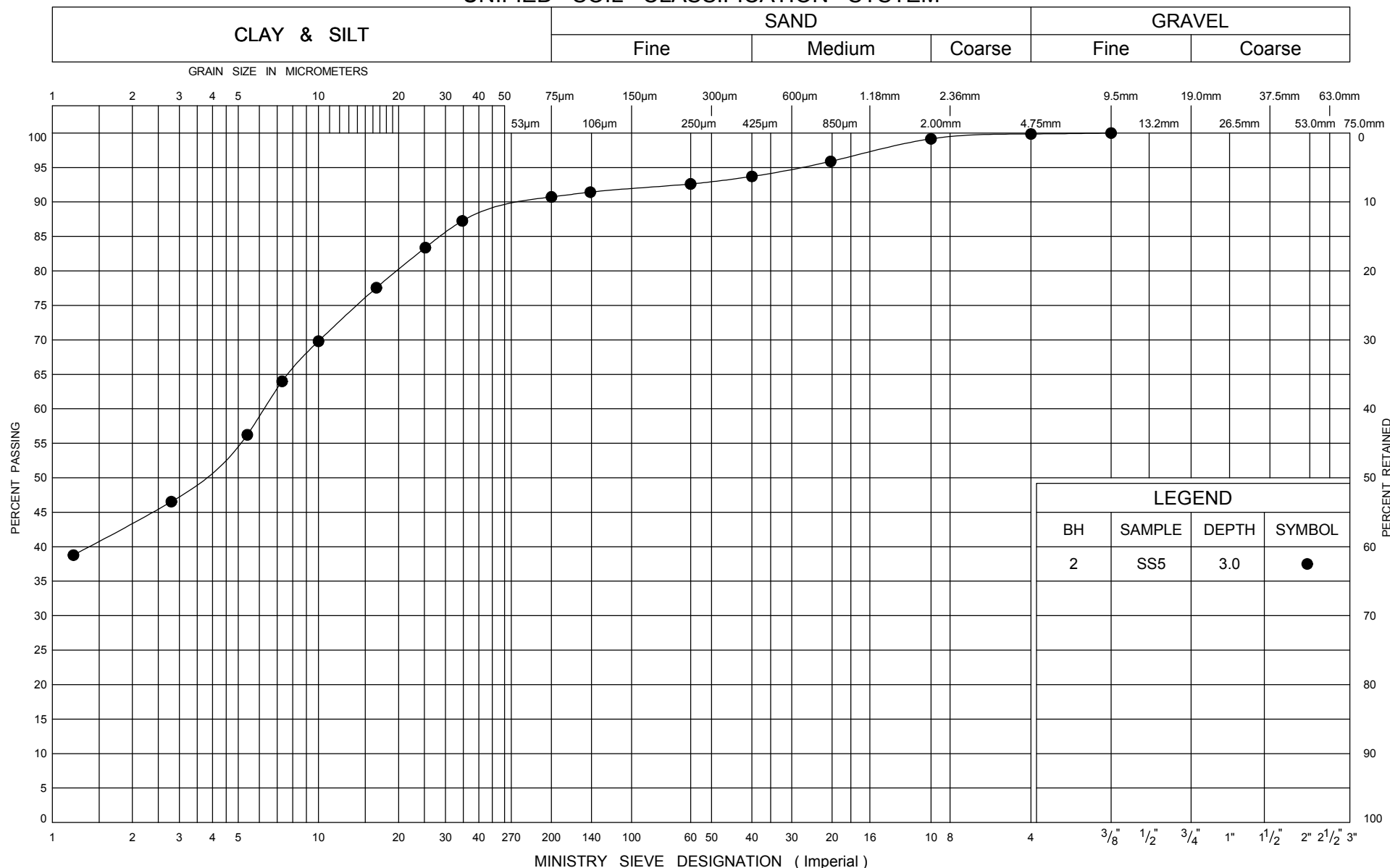


Unnamed Creek Culvert (39E - 0213/C0)

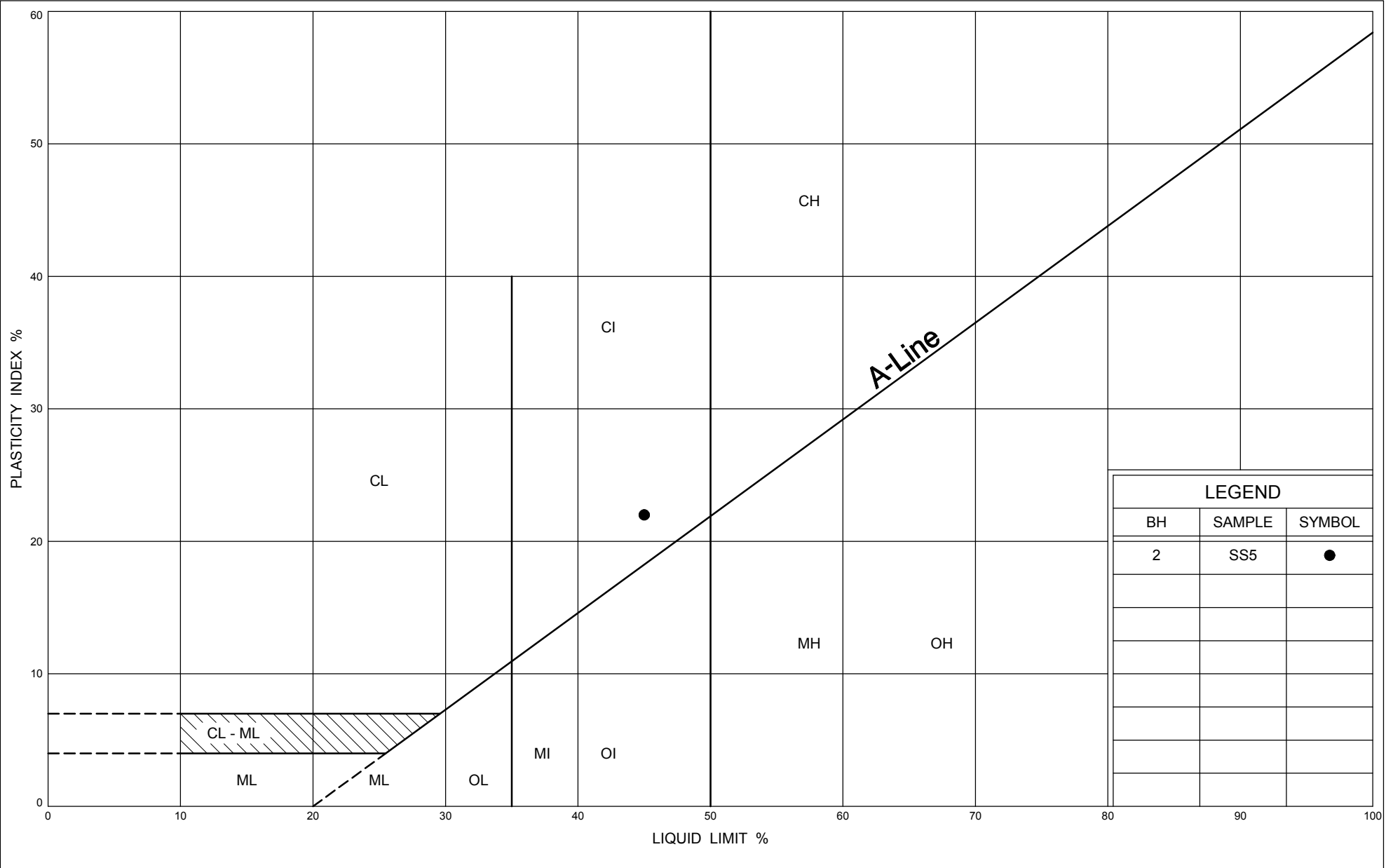
# UNIFIED SOIL CLASSIFICATION SYSTEM



# UNIFIED SOIL CLASSIFICATION SYSTEM



library: library - terraprobe.gint - copy.glb report: mib-terra-plasticity-chart file: 11-14-4066 bh logs\_unknown creek culvert (39e-213)-rev 2 - copy.gpl



**UNNAMED CREEK CULVERT (Site 39E-0213/C0)**  
Silty Clay to Clay



**BH1 SS9**



**BH3 SS10**



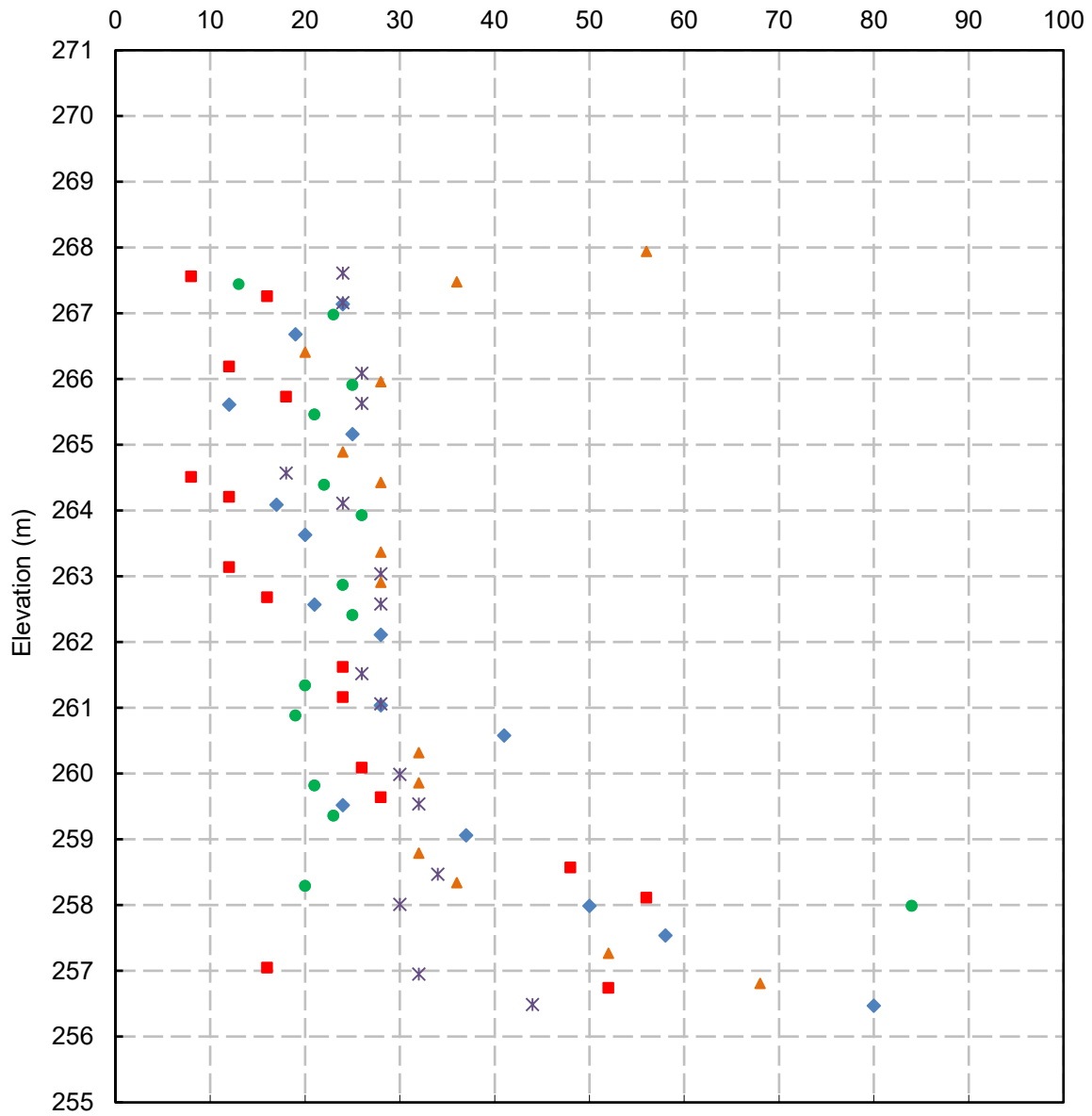
# UNDRAINED SHEAR STRENGTH

FIGURE B6

UNNAMED CREEK CULVERT (Site 39E-0213/C0)

