Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

FOUNDATION INVESTIGATION AND DESIGN REPORT
SHILLINGTON CREEK CULVERT REPLACEMENT
SITE 39E-0214/C0 - HIGHWAY 101
ASSIGNMENT No. 5016-E-0023 - G.W.P. 5165-12-00
MINISTRY OF TRANSPORTATION, ONTARIO
NORTHEASTERN REGION
GEOCRES NO. 42A-141

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

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

Distribution:

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

Terraprobe Inc.

Greater Toronto	Hamilton – Niagara	Central Ontario	Northern Ontario
11 Indell Lane	903 Barton Street, Unit 22	220 Bayview Drive, Unit 25	1012 Kelly Lake Rd., Unit 1
Brampton, Ontario L6T 3Y3	**Stoney Creek**, Ontario L8E 5P5	**Barrie**, Ontario L4N 4Y8	**Sudbury**, Ontario P3E 5P4
(905) 796-2650 Fax: 796-2250	(905) 643-7560 Fax: 643-7559	(705) 739-8355 Fax: 739-8369	(705) 670-0460 Fax: 670-0558
brampton@terraprobe.ca	stoneycreek@terraprobe.ca	barrie@terraprobe.ca	sudbury@terraprobe.ca

www.terraprobe.ca

TABLE OF CONTENTS

PART A – FOUNDATION INVESTIGATION REPORT

SHILLINGTON CREEK CULVERT REPLACEMENT, SITE 39E-0214/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 Shillington 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 Shillington Creek culvert Site 39E-0214/C0 on Highway 101, Township of Taylor, District of Cochrane, Ontario.

2.0 SITE DESCRIPTION

The site (Latitude 48.538º; Longitude -80.673º) is located on Highway 101, approximately 600 m east of the highway's intersection with Highway 577 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 20.0 m long and its internal cross-section is 4.9 m wide and 1.2 m high. The Highway 101 embankment is approximately 2 m high at the site with a pavement centre line elevation of 267.4 ±m.

The culvert conveys creek flows from south to north 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 21.8 m to 25.8 m below ground surface between July 14 and August 14, 2014. Under this current assignment, two additional boreholes numbered Boreholes 4 and 5 were drilled and sampled to depths of 21.3 m and 22.5 m below ground surface between April 25 and May 03, 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 site features and WSP surveyors established Horizontal Control Point 100 with a geodetic elevation of 269.1 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	5 377 752.5	328 938.3	266.0	22.9
BH2	5 377 733.2	328 923.6	267.2	25.8
BH3	5 377 725.6	328 926.9	266.3	21.8
BH4	5 377 736.2	328 946.0	267.5	22.5
BH5	5 377 750.1	328 924.4	265.9	21.3

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[1]. 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.

Ground water conditions in the open boreholes were observed throughout the drilling operations. Hydrostatic uplift was encountered in Borehole 3 at a depth of 19.8 m within the silty clay to clay deposit immediately above the underlaying silty sand till deposit and, artesian condition was encountered at a depth of 22.0 m below ground surface in Borehole 4 within the silty sand till deposit. These boreholes were backfilled immediately and sealed in accordance with current MTO procedures and Ontario Regulation 903 (as amended). Standpipe piezometers were installed in Boreholes 1 and 5 to permit longer term ground water level monitoring.

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

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.

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

4.2.1 Topsoil

Topsoil layers ranging from 25 mm to 150 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 lane and shoulder. The flexible pavement consists of 240 mm thick asphaltic concrete underlain by granular fill consisting of gravelly sand. 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	600	266.6
BH4	450	266.8

Standard Penetration tests carried out in the gravelly sand fill measured SPT N-values of 41 and more than 100 blows for 0.3 m of penetration indicating a dense to very dense relative density. The natural water contents of two samples of the gravelly sand fill are 2% and 5% by weight.

4.2.3 Fill – Sand

Sand fill was encountered below the gravelly sand fill in Boreholes 2 and 4. The locations, thicknesses and base elevations of the sand fill are summarized in the following table.

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH2	0.8	1.4	265.8
BH4	1.7	2.4	265.1

Standard Penetration tests carried out in the sand fill measured SPT N-values ranging from 22 to more than 100 blows for 0.3 m of penetration indicating a compact to very dense relative density. The natural water contents of a sample of the sand fill 12% by weight.

4.2.4 Fill – Silty Clay

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

Borehole No.	Fill Thickness (m)	Fill Depth (m)	Fill Base Elevation (m)
BH1	1.2	1.4	264.6
BH2	1.5	2.9	264.3
BH3	2.0	2.1	264.2
BH5	1.2	1.2	264.7

Standard Penetration tests performed in the silty clay fill measured SPT N-values ranging from 1 to 25 blows for 0.3 m of penetration indicating a very soft to very stiff consistency. The natural water content of samples of the silty clay fill range from 19% to 39% by weight.

The grain size distribution plots of two samples of the silty clay fill are depicted on Figure B1 in Appendix B. These results show a grain size distribution consisting of 0% and 2% gravel, 11% and 40% sand, 27% and 31% silt and, 29% and 60% clay sized particles.

Atterberg limits tests were also carried out on two samples of the silty clay fill and the results are presented in Figure B2 in Appendix B. These values indicate that the fill is a cohesive soil with low to intermediate plasticity (CL to CI). The results from the Atterberg limits tests are summarized below:

Liquid Limit:	28% and 46%
Plastic Limit:	15% and 21%
Plasticity Index:	13% and 25%
Natural Moisture Content:	19% and 39%

4.2.5 Peat

The fill material at Boreholes 4 and 5 are underlain by 0.2 m and 0.4 m thick layers of amorphous peat that extend to depths of 2.6 m (elevation 264.9 m) and 1.6 m (elevation 264.3 m) respectively.

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 and clayey silt ranging from 1 mm to 40 mm in thickness. Photographs illustrating the varved clay matrix are provided in Figure B3 in Appendix B. The locations, thicknesses depths and base elevations of the silty clay to clay deposit are summarized in the following table.

Borehole No.	Thickness (m)	Depth (m)	Base Elevation (m)
BH1	17.9	19.3	246.7
BH2	18.7	21.6	245.6
BH3	18.7	20.8	245.5
BH4	17.4	20.0	247.5
BH5	17.7	19.3	246.6

The N-values of Standard Penetration tests carried out in the silty clay to clay deposit range from 0 blows (weight of hammer) to 18 blows per 0.3 m of penetration and, field vane tests generally measured in-situ undrained shear strengths ranging from 12 kPa to more than 100 kPa as illustrated on Figure B4 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 very stiff. The sensitivity of the silty clay to clay generally ranges from about 1.2 to 6, 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 versus elevation is depicted in Figure B4. Stiff to very stiff silty clay to clay was encountered at Borehole 2 between elevation 263.5 m and 262.5 m with undrained shear strength values of 92 kPa and more than 100 kPa. Between elevation 262.5 m and 251.0 m, very soft to firm clay soils with undrained shear strength values ranging from 12 kPa to 40 kPa were encountered. Below elevation 251.0 m, there is a trend of increasing undrained shear strength values with depth, with shear strength values ranging from about 32 kPa to 88 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 B5 to B7 in Appendix B. The test results show a grain size distribution consisting of 0% gravel, 0% to 1% sand, 12% to 78% silt and, 21% to 88% clay sized particles.

Atterberg limits tests were carried out on seventeen samples of the silty clay to clay and the results are plotted on the plasticity charts, Figures B8 to B10 in Appendix B. The results indicate a cohesive deposit of generally intermediate to high plasticity (CI to CH) with layers of low plasticity clay (CL). The Atterberg limits test results are summarized below.

Liquid Limit:	23% to 63%
Plastic Limit:	17% to 27%
Plasticity Index:	8% to 36%
Natural Moisture Content:	23% to 73%

Grain size distribution tests were carried out on two samples of the clayey silt layers embedded within the silty clay to clay matrix and the grain size distribution curves are shown on Figure B11 in Appendix B. The grain size distribution of the clayey silt consists of 0% gravel, 0% and 1% sand, 73% and 83% silt and; 17% and 26% clay sized particles.

Atterberg limits tests were also carried out on two samples of the clayey silt layers embedded within the silty clay to clay matrix and the results are plotted on the plasticity chart, Figures B12 in Appendix B. These values indicate low plasticity (CL-ML to CL) clayey silt interbeds. The Atterberg limits test results are summarized below.

Liquid Limit:	23% and 24%
Plastic Limit:	17% and 18%
Plasticity Index:	5% and 7%
Natural Moisture Content:	28% and 30%

The Atterberg Limits tests results of the silty clay to clay deposit are also plotted against elevation in Figure B13. These results illustrate that the natural moisture contents of the tested samples are typically higher than the liquid limits. The moisture content of samples of the silty clay to clay varies between 23% and 74% and the unit weight of a tested sample is 16.8± 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 B14 to B17 in Appendix B. The results of the one-dimensional consolidation test are summarized below.

Borehole/Sample No.	Sample Depth/Elevation (m)	σ'_{vo} (kPa)	σ'_P (kPa)	C_c	C_r	e_o
BH3, Sample 6	4.7/261.7	36.7	39	0.348	0.043	1.37

Where: σ'_{vo} = effective overburden pressure
σ'_P = Preconsolidation pressure;
C_c = Compression index
C_r = Recompression index; and
e_o = Initial void ratio

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

4.2.7 Silty Sand to Sand Till

A silty sand to sand till deposit was encountered at this site. Summarized below are the explored depths and base elevations of the silty sand to sand till.

Borehole No.	Thickness (m)	Depth (m)	Base Elevation (m)
BH1	3.6	22.9*	243.1
BH2	4.2	25.8*	241.4
BH3	1.0	21.8*	244.5
BH4	2.5	22.5*	245.0
BH5	2.0	21.3*	244.6

* Borehole termination depth.

Standard Penetration tests carried out in this deposit measured N-values that range from 19 to more than 100 blows per 0.3 m of penetration indicating a compact to very dense relative density. At Boreholes 4 and 5 the Standard Penetration tests carried out in the silty sand to sand till deposit immediately below the silty clay to clay deposit measured N-values of 1. It is likely that these SPT tests were carried out in areas of the silty sand to sand till that have experienced hydrostatic uplift because of the artesian conditions present. The natural water content of samples from this stratum range from 9% to 21%.

Grain size distribution tests were carried out on two samples from this deposit and the results illustrated in Figure B18, Appendix B show a grain size distribution consisting of 2% and 10% gravel, 61% and 70%

sand, 15% and 29% silt and 5% and 8% clay sized particles. Till soils can also be expected to contain random cobble and boulder inclusions.

4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling and standpipe piezometers were installed in Boreholes 1 and 5. Hydrostatic uplift was encountered in Borehole 3 at a depth of 19.8 m within the silty clay to clay deposit immediately above the underlying silty sand till deposit and, artesian condition was encountered at a depth of 22.0 m below ground surface in Borehole 4 within the silty sand till deposit. The recorded artesian heads of water are 2.1 m and 0.9 m above ground surface corresponding to a ground water elevation of 264.8 m. The measured ground water levels in the silty clay soils are summarized in the following table:

Borehole No.	Date	Water Levels	
		Depth (m)	Elevation (m)
BH1	September 16, 2014 October 28, 2014	0.3 0.3	265.7 265.7
BH5	May 19, 2018 November 21, 2018	0.0 0.4	265.9 265.5

The ground water level at this site is estimated to be at approximately Elevation 265.5 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 can be expected to rise during wet periods of the year.

5.0 MISCELLANEOUS

The investigation was carried out using drilling equipment supplied and operated by Tatry 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.. The routine laboratory tests 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.

Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
S. DAMAVANDINEJAD MONFARED
100523519
Oct. 21, 2021
PROVINCE OF ONTARIO

Sepideh D-Monfared

Sepideh D-Monfared, P.Eng.
Geotechnical Engineer

Rehman Abdul

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
Oct. 21, 2021
PROVINCE OF ONTARIO

Rehman Abdul, P.Eng.
Designated MTO Foundations Contact

PART B – FOUNDATION DESIGN REPORT

SHILLINGTON CREEK CULVERT REPLACEMENT, SITE 39E-0214/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 Shillington 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 20.0 ±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 264.3 ±m and 264.2 ± 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 west 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 18, 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 (ψ) and geotechnical resistance factors (Φ_{gu} & Φ_{gs}) used for designs and stipulated in Clause 6.5.2 and Clause 6.9 of the CHBDC S6-19, are based on a "typical consequence level" and a "typical degree of site and prediction model understanding".

6.4 Seismic Design

6.4.1 Importance Category & Seismic Site Classification

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

6.4.2 Spectral Response Values

The CHBDC requires that the seismic hazard values associated with the design earthquake be established based on the National Building Code of Canada (NBCC). These values, Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and Spectral Acceleration (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	0.164	0.091	0.049	0.024	0.006	0.002
Site Specific Design Seismic Hazard Values Site Class E 2% Exceedance in 50 years (2,475 Year Return Period)							
0.188	0.180	0.269	0.225	0.138	0.070	0.018	0.005

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 263.6 ±m, can be supported on a 0.5 m thick OPSS Granular A engineered fill pad founded at a base elevation of 263.1 ±m.

The engineered fill pad shall be compacted to 95% Standard Proctor maximum dry density (SPMDD) at ±2% of optimum moisture content. At elevation 263.6 ±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 263.1 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 263.1 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 (ϕ').

- OPSS Granular A bedding – internal friction angle ϕ' = 35º; and
- Silty Clay to Clay – internal friction angle ϕ' = 24º.

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

6.7 Lateral Earth Pressure

6.7.1 Static Conditions

Earth pressures are generally calculated using the following expression:

$P_h = K(\gamma h + q)$
P_h = horizontal pressure on the wall (kPa)
K = lateral earth pressure coefficient
γ = unit weight of retained soil (kN/m³)
h = depth below top of fill where pressure is computed (m)
q = value of any surcharge (kPa)

Earth pressures acting on the structure should be computed in accordance with Clause 6.12 of the CHBDC S6-19 and according to Clause 6.12.3 of the CHBDC S6-19; a compaction surcharge should also be added. For soils with an angle of internal friction ranging from 30º to 35º 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 35º. 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 $\phi = 35°$; $\gamma = 22.8$ kN/m^3		OPSS Granular B Type I $\phi = 32°$; $\gamma = 21.2$ 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 ϕ. 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 ϕ = 35°; δ = 17.5° γ = 22.8 kN/m^3	OPSS Granular B Type I ϕ = 32°; δ = 16.0° γ = 21.2 kN/m^3
	Horizontal Surface Behind Wall	Horizontal Surface Behind Wall
K_{AE} (Yielding Wall)	0.30	0.34
K_{AE} (Non-Yielding Wall)	0.37	0.41

6.8 Design Frost Depth

The frost penetration depth is 2.4 m of earth cover below the lowest surrounding grade to provide adequate protection against frost penetration, as per 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%± 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%± of optimum.

6.11 Global Stability Assessments

6.11.1 General

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

6.11.2 Static Conditions (Permanent Embankments)

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

Material Type	γ (kN/m³)	Total Stress Analysis		Effective Stress	
		ϕ (degrees)	c (kPa)	ϕ' (degrees)	c' (kPa)
New Embankment Fill	21.2	32	0	32	0
Existing Embankment Fill	18.5	29	0	29	0
Upper Silty Clay to Clay	16.5	0	14	24	3
Lower Silty Clay to Clay	16.5	0	14+1.35H	24	3
Silty Sand to Sand Till	21	32	0	32	0
Design Factor of Safety	-	1.6		1.6	

The analyses indicate that the factors of safety will be 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 1.0. The results of the seismic stability analyses are presented in Appendix D, Figure D1.

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 west side of the existing culvert and perpendicular to the highway extending from the highway platform down to elevation 263.1 m (i.e., 4.3 m± high cut) where the new culvert will be installed.

The global stability of the 4.3 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 4.3 m± high cut will require a forward slope geometry of 3.5H:1V or flatter to achieve a target factor of safety of 1.3. 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 – σ'_p (kPa)	39
Overconsolidation Ratio	1.0
Compression Index - C_c	0.2 + 0.05H*, 0.4, and 0.4 – 0.024H*
Recompression Index - C_r	0.03 + 0.0063H*, 0.055, and 0.055 – 0.003H*,
Initial Void Ratio - e_o	0.8 + 0.15H*, 1.4, and 1.4 – 0.075H*
Coefficient of Consolidation - C_v (cm²/s)	$1.0\text{x}10^{-3}$

* 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 west 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 263.5 m on a 0.5 m± 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± 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	±5	±10	±12	±15
Estimated Settlement (mm)	5	5	5	75	175

* 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-east 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.

Culvert constructed at an offset distance from the existing culvert					
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 ±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 263.1± 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 4.3 m deep temporary excavation, a side slope geometry of 3.5H: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 ϕ (degrees)	Unit Weight γ (kN/m^3)	Active Earth Pressure Coefficient	At - Rest Earth Pressure Coefficient	Passive Earth Pressure Coefficient
			K_a	K_o	K_p
Existing Embankment Fill	29	18.5	0.35	0.52	2.88
Silty Clay to Clay	24	16.5	0.42	0.59	2.37
Silty Sand to 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[3]. 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 4.

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

7.0 CLOSURE

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

Terraprobe Inc.

LICENSED PROFESSIONAL ENGINEER
S. DAMAVANDINEJAD MONFARED
100523519
Oct. 21, 2021
PROVINCE OF ONTARIO

Sepideh D-Monfared.

Sepideh D-Monfared, P.Eng.
Geotechnical Engineer

LICENSED PROFESSIONAL ENGINEER
R.A. ABDUL
Oct. 21, 2021
PROVINCE OF ONTARIO

Rehman Abdul.

Rehman Abdul, P.Eng.
Designated MTO Foundations Contact

---

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

REFERENCES

Bowles, J.E., 1984. *Physical and Geotechnical Properties of Soils, Second Edition*. McGraw Hill Book Company, New York.

Canadian Geotechnical Society, 2006. *Canadian Foundation Engineering Manual*, 4th Edition. The Canadian Geotechnical Society c/o BiTech Publisher Ltd, British Columbia.

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6-19. 2019. Canadian Standard Association.

Chan, H. T., and Kenny, T. C., 1973, *Laboratory Investigation of Permeability Ratio of New Liskeard Varved Soil"* Canadian Geotechnical Journal, Vol. 10 pp. 453-472.

Chandler, R. J. 1988. *The In Situ Measurement of the Undrained Shear Strength of Clays Using the Field Vane, STP 1014, Vane Shear Strength Testing in Soils: Field and Laboratory Studies,* American Society for Testing and Materials.

Hough, B.K., 1957. *Basic Soils Engineering*. The Ronald Press Company, New York.

Kulhawy, F. H., and Mayne, P. W. 1990. *Manual on Estimating Soil Properties for Foundation Design, Report EL-6800*, Electric Power Research Institute Inc.

Lacasse, Suzanne M., Ladd, Charles C., and Barsvary, Allex K., 1977, *Undrained behaviour of embankments on New Liskeard varved clay,* Canadian Geotechnical Journal, Vol 14, pp 367-388.

Lo, K. Y., and Stermac, A. G., 1965, *Failure of an Embankment Founded on Varved Clay,* Canadian Geotechnical Journal, Vol. II, no. 3, pp. 234-253.

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

Nagaraj, T. S., and Murthy, B. R. S. 1985. *Prediction of the Preconsolidation Pressure and Recompression Index of Soils*. Geotechnical Testing Journal, American Society for Testing and Materials, Vol. 8, No. 4.

Newmark, N. (1965)., *"Effects of Earthquakes on Dams and Embankments"*, Geotechnique, Vol. 15, No. 2, pp. 139-160.

Quigley, R. M. and Ogunbadejo, T. A., 1972, *Clay Layer Fabric and Oedometer Consolidation of a Soft Varved Clay"* Canadian Geotechnical Journal, Vol. 9, pp. 165-175.

Rendon-Herrero, O. 1983. *Universal Compression Index*, Journal of Geotechnical Engineering, Vol. 109, No. 10

Stermac, A. G., Lo, K. G., and Barsvary, A. K., 1967, *The Performance of an Embankment on a Deep Deposit of Varved Clay,* Canadian Geotechnical Journal, Vol. IV, no. 1, pp 45-61.

Terzaghi, K., and Peck, R. B. (1967). *Soil Mechanics in Engineering Practice.* Wiley, New York.

Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification For Grading.
OPSS 422	Construction Specification For Precast Reinforced Concrete Box Culverts in Open Cut.
OPSS.PROV 501	Construction Specification For Compacting.
OPSS.PROV 510	Construction Specification For Removal.
OPSS.PROV 539	Construction Specification For Temporary Protection Systems.
OPSS 803	Construction Specification For Sodding.

