



Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
SHILLINGTON CREEK CULVERT REPLACEMENT
HIGHWAY 101
ASSIGNMENT No. 5013-E-0018
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 5165-12-00, SITE 39E-214C
GEOCRES NO.: 42A-103**

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PART A – FOUNDATION INVESTIGATION REPORT

**SHILLINGTON CREEK CULVERT REPLACEMENT, SITE 39E-214C
HIGHWAY 101
TOWNSHIP OF TAYLOR, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5013-E-0018, G.W.P. 5165-12-00**



1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by MMM Group Limited (MMM) on behalf of the Ministry of Transportation, Ontario (MTO) to provide foundation engineering services in support of preliminary designs for the rehabilitation of structures identified in MTO's Request for Proposal (RFP) titled *"Preliminary Design, Rehabilitation/Replacement of Twelve Structures on Highway 11, 101, 577, 579, 634 & 668, in New Liskeard Area"*, Contract Number. 5013-E-0018.

The terms of reference and scope of work for the foundation engineering services are outlined in MTO's RFP, and in Section 5.7 of MMM's *Technical Proposal* for this assignment. This report presents the factual data on the subsurface conditions at the Shillington Creek Culvert, Site 39E-214C on Highway 101, Township of Taylor, District of Cochrane, Ontario.

2.0 SITE DESCRIPTION

The site with coordinates of N 5,377,740 and E 328,930 is located on Highway 101, approximately 600 m east of the highway's west junction with Trans-Canada Highway 577 in the Township of Taylor, Ontario. Timmins is located west of the site and the main community of Matheson is situated at the intersection of Highways 11 and 101. The key plan on the Borehole Locations and Soil Strata Drawing, (Drawing 1) provides an overview of the site location.

The existing concrete culvert located at Station 19+125 is 20.0 m long and its cross-section (opening) is 4.88 m wide and 1.20 m high. The Highway 101 embankment is approximately 2 m high at the culvert site with a pavement centre line elevation of 267.5 ±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 between July 14 and August 14, 2014 and consisted of drilling and sampling three boreholes to depths ranging from 21.8 m to 25.8 m below ground surface. The approximate borehole locations are shown on Drawing 1.

Terraprobe's staff staked out the borehole locations in the field relative to on-site features and MMM surveyors established Control Point HCP 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 Details

Borehole No.	MTM NAD 83 Coordinates		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

The boreholes were drilled with truck and track-mounted CME 55 drill rigs supplied and operated by specialist drilling contractors. Samples of the overburden soils were generally obtained at intervals of

0.75 m and 1.5 m depth using a 50 mm outer diameter (O.D.) split-spoon sampler in conjunction with the Standard Penetration Testing (SPT) procedures as specified in ASTM Method D 1586¹. Relatively undisturbed samples of the clay soils were also collected with thin-walled Shelby Tube samplers. In the clay deposits an MTO 'N' vane was used to perform in-situ field vane tests, in order to determine the undrained shear strength of the soil. The field work was supervised on a full-time basis by a member of Terraprobe's staff who observed the drilling, sampling and in situ testing operations and logged the boreholes.

Ground water conditions in the open boreholes were observed during the drilling operations and a standpipe piezometer was installed in Borehole 1 to permit longer term ground water level monitoring. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

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.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

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

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

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

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.

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 gravelly sand, compact sand and soft to very stiff silty clay. The fill materials and ground surface are underlain by deposits of very soft to stiff varved silty clay to clay and compact to very dense silty sand till. A more detailed description of the subsurface conditions is provided in the following sections.

4.2.1 Topsoil

Topsoil ranging from 100 mm to 150 mm in thickness was encountered at this site. Topsoil thickness may vary between and beyond the boreholes.

4.2.2 Fill – Gravelly Sand

Borehole 2 was drilled through the shoulder of Highway 101 East Bound Lane. This borehole encountered a 600 mm thick layer of gravelly sand fill that extends to elevation 266.6 m.

A Standard Penetration test carried out in this gravelly sand fill measured an SPT N-value of 41 blows for 0.3 m of penetration suggesting a dense relative density. The moisture content (by weight) of a sample of the granular fill is 5%.

4.2.3 Fill – Sand

Sand fill was encountered below the gravelly sand fill in Borehole 2 extending to a depth of 1.4 m below ground surface or to elevation 265.8 m.

A Standard Penetration test carried out in the sand fill measured an SPT N-value of 22 blows for 0.3 m of penetration indicating a compact relative density.

4.2.4 Fill – Silty Clay

Silty clay fill was encountered beneath the sand fill in Borehole 2 and beneath the topsoil in Boreholes 1 and 3. The locations, thicknesses, depths and base elevations of the silty clay fill are summarized in the following table.

Silty Clay Fill Borehole Data

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

Standard Penetration tests performed in the silty clay fill gave N-values ranging from 2 to 25 blows for 0.3 m of penetration indicating a 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 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 25 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.

Silty Clay to Clay Borehole Data

Borehole No.	Silty Clay to Clay Thickness (m)	Silty Clay to Clay Depth (m)	Silty Clay to Clay Base Elevation (m)
BH1	17.9	19.3	246.7
BH2	18.7	21.6	245.6
BH3	18.7	20.8	245.5

The N-values of Standard Penetration tests carried out in the silty clay to clay deposit range from 0 blows (weight of hammer) to 15 blows per 0.3 m of penetration and, field vane tests generally measured in-situ undrained shear strengths ranging from 12 kPa to 88 kPa as illustrated on Figure B4 in Appendix B. Based on these results the consistency of the silty clay to clay is described as very soft to stiff. The sensitivity of the silty clay to clay generally ranges from about 1.2 to 6, indicating a low sensitivity soil class (Canadian Foundation Engineering Manual [CFEM], 2006).

The variation of undrained shear strength with elevation plot depicted in Figure B4 generally illustrates low undrained shear strength values ranging from 12 kPa to 28 kPa. Below elevation 253.0 m higher undrained shear strength values ranging from about 32 kPa to 88 kPa were recorded.

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 and B6 in Appendix B. The test results show a grain size distribution consisting of 0% gravel, 0% to 1% sand, 16% to 41% silt and 58% to 84% clay sized particles.

Atterberg limits tests were carried out on nine samples of the silty clay to clay and the results are plotted on the plasticity charts, Figures B7 and B8 in Appendix B. The results indicate a cohesive deposit of generally intermediate plasticity (CI) with occasional high plasticity clay (CH). The Atterberg limits test results are summarized below.

Liquid Limit:	36% to 63 %
Plastic Limit:	19% to 27 %
Plasticity Index:	17% to 36 %
Natural Moisture Content:	36% 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 B9 in Appendix B. The grain size distribution of the clayey silt consists of 0% gravel, 1% sand, 68% and 73% silt and; 26% and 31% 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 B10 in Appendix B. These values indicate low plasticity (CL) clayey silt. The Atterberg limits test results are summarized below.

Liquid Limit:	24% and 26 %
Plastic Limit:	17%
Plasticity Index:	7% and 9 %
Natural Moisture Content:	30% and 35 %

The Atterberg Limits tests results of the silty clay to clay deposit are also plotted against elevation in Figure B11. These results illustrate that the natural moisture contents of the tested samples are typically higher than the liquid limits. The moisture content of twenty three samples of the silty clay to clay varies between 23% and 73% and the unit weight of a tested sample is 16.9 kN/m³.

A one-dimensional consolidation test was performed on a sample of the silty clay to clay and the results are presented in Figures B12 to B15 in Appendix B. The results of the one-dimensional consolidation test are summarized below.

One-Dimensional Consolidation Test Results

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.

4.2.6 Silty Sand Till

A silty sand till deposit was encountered in all of the boreholes. Summarized below are the explored depths and base elevations of the silty sand till.

Silty Sand Till Borehole Data

Borehole No.	Silty Sand Till Depth of Deposit (m)	Silty Sand Till Base Elevation (m)
BH1	22.9*	243.1
BH2	25.8*	241.4
BH3	21.8*	244.5

* 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. The natural water content of samples from this stratum range from 9% to 12%.

A grain size distribution test was carried out on a sample from this deposit and the results illustrated in Figure B16, Appendix B show a grain size distribution consisting of 2% gravel, 61% sand, 29% silt 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. A standpipe piezometer was installed in Borehole 1. The ground water levels measured in the piezometer are summarized in the following table:

Ground Water Level Data

Borehole No.	Date	Water Levels	
		Depth (m)	Elevation (m)
BH1	September 16, 2014	0.3	265.7
	October 28, 2014	0.3	265.7

The ground water level at this site is estimated to be at approximately Elevation 265.7 m based on the soil moisture conditions, measured ground water levels and creek water levels. The ground water levels are expected to fluctuate seasonally and are expected to rise during wet periods of the year.



5.0 MISCELLANEOUS

The investigation was carried out using drilling equipment supplied and operated by Tattr Environmental Drilling of Timmins, Ontario and Landcore Drilling of Chelmsford, Ontario. The field operations were supervised by Mr. Satyajit Manani, C.E.T. and the routine laboratory and one-dimensional consolidation testing was carried out at Terraprobe's Brampton laboratory.

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

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PART B – FOUNDATION DESIGN REPORT

**SHILLINGTON CREEK CULVERT REPLACEMENT, SITE 39E-214C
HIGHWAY 101
TOWNSHIP OF TAYLOR, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5013-E-0018, 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 preliminary geotechnical design recommendations to assist the design team to select a preferred alternative for a 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 for planning and preliminary design purposes only, as part of the assessment of the feasibility and constructability of potential alternatives.

When designing culverts, the economic analysis usually includes factors such as estimated service life, construction cost, maintenance cost, replacement cost, risk of failure, and risk of property damage. Fish passage is also an important factor that affects the choice of culvert type.

The material choice includes steel, concrete, high density polyethylene and polyvinyl chloride. Some factors that are to be considered when choosing the material type include:

- Steel and plastic culverts have the advantage of simpler and quicker construction, which is especially advantageous in remote areas. Steel also has added advantage of often being at least partly salvageable;
- A well designed concrete culvert is extremely durable under a wide range of conditions;
- Precast concrete and smooth walled plastic pipes provide more efficient inlets than sharp edged inlets on metal culverts;
- The greater roughness of corrugated interiors may be an advantage for fish passage and for other situations where barrel or outlet velocities must be reduced; and
- Flexible pipe culverts may have an advantage over concrete box culverts in certain unfavourable foundation soil conditions.

The most economical culvert is neither the one with the lowest initial cost nor the culvert with the longest service life. Short and long term costs should be considered in both the original designs and in repairs or replacements. Normally where weak foundation soils and/or settlement sensitive soils exist (as encountered on this project) concrete box culverts or pipe culverts may provide better solutions.

The existing concrete culvert opening is 4.88 m wide and 1.2 m high measuring about 20.0 m in length and it will be replaced with a longer 25± m culvert in order to prevent spillage of the embankment fill into the culvert's inlet and outlet. The upstream and downstream streambed elevations are 264.32 m and 264.2 m respectively and, these elevations will be maintained for the new culvert. The maximum height of embankment fill at the site is approximately 2.0 m.

Listed below are the four alternative culvert types that were considered.

- A twin 2.44 m x 1.80 m concrete box culvert;
- A single cell 4.5 m x 1.80 m concrete box culvert;
- A triple cell 2.5 m diameter circular CSP culvert; and
- A single-span 6.086 m x 1.830 m precast rigid frame (open footing) culvert.

6.2 Foundation Alternatives

The advantages, disadvantages, risks and consequences of practical foundation options for supporting a structural culvert are presented in Table 1. These foundation alternatives are summarized below.

- Geogrid reinforced culvert bedding (box culverts);
- Driven piles (open footing culverts); and
- Small diameter displacement piles (open footing culverts).

6.2.1 Geotechnical Resistances

6.2.1.1 Geogrid Reinforced Culvert Bedding (Granular A)

It is not practical to support a new culvert directly on the very soft to firm silty clay to clay because of the deposit's low bearing capacity. However, a box culvert founded at an invert elevation of 264 m approximately, can be supported on 0.5 m thick OPSS Granular A bedding constructed at a base elevation of 263.5 m. At elevation 264 m the bedding should extend at least 1 m horizontally beyond the culvert sides and should extend down to elevation 263.5 m at a 1 Horizontal to 1 Vertical (1H:1V) side slope.

A 5.0± m wide box culvert supported on 0.5 m thick OPSS Granular A bedding that is founded at an elevation of 263.5 m on the undisturbed silty clay to clay deposit, can be designed for a factored geotechnical resistance at the Ultimate Limit State (ULS) of 85 kPa. It is understood that a geotechnical reaction at the Serviceability Limit State (SLS) of 65 kPa is required. For an SLS of 65 kPa, the settlement will be greater than 25 mm as outlined in Section 6.8 of this Report.

The factored geotechnical resistance value of 85 kPa at ULS and 65 kPa at SLS is for vertical, concentric loads only. Effects of load inclination and eccentricity should be taken into account as illustrated in the *Canadian Highway Bridge Design Code (CHBDC) 2006*, Clause 6.7.3 and Clause 6.7.4.

Since the silty clay to clay soils will be susceptible to loosening/softening and degradation on exposure to water and construction traffic; the culvert bedding material should be placed expeditiously to avoid disturbance of the foundation bearing surfaces. A geotextile fabric should be installed at the silty clay/bedding interface to prevent soil migration, and the culvert bedding should also be reinforced with a biaxial geogrid to provide stability during bedding placement and compaction.

Resistance to lateral forces/sliding resistance between the concrete box and the subgrade soils should be evaluated in accordance with the CHBDC 2006. The following ultimate coefficient of friction values are recommended between concrete and the bedding material or subgrade soils:

- OPSS Granular A bedding – ultimate coefficient of friction of 0.7, and
- Silty Clay to Clay – ultimate coefficient of friction of 0.46.

Settlement will be greater than 25 mm and the culvert will have to be designed as an articulated structure with a camber to accommodate this settlement. The culvert sections will have to be installed with a gap that can close progressively as settlement occurs. The estimated magnitude of settlement is provided in Section 6.8.

6.2.1.2 Driven Piles

The subsurface conditions at the site are considered suitable for supporting an open footing culvert on steel H-piles driven into the underlying very dense silty sand till. An HP 310x79 pile section is considered suitable for supporting the culvert and; the concentric axial geotechnical design resistance at ULS, the foundation load at SLS, and estimated pile tip elevations are provided in the following table.

Axial Resistance of Driven Piles

PILE TYPE – HP 310 x 79				
Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)
Borehole 1	244.5±	Silty Sand Till	800	625
Borehole 2	243.0±	Silty Sand Till		
Borehole 3	243.0±*	Silty Sand Till		

* Estimated based on Borehole 1 and Borehole 2 data.

Pile installation should be carried out in accordance with OPSS 903, November 2009 and the tips of all piles should be fitted with a Steel H-Pile Driving Shoe conforming to OPSD 3000.100. Pile driving should be controlled by the Hiley Formula and an Ultimate Pile Resistance (R) to be specified by the structural engineer in accordance with Clause 3.3.2 (b) Construction Stage of the Structural Manual. The appropriate pile driving note is “Piles to be driven in accordance with Standard SS 103-11 using an ultimate resistance of “R” kN per pile”. The ultimate resistance “R” shall be determined during detail design based on inputs from the structural engineer. For preliminary design we recommend an ultimate resistance “R” of 1600 kN.

6.2.1.3 Small Diameter Displacement Piles

Small diameter displacement piles can also be used to support an open footing culvert. This type of piling system consists of ductile cast iron pipes that are joined by a spigot and collar arrangement to form a continuous pile length. The cast iron pipes are driven into the ground with a lightweight excavator equipped with a hydraulic hammer. A pile shoe (either flat or conical) is attached to the tip of the pile and during driving; the pile shoe forms an annular space along the entire pile that is continuously filled with pumped concrete.

A 118 mm diameter displacement pile driven into the silty sand till at least to a pile tip elevation of about 244 m can be designed for a factored axial resistance at ULS of 360 kN. Also, the foundation load at SLS for 25 mm settlement should not exceed 360 kN.

6.2.1.4 Ground Improvement

Ground improvement options such as concrete columns and aggregate piers were considered. However, these techniques are not practical because the very soft to stiff silty clay to clay deposit is too thick. These options were therefore ruled out from this study.

6.2.2 Recommended Foundation Scheme

From a geotechnical point of view, a box culvert supported on a geogrid reinforced culvert bedding is recommended. Based on the advantages, disadvantages, risks and consequences, this foundation scheme is reliable, has a low risk associated with settlement (provided that there are no highway grade raises), and is also an economical foundation solution compared to pile foundations.

6.2.3 Design Frost Depth

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.

Strip footings or pile caps for an open footing culvert and for any associated retaining walls, should be founded at a minimum depth of 2.5 m of earth cover below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSD 3090.100. In addition, the footings should extend below any existing fill and surficial organic materials, where present.

6.3 Lateral Earth Pressure

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 6.9 of the CHBDC 2006 and according to Clause 6.9.3 of the CHBDC 2006; a compaction surcharge should also be added. For soils with an angle of internal friction ranging from 30° to 35° the magnitude should be 12 kPa at the top of the fill decreasing linearly to 0 kPa at a depth of 1.7 m; or decreasing linearly to 0 kPa at a depth of 2.0 m for soils with an angle of internal friction that exceeds 35°. Compaction equipment including hand operated vibratory equipment should be in accordance with OPSS.PROV 501.

Earth pressure coefficients for backfill to the culvert and retaining walls are dependent on the material used as backfill and typical values are provided in the following table.

Lateral Earth Pressure Coefficients

Wall Condition	Lateral Earth Pressure Coefficient (K)					
	Existing Earth Fill $\phi = 30^\circ; \gamma = 19.0 \text{ kN/m}^3$		OPSS Granular A or OPSS Granular B Type II $\phi = 35^\circ; \gamma = 22.8 \text{ kN/m}^3$		OPSS Granular B Type I $\phi = 32^\circ; \gamma = 21.2 \text{ kN/m}^3$	
	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)	Horizontal Surface Behind Wall	Sloping Surface Behind Wall (2H:1V)
Active (Unrestrained Wall)	0.33	0.54*	0.27	0.38*	0.30	0.46*
At rest (Restrained Wall)	0.50	-	0.43	-	0.47	-
Passive (Movement Towards Soil Mass)	3.00	-	3.70	-	3.30	-

* For retaining walls.

The lateral earth pressure coefficients 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 C6.16 in the Commentary to the CHBDC, 2006.