Silty Clay to Clay

Cu (kPa)



◆ BH 1

■ BH 2

● BH 3

✱ BH 4

▲ BH 5

Project No. : 1-17-0822

Date : August, 2018

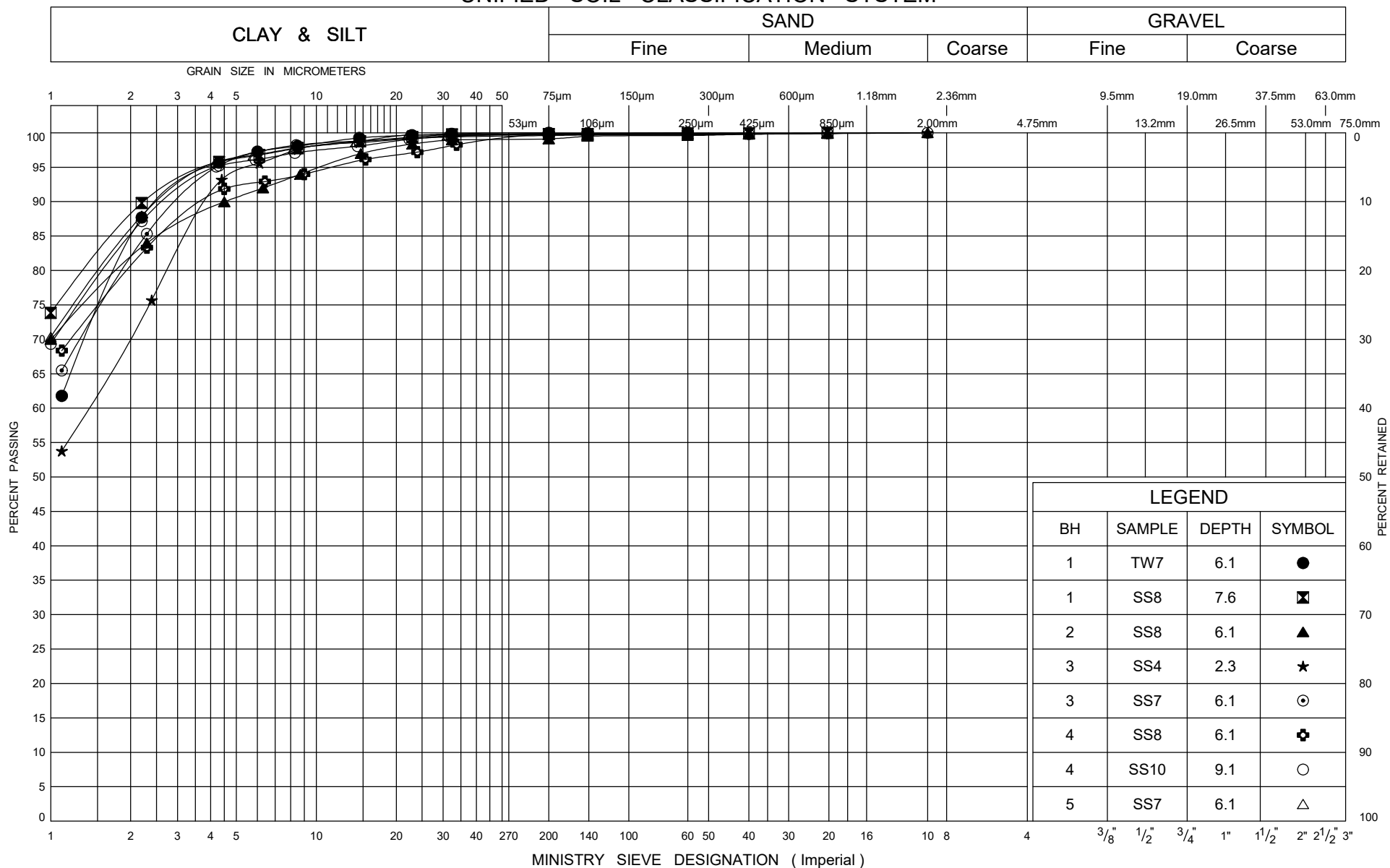


**Terraprobe Inc.**

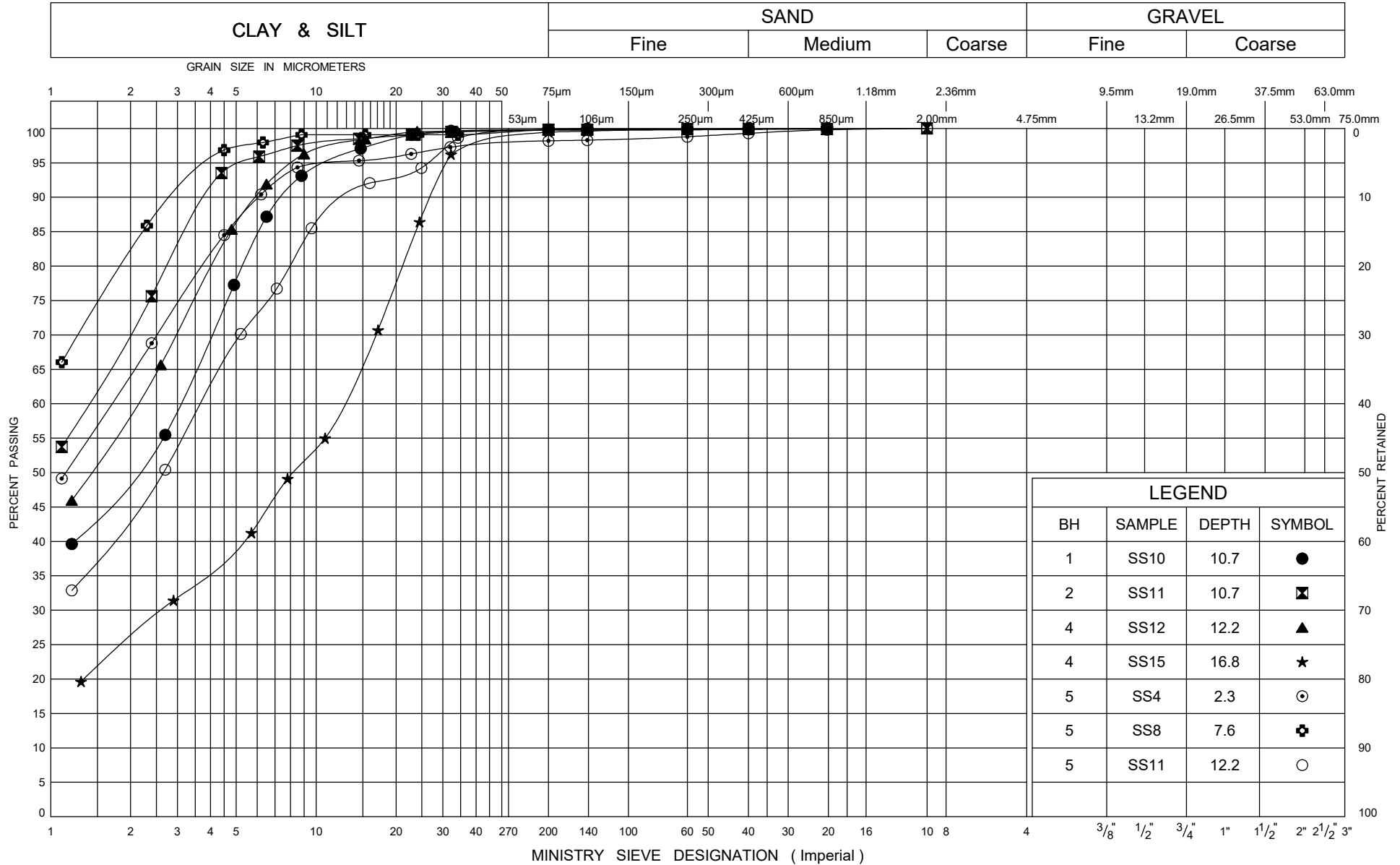
Prepared by : SD

Checked by : RA

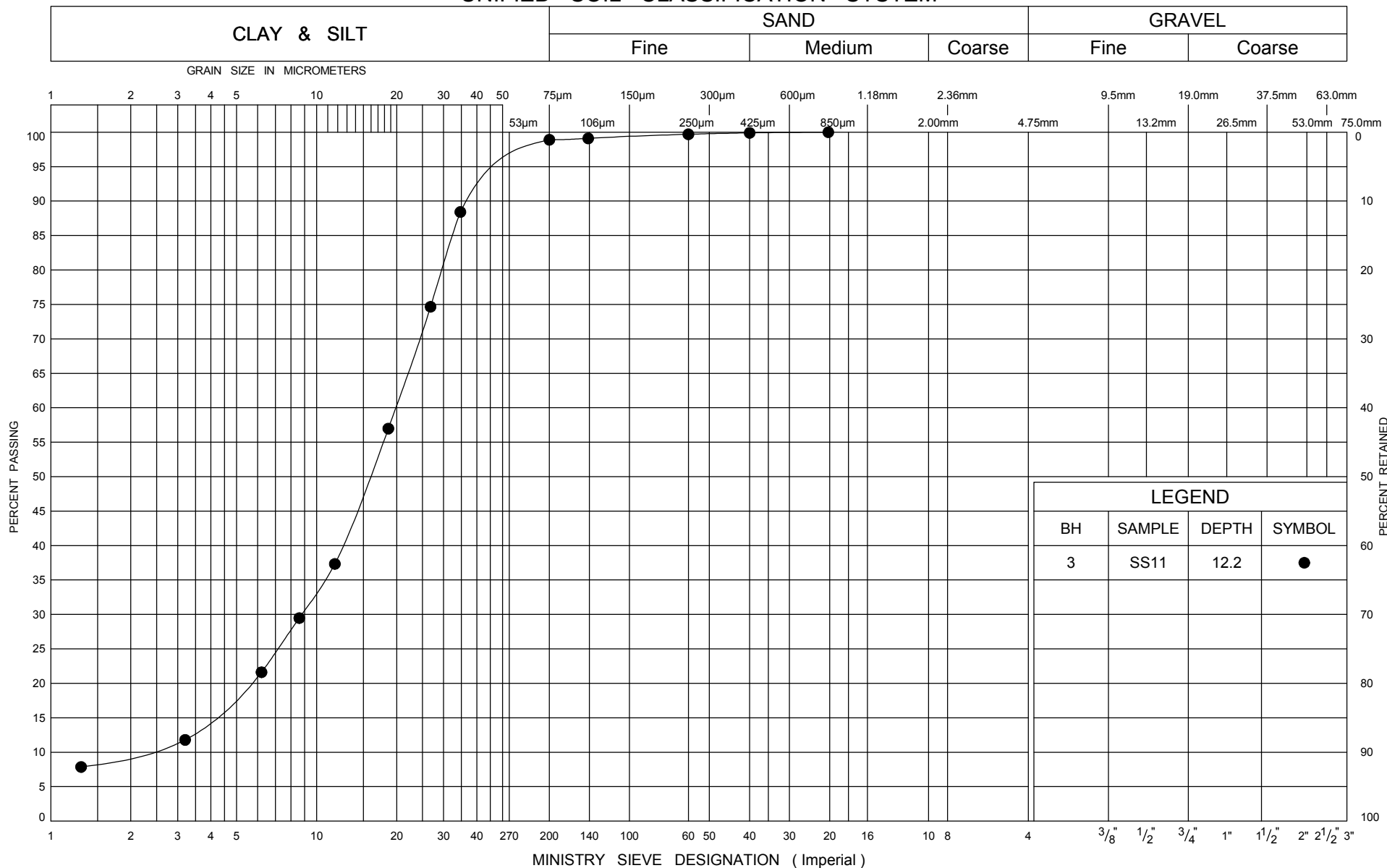
# UNIFIED SOIL CLASSIFICATION SYSTEM

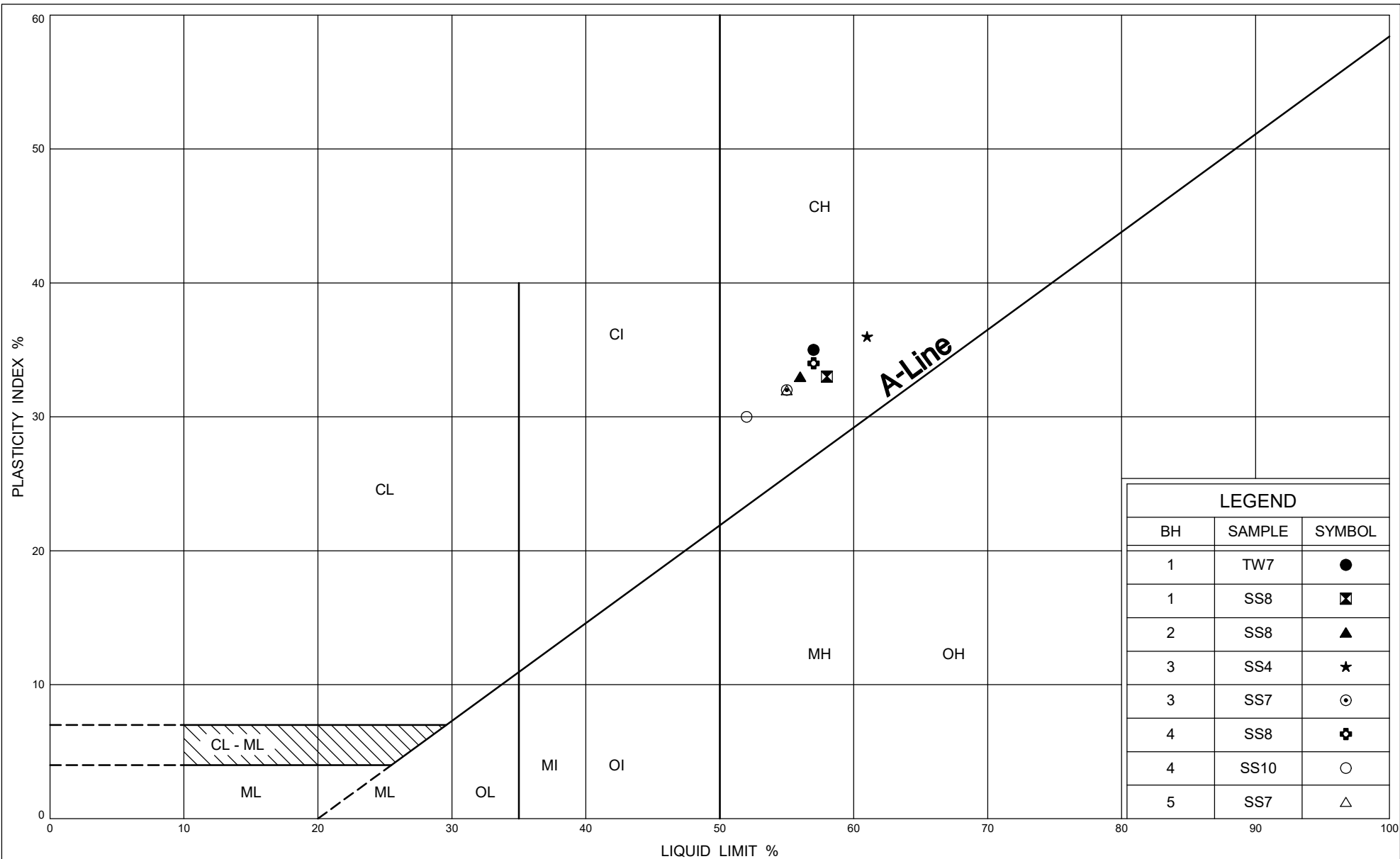


# UNIFIED SOIL CLASSIFICATION SYSTEM



# UNIFIED SOIL CLASSIFICATION SYSTEM





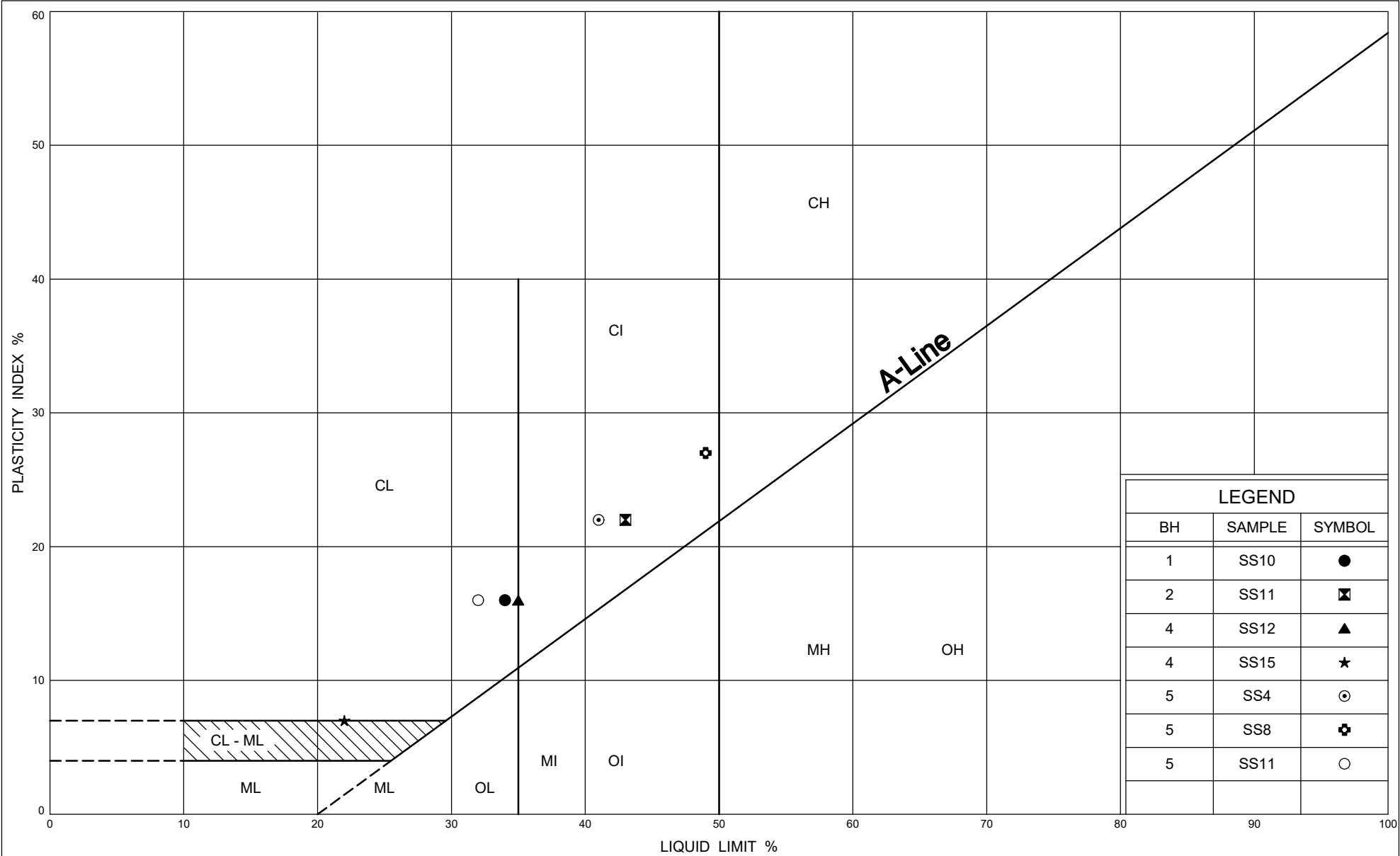
Ministry of  