OPSS.PROV 804	Construction Specification For Seed and Cover.
OPSS 805	Construction Specification For Temporary Erosion And Sediment Control Measures.
OPSS.PROV 902	Construction Specification For Excavating and Backfilling – Structures.
OPSS.PROV 1010	Material Specification for Aggregates – Base, Subbase, Select Subgrade and Backfill Material.
OPSS.PROV 1205	Material Specification For Clay Seal

Ontario Provincial Standard Drawings (OPSD)

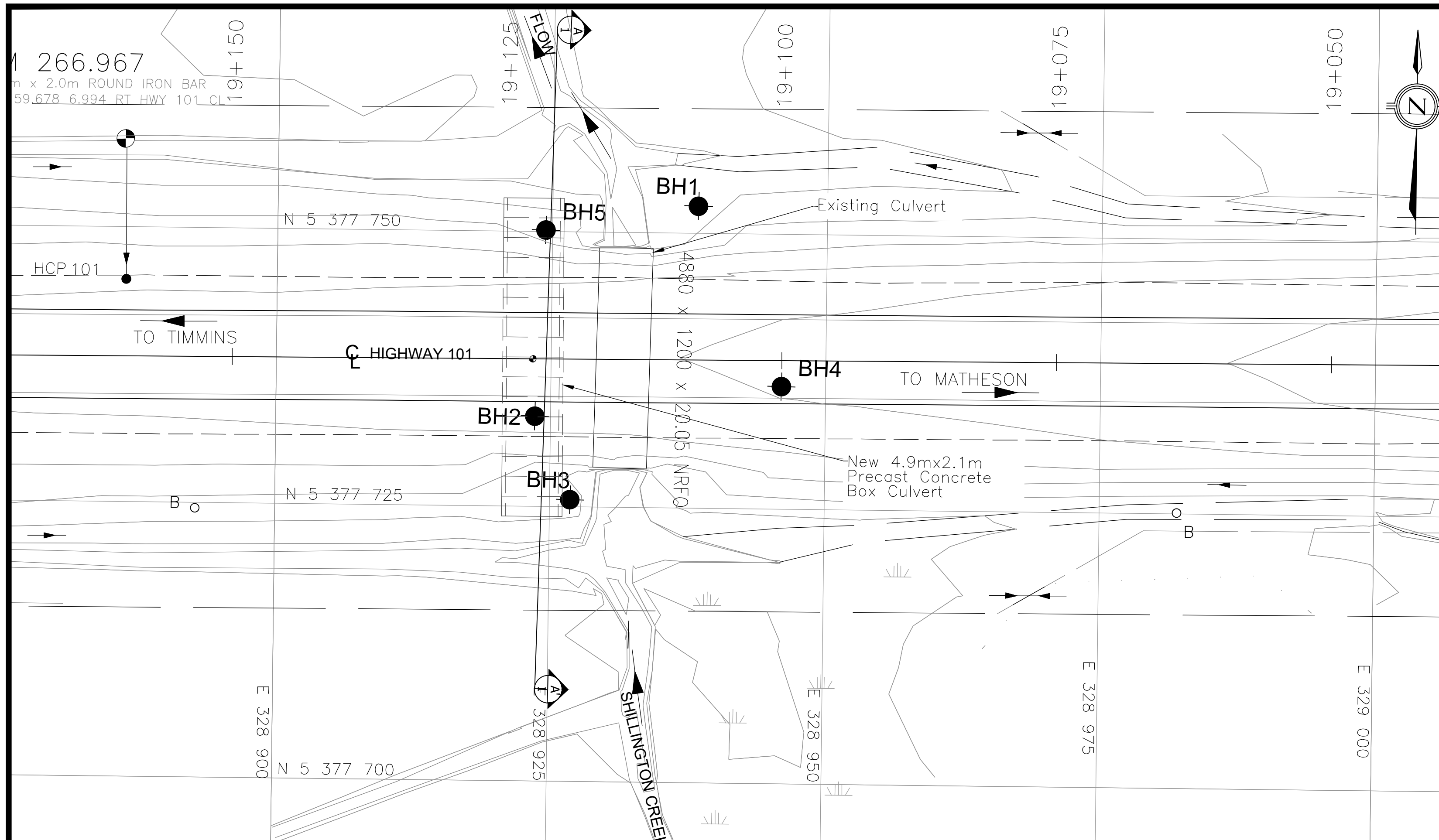
OPSD 208.010	Benching Of Earth Slopes.
OPSD 803.010	Backfill And Cover For Concrete Culverts With Spans Less Than Or Equal To 3.0 m
OPSD 3090.100	Foundation, Frost Penetration Depths For Northern Ontario

DRAWINGS

&

SITE PHOTOGRAPHS

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

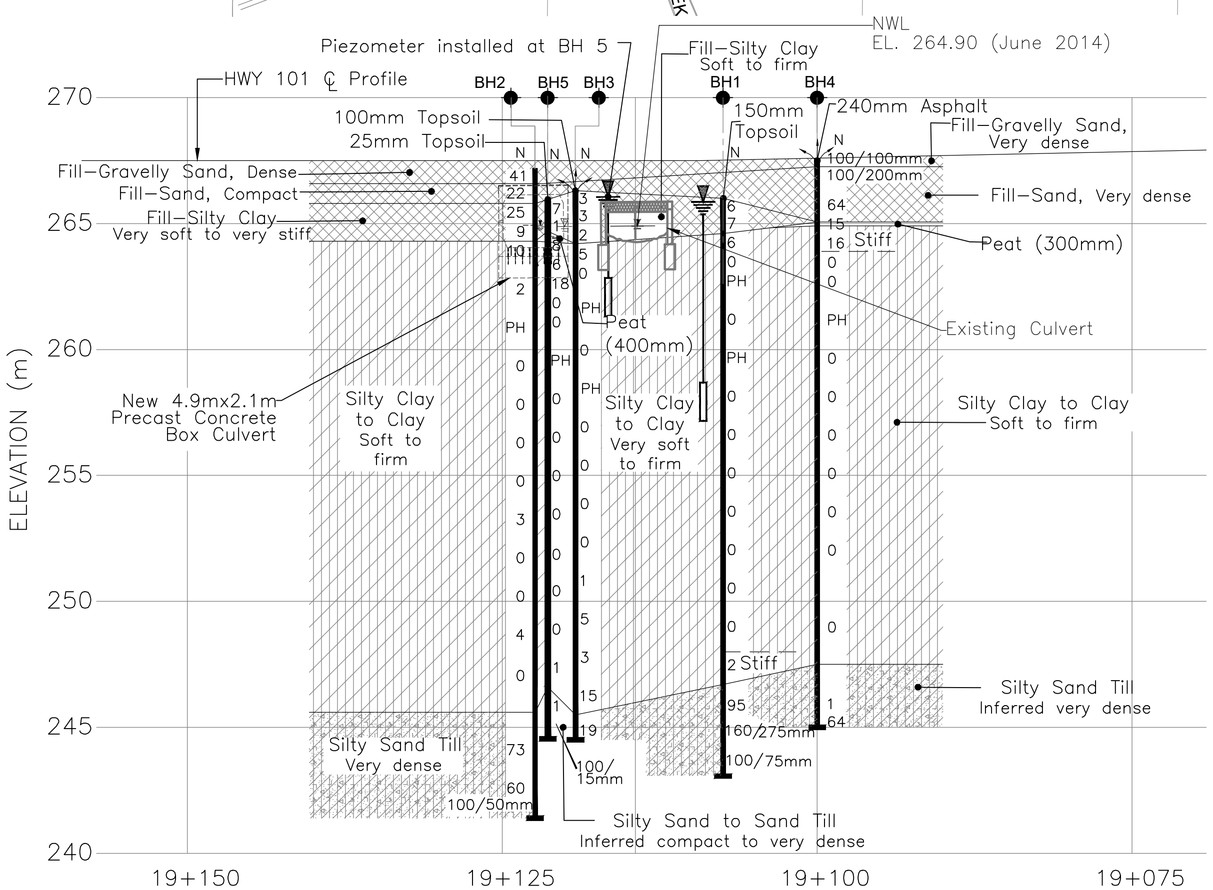


PLAN

SCALE 5 0 5 10m

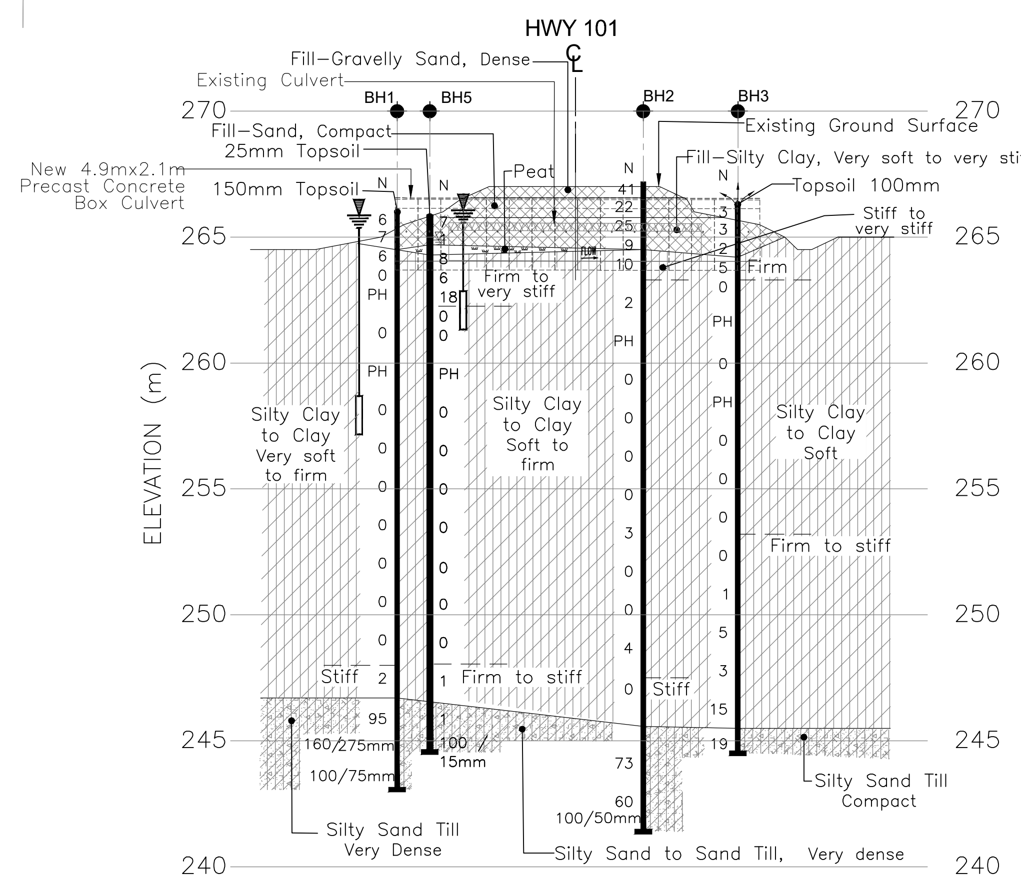
LICENSED PROFESSIONAL ENGINEER
R A ABDUL
10/21/2021
PROVINCE OF ONTARIO

LICENSED PROFESSIONAL ENGINEER
S.DAMAVANDINEJAD MONFARED
100523519
10/21/2021
PROVINCE OF ONTARIO



HIGHWAY 101 - CENTRELINE PROFILE

Horiz. Scale 5 0 5 10m
Vert. Scale 2.5 0 2.5 5m



SECTION A-A'

CONT. No.
WP No. 5166-12-01

SHILLINGTON CREEK CULVERT REPLACEMENT

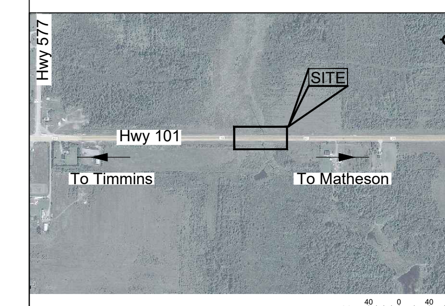
BOREHOLE LOCATION AND SOIL STRATA

SHEET 47

wsp

METRIC

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



KEY PLAN

LEGEND

- Bore Hole
- 'N' Blows/0.3m (Std Pen Test, 475 J/blow)
- WL at Time of Investigation
- WL in Piezometer (Oct. 2014 & Nov. 2018)
- Piezometer
- A/R Auger Refusal
- Artesian Condition

BH No.	ELEV. (m)	COORDINATES (MTM, ZONE 12) NORTHING (m)	EASTING (m)
1	266.0	5 377 752.5	328 938.3
2	267.2	5 377 733.2	328 923.6
3	266.3	5 377 725.6	328 926.9
4	267.5	5 377 736.2	328 946.0
5	265.9	5 377 750.1	328 924.4

NOTE

This drawing is for subsurface information only. The proposed structure details/works if shown are for illustration purposes only and may not be consistent with final design configuration as shown elsewhere in the contract documents.

The boundaries between soil strata have been established only at borehole locations. Between boreholes the boundaries are assumed from geological evidence.

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

REFERENCE

Drawing provided in digital format by WSP, drawing file Hwy 101–Hwy 577 Base Mapping.dwg received Feb 27, 2018 via email and contract drawings emailed June 11, 2021.

REVISIONS

DATE	BY	DESCRIPTION

HWY. 101	PROJECT No. 1-17-0822	GEOCRES No. 42A-141
SUBM'D. SD CHKD. SD	DATE: OCT. 2021	SITE: 39E-0214/CO
DRAWN: KC CHKD. SD	APPD: RA	DWG. 2

\\192.168.1.16\tss\1-Project Files\2017\1-17-0822 - Hwy 101, Anthony\1- Culvert Replacements,MTO\A. Dwgs, Logs\AutoCad\Shillington Creek\1-17-0822 Shillington Creek Culvert (2021-10-06).dwg, DWG To PDF.pc3, Kamal, Kamal

Hwy. 101 at Sta. 19+125 - North Side of Hwy. 101 - Looking East - July, 2019

Hwy. 101 at Sta. 19+125 - South Side of Hwy. 101 - Looking East - July, 2019

South Side of Hwy. 101 - Looking at Shillington Creek Culvert Inlet - April, 2018

North Side of Hwy. 101 - Looking at Shillington Creek Culvert Outlet - April, 2018

Terraprobe Inc. Consulting Geotechnical & Environmental Engineering Construction Materials, Inspection & Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	HWY 101 SHILLINGTON CREEK CULVERT REPLACEMENT, SITE 39E-0214/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 ú.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 5165-12-00	LOCATION Coords: E:328938.3 N:5377752.5		ORIGINATED BY S.M
DIST	HWY 101	BOREHOLE TYPE CASING AND WASH BORING	COMPILED BY S.D
DATUM GEODETIC	DATE 2014-8-13		CHECKED BY R.A

SOIL PROFILE

ELEV DEPTH (m)	DESCRIPTION
266.0	**GROUND SURFACE**
265.8 / 0.2	150mm **TOPSOIL**
	FILL, silty clay, some sand, trace gravel, containing organics, firm, dark brown, wet
264.6 / 1.4	**SILTY CLAY to CLAY** (varved), soft to firm, grey, wet
	trace sand, very soft to firm, brown, wet
	...
	...
	containing 4mm to 20mm thick silt layers

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE
1	SS	6	266
2	SS	7	265
3	SS	6	264
4	SS	0*	
5	TW	PH	263
6	SS	0*	262–261
7	TW	PH	260
8	SS	0*	259–258
9	SS	0*	257
10	SS	0*	256–255
11	SS	0*	254–253
12	SS	0*	252

GROUND WATER CONDITIONS

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values (top to bottom): 5.0, 6.0, 6.0, 4.7, 6.0, 4.0, 5.0, 2.7, 5.0, 5.6, 4.7, 3.5, 3.5, 4.0, 2.7

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

WATER CONTENT (%): 10 20 30

Water content annotations: 52, LL=41; 69; 59, LL=48; LL=46

UNIT WEIGHT γ kN/m^3

REMARKS & GRAIN SIZE DISTRIBUTION (%)

Sample	GR	SA	SI	CL
4	0	1	41	58
8	0	0	19	81
9	0	0	29	71

Continued Next Page

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

Ministry of Transportation
Ontario

Terraprobe

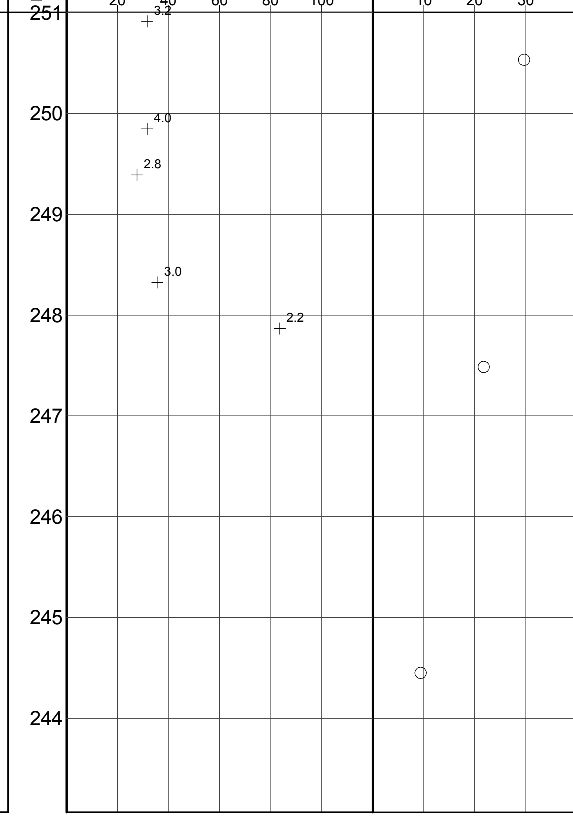
RECORD OF BOREHOLE No 1

METRIC

G.W.P.	5165-12-00	LOCATION	Coords: E:328938.3 N:5377752.5	ORIGINATED BY	S.M
DIST		HWY 101	BOREHOLE TYPE: CASING AND WASH BORING	COMPILED BY	S.D
DATUM	GEODETIC	DATE	2014-8-13	CHECKED BY	R.A

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

ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE
	(continued) **SILTY CLAY to CLAY** (varved), containing 4mm to 20mm thick silt layers, firm, grey, wet		13	SS	0*
			14	SS	0*
	... stiff, moist		15	SS	2
246.7 / 19.3	**SILTY SAND**, trace clay, trace gravel, very dense, grey, moist to wet (GLACIAL TILL)		16	SS	95
			17	SS	160 / 275mm
243.1 / 22.9	**END OF BOREHOLE**		18	SS	100 / 75mm



END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

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

Piezometer installation consists of a 25mm diameter schedule 40 PVC pipe with a 1.52m slotted screen.