6.4 Culvert Bedding and Backfill

Structural backfill and cover around the culvert should be placed in accordance with the limits illustrated in MTOD 803.021 (precast concrete box culverts). 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. The backfill should consist of free-draining, non-frost susceptible granular materials in accordance with OPSS.PROV 1010. All structural backfill should be placed in loose lifts not exceeding 150 mm thick and should be compacted to at least 95 % of the materials Standard Proctor Maximum Dry Density (SPMDD).

During all stages of backfill placement the differential backfill height shall not be greater than 400 mm and, backfilling operations should be carried out in accordance with OPSS 902. 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.

Bedding material should consist of OPSS Granular “A” material placed and compacted to 95% of the materials SPMDD in accordance with Section 422.07.07 of OPSS.PROV 422. Additional bedding requirements that may be imposed by the supplier must also be followed.

The silty clay to clay deposit at the base of the excavation is soft to firm and a geotextile fabric should be placed on its surface prior to placing bedding material to prevent soil migration. It is also recommended that the bedding be reinforced with a biaxial geogrid to provide reinforcement and stability during placement and compaction.

To achieve the specified compaction, soils used for non-structural backfill must neither be too wet nor too dry of their optimum moisture content. Soils that are too wet (such as the silty clay to clay) cannot be used immediately because the material will have to be dried to a moisture content of $2\pm$ % of optimum. If the construction operations are time sensitive, the use of imported earth fill may be considered. Soils that are dry of optimum can be used immediately provided that the material is moisture conditioned (i.e. water

added) to achieve a moisture content of $2\pm$ % of optimum. The existing embankment fill (sand and gravelly sand) can be re-used for non-structural backfill provided they are free of organics and other deleterious material.

6.5 Erosion Protection

Erosion protection should be provided at the culvert inlet and outlet (including the slopes and sides). At the inlet area a clay seal can 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 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 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.

6.6 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 261.5 m), the OHSA soil classifications are:

- Embankment fill – Type 3 soils; and
- Silty Clay to Clay – Type 4 soils.

The side slopes of temporary excavations may be formed no steeper than 1H:1V for Type 3 soils and 3H:1V or flatter for Type 4 soils. Excavations should be undertaken in accordance with OPSS 902.

6.7 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. Around the perimeter of the excavation, a cofferdam and an interceptor perimeter trench should also be installed to prevent surface water from entering the excavation.

The design, installation, operation and maintenance of the dewatering 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 dewatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

6.8 Settlement (Culvert and Embankment)

To predict the magnitude and time rate of settlement of the underlying silty 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 as well as empirical correlations of undrained shear strengths, laboratory index tests and soil moisture contents. These deformation parameters are provided in Figures C1, C2, C3 and C4. The preconsolidation pressure (σ'_p) 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 are summarized in the following table.

Silty Clay to Clay Deformation Parameters	
Parameter	Lower Silty Clay
Overconsolidation Ratio	1.0
Compression Index - C_c	0.3 – 0.2
Recompression Index - C_r	0.045 – 0.040
Initial Void Ratio - e_o	1.45 – 1.40
Coefficient of Consolidation - C_v (m^2/s)	3.6×10^{-8}

The settlement analysis is based on an SLS of 65 kPa, (applied only in the area of the pavement platform), a reconstructed embankment side slope geometry of 2H:1V and the assumption that the embankment height will remain unchanged i.e. no grade raise of Hwy. 101. Beyond the pavement platform the settlement analysis is based only on the culvert self-weight and embankment load. Two scenarios were examined for a culvert supported on a geogrid reinforced culvert bedding namely:

- Culvert installed at an offset distance of 6.0 m east from the existing culvert centreline; and
- Culvert installed in the footprint area of the existing culvert.

If the replacement culvert is installed below Highway 101 at an offset distance from the existing culvert, the estimated total settlement along the culvert centre line will be:

**Total Estimated Settlement Along Culvert Alignment
Culvert Installed at an Offset Distance**

Offset Distance (m)*	-13	-10	-7	-5	0	5	7	10	13
Estimated Settlement (mm)	0	0	20	45	55	45	20	0	0

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

The settlement data tabulated above indicates that a culvert camber is required and the time required to achieve a post construction settlement criterion of 25 mm below the pavement platform is about one month. The existing culvert will be removed and, fill placed in this area will induce approximately 95 mm

of settlement in the underlying silty clay to clay deposit below the pavement platform. It is estimated that most of this settlement will be complete in one month and the remaining post construction settlement will be less than 25 mm.

If the replacement culvert is installed in the footprint area of the existing culvert, the estimated total settlement along the culvert centre line will be.

**Total Estimated Settlement Along Culvert Alignment
Culvert Installed In The Footprint Area Of The Existing Culvert**

Offset Distance (m)*	-13	-10	-7	-5	0	5	7	10	13
Estimated Settlement (mm)	190	70	10	30	30	30	10	70	190

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

The settlement data tabulated above indicates that a culvert camber is required and the time required to achieve a post construction settlement criterion of 25 mm is about two months.

The estimated differential settlement at offset distances of 10 m to 13 m from the highway centre line is quite large because this area has not experienced the benefit of any preloading from the current embankment. Therefore, it is imperative that the culvert joints be designed to accommodate this settlement. Alternatively, it may be preferable to install a temporary culvert extension (such as a CSP) and construct a temporary embankment to preload the area over a two month period prior to installing the new culvert.

It is understood that the embankment will not be widened and the highway grade will not be raised. Therefore, settlement of the underlying silty clay to clay deposit due to embankment loads will be negligible. However, embankments constructed with local granular or earth fill will settle during construction (fill compression) and; the magnitude of this settlement is expected to be about 1% of the fill height.

6.9 Embankments

6.9.1 Embankment Stability

The global, internal and surficial stability of the embankment will depend on the slope geometry and also to a large degree on the material used to construct the embankment. For the purpose of embankment stability analyses, the commercially available slope stability program Slide 6.0 developed by Rocscience Inc. was used.

The Morgenstern-Price and Spencer methods for stability analysis were employed and a minimum target factor of safety of 1.3 was established. The soil parameters used for the slope stability analyses and the factors of safety that were obtained are provided in the following table. The slope stability models depicting the corresponding factors of safety are provided in Figures D1 and D2 in Appendix D. Our analyses indicate that the factors of safety will be equal to or greater than the target factor of safety of 1.3 provided that the embankment is constructed at a minimum 2H:1V side slope or flatter.

Slope Stability Design Parameters and Results

Material Type	Total Stress Analysis		Effective Stress Analysis		Unit Weight
	ϕ (degrees)	c (kPa)	ϕ' (degrees)	c' (kPa)	γ (kN/m ³)
New Embankment Fill	30	0	30	0	20
Culvert Bedding	32	0	32	0	21
Silty Clay to Clay	0	16 to 25*	25	0	16.5
Silty Sand Till	32	0	32	0	21
Design Factors of Safety	1.6 to 1.8		1.3 to 1.6		

* Refer to Figure C5 in Appendix C.

6.9.2 Embankment Construction

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 SPMD. Embankment construction should be carried out in accordance with OPSS.PROV 209, OPSS.PROV 501 and OPSS.PROV 206. Borrow material must meet the requirements of OPSS.PROV 212 and bonding between existing fill and new fill should be carried out by benching in accordance with OPSS 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 and embankment slopes must be reinstated with permanent erosion protection in accordance with OPSS 803 and OPSS.PROV 804.

6.10 Temporary Protection Systems

Decisions regarding shoring methods and sequencing are the responsibility of the Contractor. Temporary protection systems 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 temporary protection system depends upon the type of soil to be encountered and the amount of movement that can be permitted. The protection 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). It should be noted that the lateral

earth pressure coefficients chosen for design require certain movements for the active and passive conditions to be mobilized.

The appropriate lateral earth pressure parameters for use in the design temporary protection systems are provided in the following table. The lateral earth pressure coefficients are based on the assumption that the ground surface behind the temporary protection 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 C6.17 and C6.18 of the CHBDC 2006.

Temporary Protection System Design Parameters

Stratigraphic Unit	Friction Angle ϕ (degrees)	Unit Weight γ (kN/m)	Active Earth Pressure Coefficient	At - Rest Earth Pressure Coefficient	Passive Earth Pressure Coefficient
			K_a	K_o	K_p
Existing Fill Soils	30	19	0.33	0.50	3.00
Silty Clay to Clay	25	16.5	0.40	0.58	2.46
Silty Sand Till	32	21	0.31	0.47	3.25

For the design of 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. For preliminary design purposes the following values of c_u are provided:

- From elevation 264.5 m to 254 m $c_u = 16$ kPa; and
- From elevation 254 m to the base of the silty clay to clay deposit $c_u = 25$ kPa.

The commercially available slope stability program Slide 6.0 (developed by Rocscience Inc.) was used to perform preliminary global stability analyses for a vertically supported open excavation extending to elevation 263.5 m i.e. the base of the geogrid reinforced culvert bedding. The Morgenstern-Price and Spencer methods of stability analysis were employed. The stability analyses indicate that an 11 m long sheet pile arrangement (measured from the current highway grade to the pile tip), is required along the highway centre line to provide a target factor of safety of 1.3 with respect to global stability.

For a pile foundation scheme, the excavation will extend approximately to elevation 261.5 m. The stability analyses indicates that a 16 m long sheet pile arrangement (measured from the current highway grade to the pile tip), is required along the highway centre line to provide a target factor of safety of 1.3 with respect to global stability.

6.11 Retained Soil System (RSS) Temporary Detour

Geotechnical recommendations for a Highway 101 temporary detour around the construction site using a low performance retained soil system (RSS) are provided in the following sections. The RSS will be located on the south side of Highway 101 and will be approximately 65 m long.

6.11.1 RSS Geotechnical Resistances

The RSS can be founded on the existing soils or compacted fill. A 3 m high RSS founded on the undisturbed silty clay to clay deposit, can be designed for a factored geotechnical resistance at ULS of 60 kPa. An SLS value is not applicable since the RSS is a temporary structure, its required performance is categorized as “low”, and the estimated settlement will be greater than 25 mm for a 3 m high RSS.

6.11.2 RSS Global Stability

The global stability of the RSS wall is dependent on the characteristics of the embankment fill and the foundation soils, the geometry of the embankment and the RSS. The RSS wall will be in the form of a rectangular block from ground surface extending to a maximum wall height of 3 m.

Stability analyses were carried out on a selected RSS configuration considering the following variables:

- RSS base founded at a design elevation of 264 ±m;
- Fill behind the RSS is horizontal;
- Water level at elevation 265.5 ±m; and
- RSS block reinforcement length required to achieve a target factor of safety of 1.3.

Analysis carried out on a 3 m high RSS wall indicates that the RSS block requires a minimum reinforcement anchor length equivalent to 100% of the wall height to achieve a target factor of safety of 1.3 with respect to global stability.

6.11.3 RSS Settlement

The settlement of the temporary RSS wall was checked using the commercially available program Settle 3D developed by Rocscience Inc. The deformation parameters used for the analyses were established using data obtained from a one-dimensional consolidation test as well as empirical correlations of undrained shear strengths, laboratory index tests and soil moisture contents. These deformation parameters are provided in Section 6.8 of this report. The estimated total settlement of a temporary RSS is approximately 580 mm and most of this settlement will be complete in about 2 months.

6.12 Seismic Requirements

The site is treated as lying in Seismic Zone 1. Reference to Annex A3.1 of the CHBDC 2006 indicates that the following seismic parameters (Matheson) should be used for design:

- | | |
|---------------------------------------|--------------------------|
| ■ Velocity Related Seismic Zone | 1 |
| ■ Zonal Velocity Ratio | 0.05 |
| ■ Acceleration Related Seismic Zone | 1 |
| ■ Zonal Acceleration Ratio | 0.05 |
| ■ Peak Horizontal Ground Acceleration | 0.08 g (10% in 50 years) |

The soil profile type at this site has been classified as Type IV and the Site Coefficient "S (ground motion amplification factor) that should be used in seismic design as per Clause 4.4.6.1, Table 4.4 of the CHBDC is 2.0. Culverts should be designed in accordance with Clause 7.5.5 of the CHBDC for a seismic event having a 10% probability of being exceed in 50 years. The vertical component of the earthquake acceleration ratio (A_v) shall be two-thirds of the horizontal ground acceleration ratio (A_h) and A_h shall be set equal to the zonal acceleration ratio.

6.13 Additional Studies

It is recommended that the following issues be considered during the future detail design studies.

- Confirm and further refine the preliminary geotechnical recommendations based on the selected option;
- Carry out detail level foundation investigations if deemed necessary; and
- Complete a more rigorous assessment of settlement and stability analyses of the preferred alternative.

7.0 CLOSURE

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

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Ontario Provincial Standard Specifications (OPSS)

OPSS.PROV 206	Construction Specification For Grading.
OPSS.PROV 209	Construction Specification For Embankments Over Swamps And Compressible Soils.
OPSS.PROV 212	Construction Specification For Earth Borrow.
OPSS 422	Construction Specification For Precast Reinforced Concrete Box Culverts And Box Sewers In Open Cut.
OPSS.PROV 501	Construction Specification For Compacting.
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 902	Construction Specification For Excavating and Backfilling – Structures.
OPSS 903	Construction Specification For Deep Foundations.
OPSS.PROV 1004	Material Specification For Aggregates – Miscellaneous.
OPSS.PROV 1010	Material Specification For Aggregates – Base, Subbase, Select Subgrade and Backfill Material.
OPSS 1205	Material Specification For Clay Seal.

Ontario Provincial Standard Drawings (OPSD)

MTOD 803.021	Bedding and Backfill For Precast Concrete Box Culverts.
OPSD 208.010	Benching Of Earth Slopes.
OPSD 3000.100	Foundation, Piles, Steel H-Pile Driving Shoe.
OPSD 3090.100	Foundation, Frost Penetration Depths For Northern Ontario.

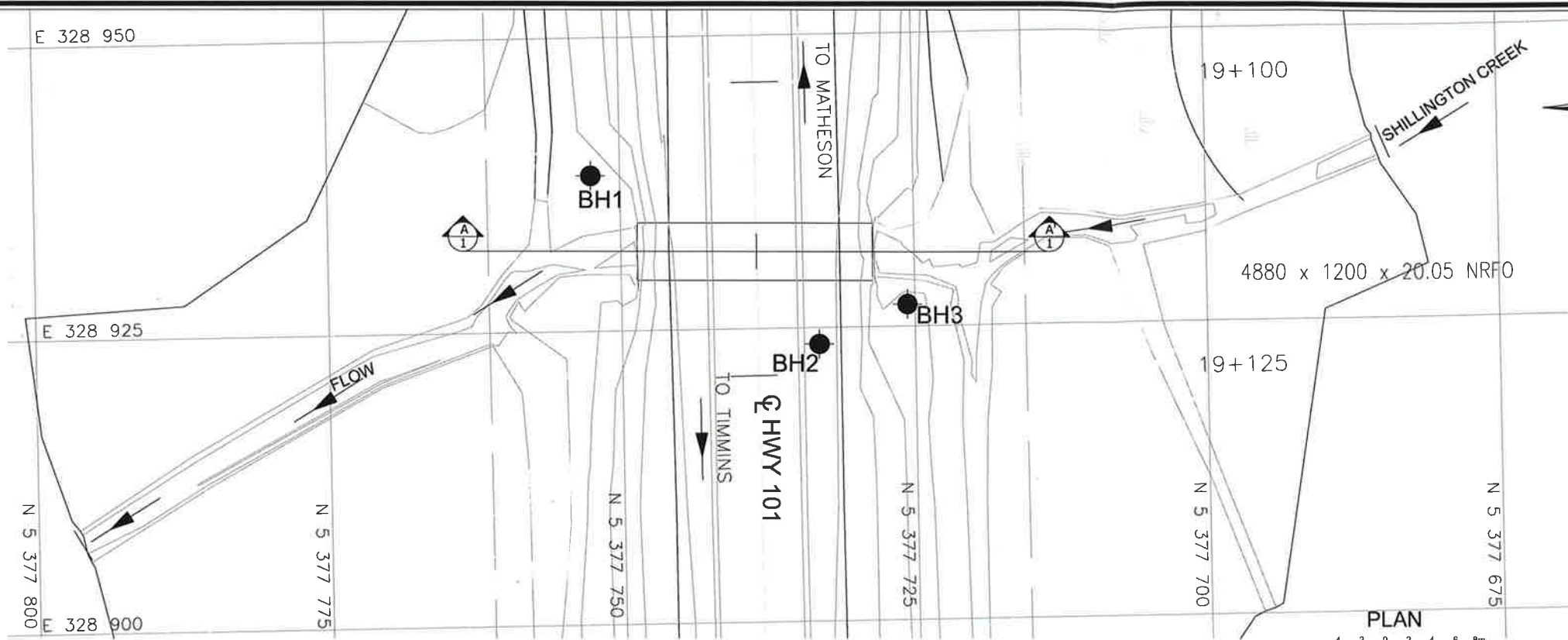


TABLE 1
COMPARISON OF CULVERT FOUNDATION ALTERNATIVES

Geogrid Reinforced Culvert Bedding (Box Culvert)	Pile Foundations (Open Footing Culvert)	Small Diameter Displacement Piles (Open Footing Culvert)
<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Precast culvert units can be used facilitating easy transportation, handling and placement.Shallower excavation depth, reduced excavation volume and reduced dewatering requirements compared to other foundation alternatives. <p>Disadvantages:</p> <ul style="list-style-type: none">Settlement will be greater than 25 mm and a culvert camber is required.Good construction techniques required in order to ensure leak proof reliable joints between units during and after settlement.Higher level of construction effort required to construct camber to accommodate settlement.Dewatering required.	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Precast culvert units can be used facilitating easy transportation, handling and placement.Total and differential settlement will be less than 25 mm and 20 mm respectively.Culvert camber not required. <p>Disadvantages:</p> <ul style="list-style-type: none">More expensive than geogrid reinforced culvert bedding.Requires a relatively heavy pile driving rig to install piles.Requires struts between pile caps to provide lateral structural resistance.Dewatering required and a working mat is required to support pile driving equipment in areas where weak soils exist.Relatively large and deep excavation required compared to a geogrid reinforced culvert bedding.	<p>Advantages:</p> <ul style="list-style-type: none">Reliable performance expected.Precast culvert units can be used facilitating easy transportation, handling and placement.Total and differential settlement will be less than 25 mm and 20 mm respectively.Culvert camber not required.Minimal vibrations created during the pile driving process such that piles can be installed as close as 450 mm away from existing culvert.Light excavating equipment can be used compared to heavy pile driving equipment.Lower equipment and set-up costs compared to a conventional pile driving rig.High productivity, up to 400 m per piling rig per day.Pile load bearing capacity proven during the pile driving process for every pile installed.Innovative solution. <p>Disadvantages:</p> <ul style="list-style-type: none">More expensive than geogrid reinforced culvert bedding.Requires braces between pile caps to provide lateral structural resistance.
<p>Risks/Consequences</p> <ul style="list-style-type: none">Low risk of bearing capacity failure.Low risk that actual settlement may be greater or less than predicted and may not be equal to design culvert camber.	<p>Risks/Consequences</p> <ul style="list-style-type: none">Very low risk of bearing capacity failure.Very low risk that total settlement will exceed 25 mm.	<p>Risks/Consequences</p> <ul style="list-style-type: none">Very low risk of bearing capacity failure.Very low risk that total settlement will exceed 25 mm.
<p>Approximate Cost*</p> <p>\$20,000.00</p>	<p>Approximate Cost*</p> <p>\$160,000.00</p>	<p>Approximate Cost*</p> <p>\$160,000.00</p>