Transportation

# PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B10

G W P 5165-12-00

Unnamed Creek Culvert (39E-0213/C0)



file: unknown creek and combined 11-14-4066.gpj



Ministry of  
Transportation

# PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B11  
G W P 5165-12-00  
Unnamed Creek Culvert (39E-0213/C0)

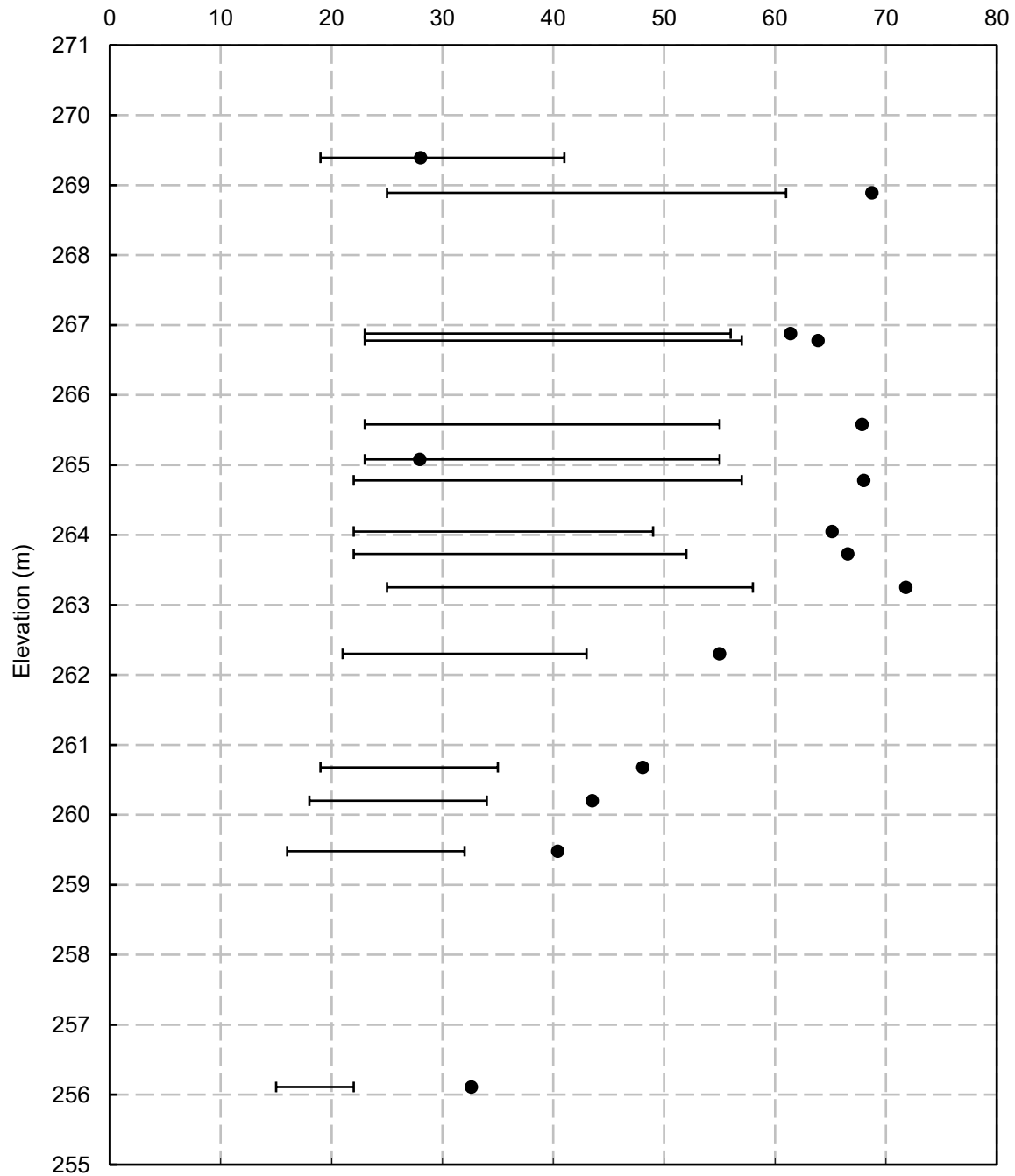
# ATTERBERG LIMITS AND WATER CONTENTS

FIGURE B12

## UNNAMED CREEK CULVERT (Site 39E-0213/C0)

Silty Clay to Clay

Atterberg Limits & Water Contents (%)