Piezometer tip installed at 21.3m but was subsequently decommissioned the following day after ground water was observed flowing from the top of the piezometer. Piezometer reinstalled with tip at 8.8m.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 16, 2014	0.3	265.7
Oct 28, 2014	0.3	265.7

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

RECORD OF BOREHOLE No 2

METRIC

G.W.P.	5165-12-00	LOCATION	Coords: E:328923.6 N:5377733.2	ORIGINATED BY	S.M
DIST	HWY 101	BOREHOLE TYPE	HOLLOW STEM AUGERS	COMPILED BY	S.D
DATUM	GEODETIC	DATE	2014-7-14 - 2014-7-15	CHECKED BY	R.A

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

ELEV DEPTH (m)	DESCRIPTION
267.2	**GROUND SURFACE**
	600mm **FILL,** gravelly sand, trace silt, dense, brown, dry
266.6 / 0.6	**FILL**, sand, some gravel, trace silt, compact, brown, dry
265.8 / 1.4	**FILL**, silty clay and sand, stiff to very stiff, brown, moist
	... containing organics, trace sand, dark brown
264.3 / 2.9	trace sand, trace organics, stiff to very stiff
	...
	SILTY CLAY to CLAY (varved), containing 1mm to 20mm thick silt layers, soft to firm, grey, wet

NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%)
1	SS	41	
2	SS	22	
3	SS	25	sampler wet at 1.5m; 0 40 31 29
4	SS	9	
5	SS	10	
6	SS	2	
7	TW	PH	
8	SS	0*	
9	SS	0*	0 0 19 81
10	SS	0*	
11	SS	0*	
12	SS	3	0 0 33 67

Elevation scale: 267, 266, 265, 264, 263, 262, 261, 260, 259, 258, 257, 256, 255, 254, 253

Field vane sensitivities: 1.2, 2.0, 1.5, 1.8, 2.0, 1.5, 2.7, 1.5, 1.6, 2.3, 1.8

Water content: 63, LL=53 (sample 9); 51 (sample 12)

July 14, 2014

July 15, 2014

Continued Next Page

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 5165-12-00 | LOCATION Coords: E:328923.6 N:5377733.2 | ORIGINATED BY S.M

DIST ____ HWY 101 | BOREHOLE TYPE HOLLOW STEM AUGERS | COMPILED BY S.D

DATUM GEODETIC | DATE 2014-7-14 - 2014-7-15 | CHECKED BY R.A

ELEV DEPTH (m)	DESCRIPTION *(continued)*	NUMBER	TYPE	SPT 'N' VALUE	Field vane shear strength (kPa) / sensitivity	Water content	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	SILTY CLAY to CLAY (varved), containing 1mm to 20mm thick silt layers, soft to firm, grey, wet	13	SS	0*	1.3		
					2.0, 2.5		
		14	SS	0*	1.8, 1.6	○	
		15	SS	4	1.6, 1.9	├─┤ ○	0 1 73 26
	... stiff	16	SS	0*	1.7	○	
245.6 / 21.6	**SILTY SAND**, trace clay, trace gravel, very dense, grey, moist to wet (GLACIAL TILL)	17	SS	73			
		18	SS	60		○	2 61 29 8
241.4 / 25.8		19	SS	100 / 50mm			

Column headings: SOIL PROFILE (ELEV DEPTH (m), DESCRIPTION, STRAT PLOT); SAMPLES (NUMBER, TYPE, SPT 'N' VALUE); GROUND WATER CONDITIONS; ELEVATION SCALE (252, 251, 250, 249, 248, 247, 246, 245, 244, 243, 242); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH (kPa) (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100); PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) (10 20 30); UNIT WEIGHT γ kN/m^3; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

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

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

RECORD OF BOREHOLE No 3

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:328926.9 N:5377725.6 ORIGINATED BY S.M

DIST HWY 101 BOREHOLE TYPE CASING AND WASH BORING COMPILED BY S.D

DATUM GEODETIC DATE 2014-8-14 CHECKED BY R.A

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

ELEV DEPTH (m)	DESCRIPTION
266.3	**GROUND SURFACE**
	100mm **TOPSOIL**
	FILL, silty clay, some sand, trace gravel, trace organics, soft, brown, wet
264.2 / 2.1	**SILTY CLAY to CLAY** (varved), soft, firm to stiff below 13.1m, grey, wet; trace sand, trace gravel, trace organics, firm ... containing 2mm to 25mm thick silt layers

Number	Type	SPT 'N' Value	Water content notes	Unit weight (kN/m³)	GR SA SI CL
1	SS	3			
2	SS	3	LL=46		2 11 27 60
3	SS	2			
4	SS	5			
5	SS	0*	46		0 0 35 65
6	TW	PH	51, LL=44	16.5	0 0 33 67
7	SS	0*	73, LL=63		0 0 34 66
8	TW	PH			
9	SS	0*	60		
10	SS	0*			
11	SS	0*	63, LL=48		0 0 16 84
12	SS	0*			

Elevation scale: 266, 265, 264, 263, 262, 261, 260, 259, 258, 257, 256, 255, 254, 253, 252

Field vane sensitivity values (top to bottom): 3.0, 3.0, 8.0, 5.0, 6.0, 5.0, 10.0, 5.0, 3.3, 3.3, 4.0, 4.7, 3.0, 3.5, 4.5

file: 11-14-4066 (39e-214c) shillington creek culvert.gpj

Continued Next Page

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 3

METRIC

G.W.P. 5165-12-00	LOCATION Coords: E:328926.9 N:5377725.6		ORIGINATED BY S.M
DIST	HWY 101	BOREHOLE TYPE CASING AND WASH BORING	COMPILED BY S.D
DATUM GEODETIC	DATE 2014-8-14		CHECKED BY R.A

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

ELEV / DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	GR SA SI CL
	SILTY CLAY to CLAY (varved), containing 2mm to 25mm thick silt layers, firm to stiff, grey, wet	13	SS	1	
		14	SS	5	0 1 68 31
		15	SS	3	
		16	SS	15	
245.5 / 20.8	**SILTY SAND**, trace clay, trace gravel, compact, grey, wet (GLACIAL TILL)	17	SS	19	
244.5 / 21.8					

Elevation scale: 251, 250, 249, 248, 247, 246, 245

Field vane sensitivity values: 6.0, 3.0, 3.0, 3.7, 5.0, 3.7, 4.0

END OF BOREHOLE

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

Consolidation performed on TW6.

Hydrostatic uplift encountered at 19.8m and water flow observed about 2.1m above ground surface. Borehole terminated at 21.8m and was sealed / grouted with bentonite slurry mixture.

file: 11-14-4066 (39e-214c) shillington creek culvert.gpj

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 4

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:328946.0 N:5377736.2 ORIGINATED BY FY

DIST ____ HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD

DATUM GEODETIC DATE 2018-4-25 CHECKED BY RA

SOIL PROFILE

ELEV / DEPTH (m)	DESCRIPTION
267.5	**GROUND SURFACE**
267.3 / 0.2	240mm **ASPHALTIC CONCRETE**
266.8 / 0.7	450mm **FILL**, gravelly sand, trace to some silt, very dense, brown, dry
265.1 / 2.4	**FILL**, sand, trace gravel, containing silty clay pockets, very dense, brown, moist to wet
264.9 / 2.6	**PEAT**, amorphous, black
	very stiff, brown ... **SILTY CLAY to CLAY** (varved), trace sand, containing 2mm to 20mm thick silt layers, very soft to firm, grey, wet

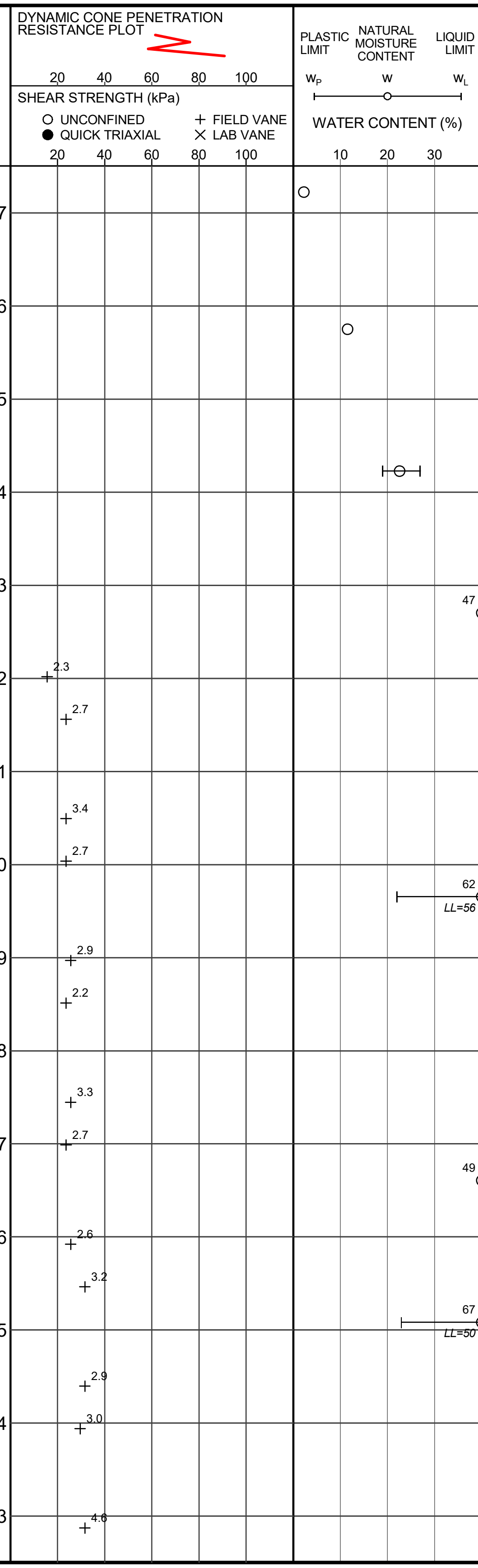
SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	100 / 100mm	resistance to augering from 0.2 m to 0.6 m and at 2.1 m
2	SS	100 / 200mm	
3	SS	64	6 64 18 12
4	SS	15	
5	SS	16	commence casing and washboring 0 1 78 21
6	SS	0*	
7	SS	0*	
8	TW	PH	
9	SS	0*	0 1 22 77
10	SS	0*	
11	SS	0*	
12	SS	0*	0 0 16 84
13	SS	0*	

GROUND WATER CONDITIONS

ELEVATION SCALE: 267, 266, 265, 264, 263, 262, 261, 260, 259, 258, 257, 256, 255, 254, 253

UNIT WEIGHT γ kN/m³



Continued Next Page

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

file: shillington creek .gpj

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 4

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:328946 N:5377736.2 ORIGINATED BY FY

DIST HWY 101 BOREHOLE TYPE SOLID STEM AUGERS / CASING AND WASH BORING COMPILED BY SD

DATUM GEODETIC DATE 2018-4-25 CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued)							
	SILTY CLAY to CLAY (varved), containing 20mm to 40mm thick silt layers, firm, grey, wet		14	SS	0*		252	0 0 47 53
							251	
							250	
			15	SS	0*		249	
247.5							248	
20.0	**SILTY SAND**, trace gravel, trace to some clay, very dense, grey, wet (GLACIAL TILL)						247	
			16	SS	1**		246	
245.0			17	SS	64		245	drilling resistance at 22.0 m
22.5								

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values plotted: 2.8, 4.0, 3.3, 3.8, 3.1

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

WATER CONTENT (%): 10 20 30

Water content value plotted: 45

UNIT WEIGHT γ kN/m^3

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

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

**Sample disturbed due to artesian conditions.

Artesian conditions encountered at 22.0 m and head of water recorded at 0.9 m above the ground upon completion of drilling. Borehole was sealed/grouted with bentonite slurry mixture.

file: shillington creek .gpj

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 5

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:328924.4 N:5377750.1 ORIGINATED BY FY

DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / CASING AND WASH BORING COMPILED BY SD

DATUM GEODETIC DATE 2018-5-3 CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	SAMPLE NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	SHEAR STRENGTH (kPa)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
265.9	**GROUND SURFACE**							
	25mm **TOPSOIL**							
	FILL, silty clay, some sand to sandy, trace gravel, trace organics, very soft to firm, brown, wet	1	SS	7				
264.7 / 1.2		2	SS	1	265			
264.3 / 1.6	**PEAT**, amorphous, black						74	
	trace organics, firm to very stiff, brown to 2.1m, grey below	3	SS	8	264			
		4	SS	6	263		42	
	...	5	SS	18	262			
	SILTY CLAY to CLAY (varved), trace sand, containing 2 mm to 40mm thick silt layers, very soft to firm, grey, wet	6	SS	0*				
		7	SS	0*	261	1.2; 2.0	68 (LL=55)	0 1 23 76
		8	TW	PH	260 / 259	2.9; 2.6		commence casing and washboring
		9	SS	0*	258 / 257	3.5; 3.5	68	
		10	SS	0*	256	4.0; 4.0		
		11	SS	0*	255 / 254	4.0; 6.0	68 (LL=56)	0 0 12 88
		12	SS	0*	253	4.0; 4.0		
		13	SS	0*	252 / 251	2.6	41	

Continued Next Page

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

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH (kPa): ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; UNIT WEIGHT γ kN/m³

file: shillington creek .gpj

RECORD OF BOREHOLE No 5

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E:328924.4 N:5377750.1 ORIGINATED BY FY
DIST HWY 101 BOREHOLE TYPE HOLLOW STEM AUGERS / CASING AND WASH BORING COMPILED BY SD
DATUM GEODETIC DATE 2018-5-3 CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	Field vane shear strength (kPa) / sensitivity	Water content	Remarks & grain size distribution (%) GR SA SI CL
	(continued) **SILTY CLAY to CLAY** (varved), containing 2 mm to 40mm thick silt layers, firm, grey, wet	14	SS	0*	3.7	w_P–w_L range; w ≈ 33	0 0 65 35
					3.0; 3.2		
	... firm to stiff, containing 60mm thick silt layers	15	SS	0*	5.2; 3.7		
		16	SS	1	3.8	w_P–w_L range; w ≈ 26	0 0 83 17
246.6 / 19.3	**SILTY SAND to SAND**, trace to some gravel, trace clay, containing cobbles, very dense, grey, wet (GLACIAL TILL)	17	SS	1**		w ≈ 18	10 70 15 5
							drilling resistance from 20.7 m to 21.3 m
244.6 / 21.3		18	SS	100 / 15mm			

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

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

**Sample disturbed due to artesian conditions.

Piezometer installation consists of a 50 mm diameter PVC pipe with a 1.5 m slotted screen.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
May 19, 2018	0.0	265.9

Elevation scale: 250, 249, 248, 247, 246, 245

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

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

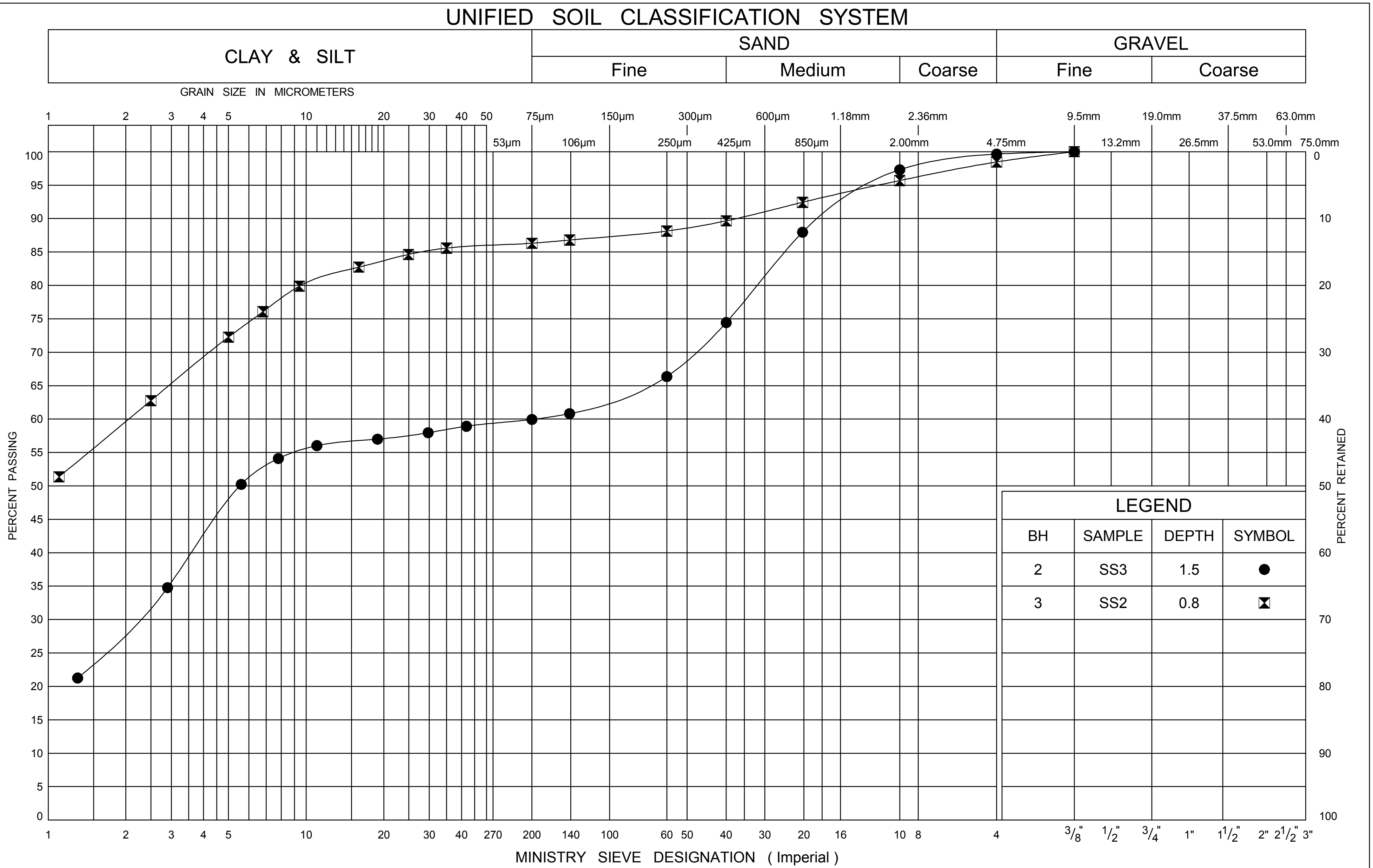
APPENDIX B

Field & Laboratory Test Results

&

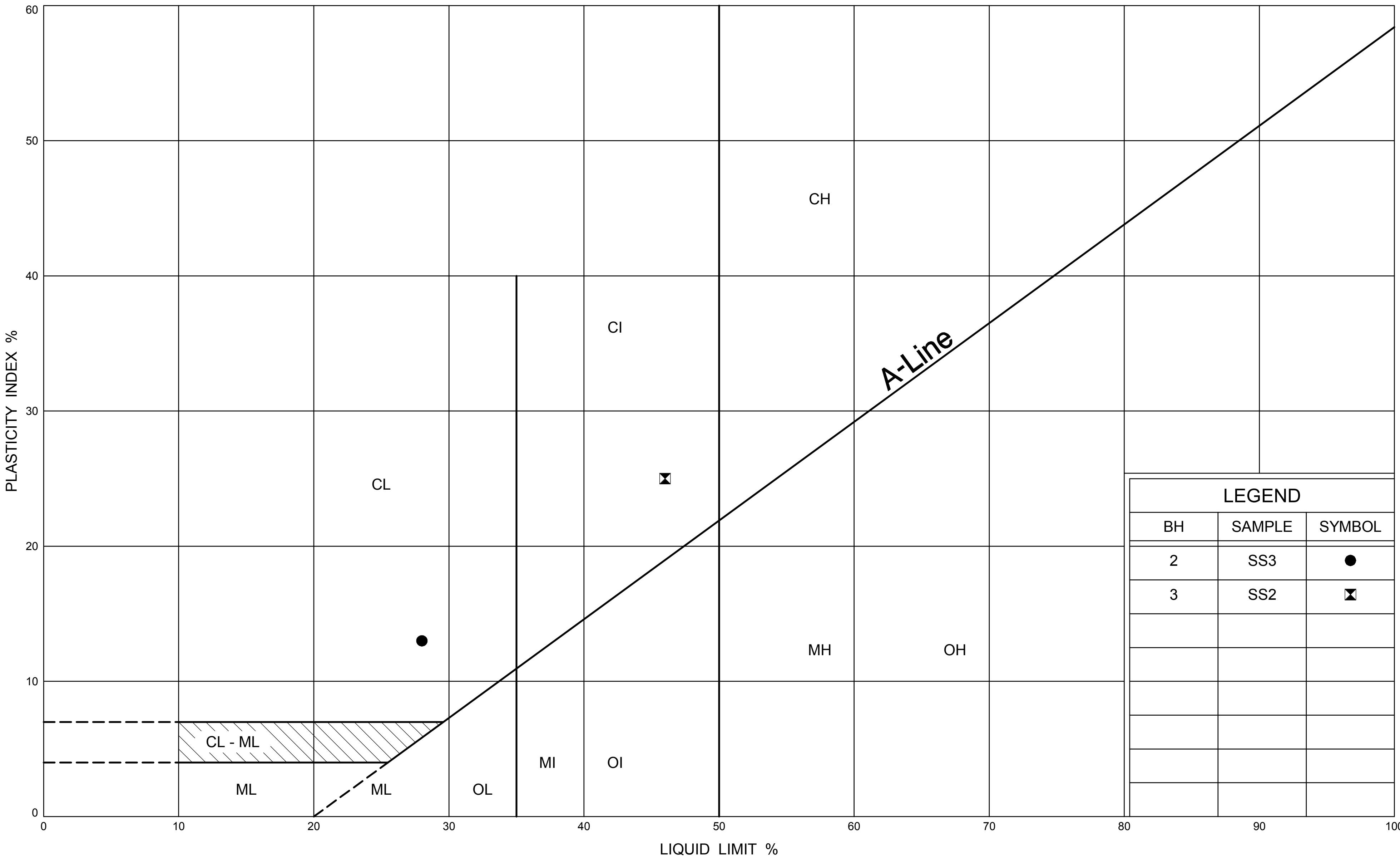
Photographs

Terraprobe



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION FILL-SILTY CLAY	FIG No B1 G W P 5165-12-00 Shillington Creek Culvert (39E-0214/C0)

Terraprobe



LEGEND

BH	SAMPLE	SYMBOL
2	SS3	●
3	SS2	⧗

Ministry of Transportation
Ontario

PLASTICITY CHART

FILL-SILTY CLAY

FIG No B2

G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

VARVED SILTY CLAY TO CLAY SAMPLES	FIGURE B3

SHILLINGTON CREEK CULVERT(Site 39E-0214/C0)

Silty Clay to Clay

BH2 SS12

BH3 SS12

Project No. : 1-17-0822

Date : December 2014

Terraprobe Inc.