DRAWINGS





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

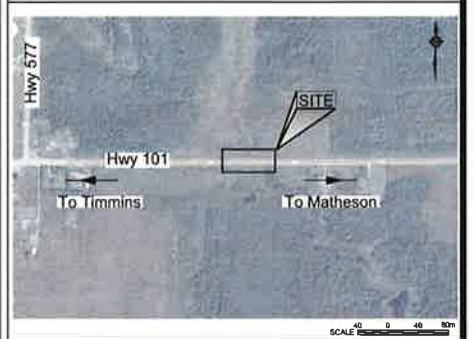
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HWY 101
SHILLINGTON CREEK CULVERT
BOREHOLE LOCATIONS AND SOIL STRATA

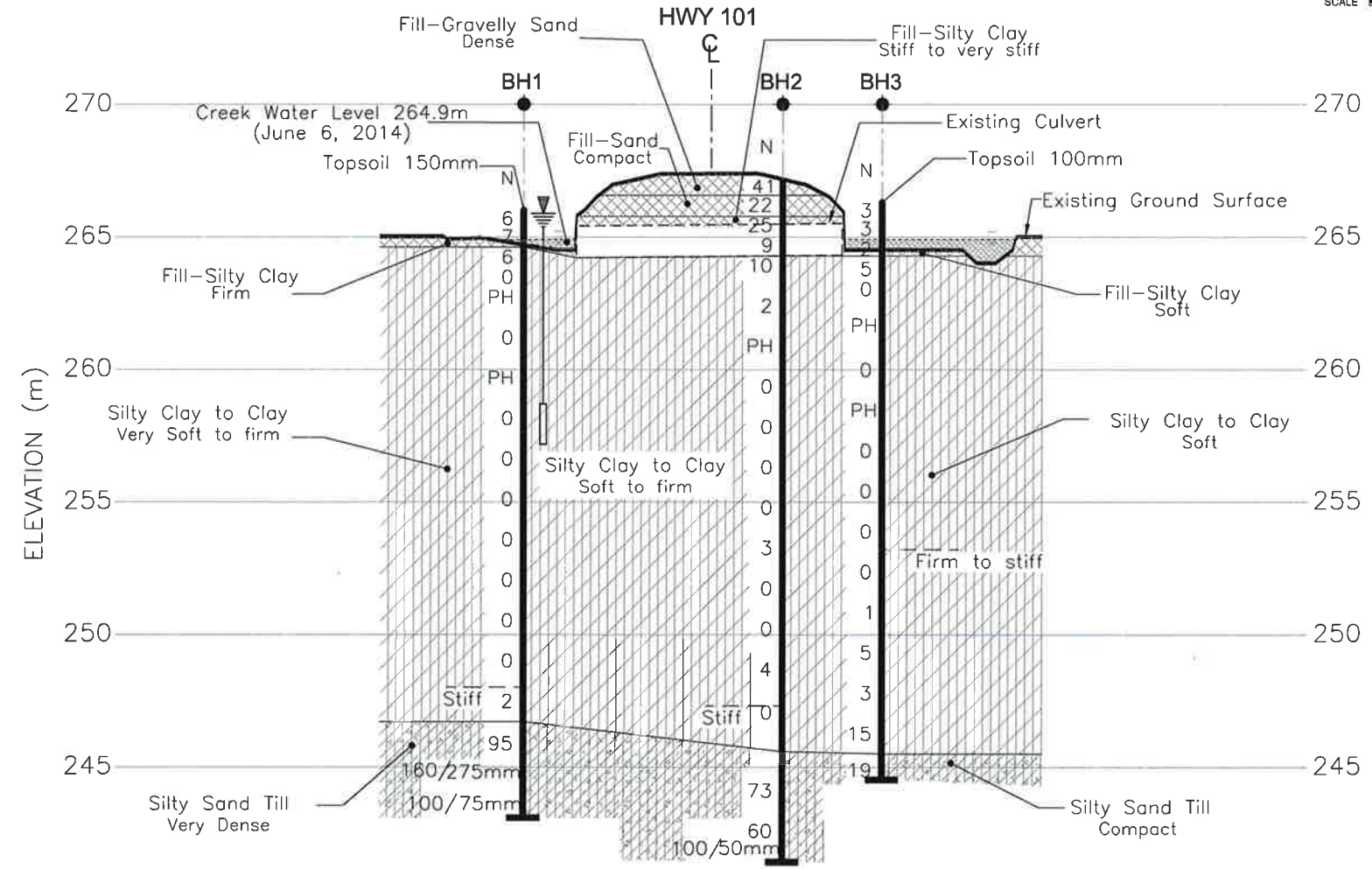
SHEET

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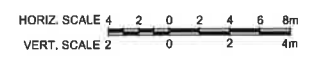
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KEY PLAN



SECTION A-A' CULVERT AT STATION 19+114
HIGHWAY 101



LEGEND	
	Bore Hole
	Dynamic Cone Penetration Test
	Bore Hole And Cone
	Blows/0.3m (Std Pen Test, 475 J/blow)
	Blows/0.3m (60' Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer (October 2014)
	Piezometer
	90% Rock Quality Designation
	Auger Refusal

No	ELEV.	COORDINATES	
		NORTHING	EASTING
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

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 2.01 of OPS General Conditions.

REFERENCE
Drawings provided in digital format by MMM Group Ltd. by CD (Assignment 5013-E-0018 Preliminary Design for Rehab/Replacement of 12 Structures on Highways in New Liskeard Area) drawing files B7770101002, DTM7770101002, received September 11, 2014.

REVISIONS		
DATE	BY	DESCRIPTION

HWY. 101	PROJECT No. 11-14-4066	GEORES No. 42A-103
SUBM'D. HA	CHKD. RA	DATE: November 2014
DRAWN: KC	CHKD. RA	APPD: MT

APPENDIX A

Record of Borehole Sheets



EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No 1

1 of 2

METRIC

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		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT		UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)		WATER CONTENT (%)			
								<div><div>○ UNCONFINED</div><div>● QUICK TRIAXIAL</div></div>	<div><div>+ FIELD VANE</div><div>× LAB VANE</div></div>	<div><div>W_p</div><div>W</div><div>W_L</div></div>	<div><div>52</div><div>LL=41</div></div>		
266.0	GROUND SURFACE						20 40 60 80 100		10 20 30				
265.8	150mm TOPSOIL		1	SS	6								GR SA SI CL
0.2	FILL, silty clay, some sand, trace gravel, containing organics, firm, dark brown, wet		2	SS	7								
264.6	trace sand, very soft to firm, brown, moist to wet		3	SS	6								
1.4	...		4	SS	0*								0 1 41 58
	SILTY CLAY to CLAY (varved), soft to firm, grey, wet		5	TW	PH								
			6	SS	0*								
			7	TW	PH								
			8	SS	0*								0 0 19 81
			9	SS	0*								0 0 29 71
	containing 4mm to 20mm thick silt layers		10	SS	0*								
			11	SS	0*								
			12	SS	0*								

Continued Next Page


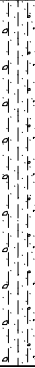
+³, ×³: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

RECORD OF BOREHOLE No 1

2 of 2

METRIC

G.W.P. 5165-12-00 LOCATION Coords: E: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			PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)			WATER CONTENT (%)						
								20	40	60	80	100	W _p			W	W _L
	(continued)																
	SILTY CLAY to CLAY (varved), containing 4mm to 20mm thick silt layers, soft to firm, grey, wet		13	SS	0*												
					14	SS	0*										
	...																
	stiff, moist																
246.7																	
19.3	SILTY SAND, trace clay, trace gravel, very dense, grey, moist to wet (GLACIAL TILL)																
					16	SS	95										
243.1																	
22.9																	
			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

RECORD OF BOREHOLE No 2

1 of 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	PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)						WATER CONTENT (%)			
								20	40	60	80			100	W _P	W	W _L
267.2	GROUND SURFACE							○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE								
266.6	600mm FILL-GRAVELLY SAND, trace silt, dense, brown, dry		1	SS	41		267										
0.6	FILL, sand, some gravel, trace silt, compact, brown, dry		2	SS	22												
265.8	FILL, silty clay and sand, stiff to very stiff, brown, moist		3	SS	25		266										
1.4	... containing organics, trace sand, dark brown		4	SS	9		265										
264.3	trace sand, trace organics, stiff to very stiff		5	SS	10		264										
2.9	...		6	SS	2		263										
			7	TW	PH		262										
	SILTY CLAY to CLAY (varved), containing 1mm to 20mm thick silt layers, soft to firm, grey, wet		8	SS	0*		261										
			9	SS	0*		260										
			10	SS	0*		259										
			11	SS	0*		258										
			12	SS	3		257										
							256										
							255										
							254										
							253										

Continued Next Page

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

RECORD OF BOREHOLE No 3

1 of 2

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		PLASTIC LIMIT NATURAL MOISTURE CONTENT		UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100	20 40 60 80 100	W _p W	W _L		
266.3	GROUND SURFACE												
	100mm TOPSOIL		1	SS	3		266						
	FILL , silty clay, some sand, trace gravel, trace organics, soft, brown, wet		2	SS	3								2 11 27 60
			3	SS	2		265						
264.2			4	SS	5		264						
2.1	trace sand, trace gravel, trace organics, firm ...		5	SS	0*		263						0 0 35 65
			6	TW	PH		262						0 0 33 67
	SILTY CLAY to CLAY (varved), soft, firm to stiff below 13.1m, grey, wet		7	SS	0*		261						
			8	TW	PH		260						0 0 34 66
			9	SS	0*		259						
			10	SS	0*		258						
	containing 2mm to 25mm thick silt layers		11	SS	0*		257						0 0 16 84
			12	SS	0*		256						
							255						
							254						
							253						
							252						

Continued Next Page

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

METRIC

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SPT 'N' VALUE	SHEAR STRENGTH (kPa)						WATER CONTENT (%)			
								20	40	60	80			100	W _p	W	W _L
	(continued)																
	SILTY CLAY to CLAY (varved), containing 2mm to 25mm thick silt layers, firm to stiff, grey, wet		13	SS	1		251										
							250										
			14	SS	5		249										
							248										
			15	SS	3		247										
							246										
			16	SS	15		245										
245.5 20.8	SILTY SAND, trace clay, trace gravel, compact, grey, wet (GLACIAL TILL)																
244.5 21.8			17	SS	19												

Artesian conditions 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.

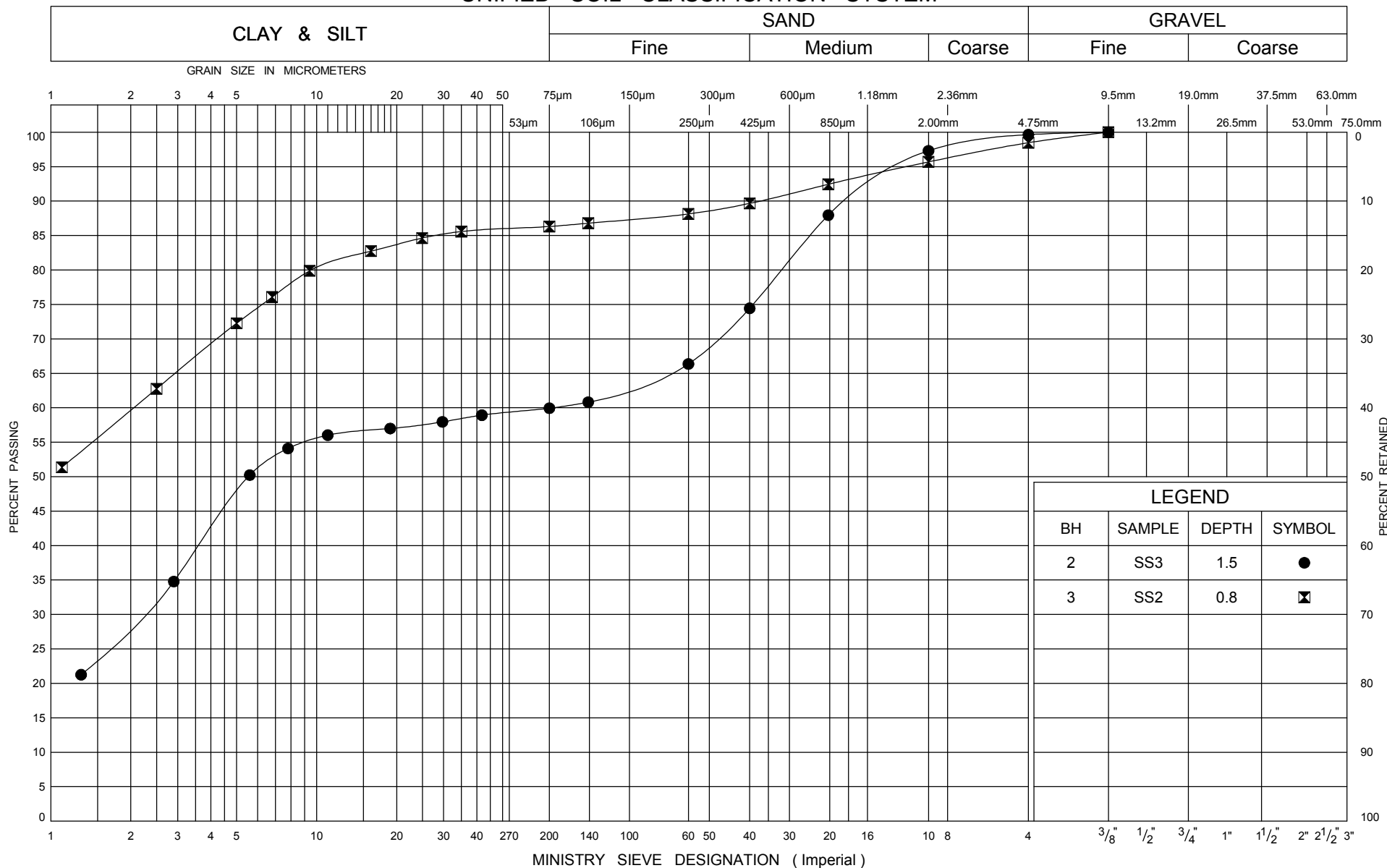
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APPENDIX B

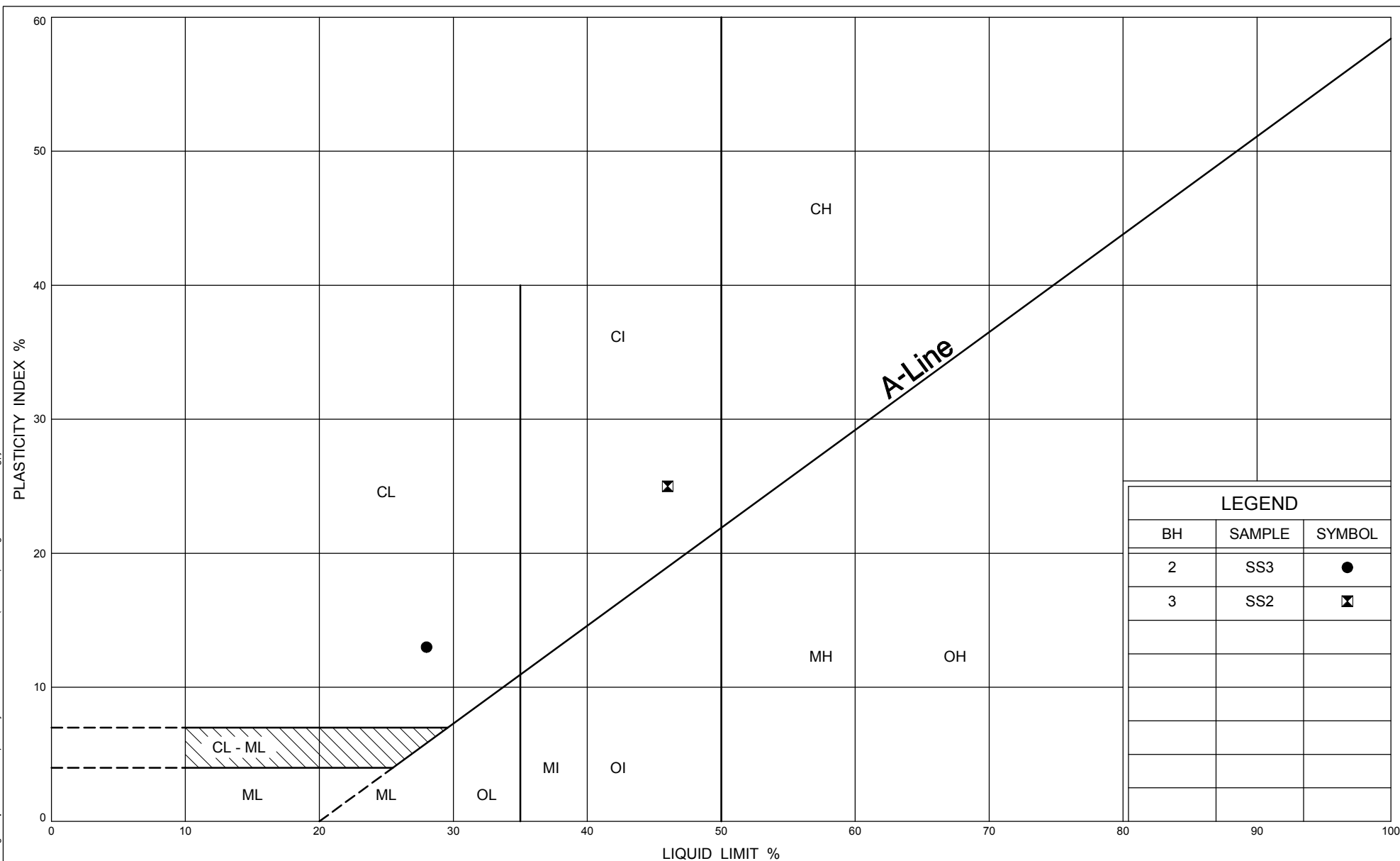
Field & Laboratory Test Results & Photographs



UNIFIED SOIL CLASSIFICATION SYSTEM



library: library - terraprobe.gint - md.glb report: mto-terra-plasticity-chart file: 11-14-4066 (39e-214c) shillington creek culvert.gpj



Ministry of
Transportation

PLASTICITY CHART

FILL-SILTY CLAY

FIG No B2

G W P 5165-12-00

Shillington Creek Culvert (39E-214C)

VARVED SILTY CLAY TO CLAY SAMPLES

FIGURE B3

SHILLINGTON CREEK CULVERT(Site 39E-214C)

Silty Clay to Clay



BH2 SS12



BH3 SS12

Z:\1-Project Files\11-Geo\2014\11-14-4066 New Likard Area\5- Shillington Creek Culvert Hwy 101 (39E-214)\Eng Analysis\Spread Sheets\le0-Pc-Cc-Cr-Cu.xls

Project No. : 11-14-4066

Date : December 2014



Terraprobe Inc.