E:\Projects\Analysis\1-17-0822\Unnamed Creek\0-Pc-Cc-Cr-Cu (1-17-0822 un).xls

Project No. : 1-17-0822

Date : August, 2018



Prepared by : SD

Checked by : RA





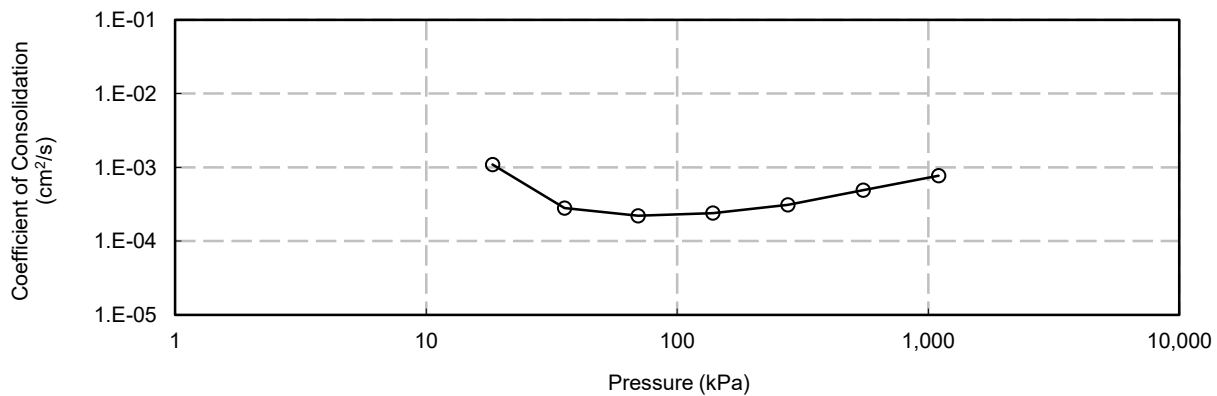
# CONSOLIDATION TEST

FIGURE B1(

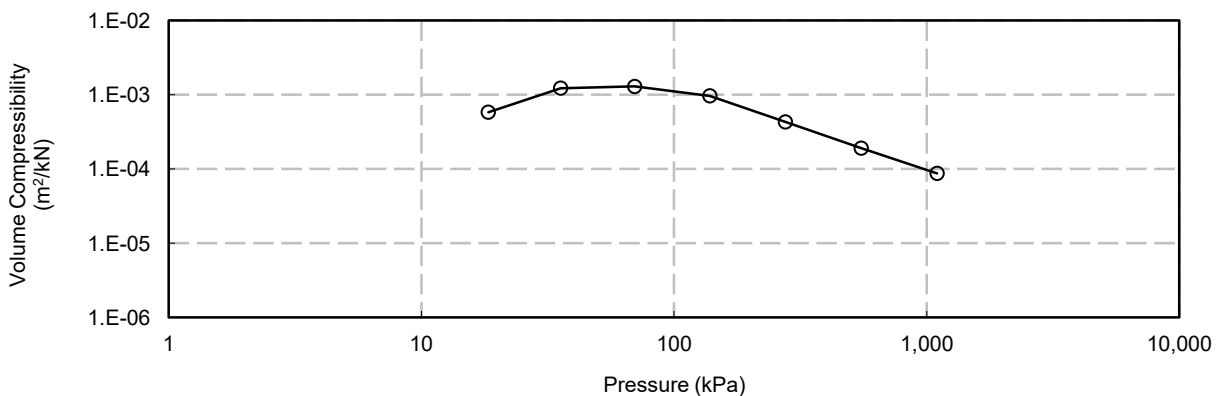
## UNNAMED CREEK CULVERT (Site 39E-0213/C0)

BH 1, TW 7

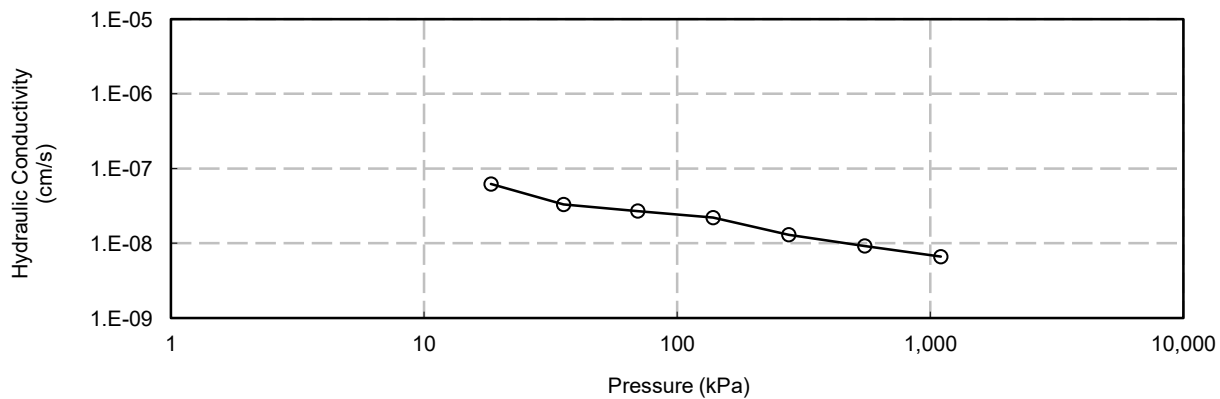
Cv vs Pressure



mv vs Pressure



k vs Pressure



Project No. : 1-17-0822  
Date : December 2014



Prepared By : SD  
Checked By : RA

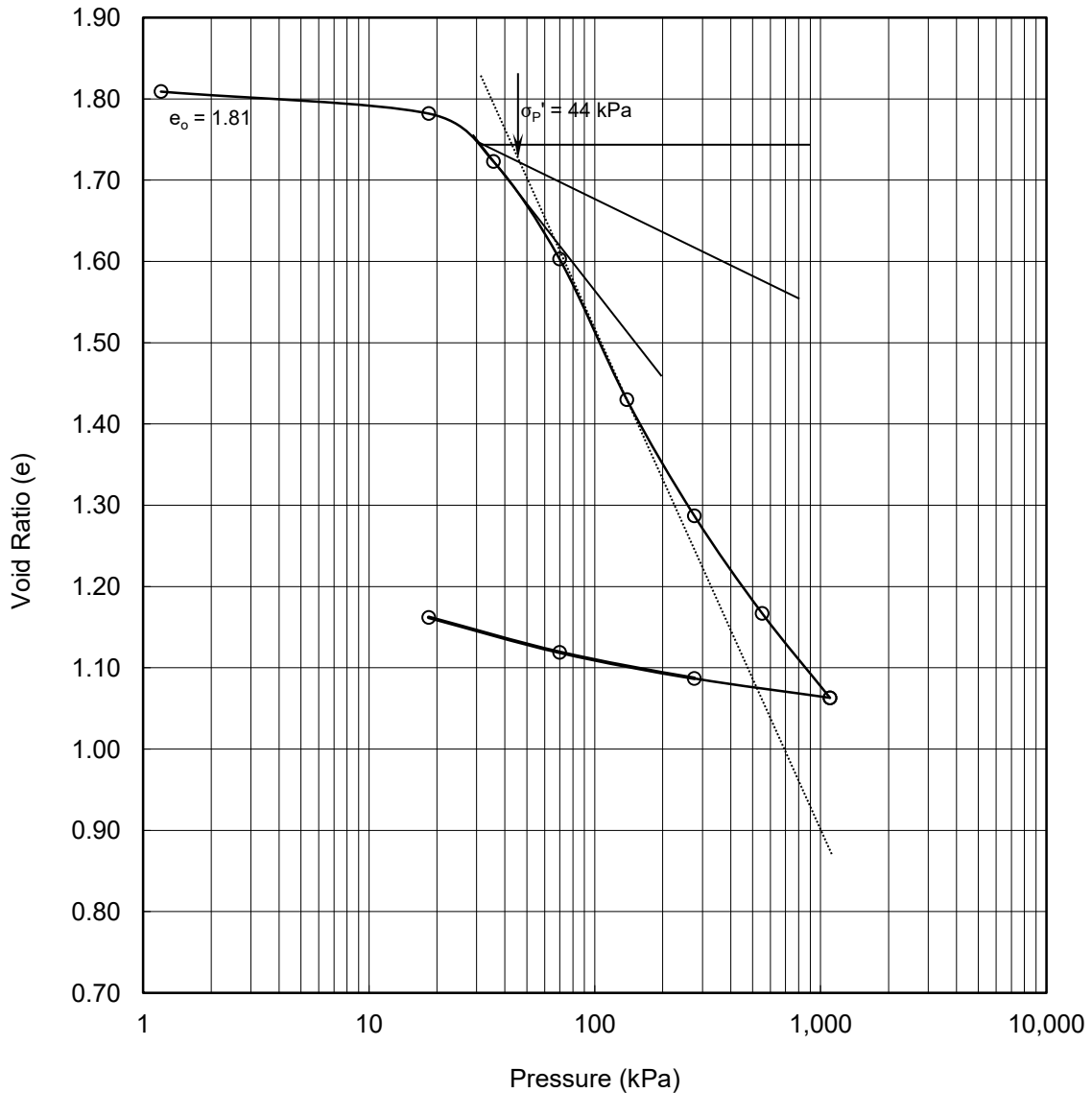
# CONSOLIDATION TEST

FIGURE B1)

## UNNAMED CREEK CULVERT (Site 39E-0213/C0)

BH 1, TW 7

### Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

$e_o =$	1.81	$\omega_L =$	57%	$\sigma_{v0}' =$	43.6 kPa
$\omega =$	68%	$\omega_P =$	22%	$\sigma_P' =$	44.0 kPa
$\gamma =$	16.0* kN/m <sup>3</sup>	PI =	35%		
Gs =	2.70				

\*Unit weight test carried out on sample trimmings.