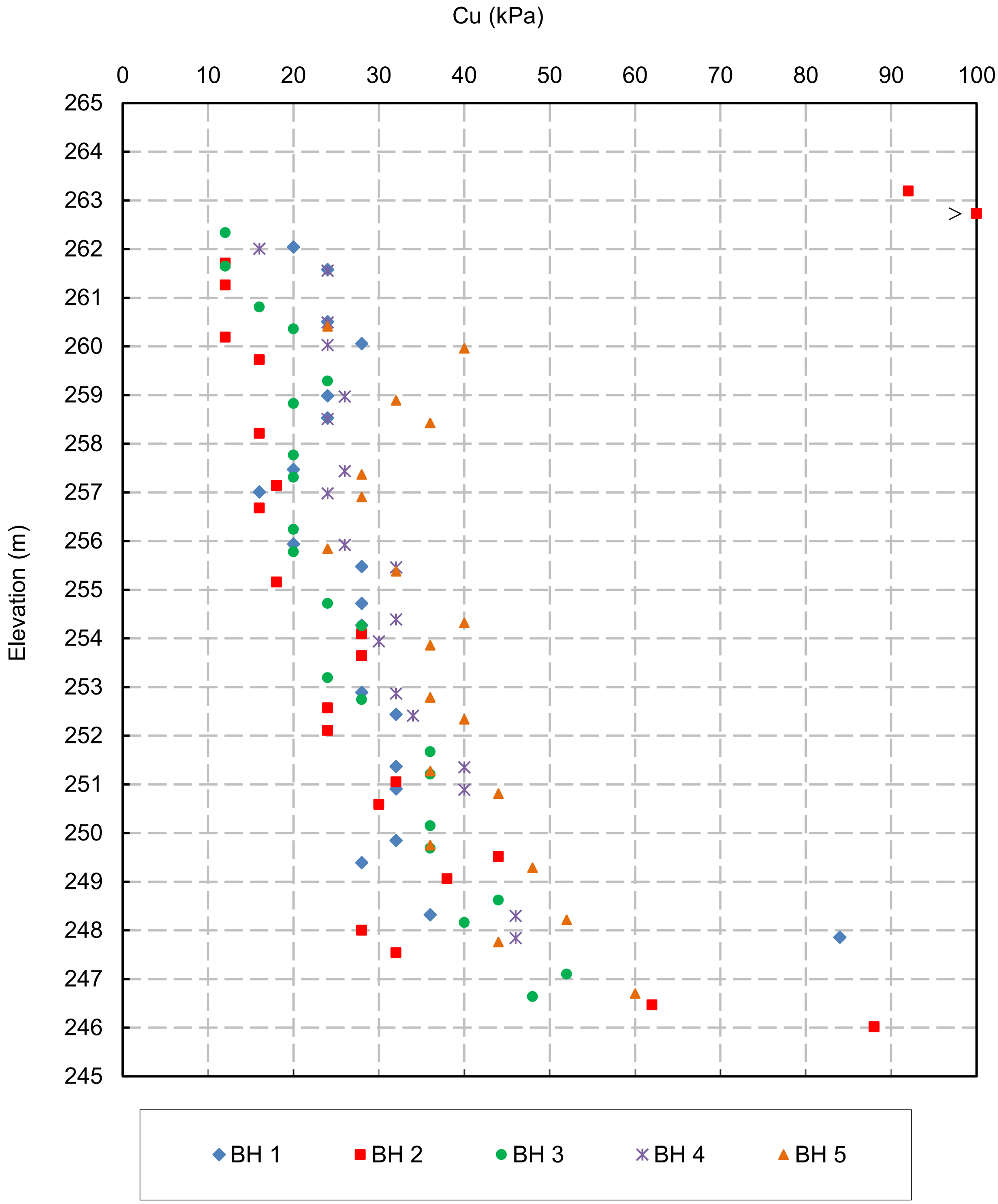
Prepared by : SD

Checked by : RA

UNDRAINED SHEAR STRENGTH	FIGURE B4

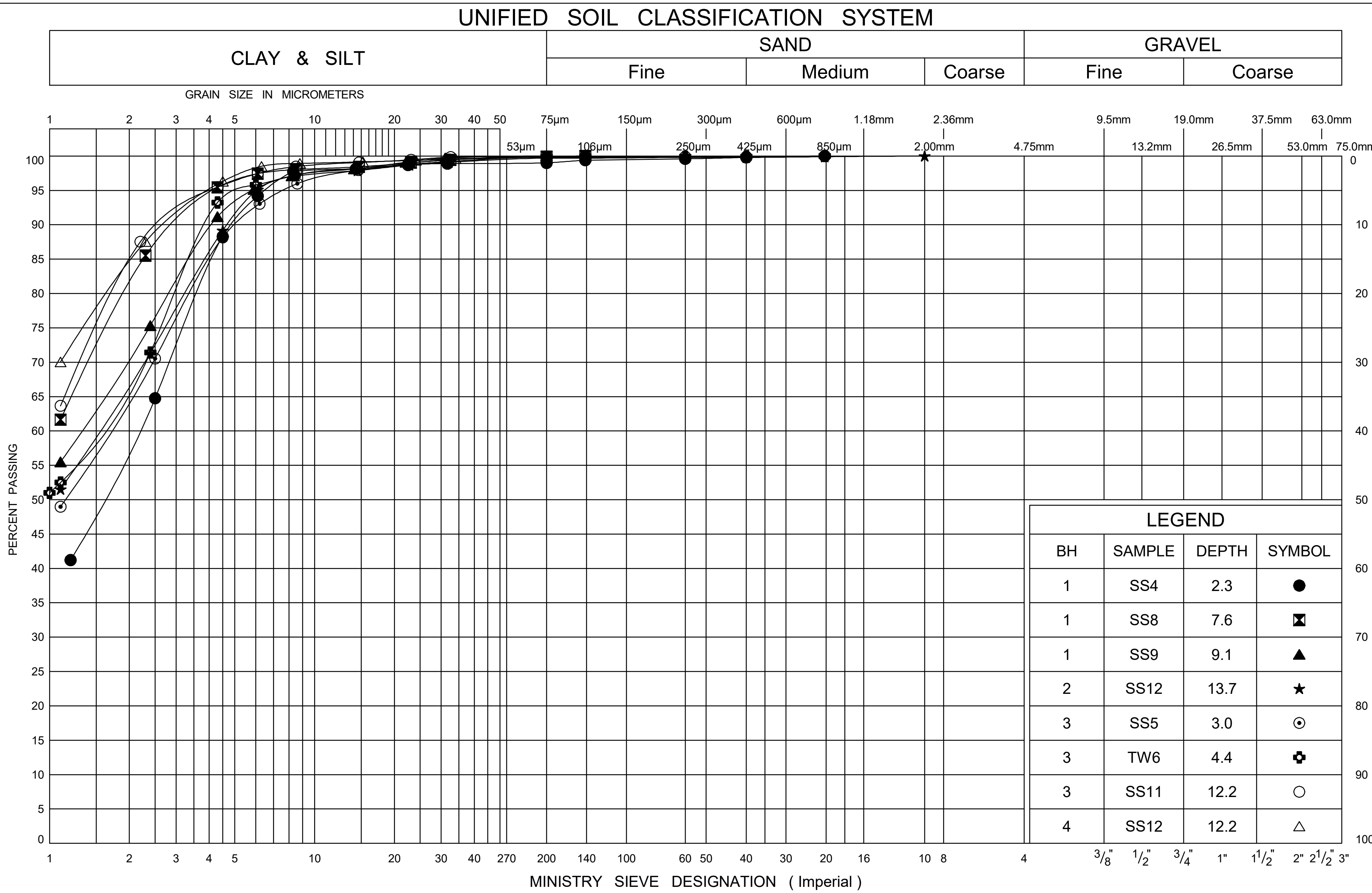
SHILLINGTON CREEK CULVERT(Site 39E-0214/C0)

Silty Clay to Clay



Project No. :	1-17-0822	Terraprobe Inc.	Prepared by :	SD
Date :	May, 2018		Checked by :	RA

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
1	SS4	2.3	●
1	SS8	7.6	⊠
1	SS9	9.1	▲
2	SS12	13.7	★
3	SS5	3.0	⊙
3	TW6	4.4	✿
3	SS11	12.2	○
4	SS12	12.2	△

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

FIG No B5

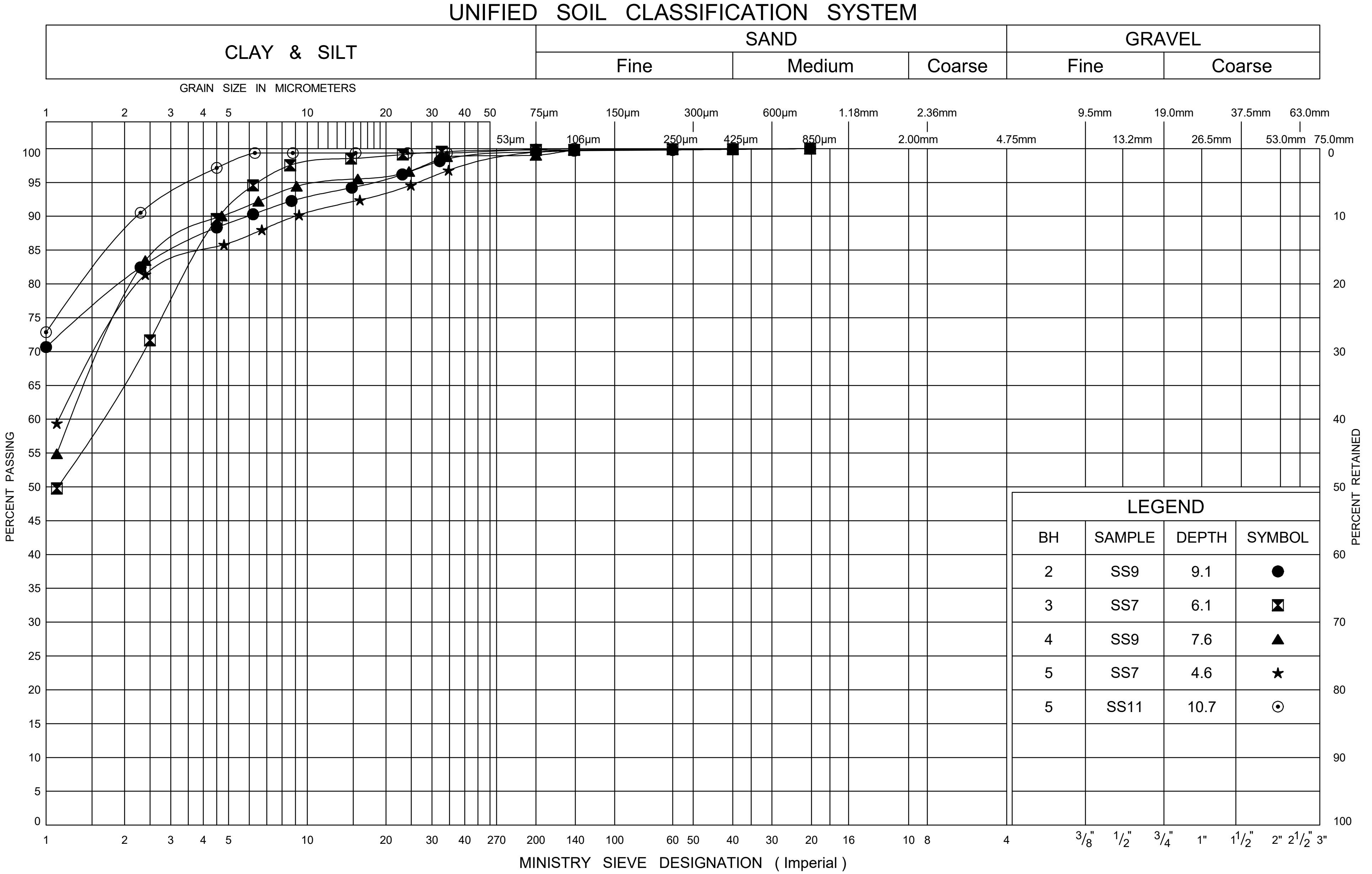
G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

file: shillington creek - combined.gpj

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
2	SS9	9.1	●
3	SS7	6.1	⊠
4	SS9	7.6	▲
5	SS7	4.6	★
5	SS11	10.7	⊙

Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY TO CLAY

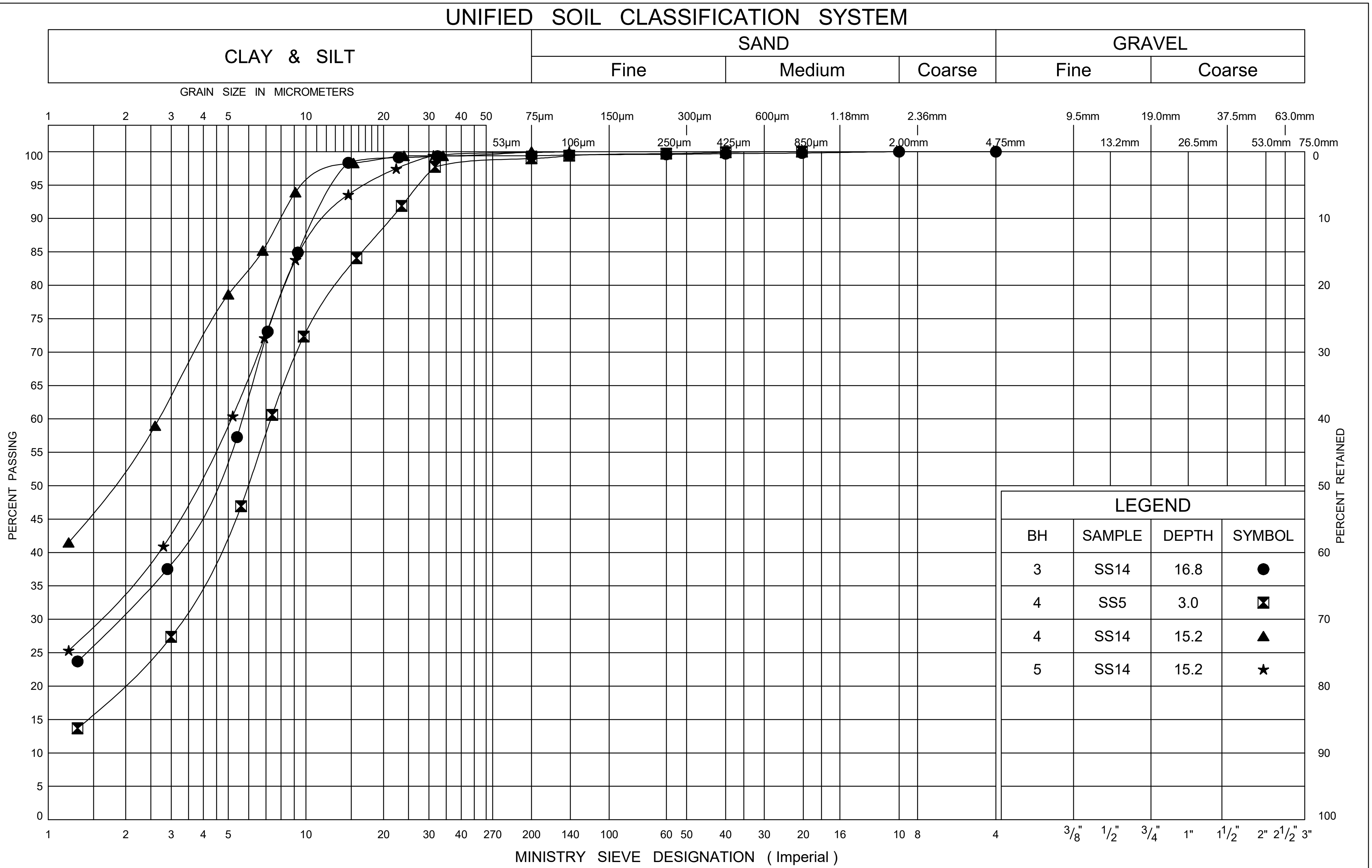
FIG No B6

G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

file: shillington creek - combined.gpj

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
3	SS14	16.8	●
4	SS5	3.0	⊠
4	SS14	15.2	▲
5	SS14	15.2	★