Prepared by : SD

Checked by : RA

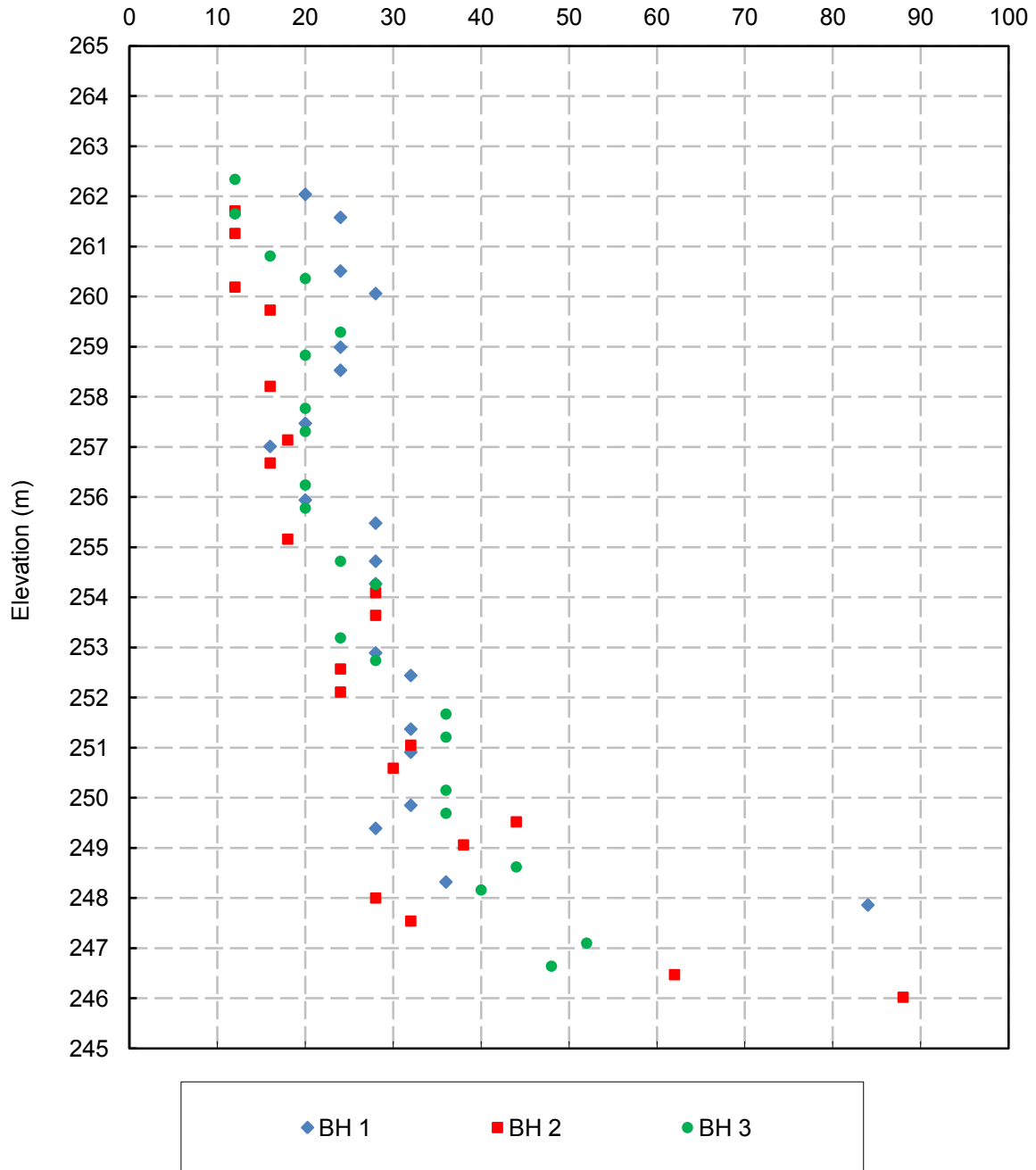
UNDRAINED SHEAR STRENGTH

FIGURE B4

SHILLINGTON CREEK CULVERT (Site 39E-214C)

Silty Clay to Clay

Cu (kPa)



Project No. : 11-14-4066

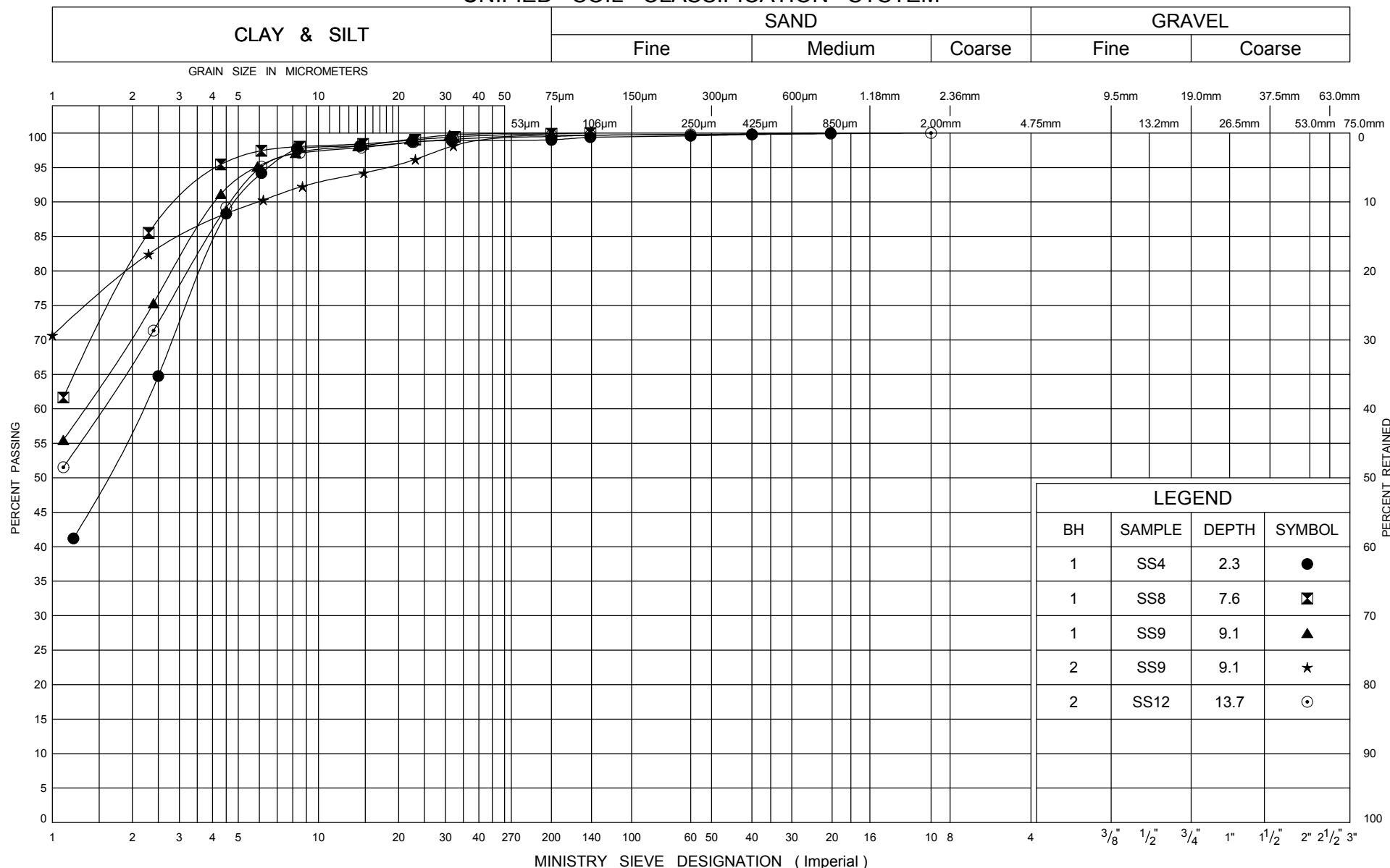
Date : December, 2014



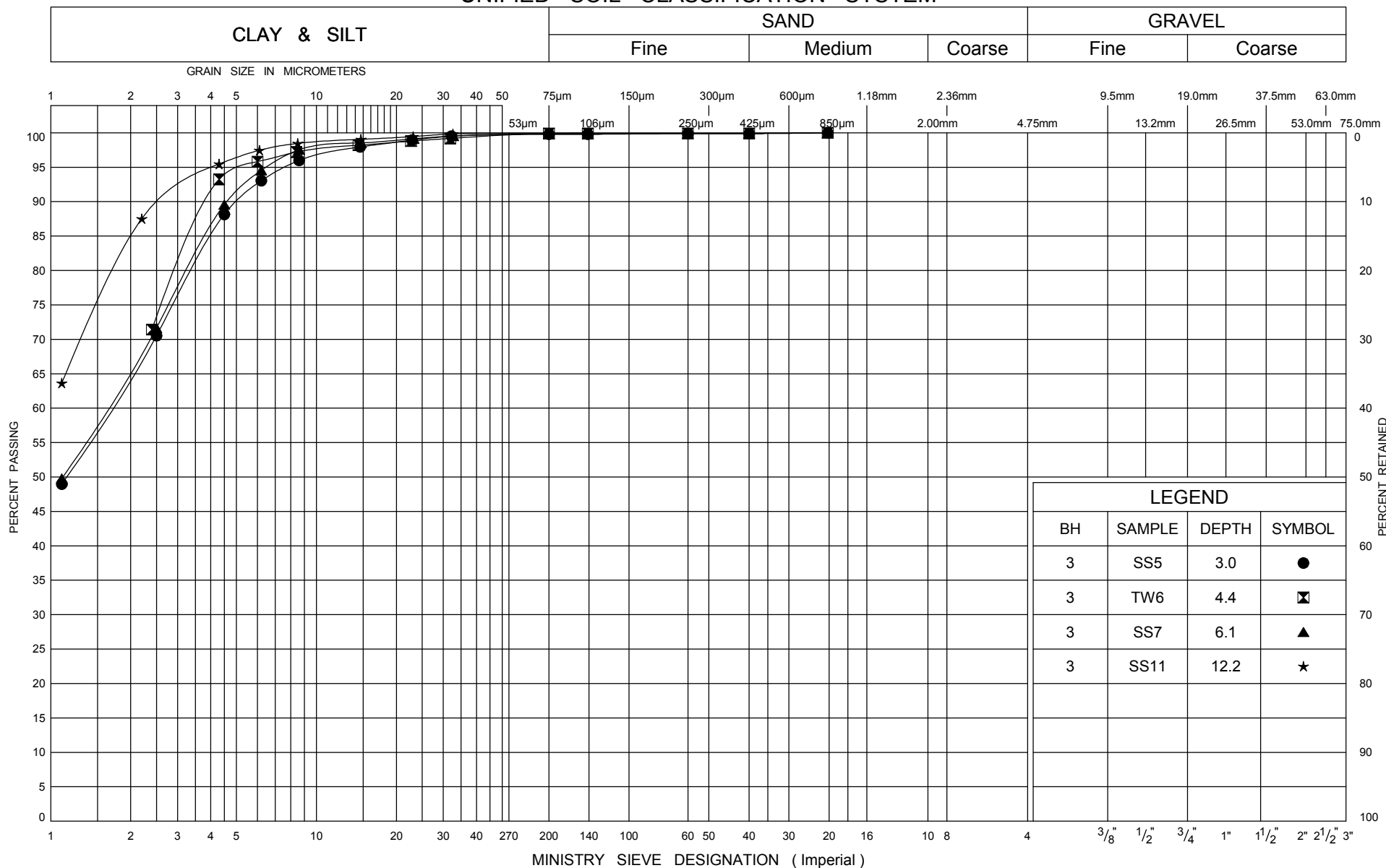
Prepared by : SD

Checked by : RA

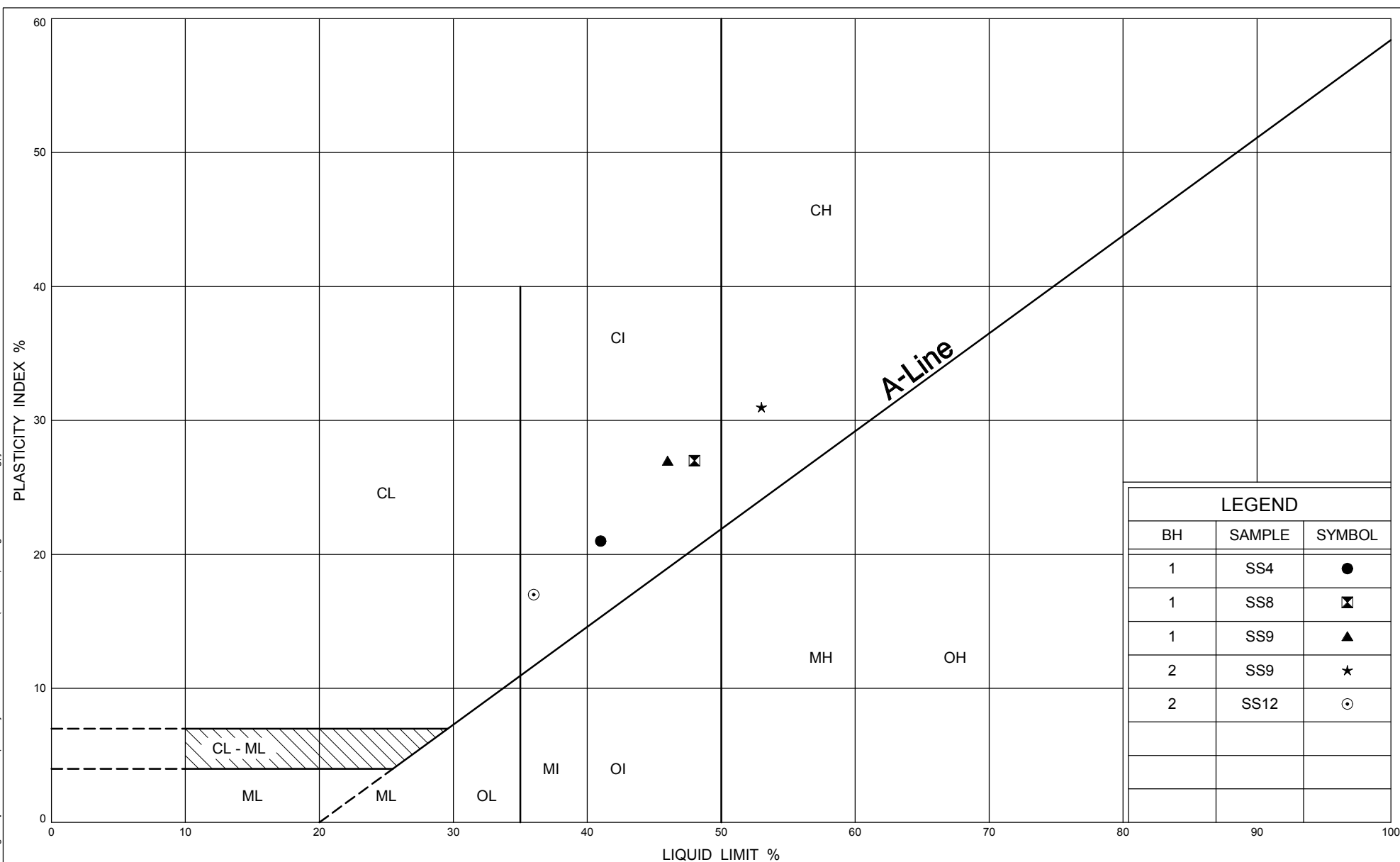
UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



library: library - terraprobe.gint - md.glb report: mto-terra-plasticity-chart file: 11-14-4066 (39e-214c) shillington creek culvert.gpj