Project No. : 1-17-0822  
Date : December 2014



Prepared By : SD  
Checked By : RA

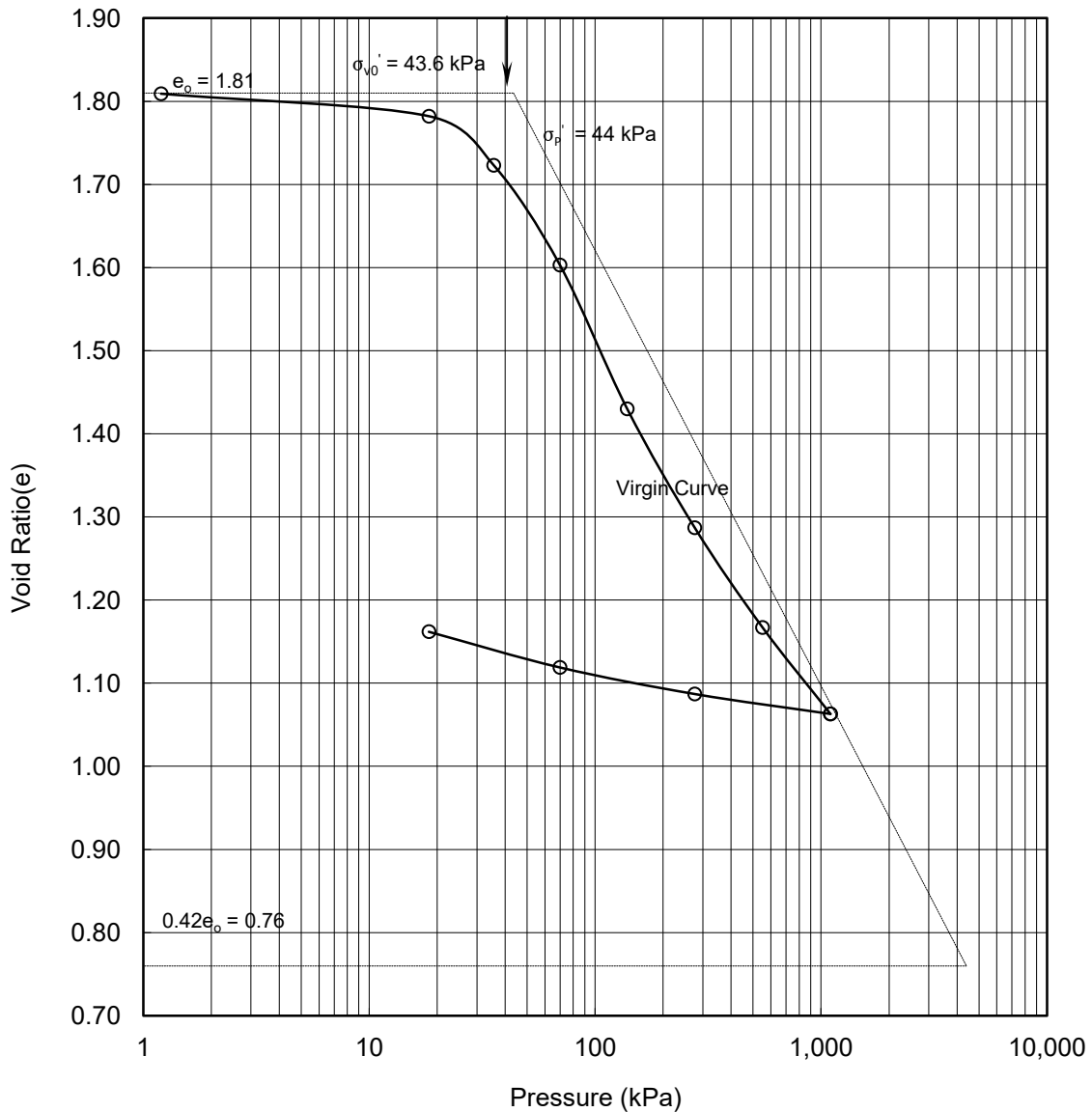
# CONSOLIDATION TEST

FIGURE B1\*

## UNNAMED CREEK CULVERT (Site 39E-0213/C0)

BH 1, TW 7

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

$e_0 =$	1.81	$\omega_L =$	57%	$\sigma_{v0}' =$	43.6 kPa
$\omega =$	68%	$\omega_P =$	22%	$\sigma_p' =$	44.0 kPa
$\gamma =$	16.0* kN/m <sup>3</sup>	PI =	35%	$C_c =$	0.524
Gs =	2.70			$C_r =$	0.056

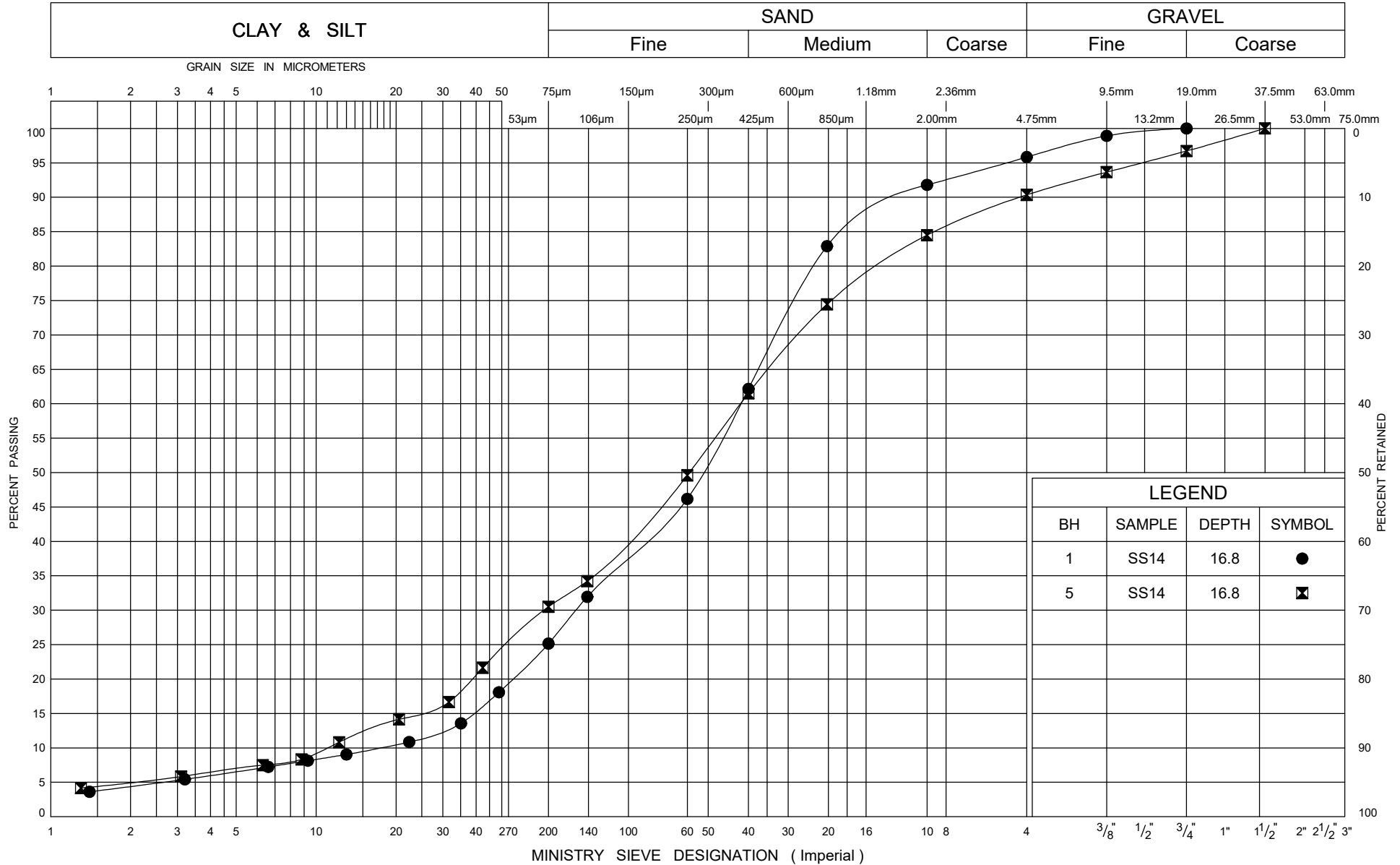
\*Unit weight test carried out on sample trimmings.

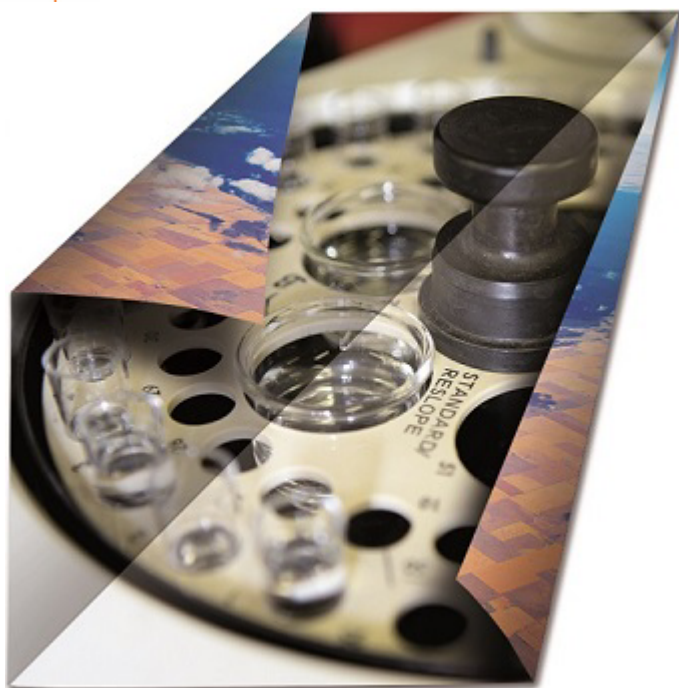
Project No. : 1-17-0822  
Date : December 2014



Prepared By : SD  
Checked By : RA

# UNIFIED SOIL CLASSIFICATION SYSTEM





## FINAL REPORT

CA14436-MAY18 R1

1-17-0822 Unnamed Creek

Prepared for

**Terraprobe Inc**

## First Page

### CLIENT DETAILS

Client Terraprobe Inc

Address 11 Indell Lane  
Brampton, ON  
L6T 3Y3, Canada

Contact Sepideh D\_Monfared

Telephone (905) 796-2650

Facsimile (905) 796-2250

Email smonfared@terraprobe.ca

Project 1-17-0822 Unnamed Creek

Order Number

Samples Soil (1)

### LABORATORY DETAILS

Project Specialist Brad Moore Hon. B.Sc

Laboratory SGS Canada Inc.

Address 185 Concession St., Lakefield ON, K0L 2H0

Telephone 705-652-2143

Facsimile 705-652-6365

Email brad.moore@sgs.com

SGS Reference CA14436-MAY18

Received 05/15/2018

Approved 05/22/2018

Report Number CA14436-MAY18 R1

Date Reported 09/18/2019

### COMMENTS

Temperature of Sample upon Receipt: 7 degrees C

Cooling Agent Present: No

Custody Seal Present: No

Corrosivity Index is based on the American Water Works Corrosivity Scale according to AWWA C-105. An index greater than 10 indicates the soil matrix may be corrosive to cast iron alloys.