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILTY CLAY TO CLAY

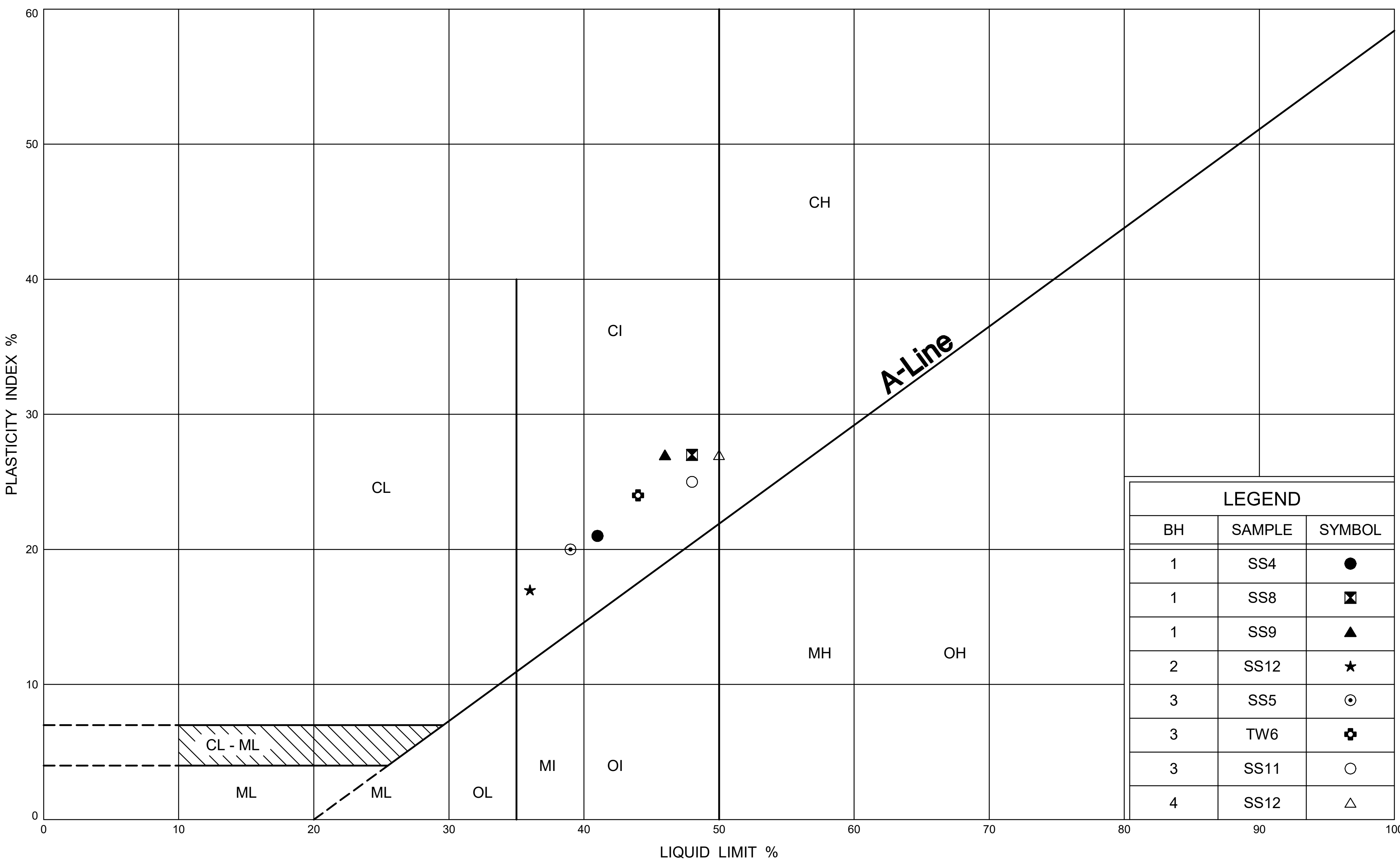
FIG No B7

G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

file: shillington creek - combined.gpj

Terraprobe



LEGEND

BH	SAMPLE	SYMBOL
1	SS4	●
1	SS8	⧗
1	SS9	▲
2	SS12	★
3	SS5	⊙
3	TW6	✤
3	SS11	○
4	SS12	△

Ministry of Transportation
Ontario

PLASTICITY CHART

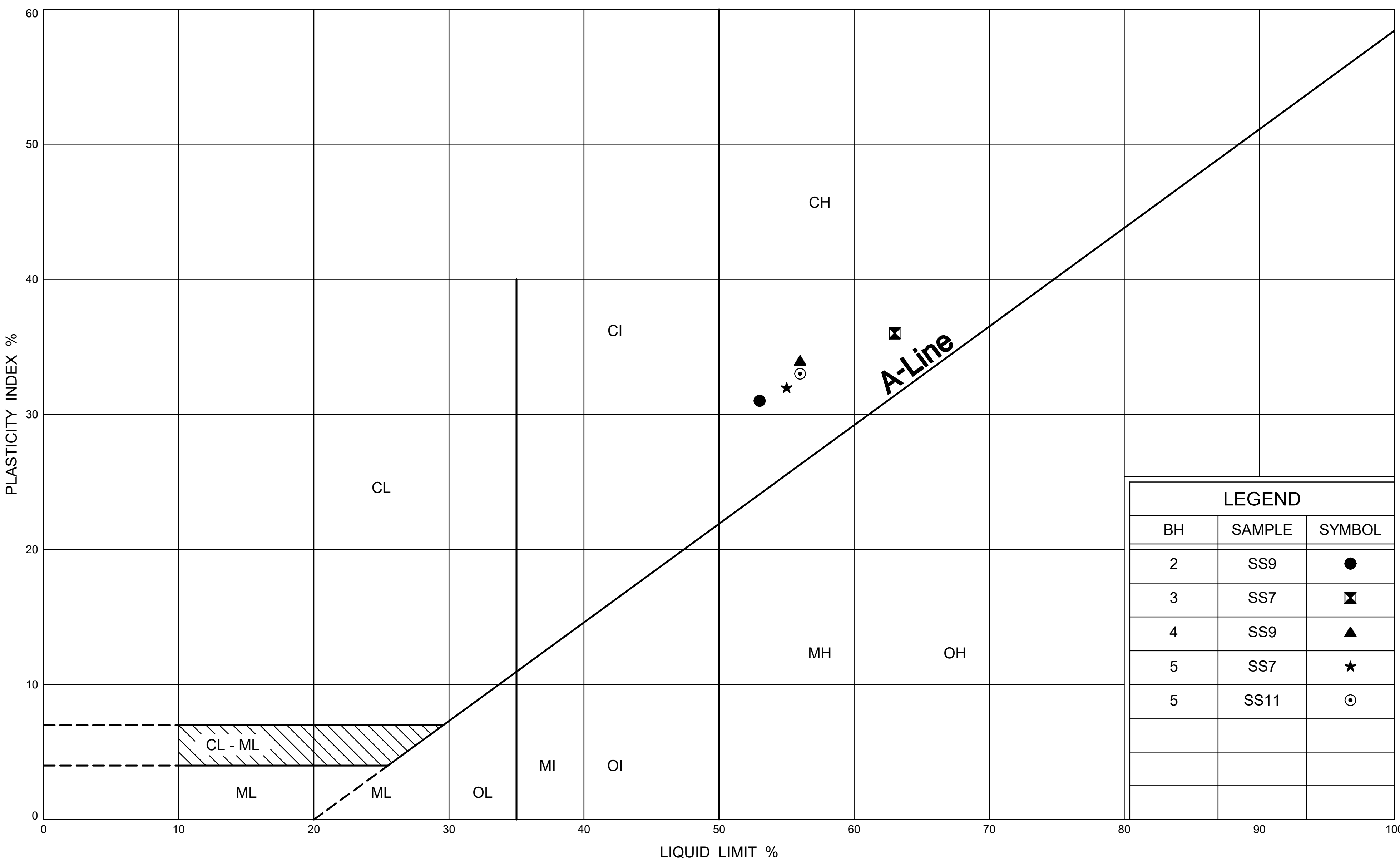
SILTY CLAY TO CLAY

FIG No B8

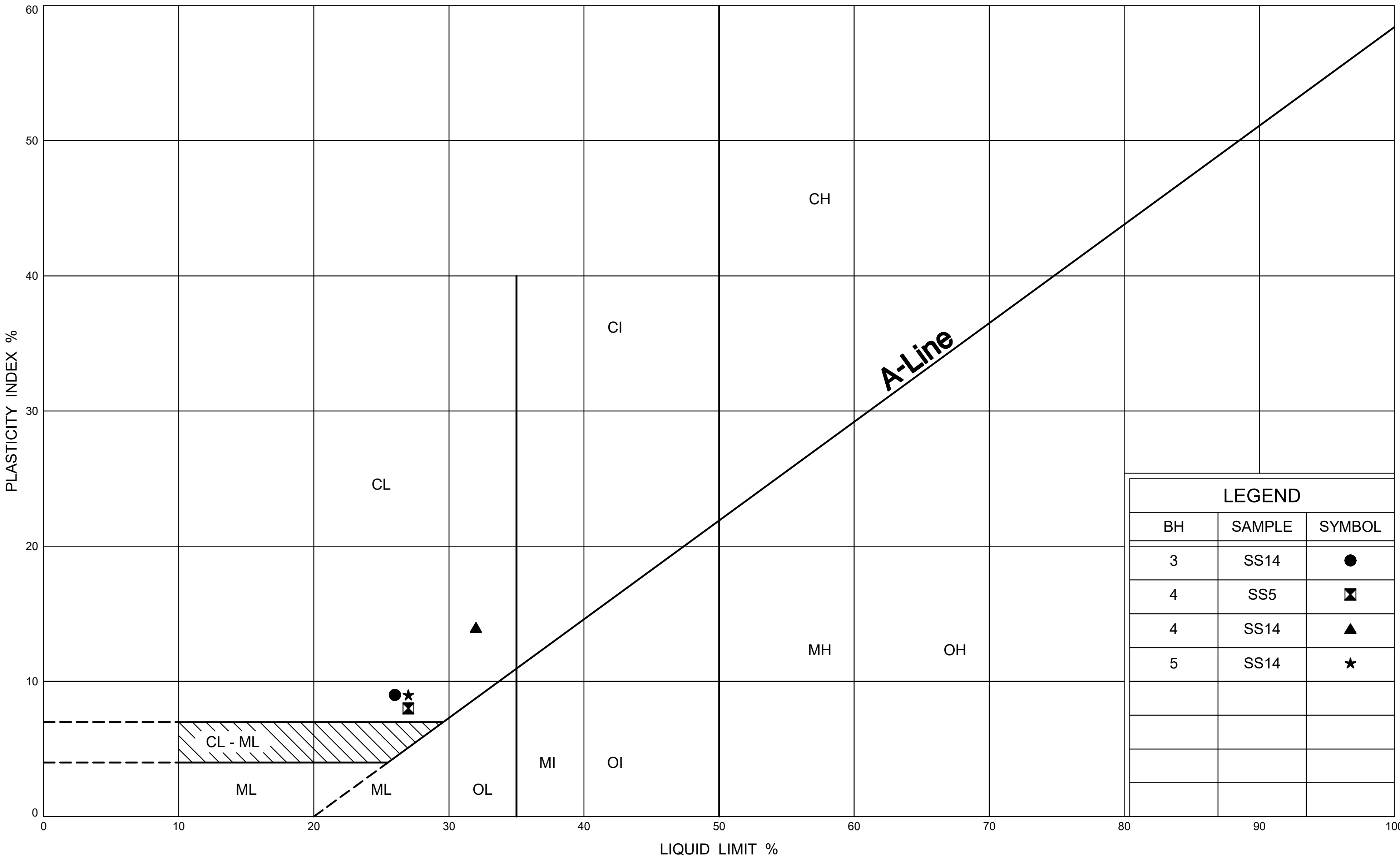
G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

file: shillington creek - combined.gpj



Ministry of Transportation Ontario	PLASTICITY CHART SILTY CLAY TO CLAY	FIG No B9 G W P 5165-12-00 Shillington Creek Culvert (39E-0214/C0)



LEGEND

BH	SAMPLE	SYMBOL
3	SS14	●
4	SS5	⧗
4	SS14	▲
5	SS14	★

Ministry of Transportation
Ontario

PLASTICITY CHART
SILTY CLAY TO CLAY

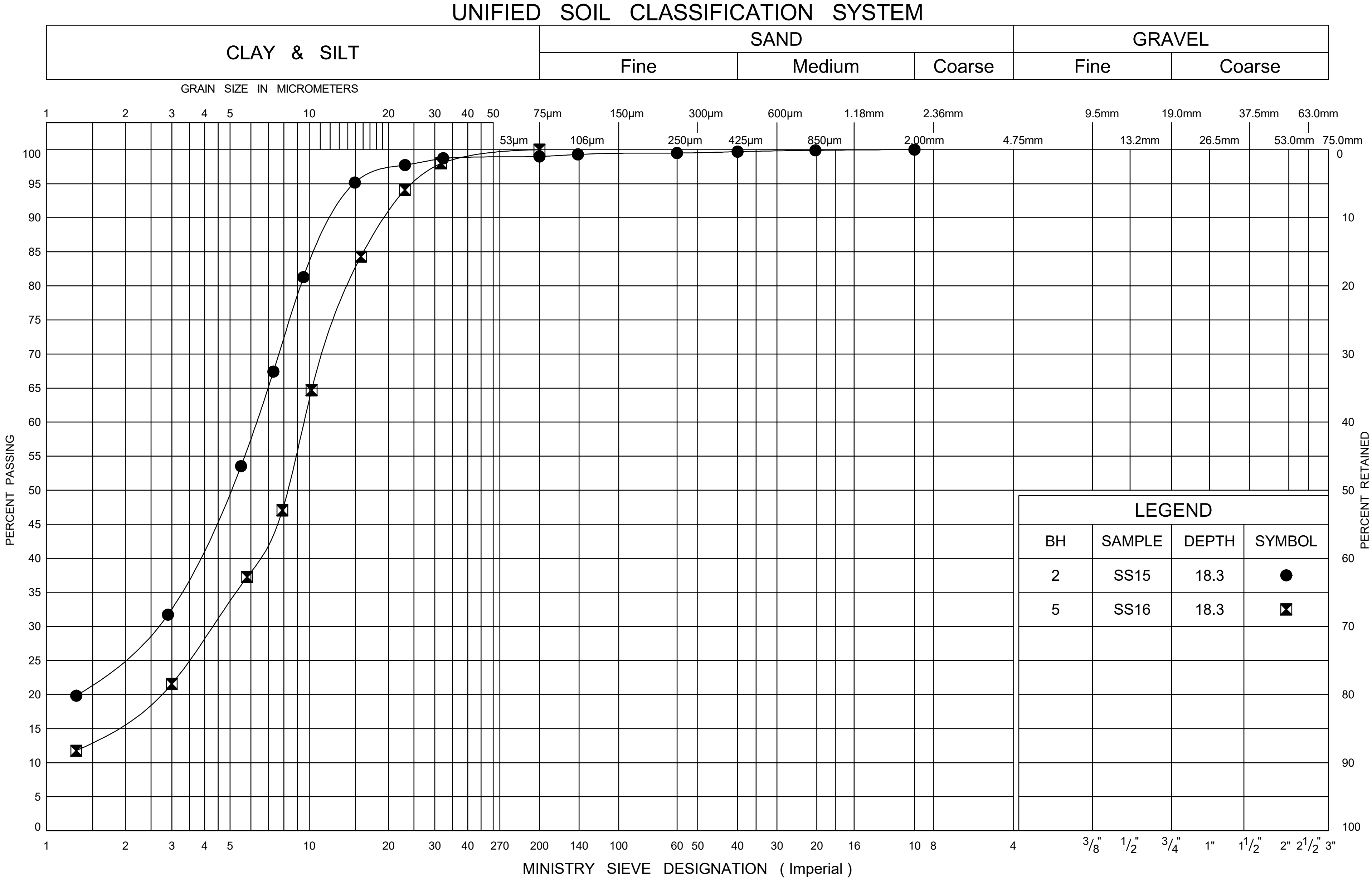
FIG No B10

G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

file: shillington creek - combined.gpj

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
2	SS15	18.3	●
5	SS16	18.3	⧗

Ministry of Transportation
Ontario

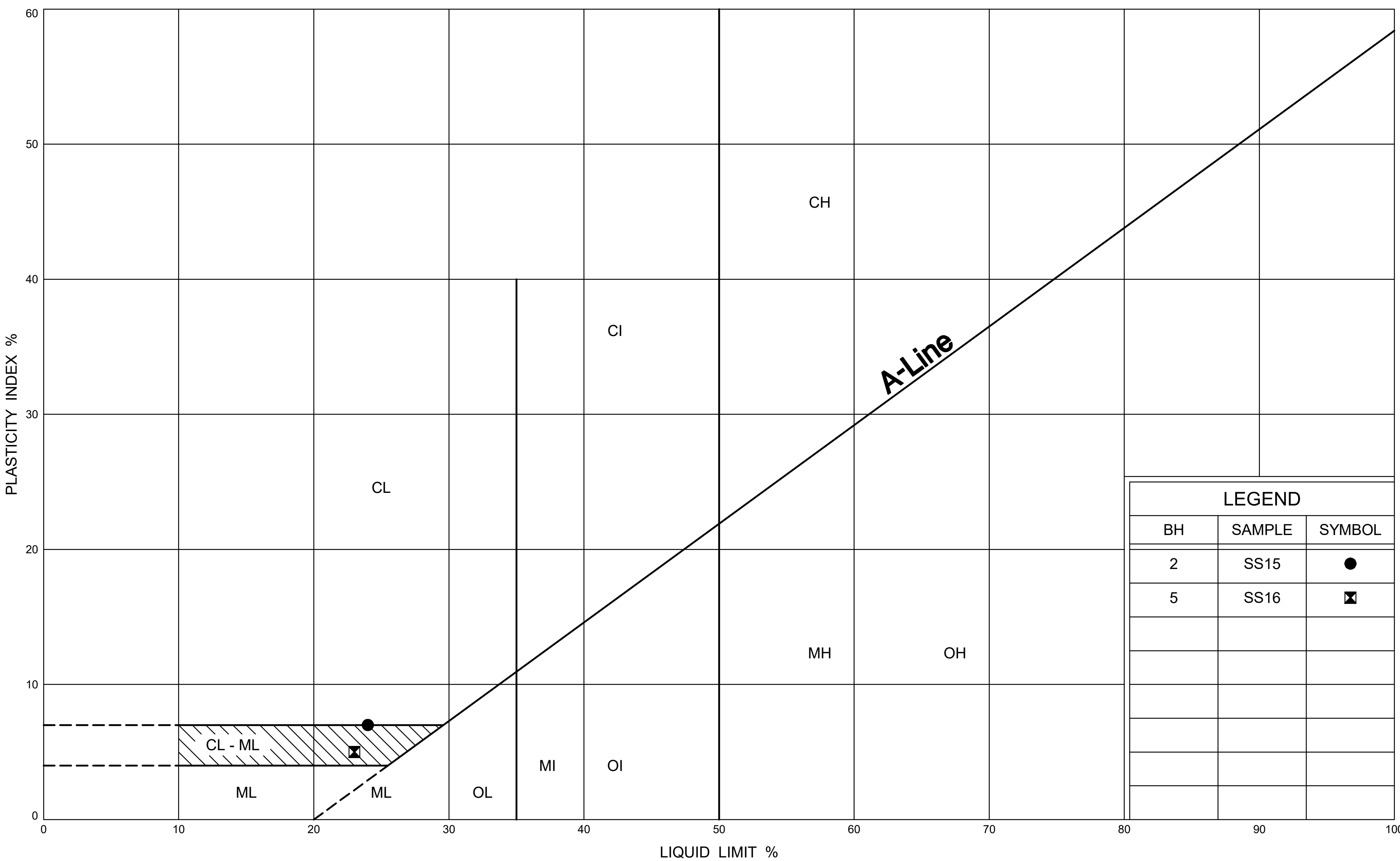
GRAIN SIZE DISTRIBUTION
CLAYEY SILT

FIG No B11

G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

Terraprobe



Ministry of Transportation
Ontario

PLASTICITY CHART

CLAYEY SILT

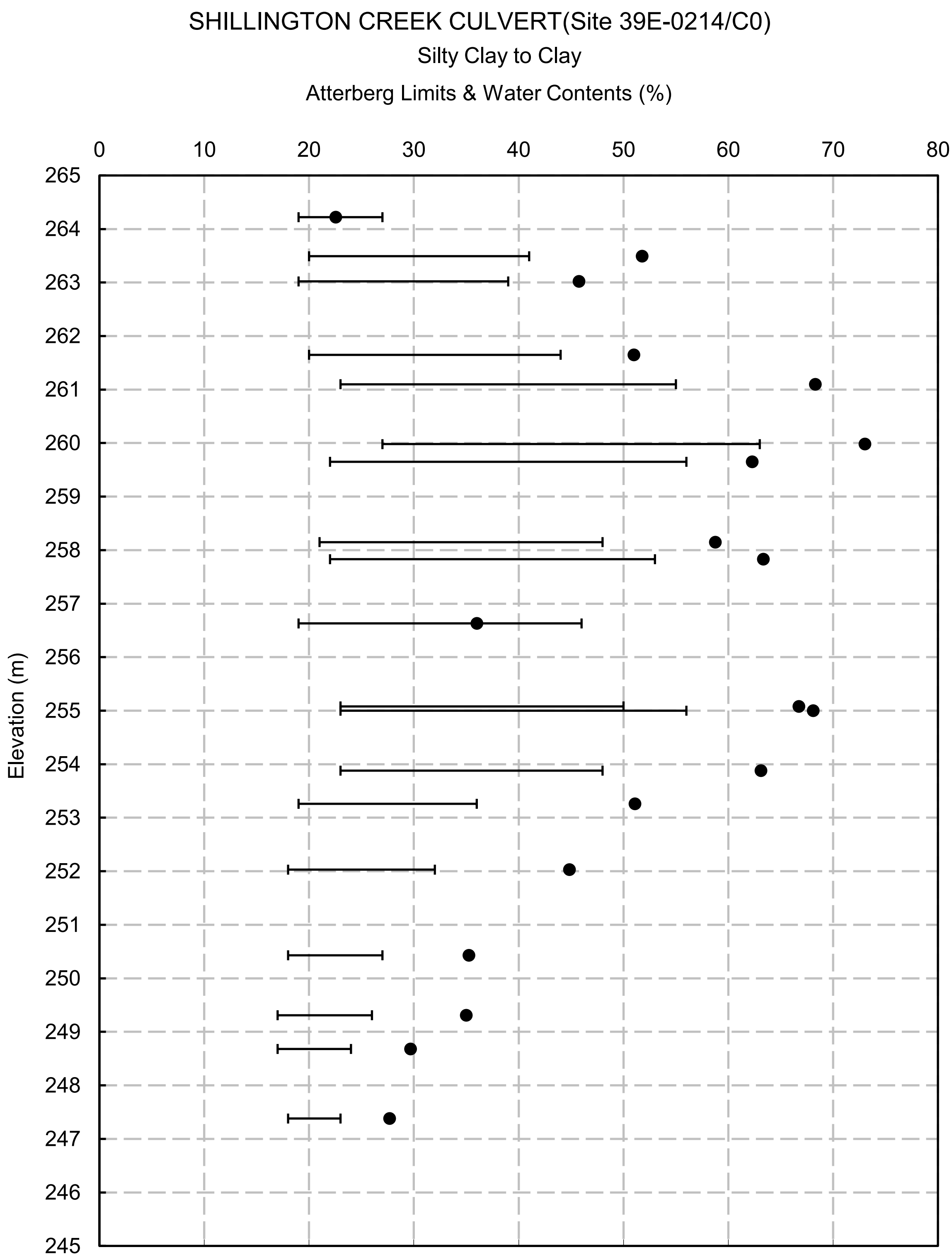
FIG No B12

G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

file: shillington creek - combined.gpj

ATTERBERG LIMITS AND WATER CONTENTS	FIGURE B13



Project No. : 1-17-0822

Date : May, 2018

Terraprobe Inc.