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Transportation

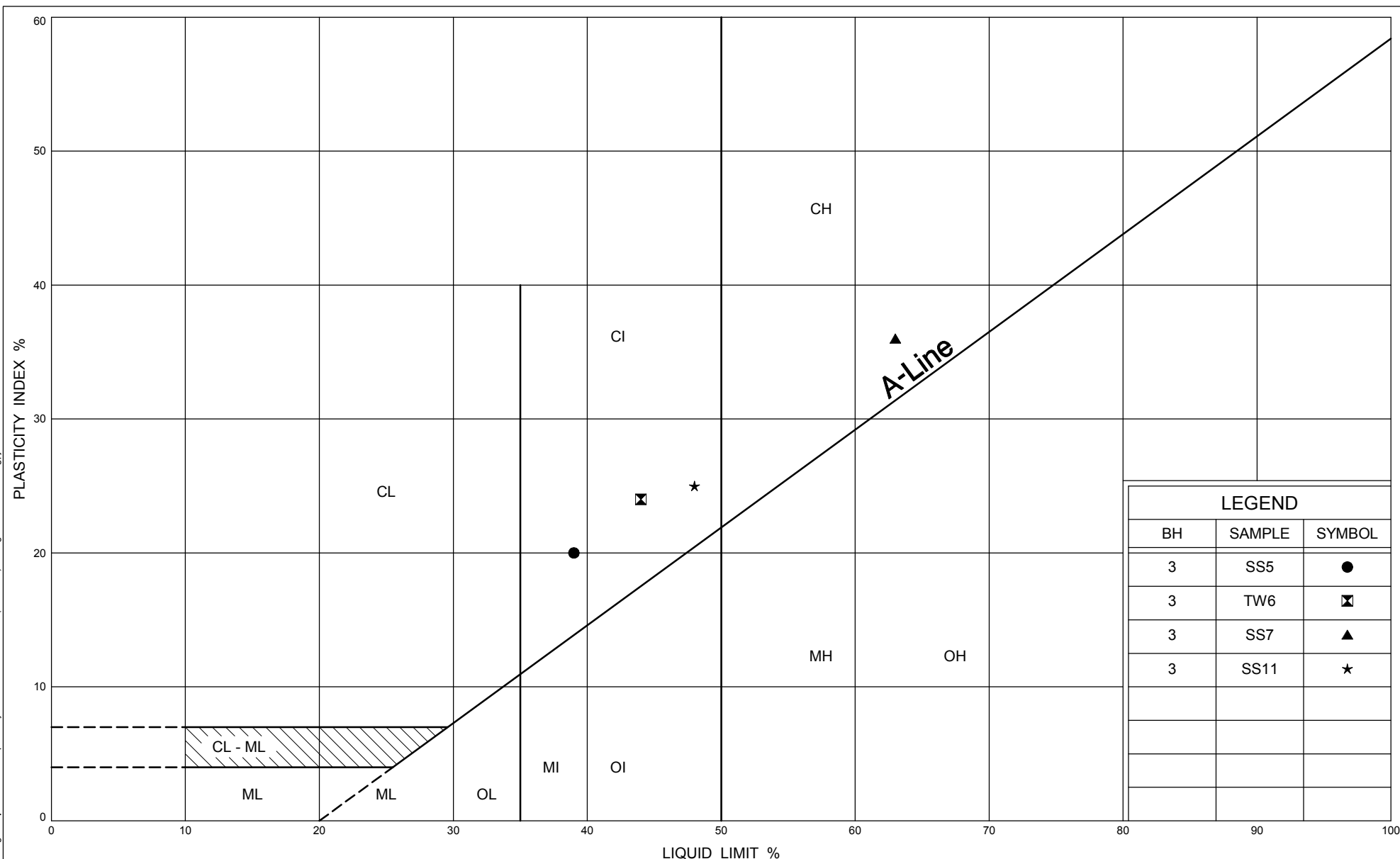
PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B7

G W P 5165-12-00

Shillington Creek Culvert (39E-214C)

library: library - terraprobe.gint - md.glb report: mto-terra-plasticity-chart file: 11-14-4066 (39e-214c) shillington creek culvert.gpj



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Transportation

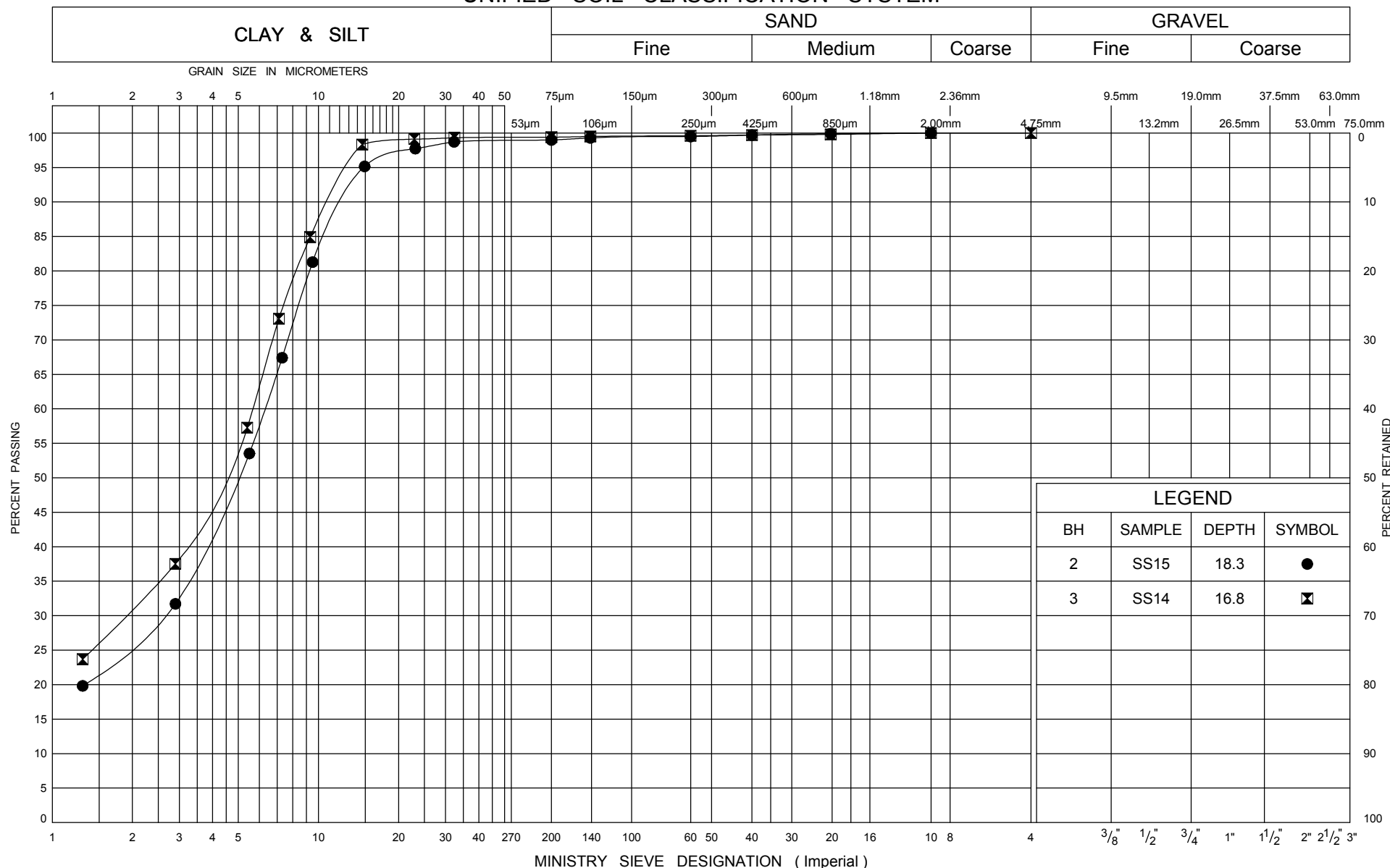
PLASTICITY CHART SILTY CLAY TO CLAY

FIG No B8

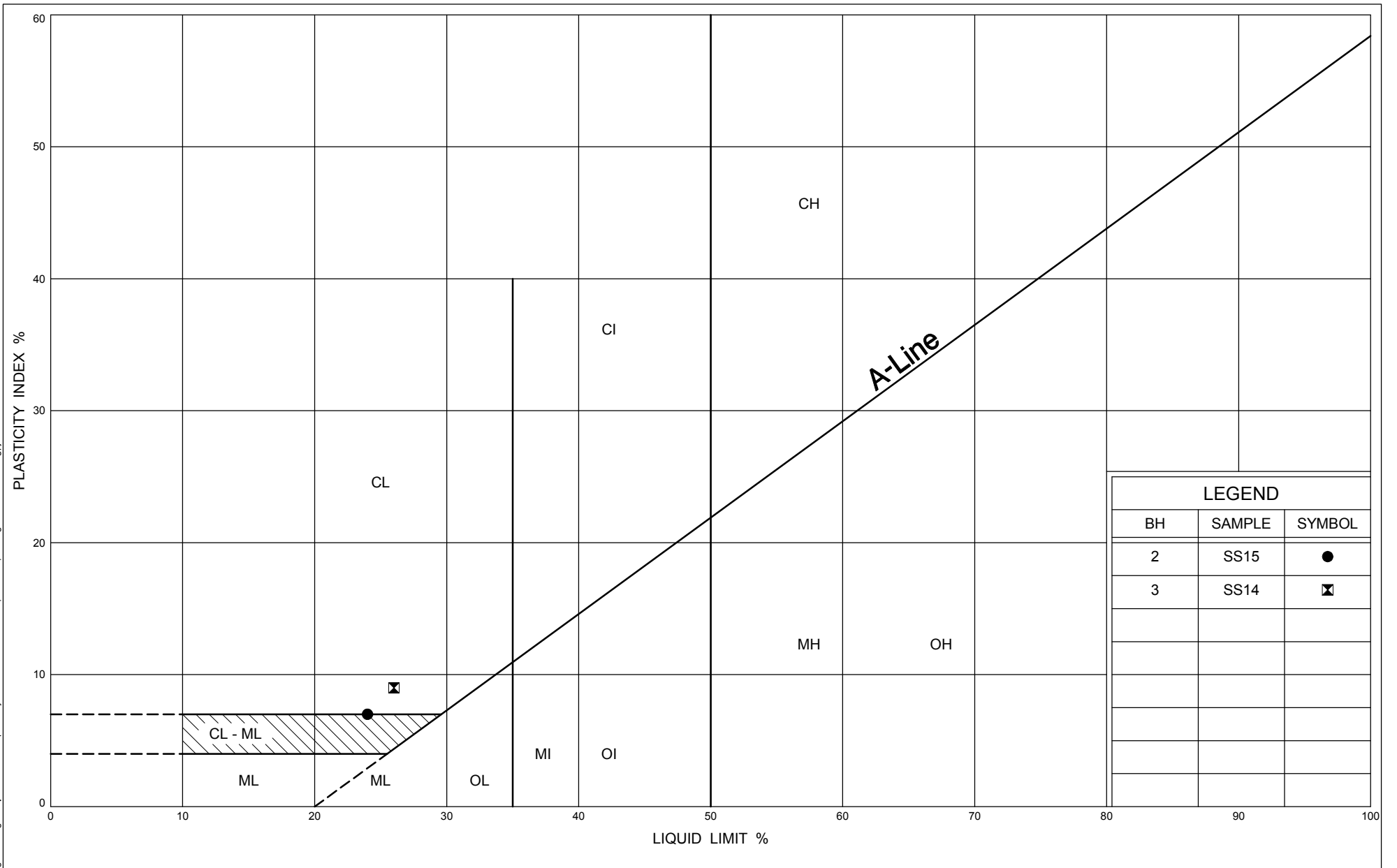
G W P 5165-12-00

Shillington Creek Culvert (39E-214C)

UNIFIED SOIL CLASSIFICATION SYSTEM



library: library - terraprobe.gint - md.glb report: mto-terra-plasticity-chart file: 11-14-4066 (39e-214c) shillington creek culvert.gpj



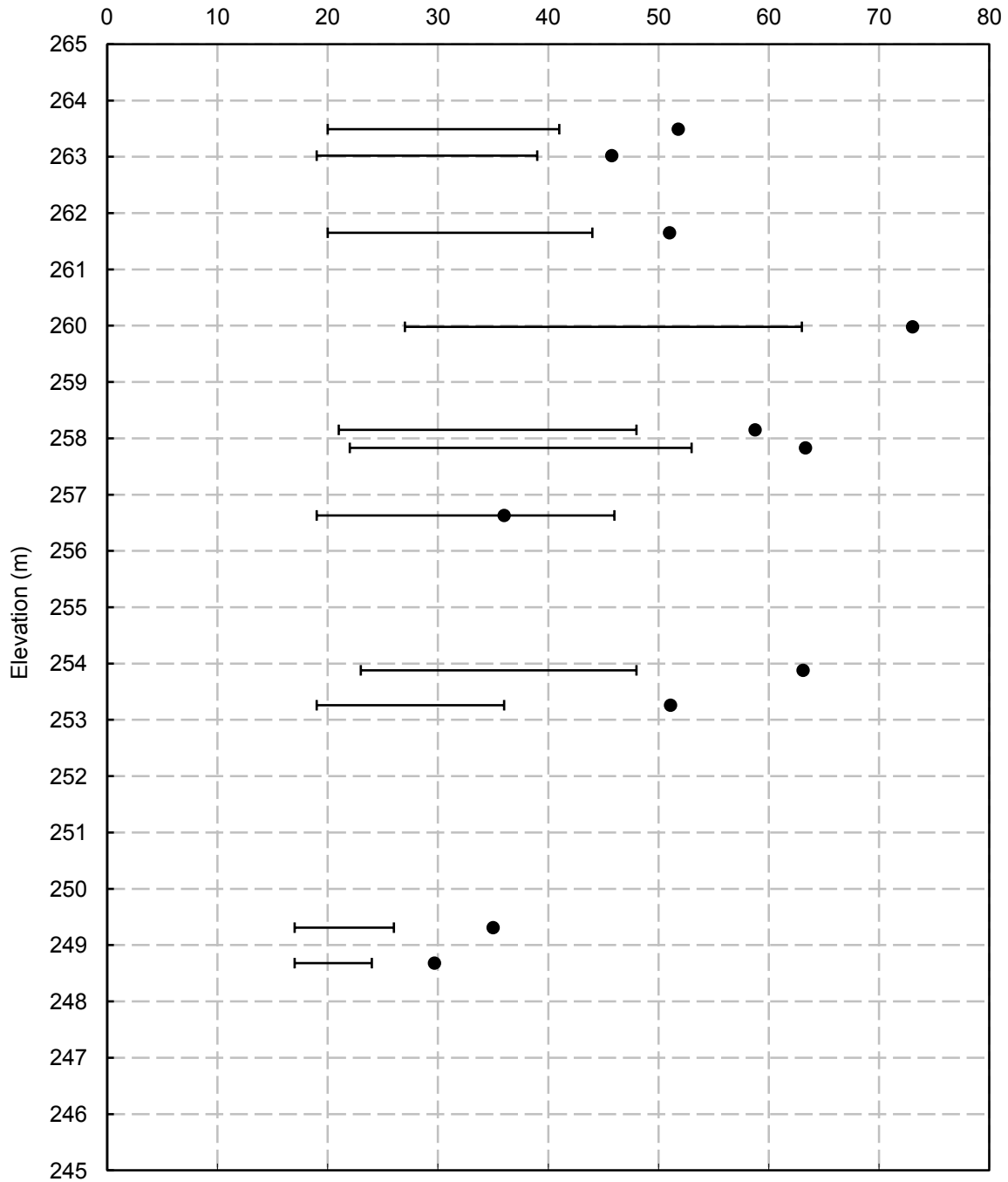
ATTERBERG LIMITS AND WATER CONTENTS

FIGURE B11

SHILLINGTON CREEK CULVERT(Site 39E-214C)

Silty Clay to Clay

Atterberg Limits & Water Contents (%)



Project No. : 11-14-4066

Date : December 2014



Prepared by : SD

Checked by : RA

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CONSOLIDATION TEST SUMMARY				FIGURE B12			
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	Initial Height	Final Height	Void Ratio	t ₉₀	C _v	m _v	k
(kPa)	(mm)	(mm)		(min)	(cm ² /s)	(m ² /kN)	(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. : 11-14-4066				Prepared By :		SD	
Date : December 2014				Checked By :		RA	



Terraprobe Inc.

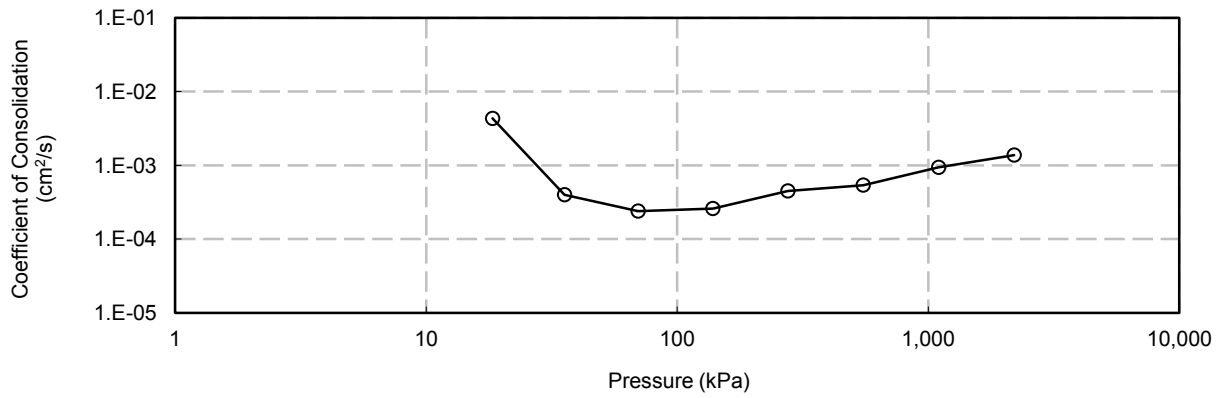
CONSOLIDATION TEST

FIGURE B13

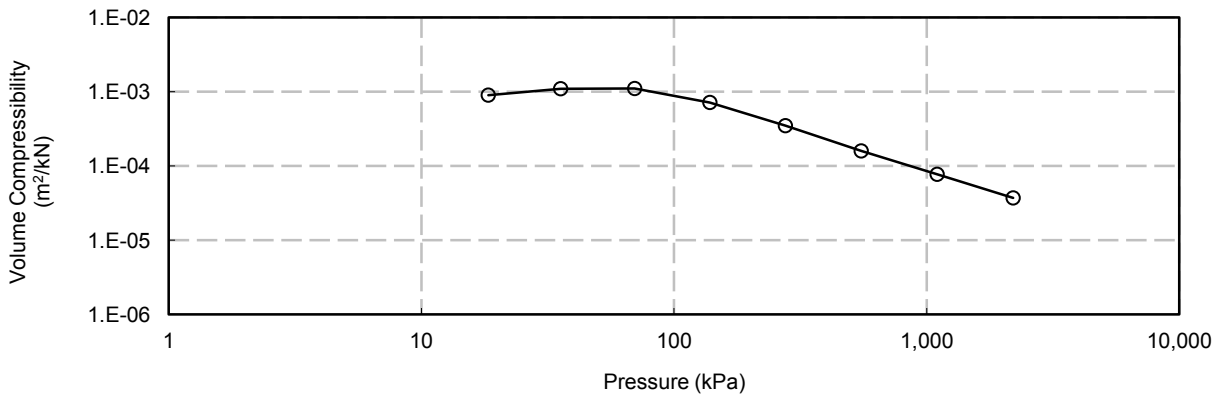
SHILLINGTON CREEK CULVERT (Site 39E-214C)

BH 3, TW 6

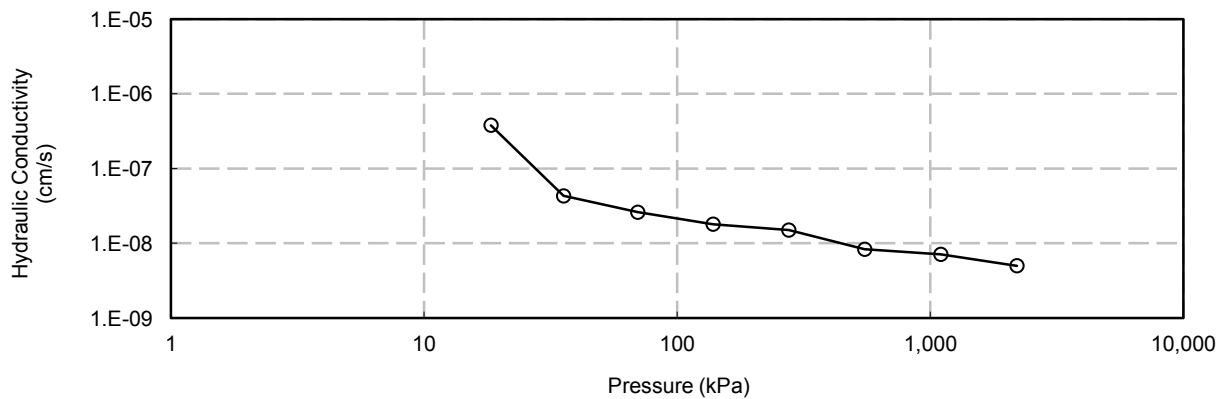
Cv vs Pressure



mv vs Pressure



k vs Pressure



Project No. : 11-14-4066
Date : December 2014



Terraprobe Inc.