### SIGNATORIES

Brad Moore Hon. B.Sc

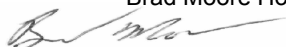




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# FINAL REPORT

CA14436-MAY18 R1

**Client:** Terraprobe Inc

**Project:** 1-17-0822 Unnamed Creek

**Project Manager:** Sepideh D\_Monfared

**Samplers:** Sepideh D\_Manfared

## PACKAGE: - Corrosivity Index (SOIL)

**Sample Number** 5  
**Sample Name** Unnamed Creek,  
BH5, SS4,  
(71/2-9)  
**Sample Matrix** Soil  
**Sample Date** 02/05/2018

Parameter	Units	RL	Result
<b>Corrosivity Index</b>			
Soil Redox Potential	mV	-	97
Sulphide	%	0.02	< 0.02
pH	no unit	0.05	8.44
Resistivity (calculated)	ohms.cm	-9999	2840

## PACKAGE: - General Chemistry (SOIL)

**Sample Number** 5  
**Sample Name** Unnamed Creek,  
BH5, SS4,  
(71/2-9)  
**Sample Matrix** Soil  
**Sample Date** 02/05/2018

Parameter	Units	RL	Result
<b>General Chemistry</b>			
Conductivity	uS/cm	2	352

## PACKAGE: - Metals and Inorganics (SOIL)

**Sample Number** 5  
**Sample Name** Unnamed Creek,  
BH5, SS4,  
(71/2-9)  
**Sample Matrix** Soil  
**Sample Date** 02/05/2018

Parameter	Units	RL	Result
<b>Metals and Inorganics</b>			



FINAL REPORT

CA14436-MAY18 R1

Client: Terraprobe Inc  
Project: 1-17-0822 Unnamed Creek  
Project Manager: Sepideh D\_Monfared  
Samplers: Sepideh D\_Manfared

PACKAGE: - Metals and Inorganics (SOIL)

Sample Number 5  
Sample Name Unnamed Creek,  
BH5, SS4,  
(71/2-9)  
Sample Matrix Soil  
Sample Date 02/05/2018

Parameter	Units	RL	Result
Metals and Inorganics (continued)			
Moisture Content	%	0.1	23.5
Sulphate	µg/g	0.4	7.8

PACKAGE: - Other (ORP) (SOIL)

Sample Number 5  
Sample Name Unnamed Creek,  
BH5, SS4,  
(71/2-9)  
Sample Matrix Soil  
Sample Date 02/05/2018

Parameter	Units	RL	Result
Other (ORP)			
Chloride	µg/g	0.4	430



FINAL REPORT

CA14436-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	EWL0274-MAY18	uS/cm	2	< 0.002	ND	10	98	90	110	NA		



FINAL REPORT

CA14436-MAY18 R1

QC SUMMARY

pH  
Method: SM 4500 | Internal ref.: ME-CA-|ENVIEWL-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	EWL0274-MAY18	no unit	0.05	NA	0		99			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](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**

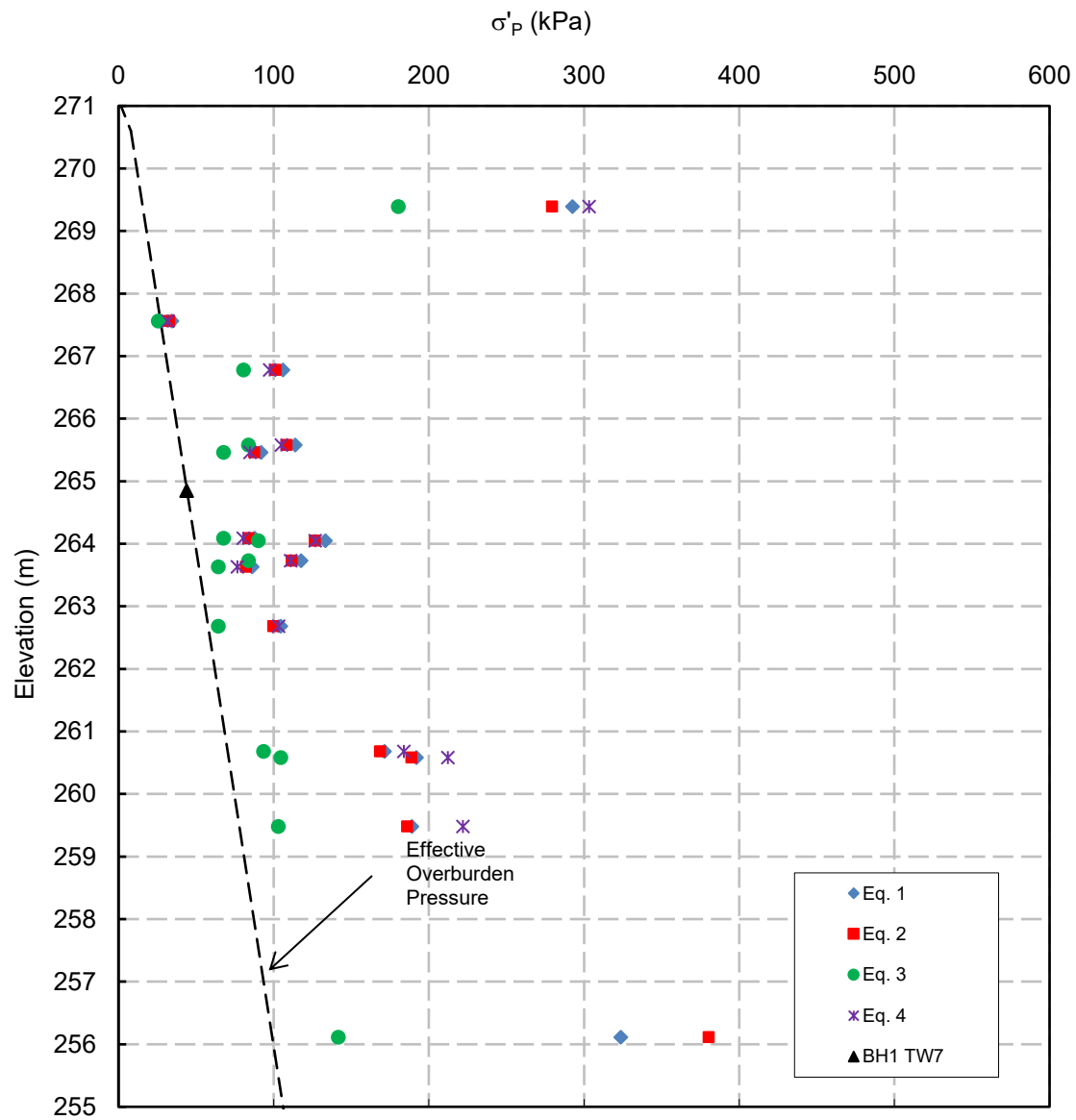


# PREDICTED AND MEASURED PRECONSOLIDATION STRESSES

FIGURE C1

## UNNAMED CREEK CULVERT (Site 39E-0213/C0)

Silty Clay to Clay



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 : August, 2018

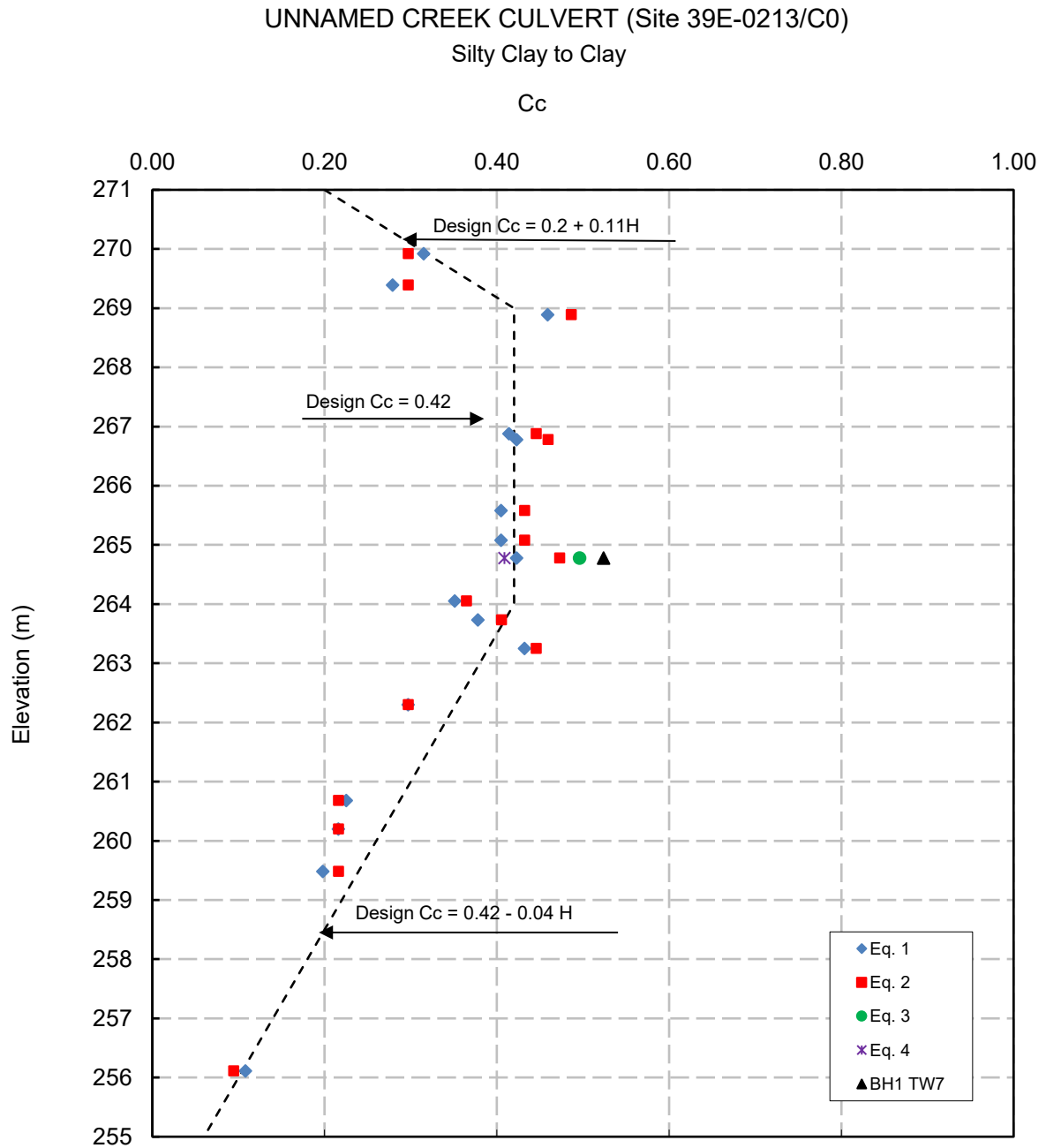


**Terraprobe Inc.**

Prepared by : SD

Checked by : RA





Eq. 1  $C_c = 0.009 * (LL - 10)$

Eq. 2  $C_c = I_p / 74$

Eq. 3  $C_c = 0.141 * G_s^{1.2} * ((1 + e_o) / G_s)^{2.38}$

Eq. 4  $C_c = 0.141 * G_s * (\gamma_w / \gamma_d)^{2.4}$

Terzaghi & Peck (1967)

Kulhaway & Mayne (1990)

Rendon - Herrero (1983)

Herrero (1983)