Prepared by : SD

Checked by : RA

CONSOLIDATION TEST SUMMARY	FIGURE B14

SAMPLE IDENTIFICATION

Borehole No. :	3	Sample No. :	TW6
		Sample Depth (m) :	4.6 - 5.0

TEST CONDITIONS

Test Type :	Laboratory Standard	Date Started :	17-Oct-14
Load Duration (hr) :	24	Date Completed :	30-Oct-14

SAMPLE DIMENSIONS AND PROPERTIES _ INITIAL

Sample Height (mm) :	25.27	Unit Weight (kN/m^3) :	16.82
Sample Diameter (mm)	63.35	Dry Unit Weight (kN/m^3) :	11.15
Area (cm^2) :	31.52	Specific Gravity :	2.70
Volume (cm^3) :	79.65	Solid Height (mm) :	10.65
Water Content (%) :	50.80	Volume of Solids (cm^3) :	33.56
Wet Mass (g) :	136.64	Volume of Voids (cm^3) :	46.10
Dry Mass (g) :	90.60	Degree of Saturation (%) :	99.88

TEST COMPUTATIONS

Stress (kPa)	Initial Height (mm)	Final Height (mm)	Void Rati	t_{90} (min)	C_v (cm^2/s)	m_v (m^2/kN)	k (cm/s)
1.2	25.27	25.27	1.37				
18.4	25.27	24.88	1.34	5.06	4.33E-03	8.98E-04	3.80E-07
35.6	24.88	24.41	1.29	52.56	4.00E-04	1.10E-03	4.30E-08
69.9	24.41	23.49	1.21	81	2.40E-04	1.10E-03	2.60E-08
138.7	23.49	22.33	1.10	68.06	2.60E-04	7.15E-04	1.80E-08
276.1	22.33	21.27	1.00	36	4.50E-04	3.48E-04	1.50E-08
551.0	21.27	20.34	0.91	27.56	5.40E-04	1.59E-04	8.30E-09
1100.7	20.34	19.48	0.83	14.44	9.40E-04	7.70E-05	7.10E-09
2200.3	19.48	18.69	0.76	9	1.38E-03	3.70E-05	5.00E-09
276.1	18.69	18.91	0.78				
69.9	18.91	19.25	0.81				
18.4	19.25	19.65	0.846				

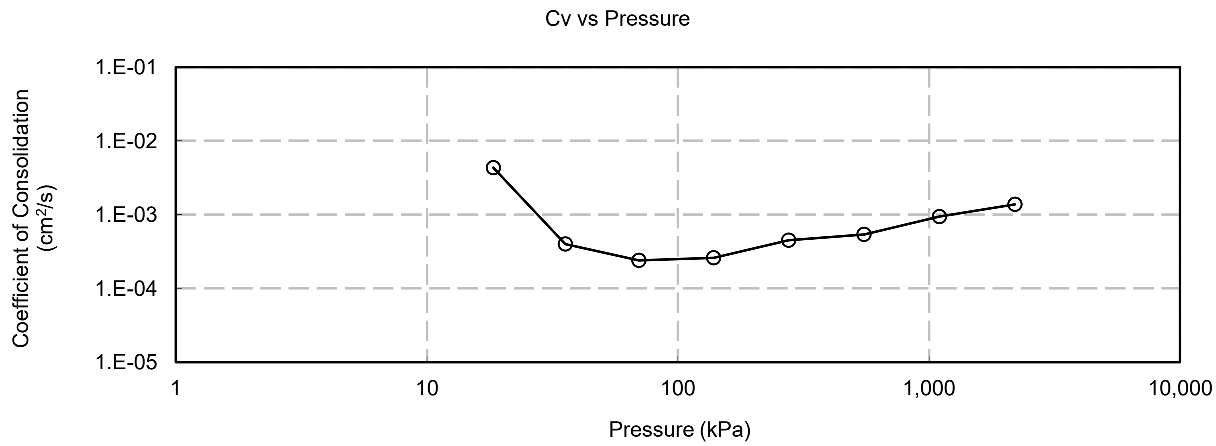
SAMPLE DIMENSIONS AND PROPERTIES _ FINAL

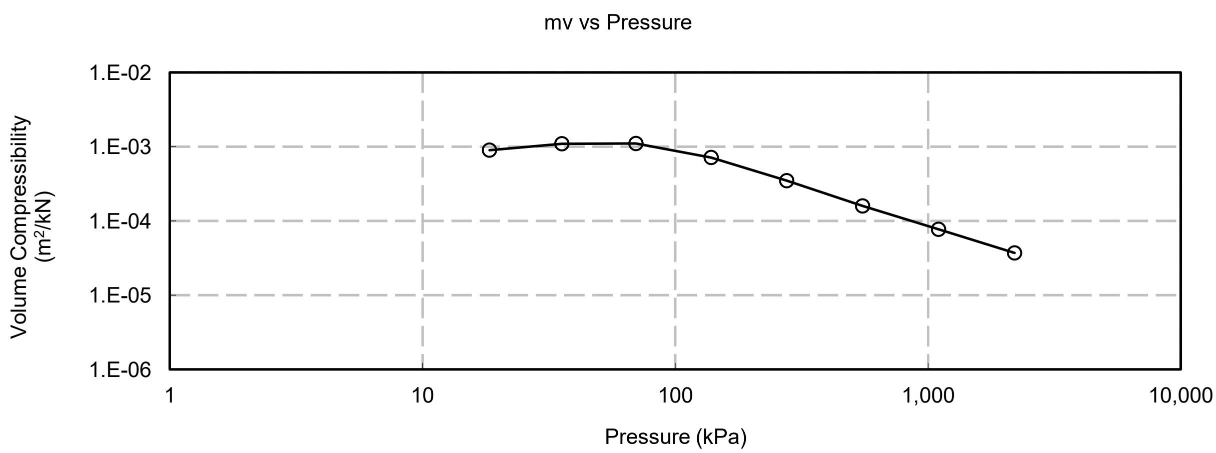
Sample Height (mm) :	19.65	Unit Weight (kN/m^3) :	18.29
Sample Diameter (mm)	63.35	Dry Unit Weight (kN/m^3) :	14.21
Area (cm^2) :	31.52	Specific Gravity :	2.70
Volume (cm^3) :	61.94	Solid Height (mm) :	10.65
Water Content (%) :	28.70	Volume of Solids (cm^3) :	33.24
Wet Mass (g) :	115.54	Volume of Voids (cm^3) :	28.70
Dry Mass (g) :	89.74		

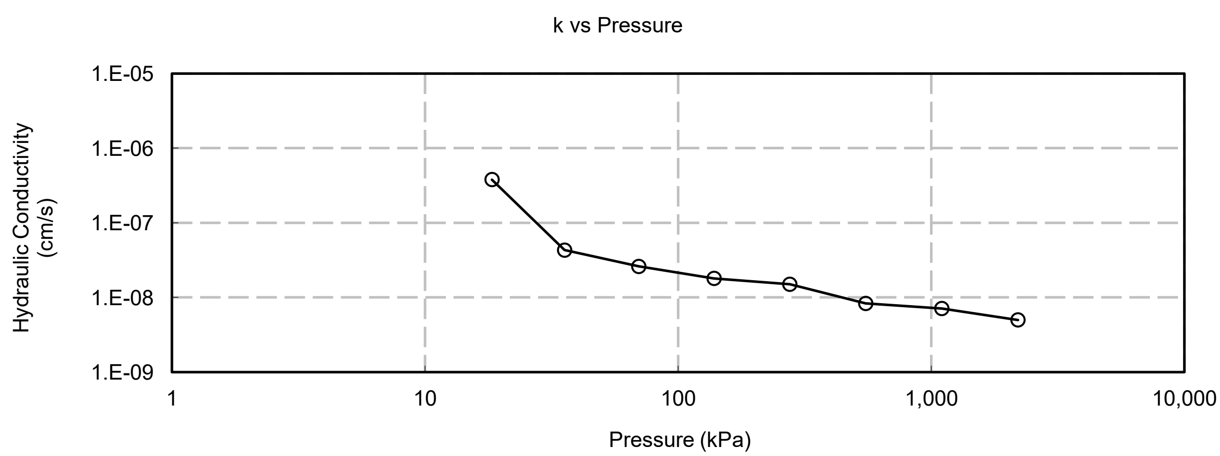
Project No. :	1-17-0822	Terraprobe Inc.	Prepared By :	SD
Date :	December 2014		Checked By :	RA

CONSOLIDATION TEST	FIGURE B15

SHILLINGTON CREEK CULVERT(Site 39E-0214/C0)
BH 3, TW 6





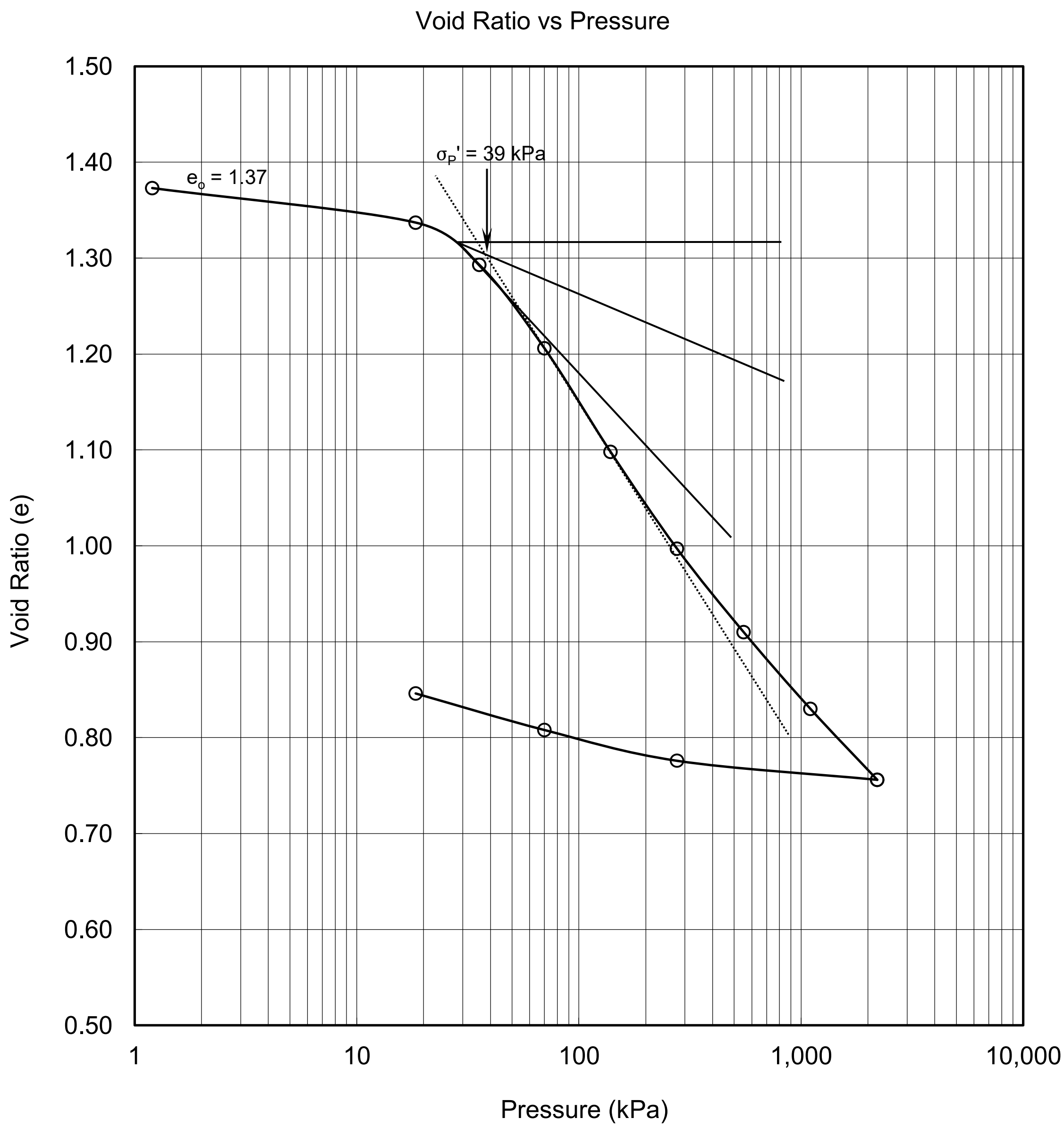


Project No. :	1-17-0822	Terraprobe Inc.	Prepared By :	SD
Date :	December 2014		Checked By :	RA

CONSOLIDATION TEST	FIGURE B16

SHILLINGTON CREEK CULVERT(Site 39E-0214/C0)

BH 3, TW 6



Soil Type : SILTY CLAY to CLAY

e_o =	1.37	ω_L =	44%	σ_{v0}' =	36.7 kPa
ω =	51%	ω_P =	20%	σ_P' =	39.0 kPa
γ =	16.8 kN/m^3	PI =	24%		
Gs =	2.70				

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

Terraprobe Inc.

Prepared By : SD
Checked By : RA

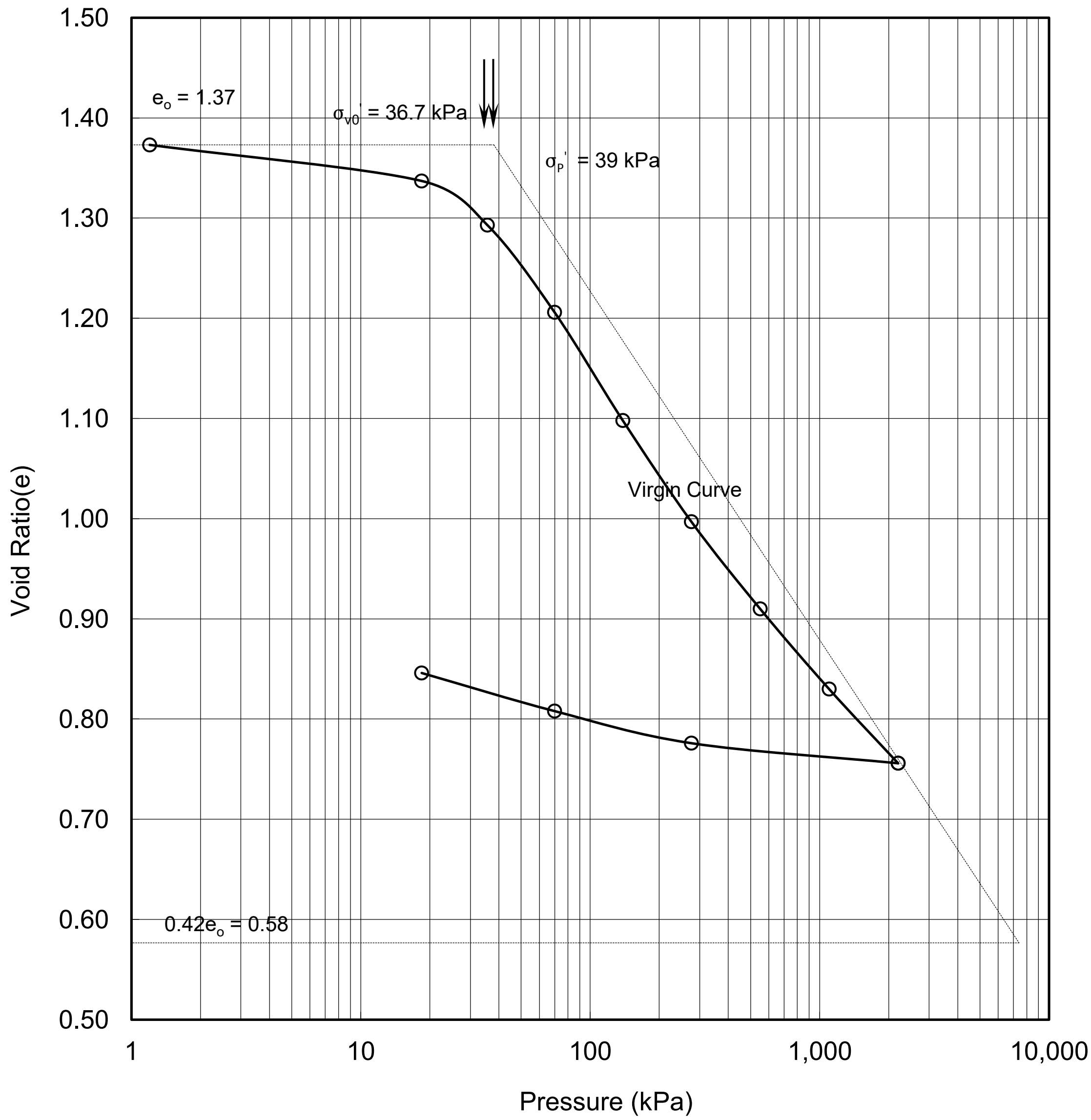
Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\5- Shillington Creek Culvert Hwy 101 (39E-214)\Eng Analysis\Spread Sheets\Consolidation Results.xls

CONSOLIDATION TEST	FIGURE B17

SHILLINGTON CREEK CULVERT(Site 39E-0214/C0)

BH 3, TW 6

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

e_o =	1.37	ω_L =	44%	σ_{v0}' =	36.7 kPa
ω =	51%	ω_P =	20%	σ_P' =	39.0 kPa
γ =	16.8 kN/m^3	PI =	24%	C_c =	0.348
Gs =	2.70			C_r =	0.043

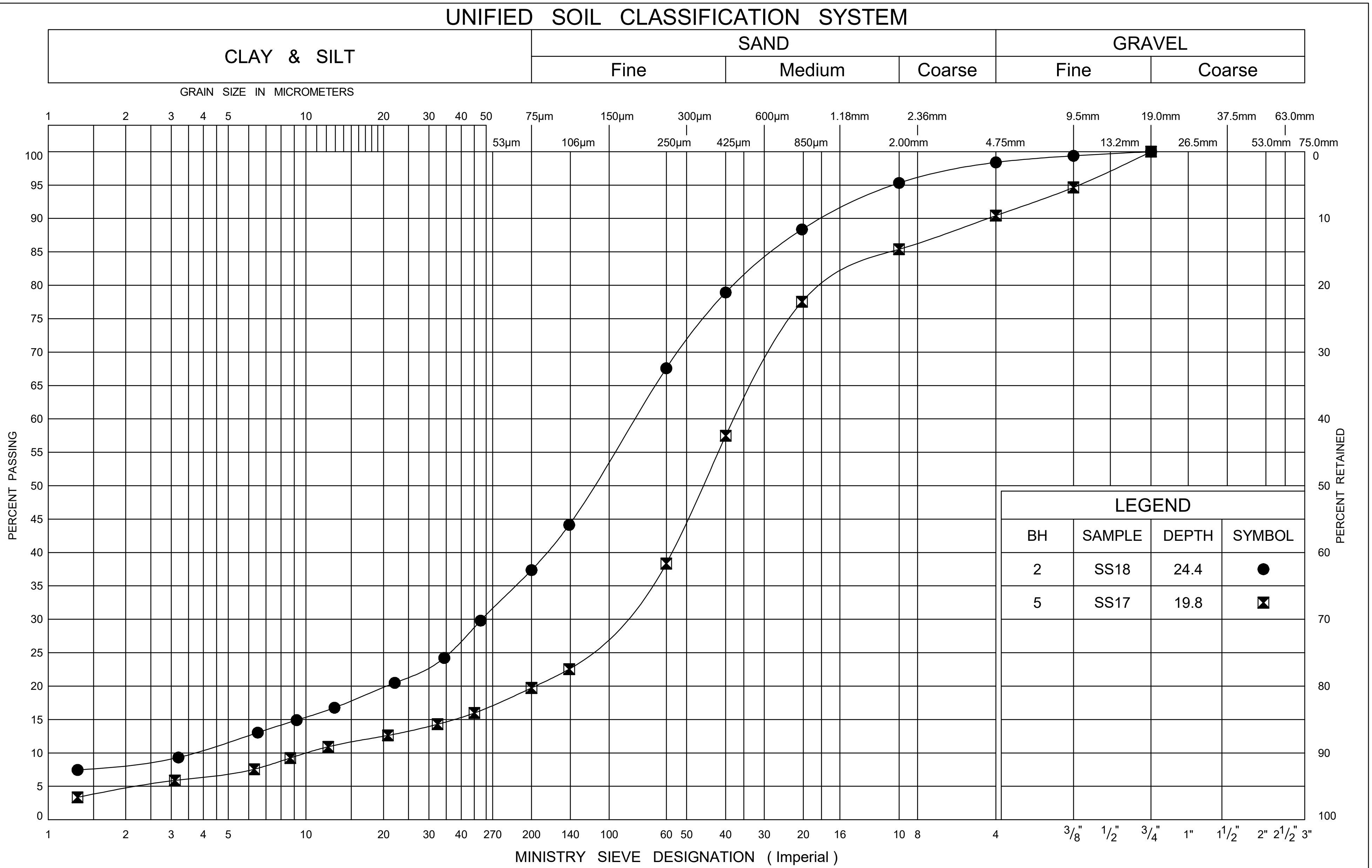
Project No. : 1-17-0822
Date : December 2014

Terraprobe Inc.

Prepared By : SD
Checked By : RA

Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likeard Area\5- Shillington Creek Culvert Hwy 101 (39E-214)\Eng Analysis\Spread Sheets\Consolidation Results.xls

78 12 M



BH	SAMPLE	DEPTH	SYMBOL
2	SS18	24.4	●
5	SS17	19.8	⧗

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILTY SAND TO SAND TILL

FIG No B18

G W P 5165-12-00

Shillington Creek Culvert (39E-0214/C0)

file: shillington creek - combined.gpj

FINAL REPORT

CA14435-MAY18 R1

1-17-0822 Shilington Creek

Prepared for

Terraprobe Inc

FINAL REPORT

First Page

CLIENT DETAILS		LABORATORY DETAILS	
Client	Terraprobe Inc	Project Specialist	Brad Moore Hon. B.Sc
		Laboratory	SGS Canada Inc.
Address	11 Indell Lane Brampton, ON L6T 3Y3, Canada	Address	185 Concession St., Lakefield ON, K0L 2H0
Contact	Sepideh D_Monfared	Telephone	705-652-2143
Telephone	(905) 796-2650	Facsimile	705-652-6365
Facsimile	(905) 796-2250	Email	brad.moore@sgs.com
Email	smonfared@terraprobe.ca	SGS Reference	CA14435-MAY18
Project	1-17-0822 Shilington Creek	Received	05/15/2018
Order Number		Approved	05/22/2018
Samples	Soil (1)	Report Number	CA14435-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

TABLE OF CONTENTS

FINAL REPORT

CA14435-MAY18 R1

Client: Terraprobe Inc

Project: 1-17-0822 Shilington Creek

Project Manager: Sepideh D_Monfared

Samplers: Sepideh D_Manfared

PACKAGE: **- Corrosivity Index** (SOIL)

Sample Number	5
Sample Name	Shillington Creek, BH5, SS4, (71/2-9)
Sample Matrix	Soil
Sample Date	03/05/2018

Parameter	Units	RL	Result
Corrosivity Index			
Soil Redox Potential	mV	-	209
Sulphide	%	0.02	< 0.02
pH	no unit	0.05	8.22
Resistivity (calculated)	ohms.cm	-9999	4520

PACKAGE: **- General Chemistry** (SOIL)

Sample Number	5
Sample Name	Shillington Creek, BH5, SS4, (71/2-9)
Sample Matrix	Soil
Sample Date	03/05/2018

Parameter	Units	RL	Result
General Chemistry			
Conductivity	uS/cm	2	221

PACKAGE: **- Metals and Inorganics** (SOIL)

Sample Number	5
Sample Name	Shillington Creek, BH5, SS4, (71/2-9)
Sample Matrix	Soil
Sample Date	03/05/2018

Parameter	Units	RL	Result
Metals and Inorganics			

Client: Terraprobe Inc

Project: 1-17-0822 Shilington Creek

Project Manager: Sepideh D_Monfared

Samplers: Sepideh D_Manfared

PACKAGE: **- Metals and Inorganics** (SOIL)

Sample Number	5
Sample Name	Shillington Creek, BH5, SS4, (71/2-9)
Sample Matrix	Soil
Sample Date	03/05/2018

Parameter	Units	RL		Result
Metals and Inorganics (continued)				
Moisture Content	%	0.1		30.2
Sulphate	µg/g	0.4		12

PACKAGE: **- Other (ORP)** (SOIL)

Sample Number	5
Sample Name	Shillington Creek, BH5, SS4, (71/2-9)
Sample Matrix	Soil
Sample Date	03/05/2018

Parameter	Units	RL		Result
Other (ORP)				
Chloride	µg/g	0.4		230

QC SUMMARY

Anions by IC

Method: EPA300/MA300-Ions1.3 | Internal ref.: ME-CA-[ENV]IC-LAK-AN-001

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%) Low	Recovery Limits (%) High	Spike Recovery (%)	Recovery Limits (%) Low	Recovery Limits (%) 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-[ENV]ARD-LAK-AN-020

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

Conductivity

Method: SM 2510 | Internal ref.: ME-CA-[ENV]EWL-LAK-AN-006

Parameter	QC batch Reference	Units	RL	Method Blank	Duplicate		LCS/Spike Blank			Matrix Spike / Ref.		
					RPD	AC (%)	Spike Recovery (%)	Recovery Limits (%) Low	Recovery Limits (%) High	Spike Recovery (%)	Recovery Limits (%) Low	Recovery Limits (%) High
Conductivity	EWL0274-MAY18	uS/cm	2	< 0.002	ND	10	98	90	110	NA		

QC SUMMARY

pH

Method: SM 4500 I Internal ref.: ME-CA-[ENV]EWL-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.