Prepared By : SD
Checked By : RA

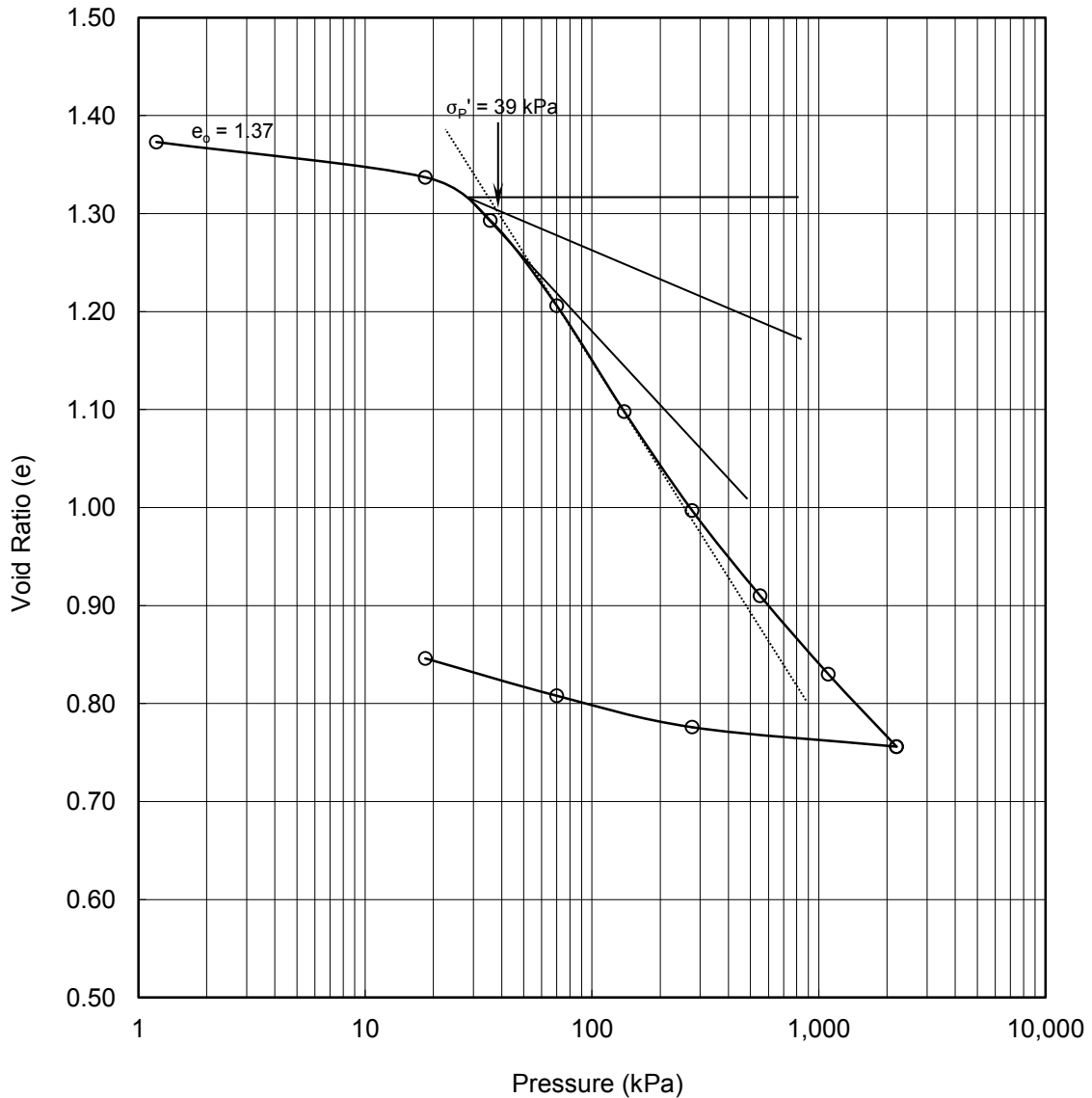
CONSOLIDATION TEST

FIGURE B14

SHILLINGTON CREEK CULVERT(Site 39E-214C)

BH 3, TW 6

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

$e_o =$	1.37	$\omega_L =$	44%	$\sigma_{v0}' =$	36.7 kPa
$\omega =$	51%	$\omega_P =$	20%	$\sigma_P' =$	39.0 kPa
$\gamma =$	16.5 kN/m ³	PI =	24%		
Gs =	2.70				

Project No. : 11-14-4066
Date : December 2014



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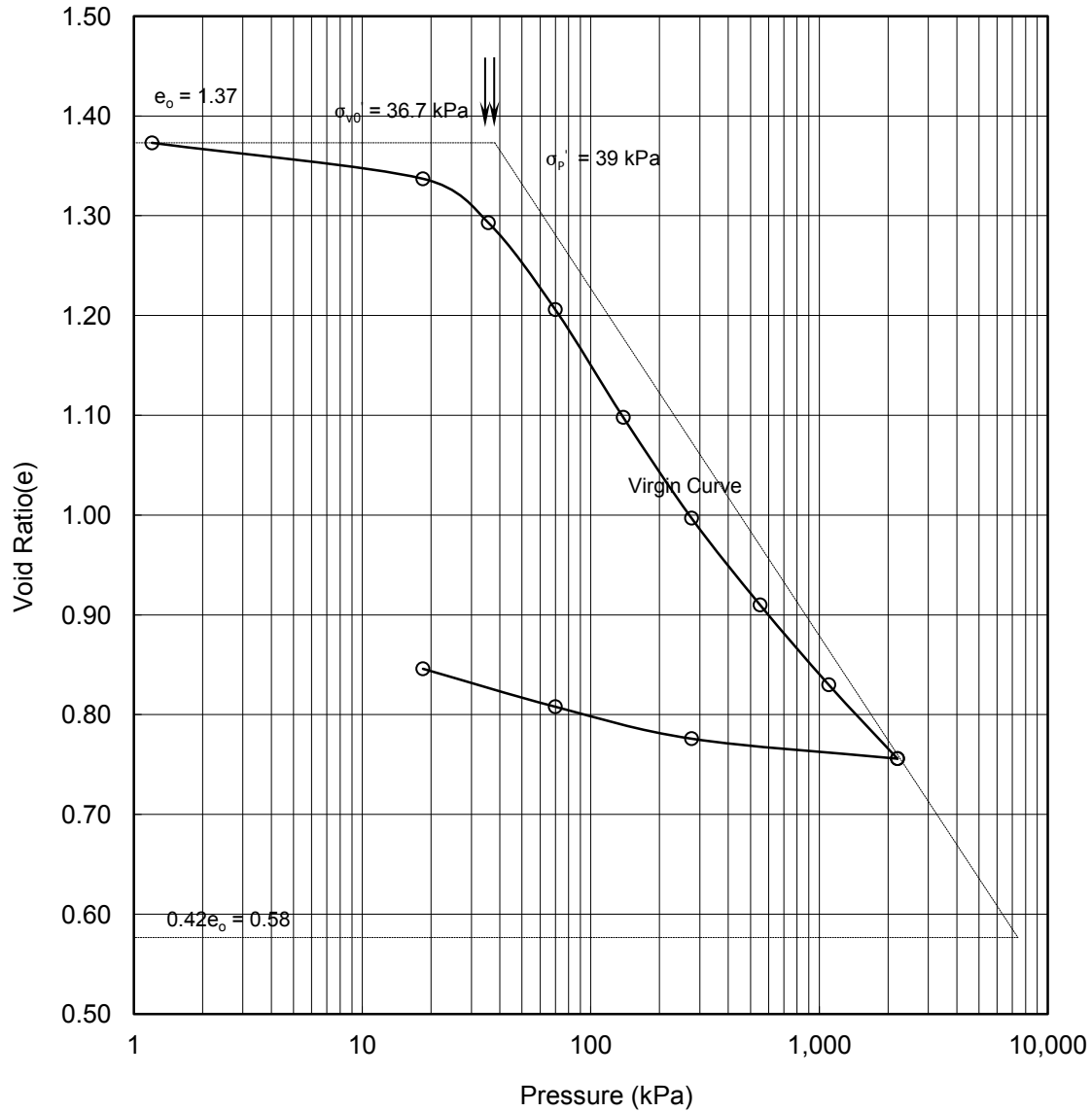
CONSOLIDATION TEST

FIGURE B15

SHILLINGTON CREEK CULVERT(Site 39E-214C)

BH 3, TW 6

Void Ratio vs Pressure



Soil Type : SILTY CLAY to CLAY

$e_o =$	1.37	$\omega_L =$	44%	$\sigma_{v0}' =$	36.7 kPa
$\omega =$	51%	$\omega_P =$	20%	$\sigma_P' =$	39.0 kPa
$\gamma =$	16.5 kN/m ³	PI =	24%	$C_c =$	0.348
Gs =	2.70			$C_r =$	0.043

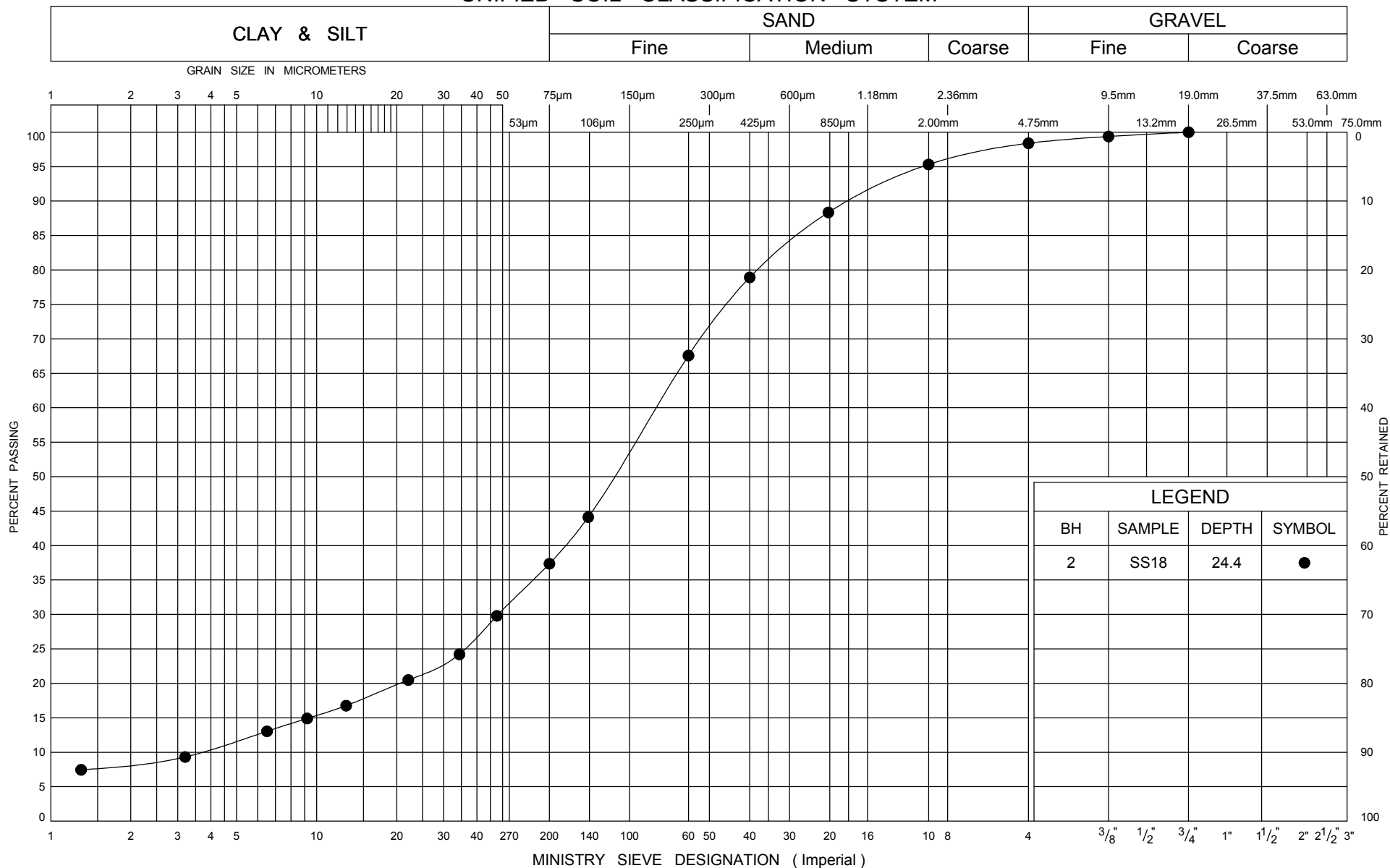
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Date : December 2014



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UNIFIED SOIL CLASSIFICATION SYSTEM



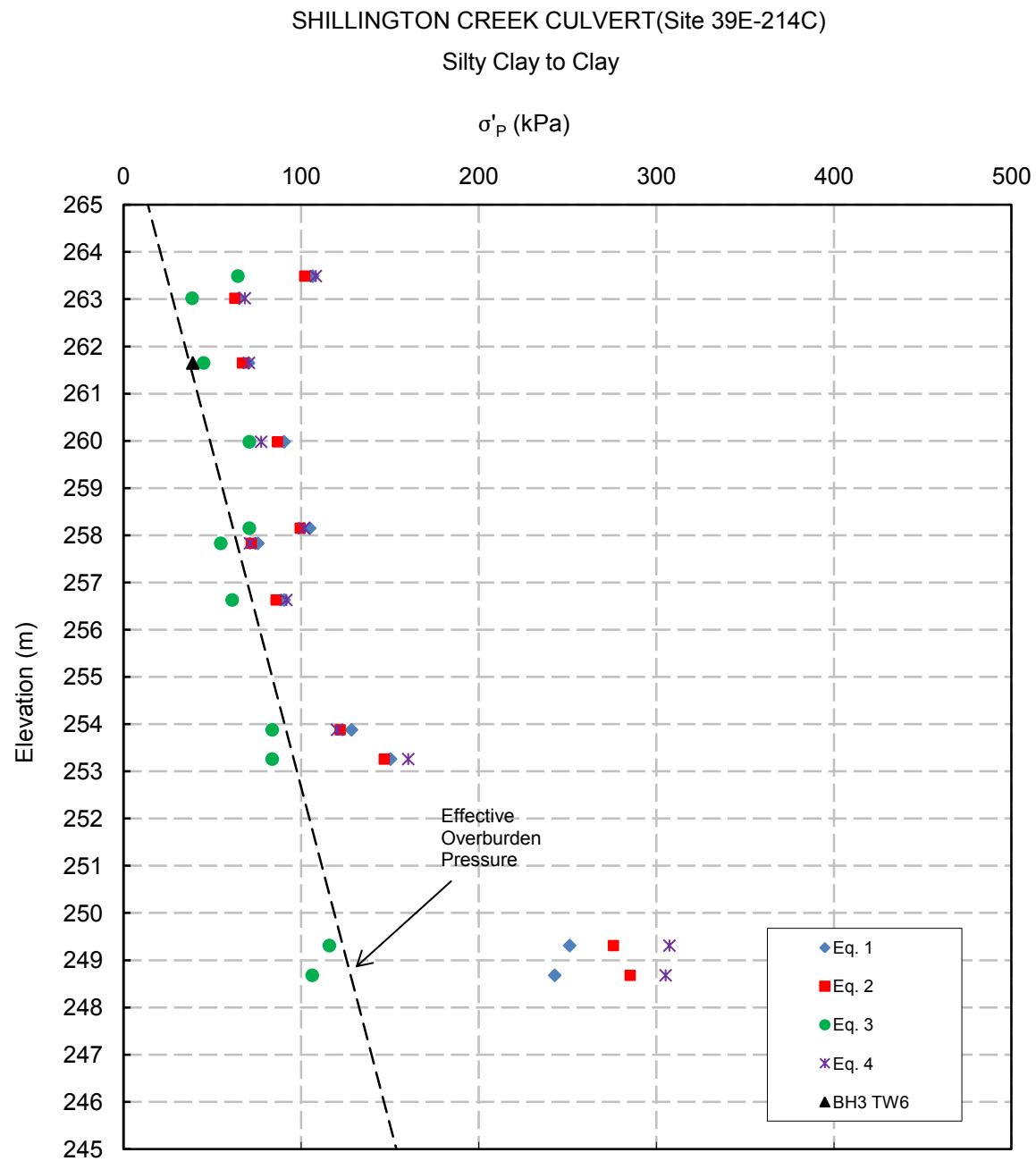
APPENDIX C

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)