Project No. : 1-17-0822

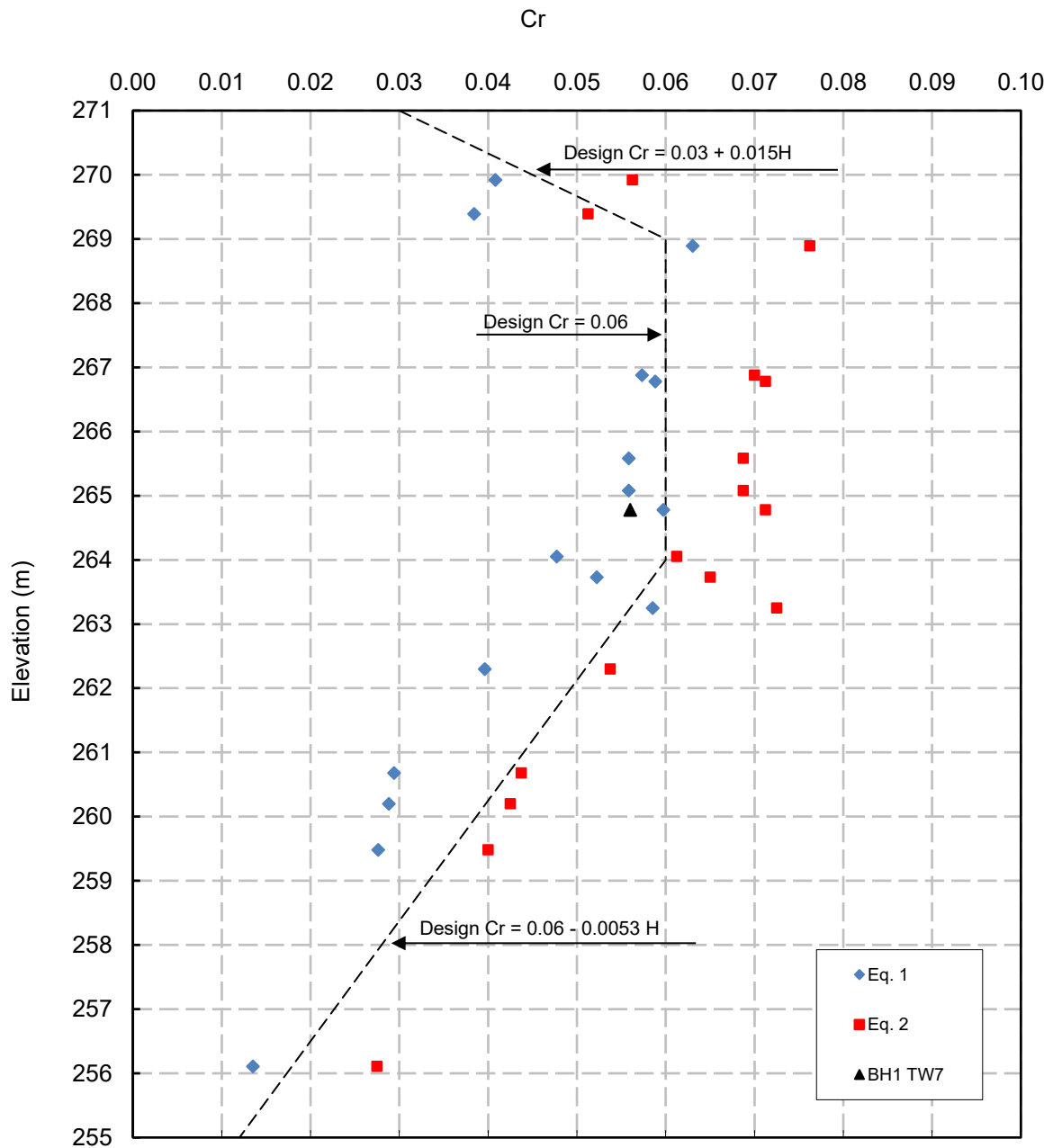
Date : August, 2018



Prepared by : SD

Checked by : RA

UNNAMED CREEK CULVERT (Site 39E-0213/C0)  
Silty Clay to Clay



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

Das (1993)

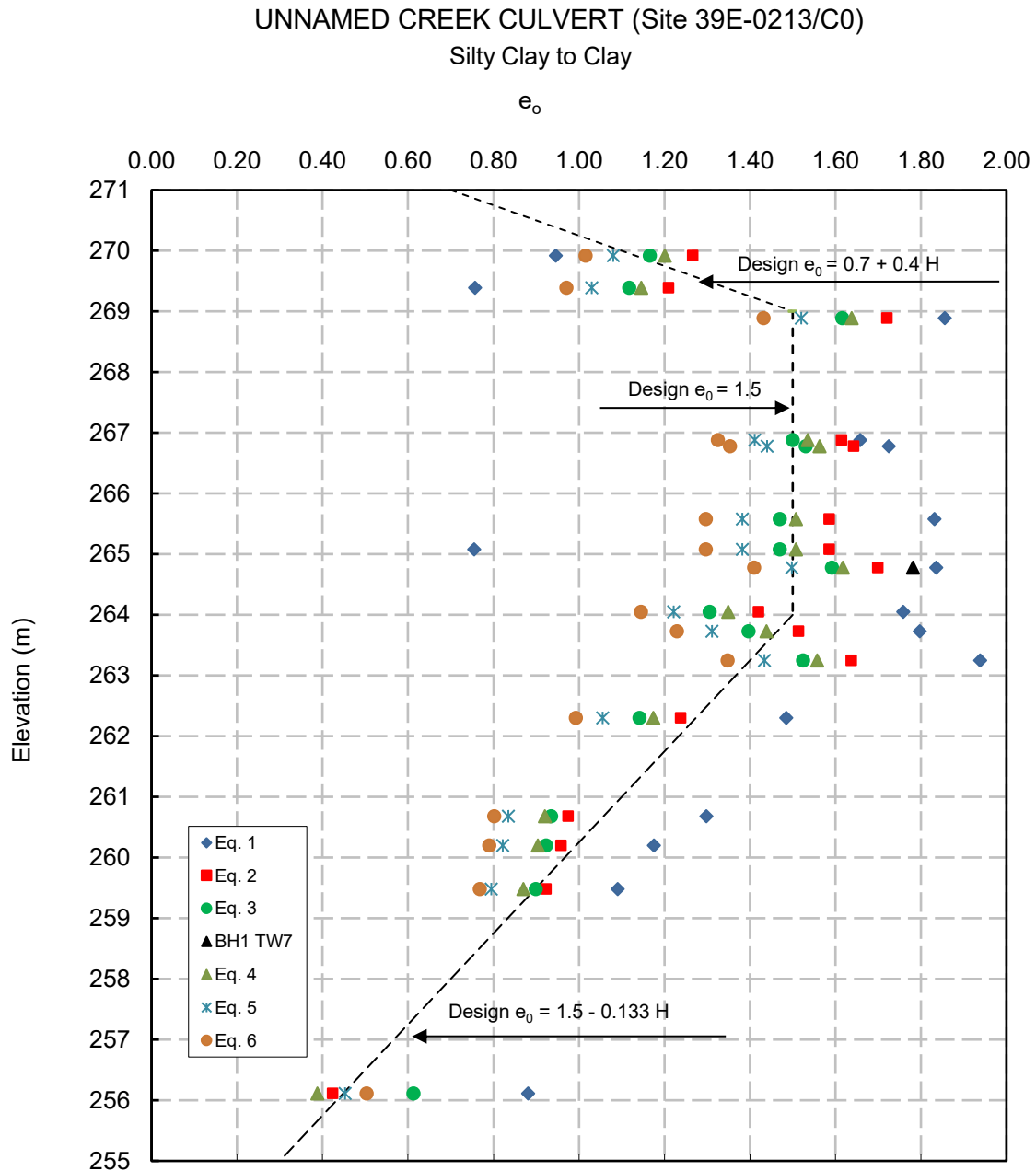
Eq. 2  $Cr = 0.000463 * LL * Gs$

Nagaraj & Murty (1985)



# PREDICTED AND MEASURED VOID RATIOS

FIGURE C4



Eq. 1  $e_o = \omega * G_s$

when saturated

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

derived from Rendon - Herrero (1983)

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

derived from Azzouz et al. (1976)

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

derived from Rendon - Herrero (1980)

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

derived from Lav & Ansal (2001)

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

derived from Lav & Ansal (2001)