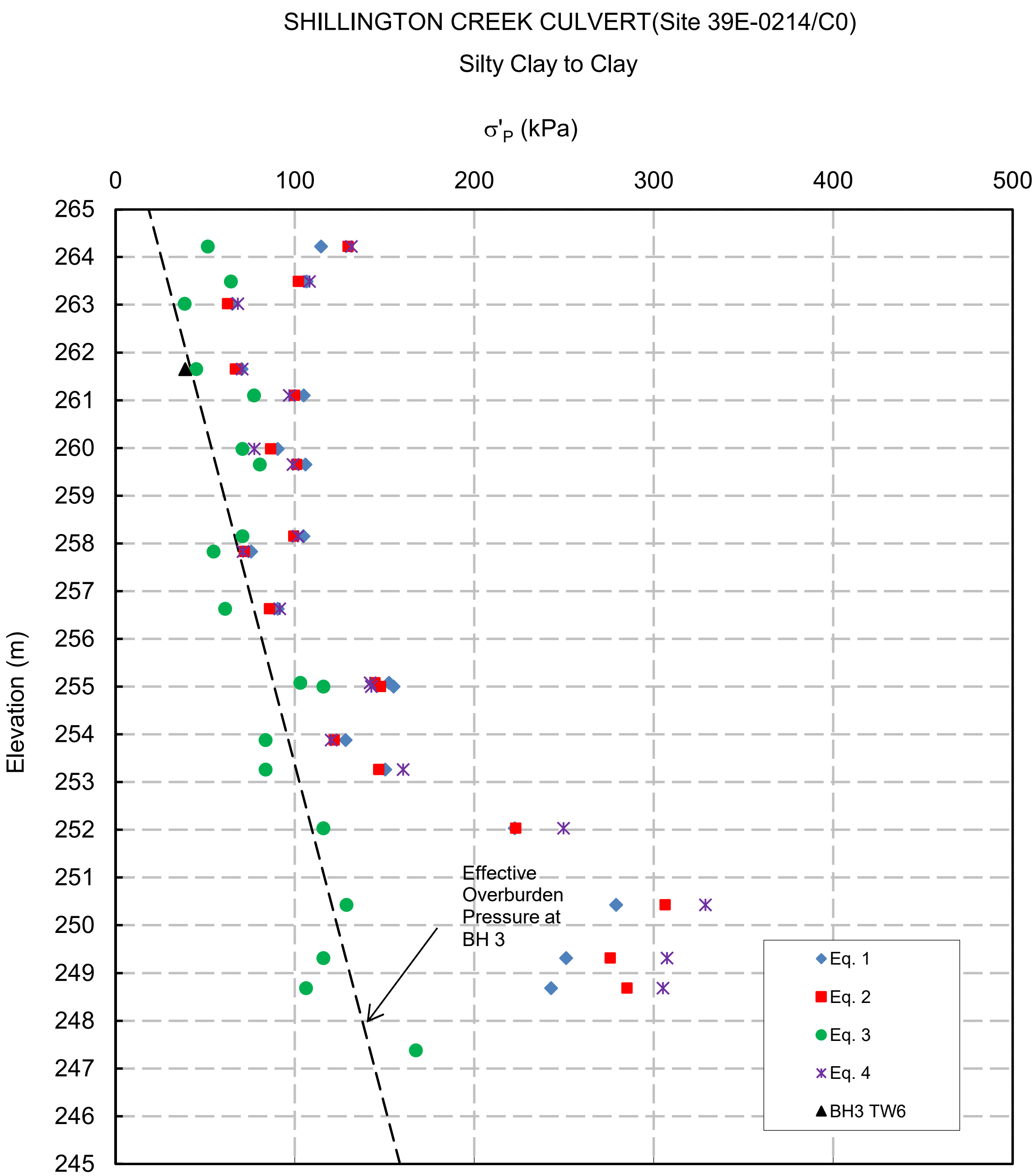
Analysis conducted on samples submitted pursuant to or as part of Reg. 153/04, are in accordance to the Protocol for Analytical Methods Used in the Assessment of Properties under Part XV.1 of the Environmental Protection Act” published by the Ministry and dated March 9, 2004 as amended.

-- End of Analytical Report --

APPENDIX C

Soil Design Parameters

PREDICTED AND MEASURED PRECONSOLIDATION STRESSES	FIGURE C1



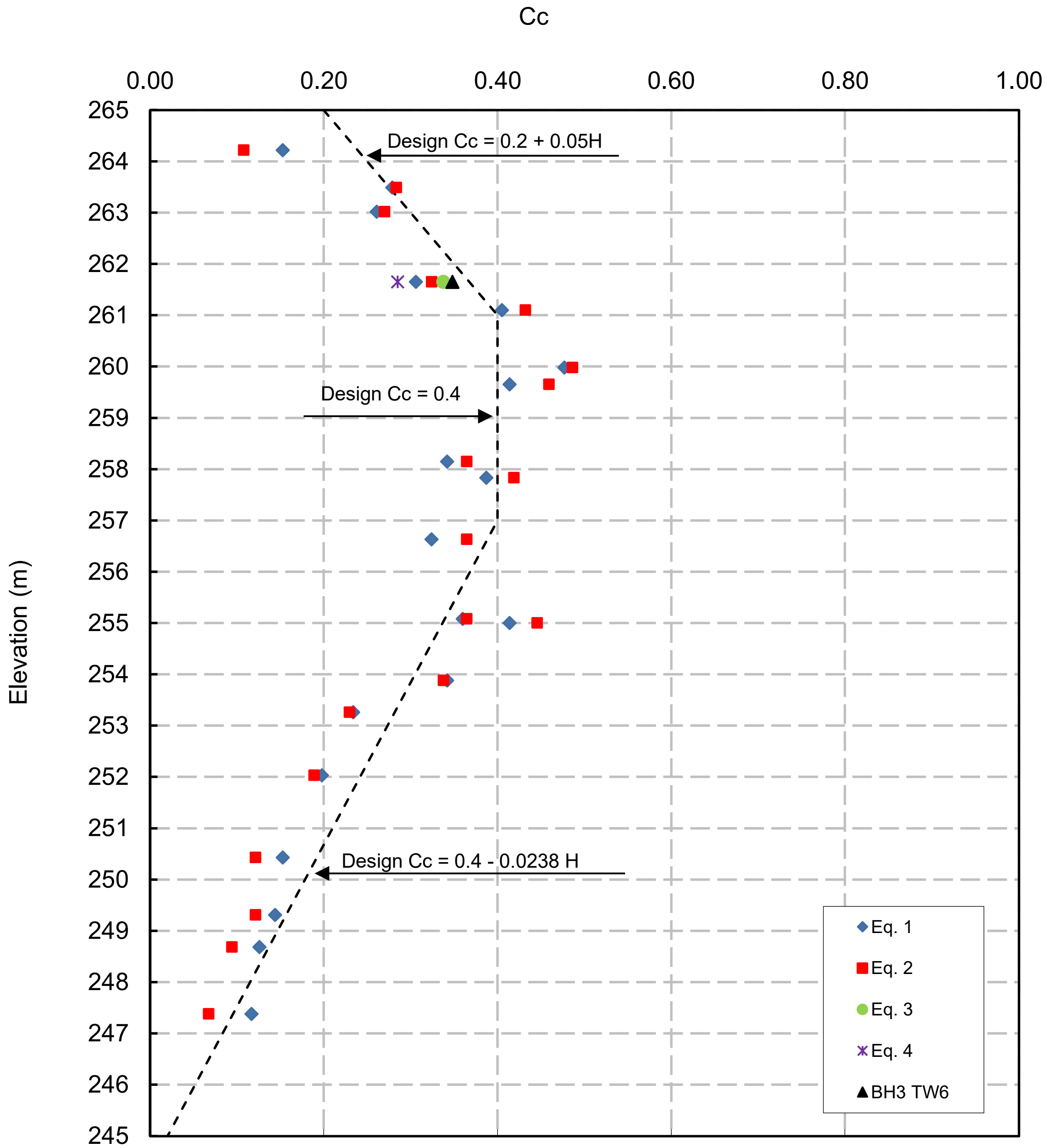
Eq. 1	$\sigma'_P = Cu / (0.11 + 0.0037 * I_P)$	Chandler (1988)
Eq. 2	$\sigma'_P = 22*(I_p^{-0.48})*Cu$	Mayne and Mitchell (1988)
Eq. 3	derived from $OCR=3.22*Cu/\sigma'_{v0}$	Mayne and Mitchell (1988)
Eq. 4	$\sigma'_P = 222 / LL * Cu$	Hansbo (1957)

Project No. :	1-17-0822	Terraprobe Inc.	Prepared by :	SD
Date :	May, 2018		Checked by :	RA

PREDICTED AND MEASURED COMPRESSION INDICES	FIGURE C2

SHILLINGTON CREEK CULVERT(Site 39E-0214/C0)

Silty Clay to Clay



Eq. 1 $Cc = 0.009 * (LL - 10)$ Terzaghi & Peck (1967)

Eq. 2 $Cc = I_p / 74$ Kulhaway & Mayne (1990)

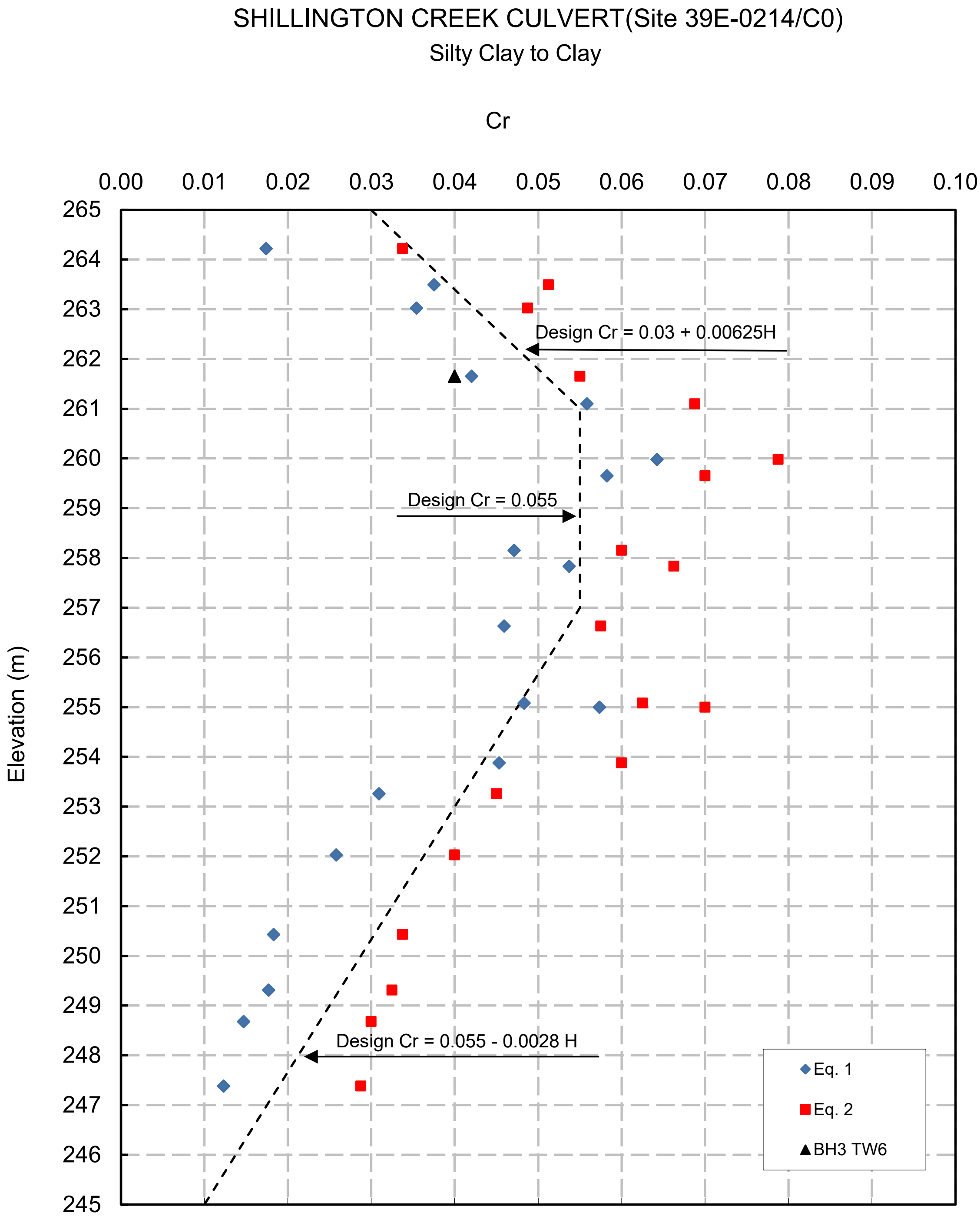
Eq. 3 $Cc = 0.141 * Gs1.2 * ((1 + eo) / Gs)2.38$ Rendon - Herrero (1983)

Eq. 4 $Cc = 0.141 * Gs * (\gamma_w / \gamma_d)^{2.4}$ Herrero (1983)

Project No. :	1-17-0822	Terraprobe Inc.	Prepared by :	SD
Date :	May, 2018		Checked by :	RA

PREDICTED AND MEASURED RECOMPRESSION INDICES

FIGURE C3



Eq. 1 Cr = Cc / 5 ~ Cc / 10 — Das (1993)

Eq. 2 Cr = 0.000463*LL*Gs — Nagaraj & Murty (1985)

Project No. : 1-17-0822

Date : May, 2018

Terraprobe Inc.

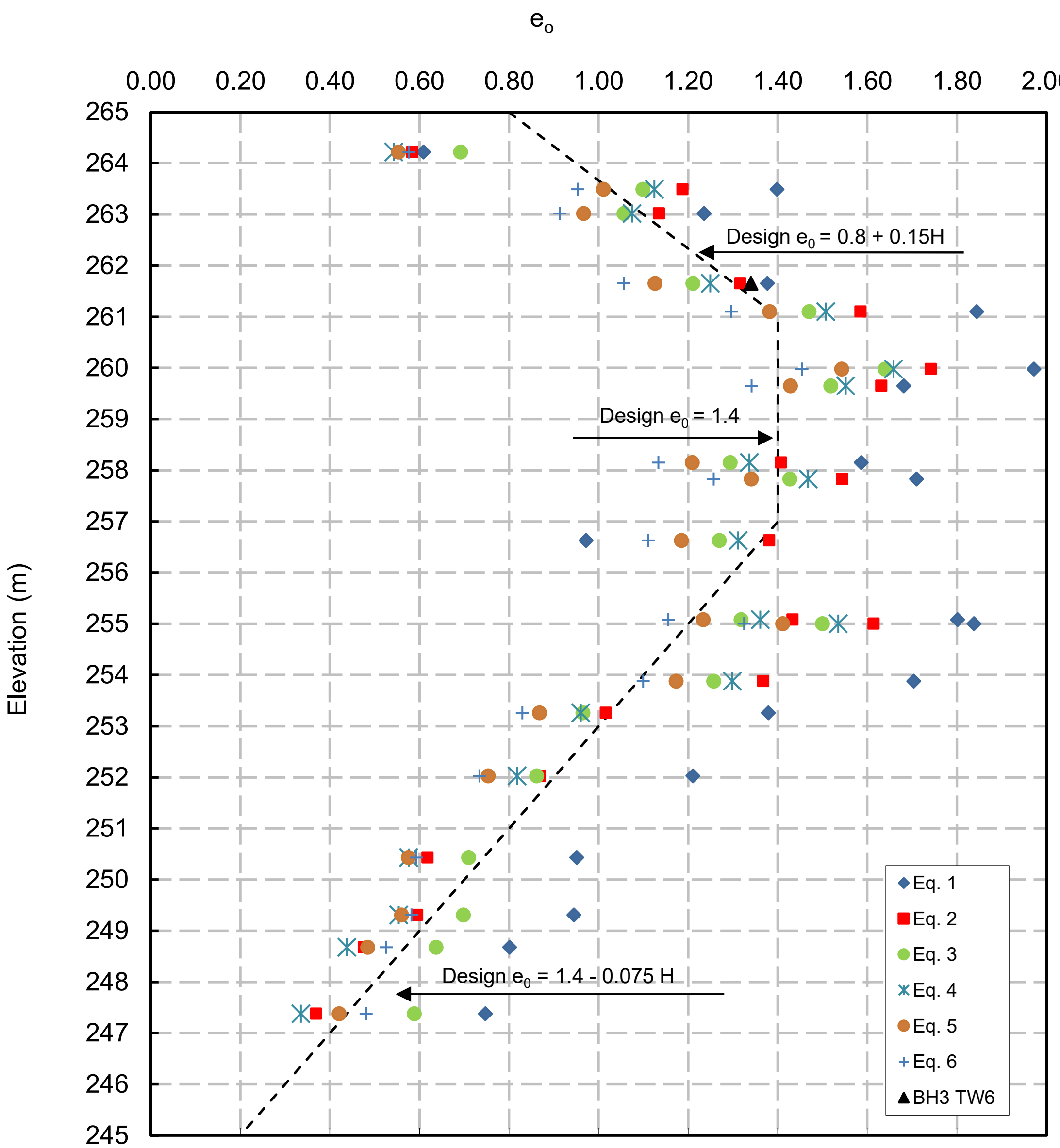
Prepared by : SD

Checked by : RA

PREDICTED AND MEASURED VOID RATIOS	FIGURE C4

SHILLINGTON CREEK CULVERT(Site 39E-0214/C0)

Silty Clay to Clay



Eq. 1	$e_o = \omega * Gs$	when saturated
Eq. 2	$e_o = (Cc / 0.141)^{0.4202} * Gs^{0.4958} - 1$	derived from Rendon - Herrero (1983)
Eq. 3	$e_o = Cc / 0.37 - 0.003 * LL - 0.0004 * \omega + 0.34$	derived from Azzouz et al. (1976)
Eq. 4	$e_o = Gs * (2Cc)^{0.4167} - 1$	derived from Rendon - Herrero (1980)
Eq. 5	$e_o = e^{(Ln\, Cc + 1.282) / 1.272}$	derived from Lav & Ansal (2001)
Eq. 6	$e_o = (Cc + 0.10) / 0.40$	derived from Lav & Ansal (2001)

Project No. : 1-17-0822

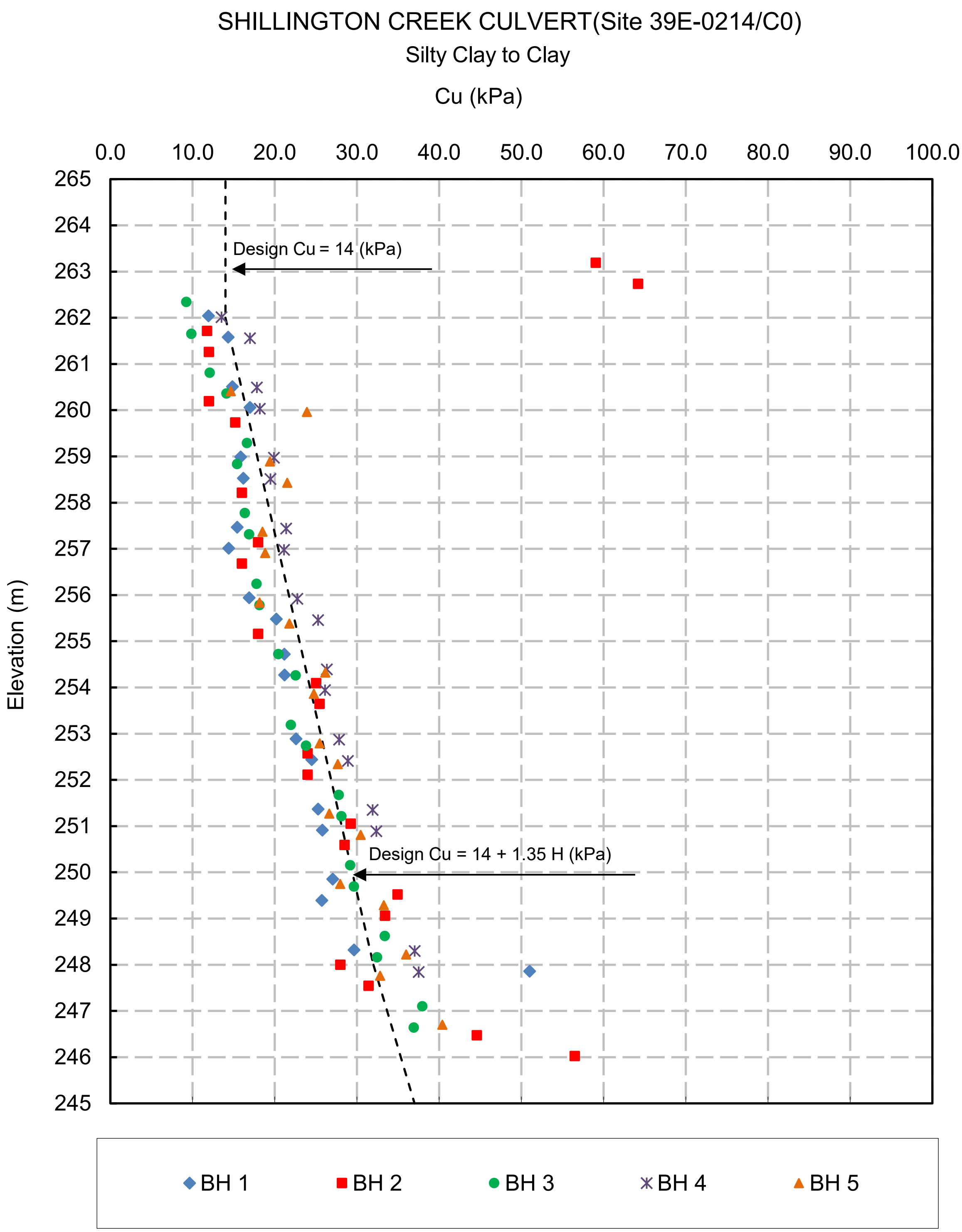
Date : May, 2018

Terraprobe Inc.