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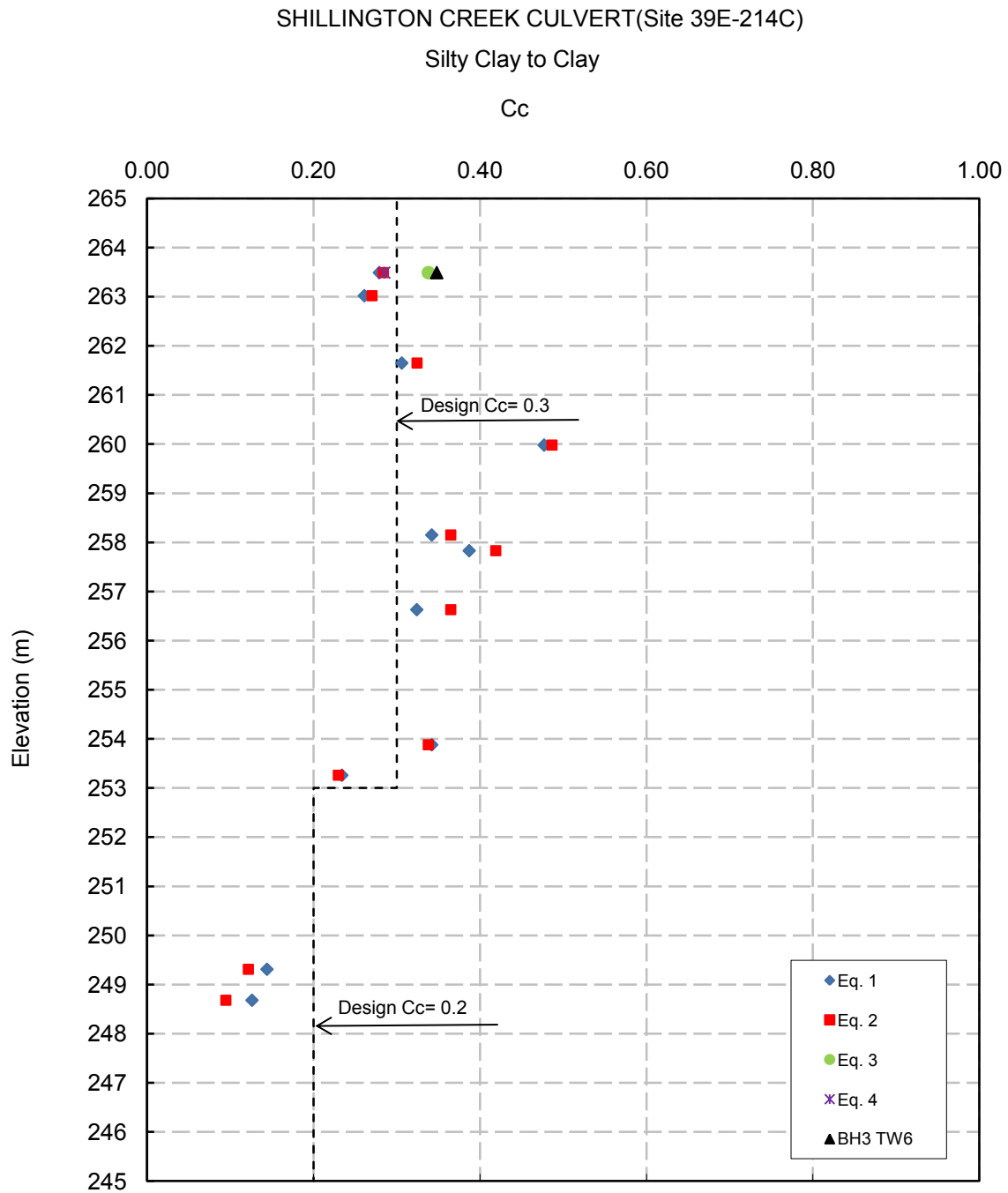
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PREDICTED AND MEASURED COMPRESSION INDICES

FIGURE C2



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

Eq. 2 $Cc = I_p / 74$

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

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

Terzaghi & Peck (1967)

Kulhaway & Mayne (1990)

Rendon - Herrero (1983)

Herrero (1983)

Project No. : 11-14-4066

Date : January, 2015



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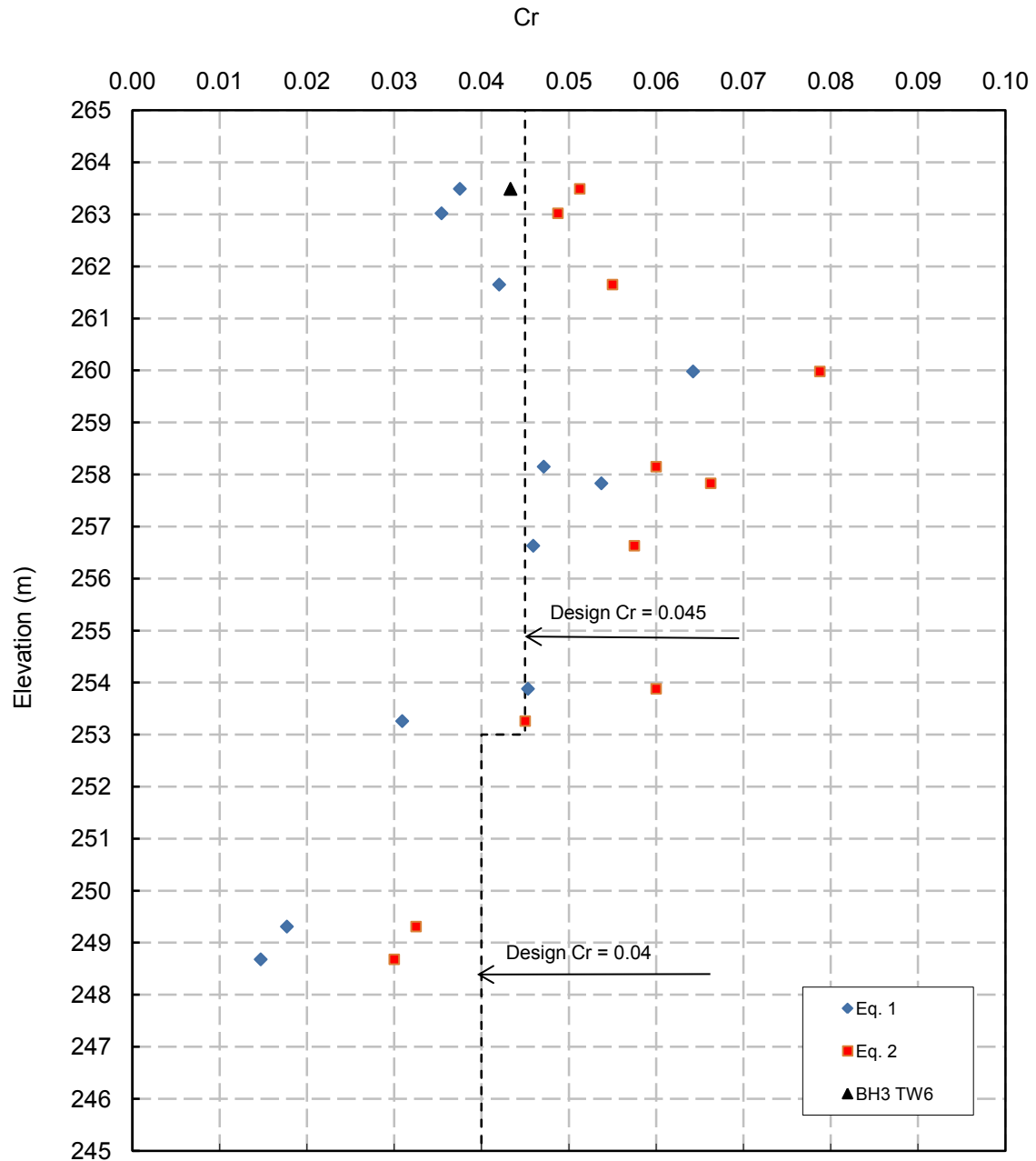
Checked by : RA

PREDICTED AND MEASURED RECOMPRESSION INDICES

FIGURE C3

SHILLINGTON CREEK CULVERT(Site 39E-214C)

Silty Clay to Clay



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

Das (1993)

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

Nagaraj & Murty (1985)

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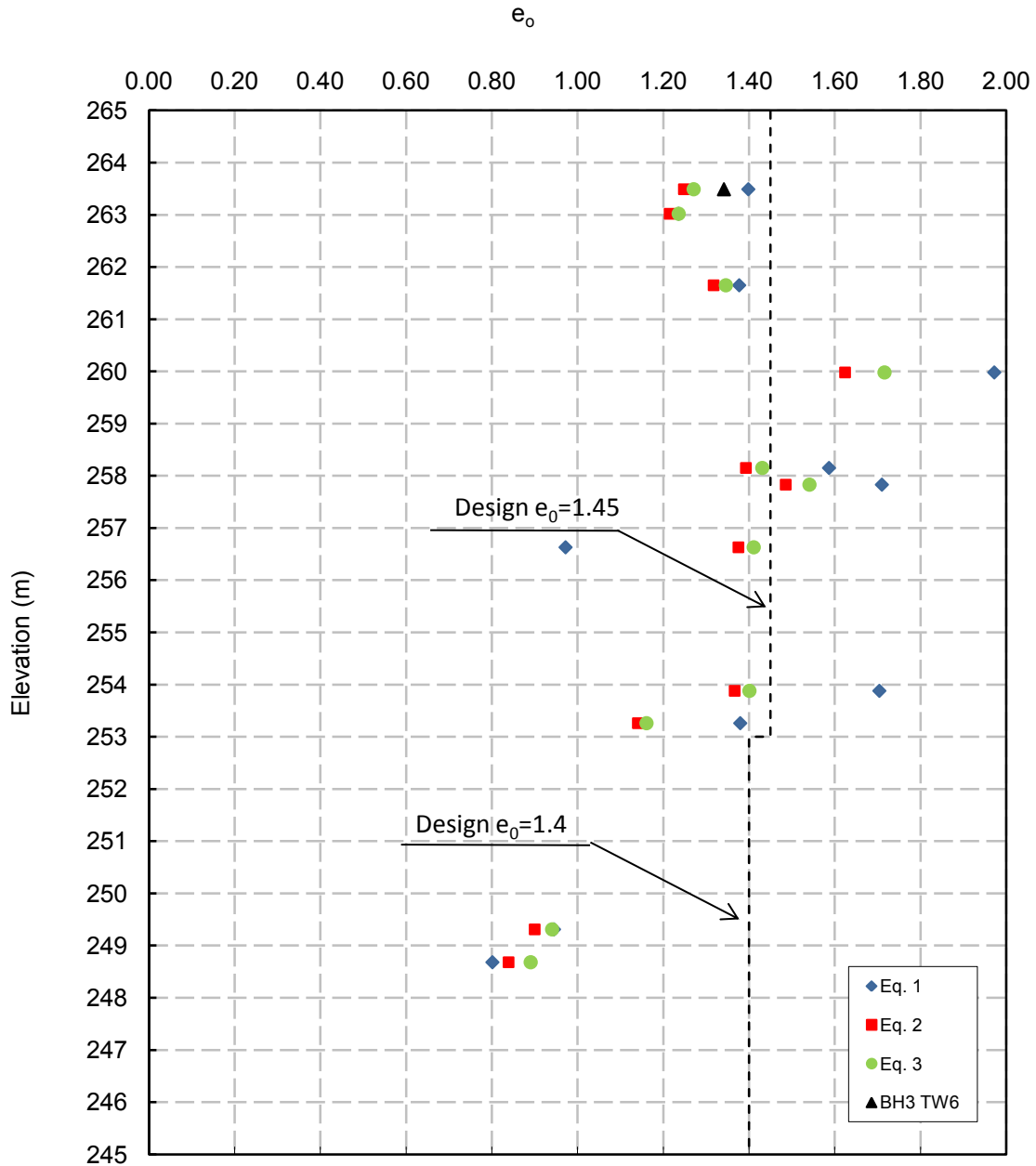
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PREDICTED AND MEASURED VOID RATIOS

FIGURE C4

SHILLINGTON CREEK CULVERT(Site 39E-214C)

Silty Clay to Clay



Eq. 1 $e_o = w * G_s$

when saturated

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

derived from Rendon - Herrero (1983)

Eq. 3 $e_o = C_c / 0.30 + 0.27$

derived from Hough (1957)

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Date : January, 2015



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Checked by : RA

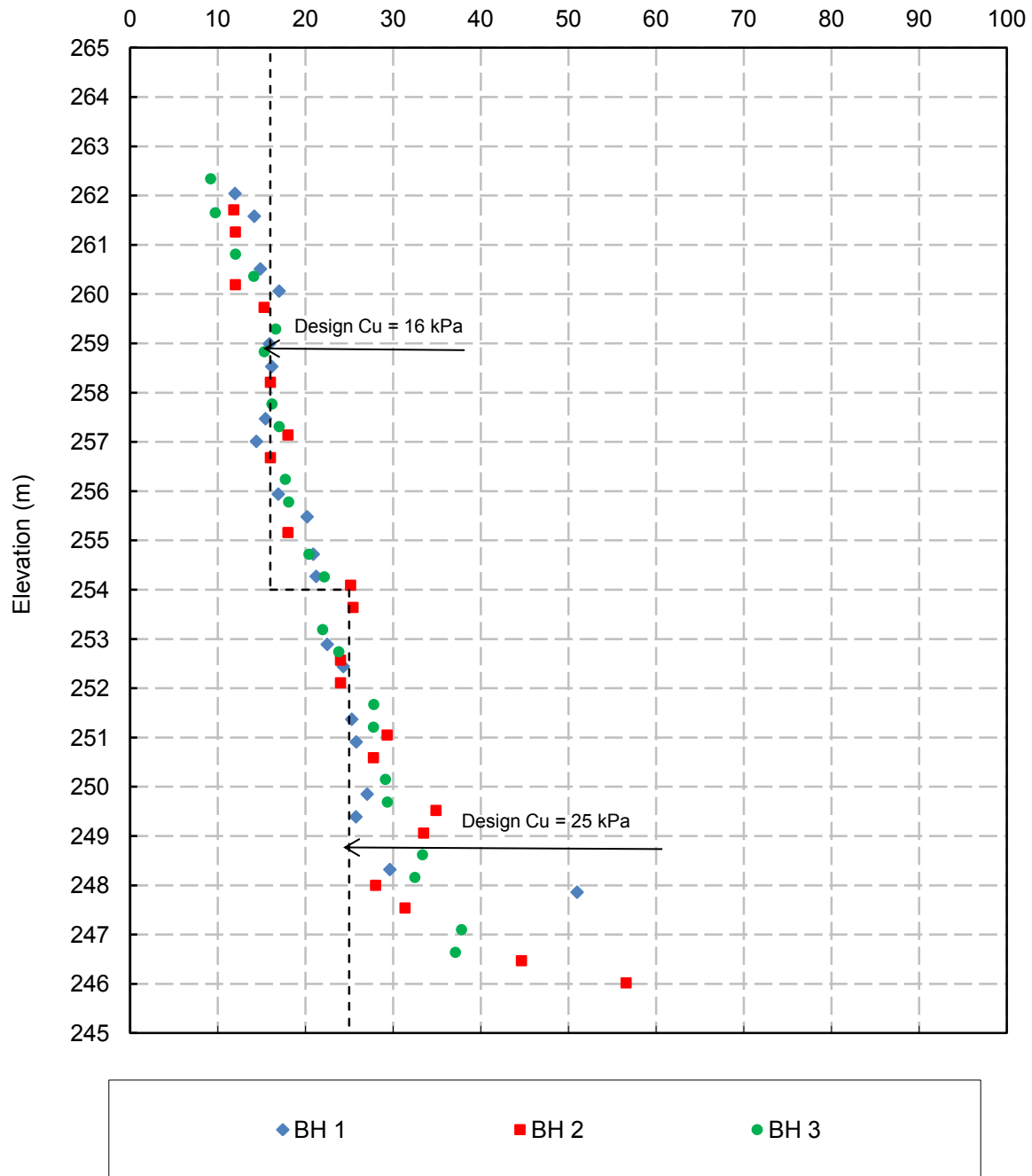
UNDRAINED SHEAR STRENGTH

FIGURE C5

SHILLINGTON CREEK CULVERT(Site 39E-214C)

Silty Clay to Clay

Cu (kPa)