Project No. : 1-17-0822

Date : August, 2018



Prepared by : SD

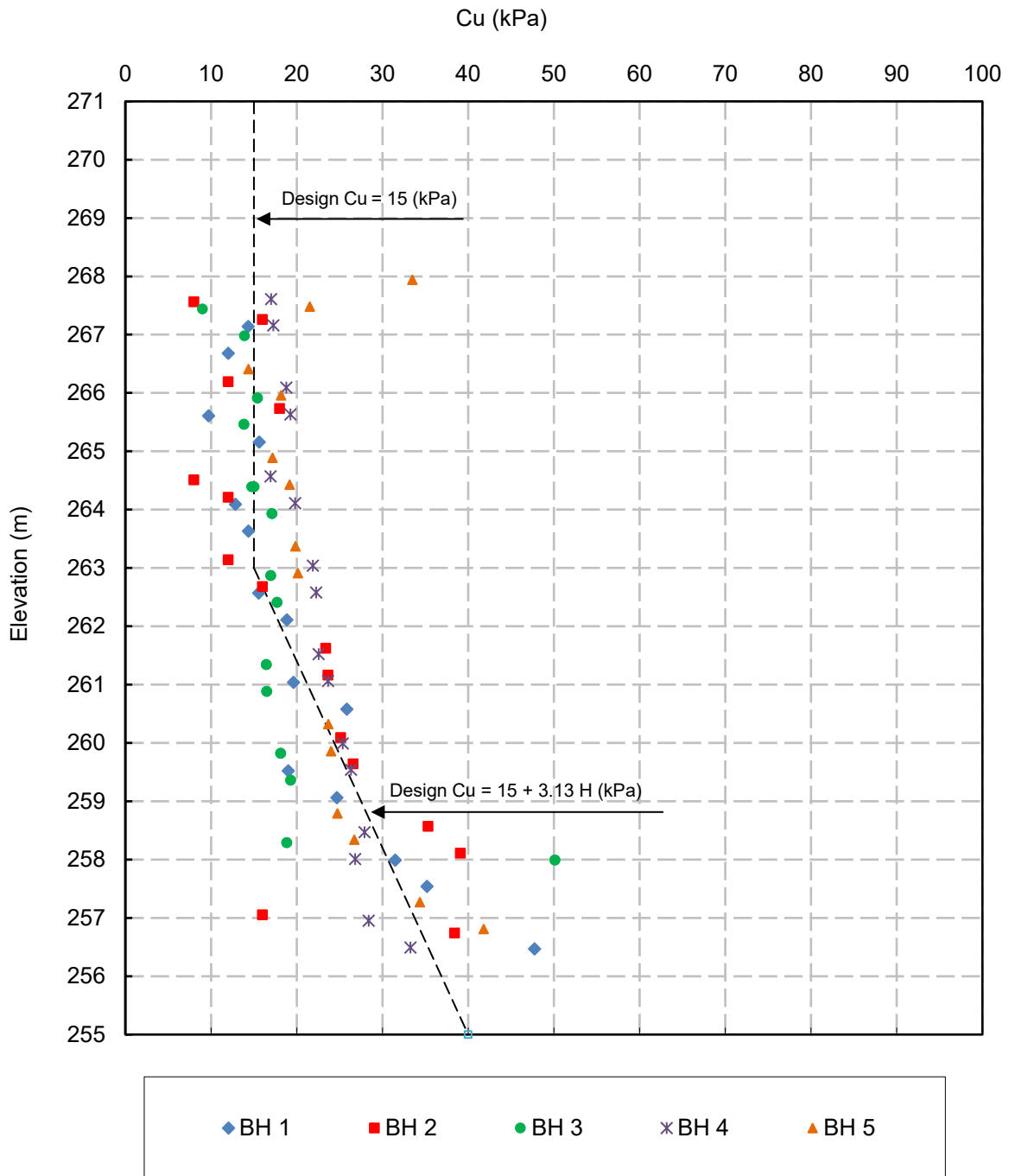
Checked by : RA

# UNDRAINED SHEAR STRENGTH

FIGURE C5

## UNNAMED CREEK CULVERT (Site 39E-0213/C0)

Silty Clay to Clay



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

Project No. : 1-17-0822

Date : August, 2018



**Terraprobe Inc.**

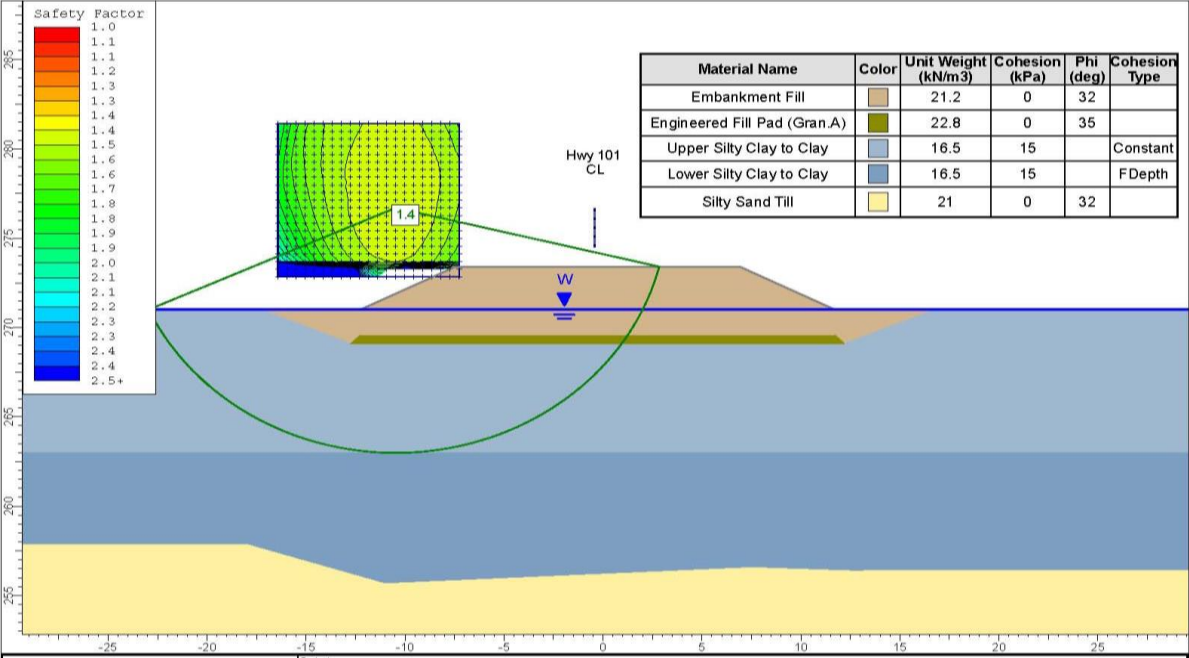
Prepared by : SD

Checked by : RA

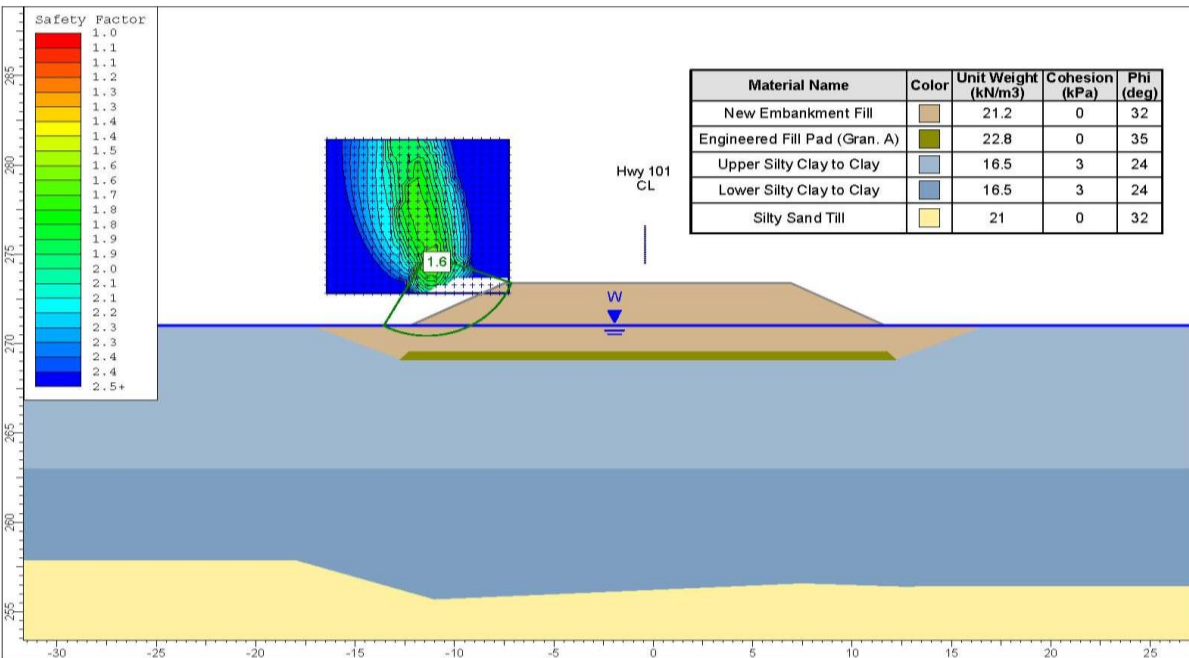
## **APPENDIX D**

### **Slope Stability Models & Results**

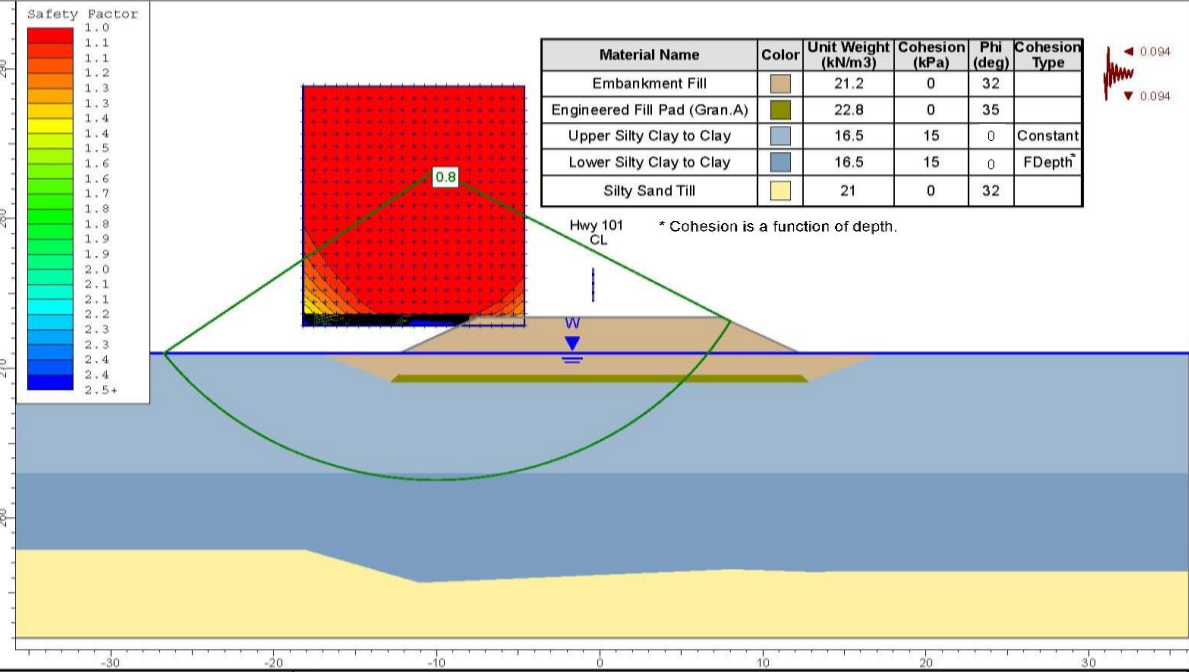




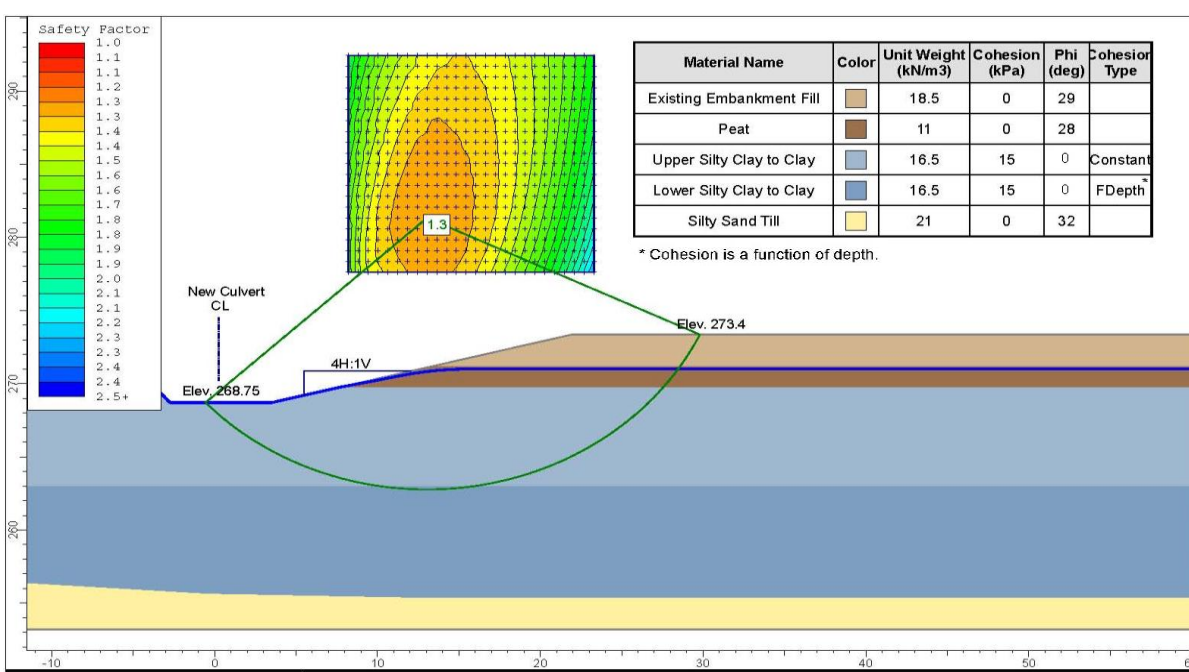
Hwy. 101 Permanent Side Slopes - Static Analysis  
(Total Stress Analysis)



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



Hwy. 101 Permanent Side Slopes - Seismic Analysis



Temporary Excavation Side Slopes - East Side of Unnamed 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**

<b>D4355 - 07</b>	Standard Test Method for Deterioration of Geotextiles by Exposure to Light, Moisture and Heat in a Xenon Arc Type Apparatus
<b>D4759 - 02(2007)</b>	Standard Practice for Determining the Specification Conformance of Geosynthetics
<b>D5818 - 11</b>	Standard Practice for Exposure and Retrieval of Samples to Evaluate Installation Damage of Geosynthetics
<b>D6637 - 10</b>	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 8 kPa to 24 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 8 kPa to 24 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.**

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Special Provision

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### **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**

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#### **Special Provision**

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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;

<b>Roadway/ Township</b>	<b>Location</b>	<b>Preload Period (months)</b>
Hwy 101 / Township of Taylor	North of Hwy 101 From 6 m Away From New Culvert Outlet to 1 m Beyond New Culvert Outlet	6 months
	South of Hwy 101 From 6 m Away From New Culvert Inlet 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**