Prepared by : SD

Checked by : RA

UNDRAINED SHEAR STRENGTH	FIGURE C5

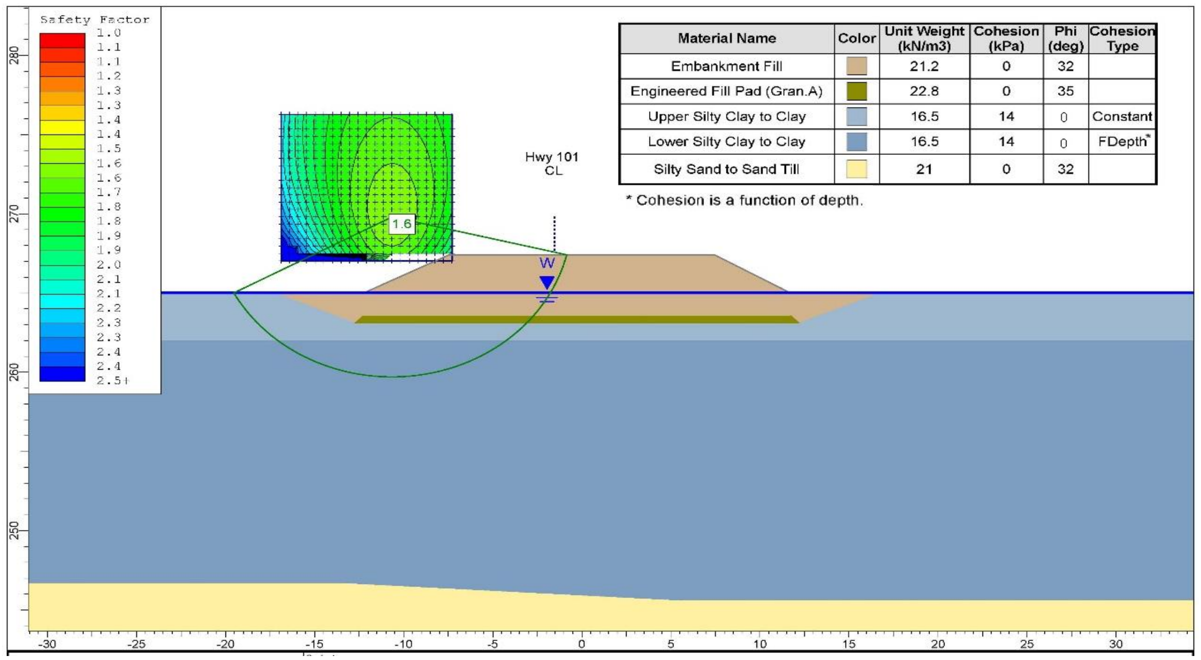


Field vane shear strengths were corrected based on Aas, et al. (1986)

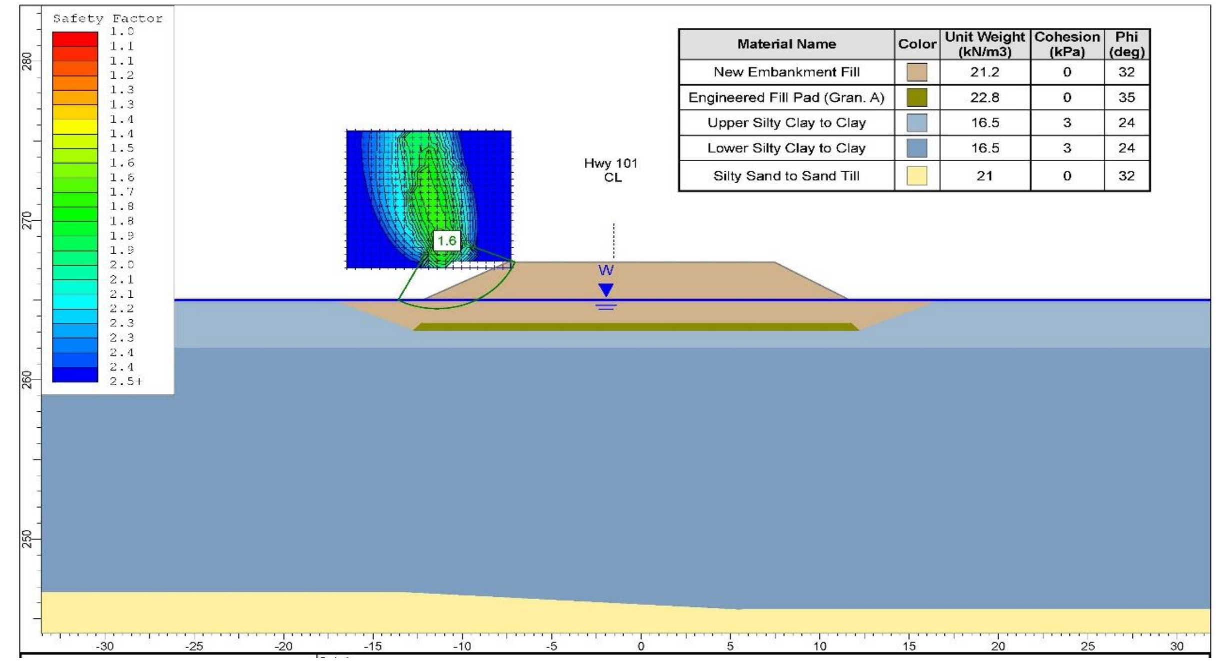
Project No. :	1-17-0822	Terraprobe Inc.	Prepared by :	SD
Date :	May, 2018		Checked by :	RA

APPENDIX D

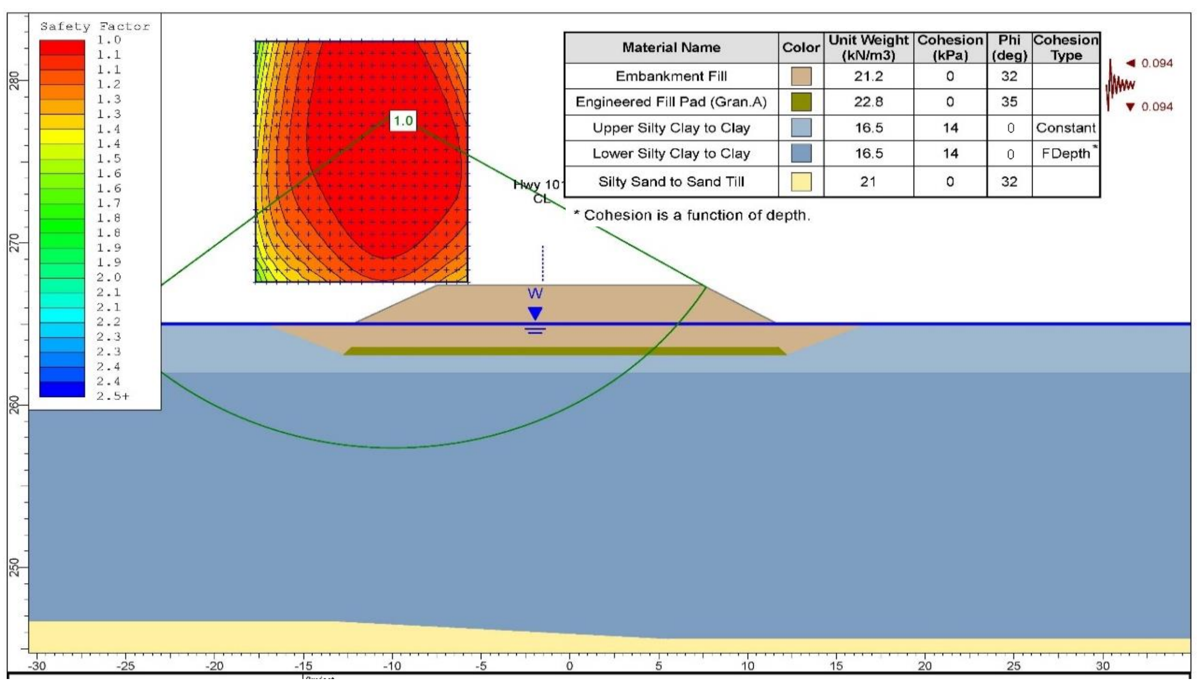
Slope Stability Models & Results



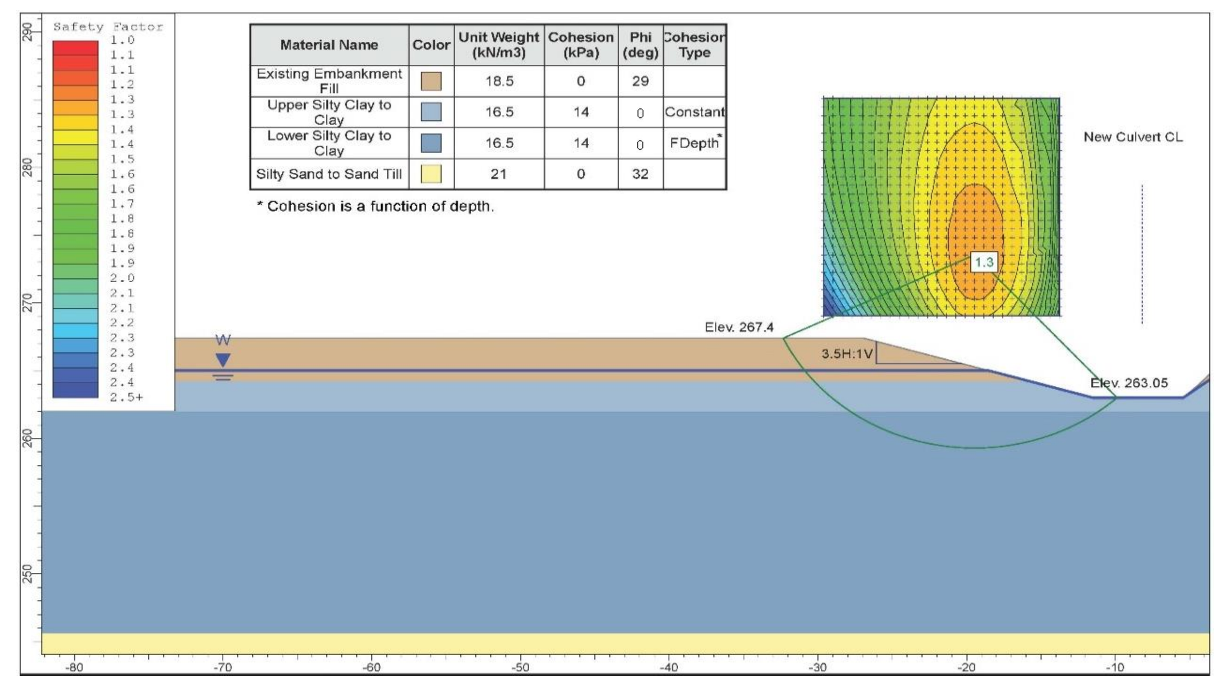
Hwy. 101 Permanent Side Slopes - Static Analysis
(Total Stress Analysis)



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



Hwy. 101 Permanent Side Slopes - Seismic Analysis



Temporary Excavation Side Slopes - West Side of Shillington Creek Culvert

Terraprobe Inc. Consulting Geotechnical & Environmental Engineering, Construction Materials, Inspection & Testing. 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	HWY 101	
	SHILLINGTON CREEK CULVERT REPLACEMENT, SITE 39E-0214/C0	
	G.W.P: 5165-12-00	DATE: January 2021
	SUBM'D: SD	CHKD: RA / APPD: RA
	Project No: 1-17-0822	Figure: D1

APPENDIX E

Special Provisions and Operational Constraint

<u>GEOGRID</u> - Item No.

Special Provision

1.0 SCOPE

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

2.0 REFERENCES

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

ASTM International

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

Ontario Provincial Standard Specifications, Construction

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

3.0 DEFINITIONS

For the purposes of this specification the following definitions apply:

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

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

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

MD means machine direction.

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

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

XMD means cross-machine direction.

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design

4.01.01 Foundation Investigation Report

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

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

4.01.02 Submission Requirements

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

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

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

The Contractor is advised that the undrained shear strength of the silty clay soils at the design geogrid elevation may vary from 12 kPa to 20 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 12 kPa to 20 kPa.

6.03 Management of Excess Materials

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

7.0 MEASUREMENT FOR PAYMENT

7.01 Geogrid

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

8.0 BASIS OF PAYMENT

8.01 Geogrid - Item

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

GEOTECHNICAL ASSESSMENT - Item No.

Special Provision

1.0 SCOPE

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

2.0 REFERENCES

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

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

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

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

3.0 DEFINITIONS – Not Used

4.0 DESIGN AND SUBMISSION REQUIREMENTS

4.01 Design Requirements

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

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

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

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

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

4.02 Submission Requirements

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

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

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

5.0 MATERIALS – Not Used

6.0 EQUIPMENT

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

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

7.0 CONSTRUCTION

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

8.0 QUALITY ASSURANCE – Not Used

9.0 MEASUREMENT FOR PAYMENT – Not Used

10.0 BASIS OF PAYMENT

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

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

<u>OPERATIONAL CONSTRAINT (GRADING)</u> - Preloading

Special Provision

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

The Contractor shall place preload material in the following areas;

Roadway/ Township	**Location**	**Preload Period (months)**
Hwy 101 / Township of Taylor	North of Hwy 101 From 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

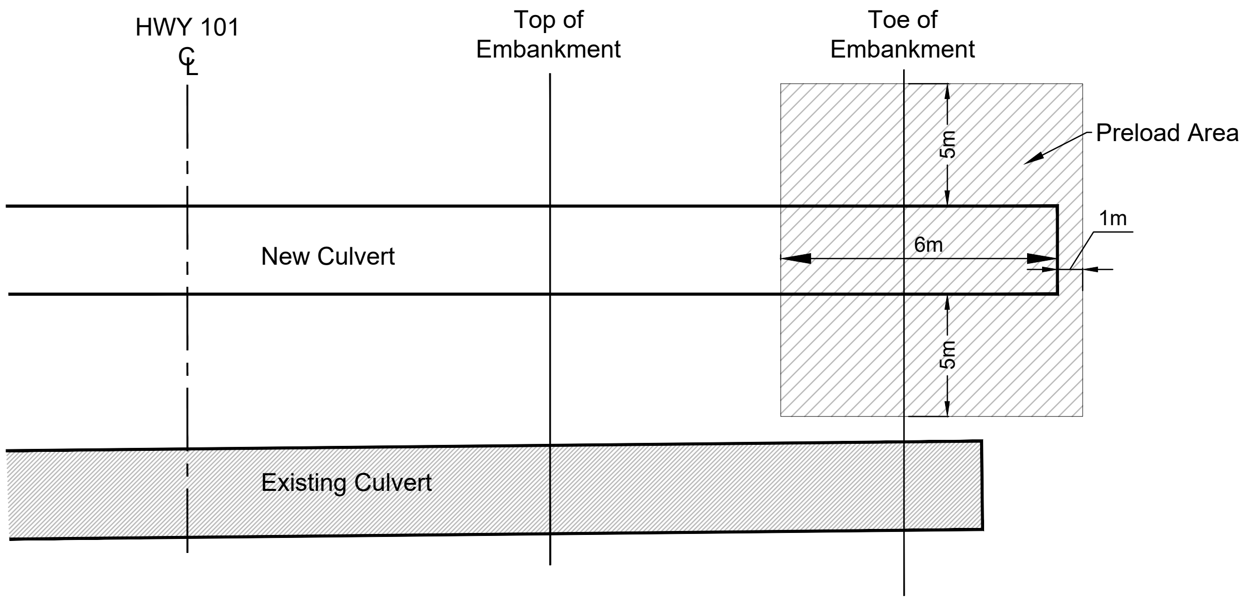
The Contractor shall not proceed with removing the preload and construction of new culvert until approval has been given by the Contract Administrator.

APPENDIX F

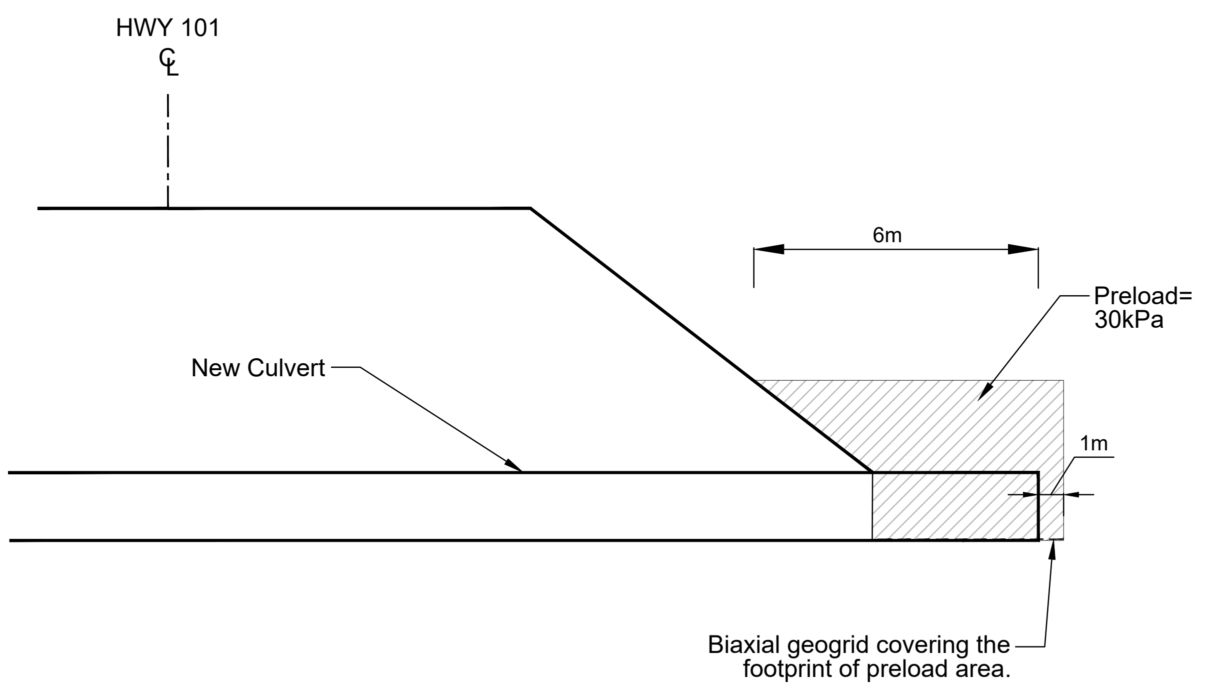
Preload Details

FIGURE F1

PRELOAD DETAILS



PLAN



PROFILE

NOT TO SCALE

Terraprobe