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

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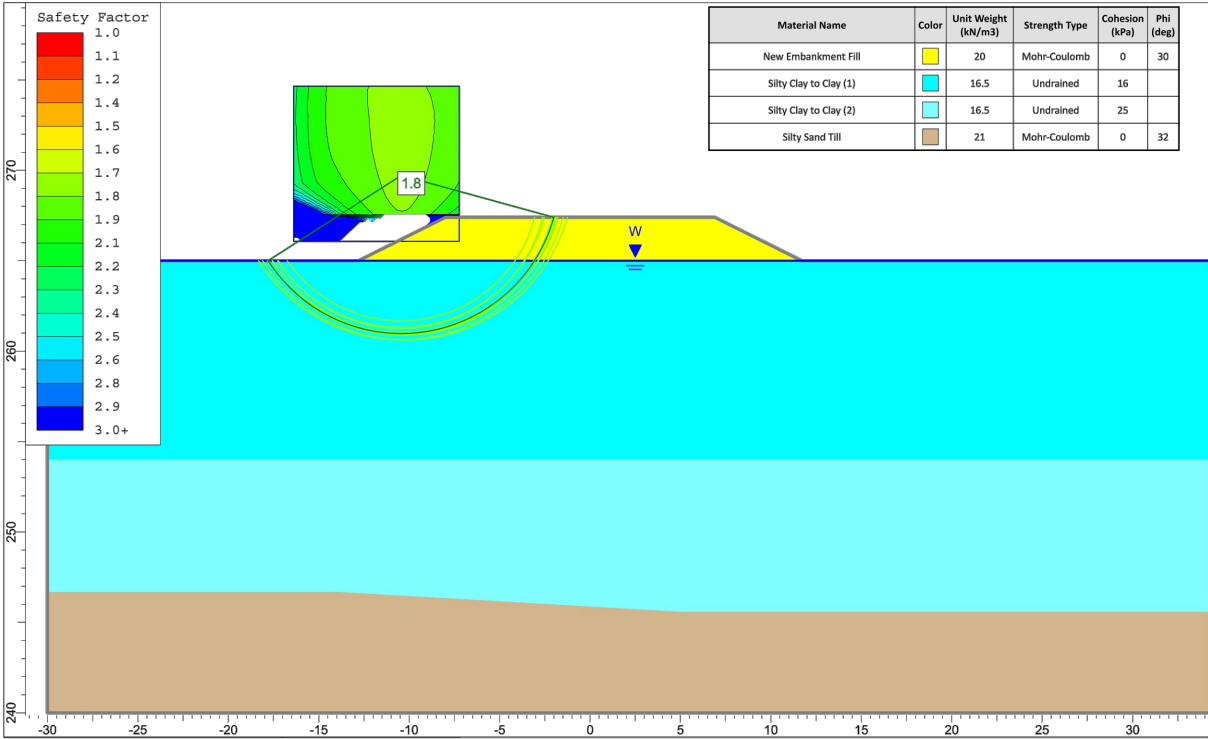
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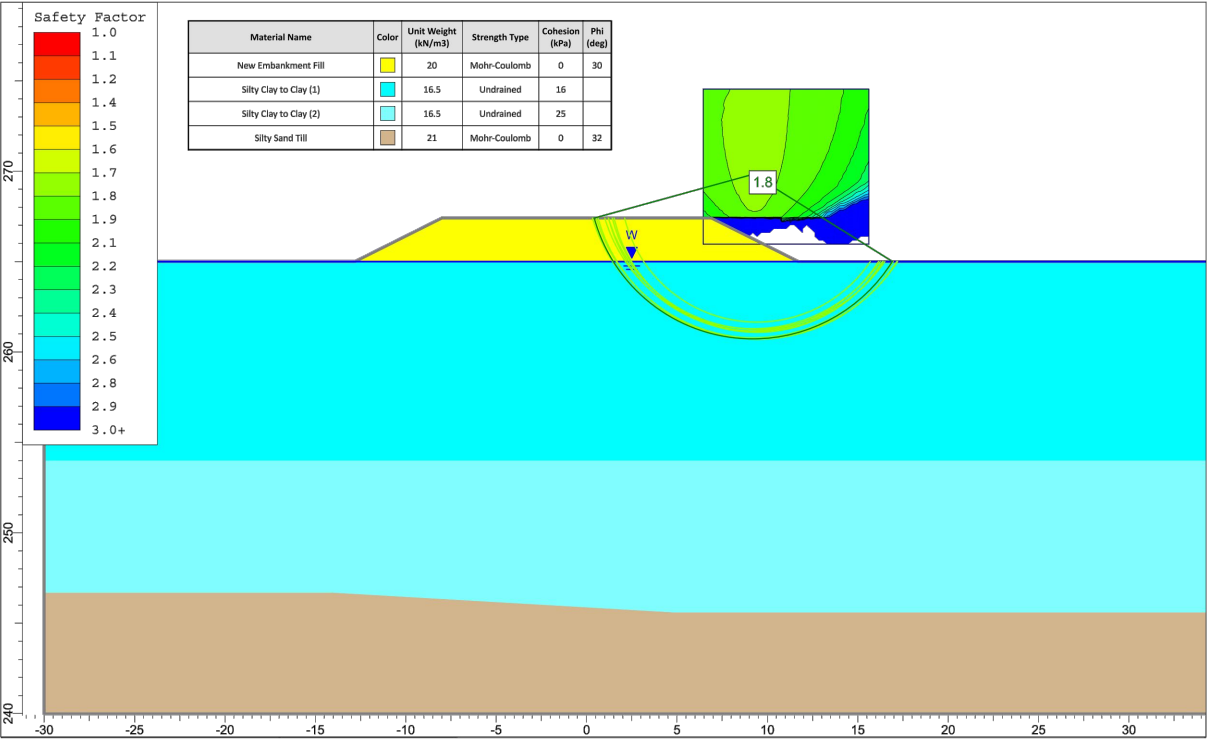
APPENDIX D

Slope Stability Models & Results

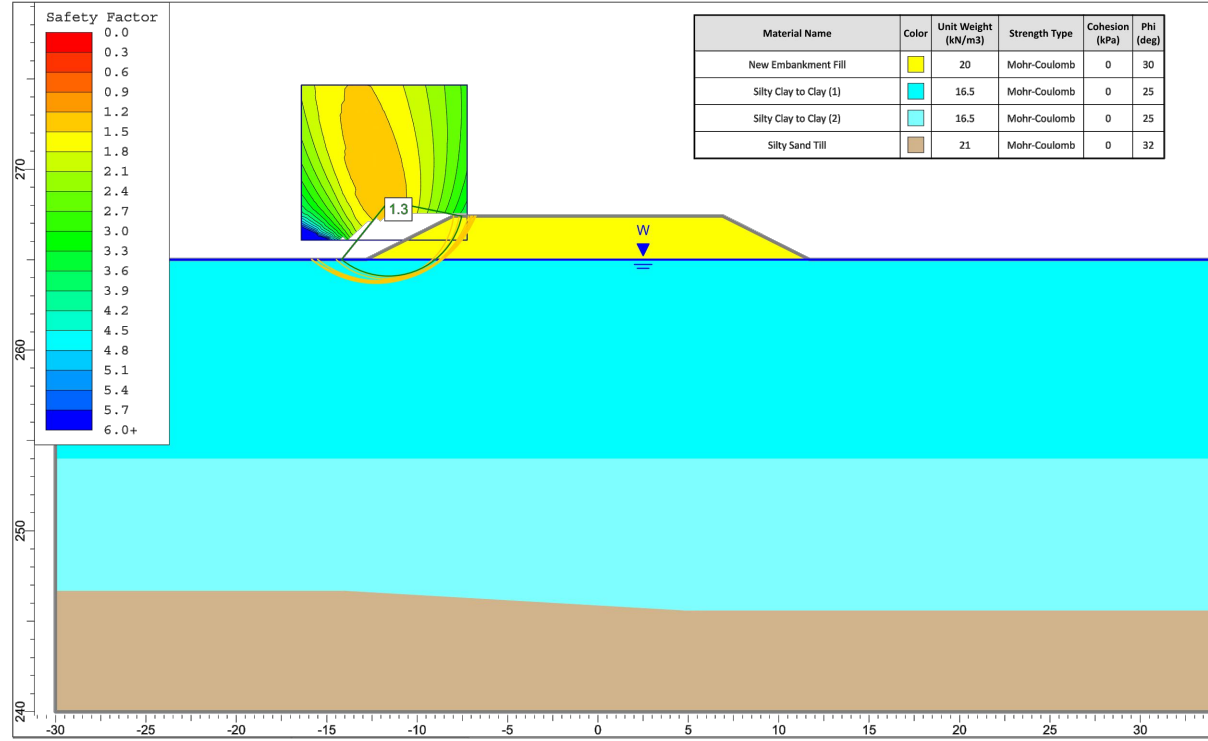




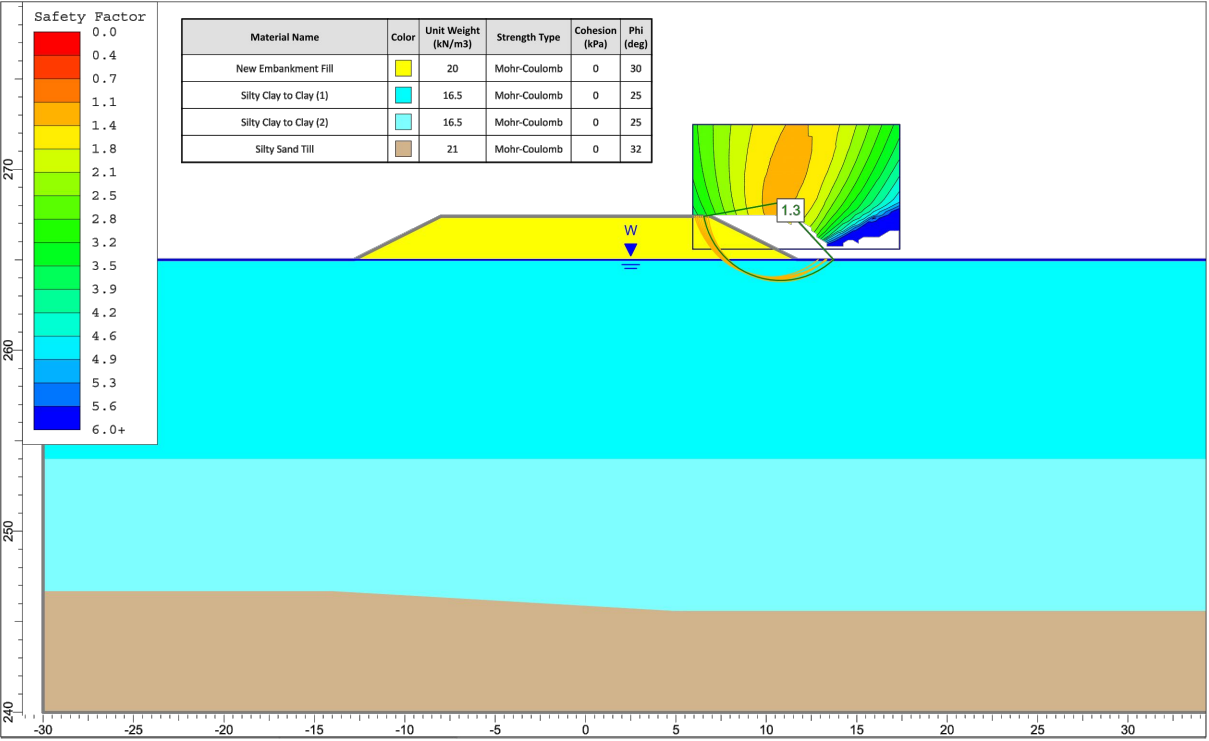
Approach Embankment-Total Stress Analysis




Approach Embankment-Total Stress Analysis



Approach Embankment-Effective Stress Analysis



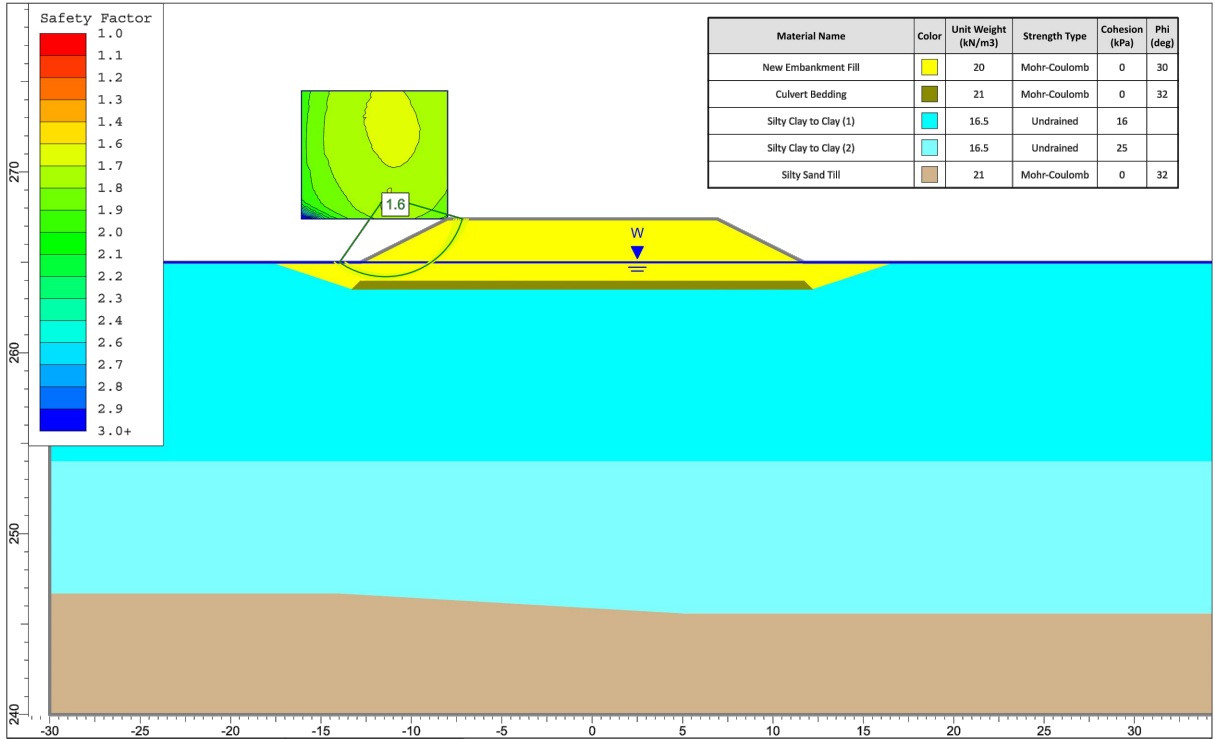
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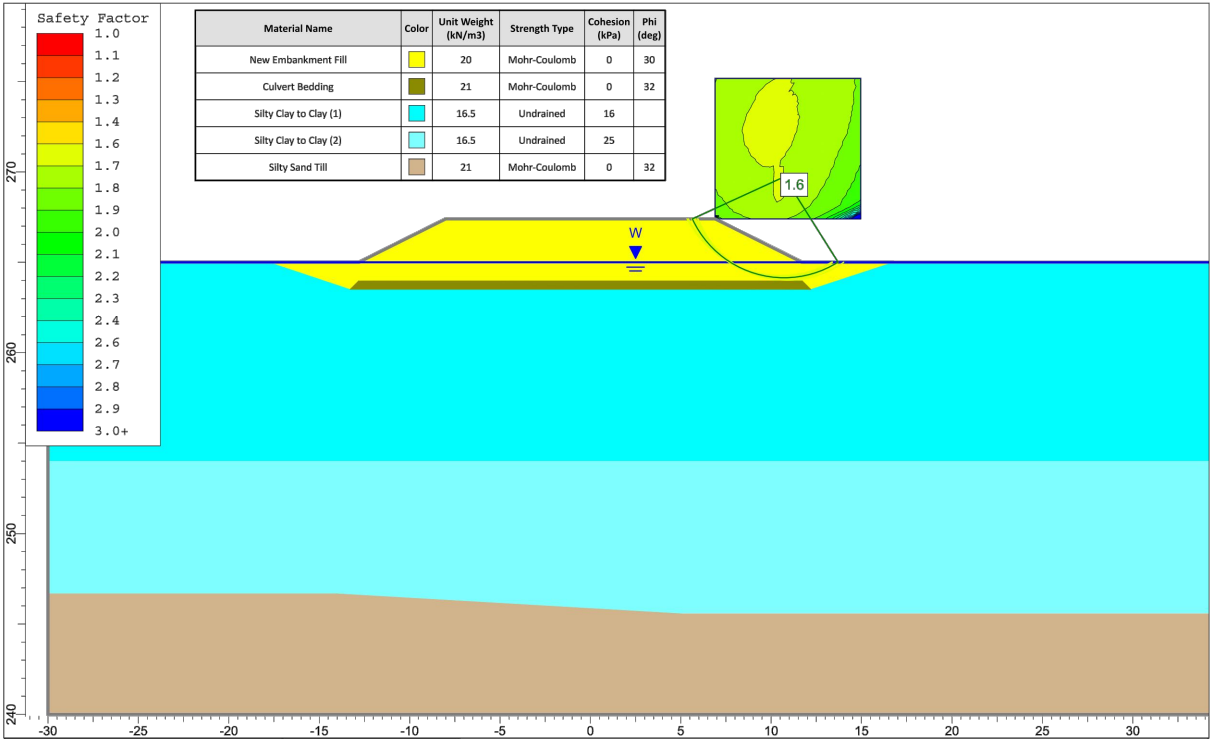
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Consulting Geotechnical & Environmental Engineering
Construction Materials, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650

HWY 101 SHILLINGTON CREEK CULVERT, SITE 39E-214			
G.W.P 5165-12-00		DATE: December 2014	
SUBM'D. AA		CHKD. RA	APPD: MT
Project No: 11-14-4066		Figure D1	

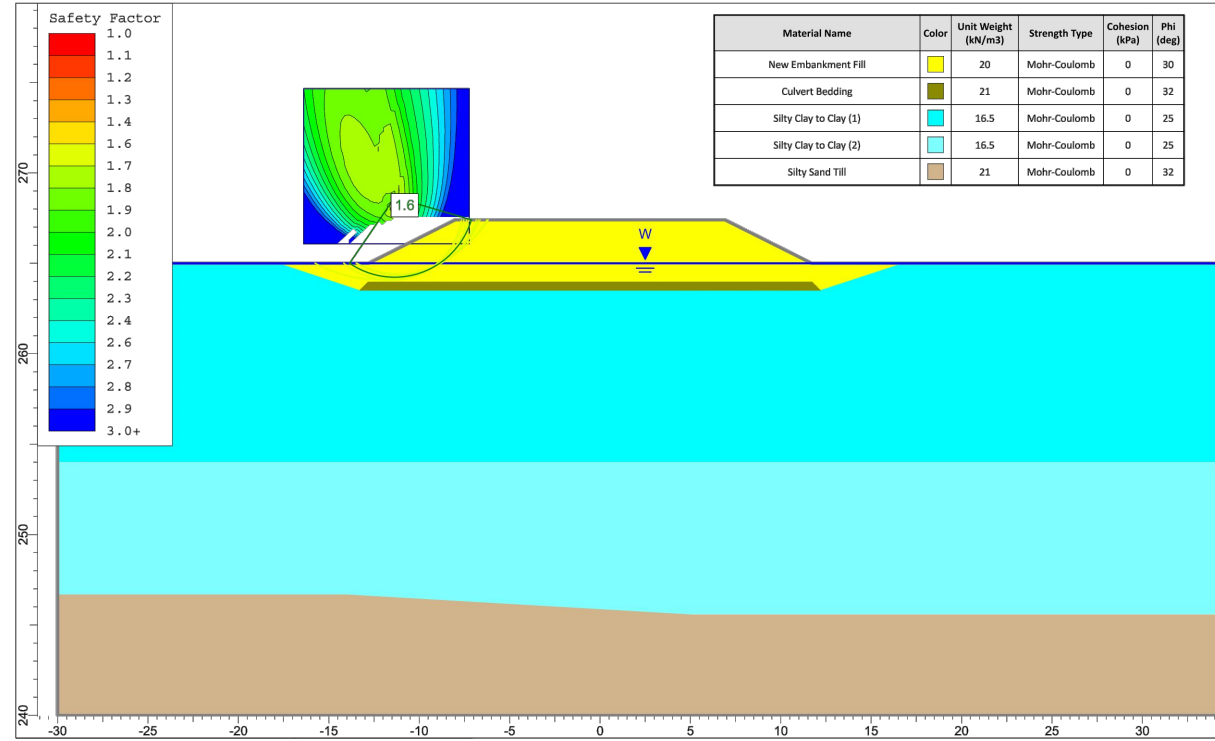
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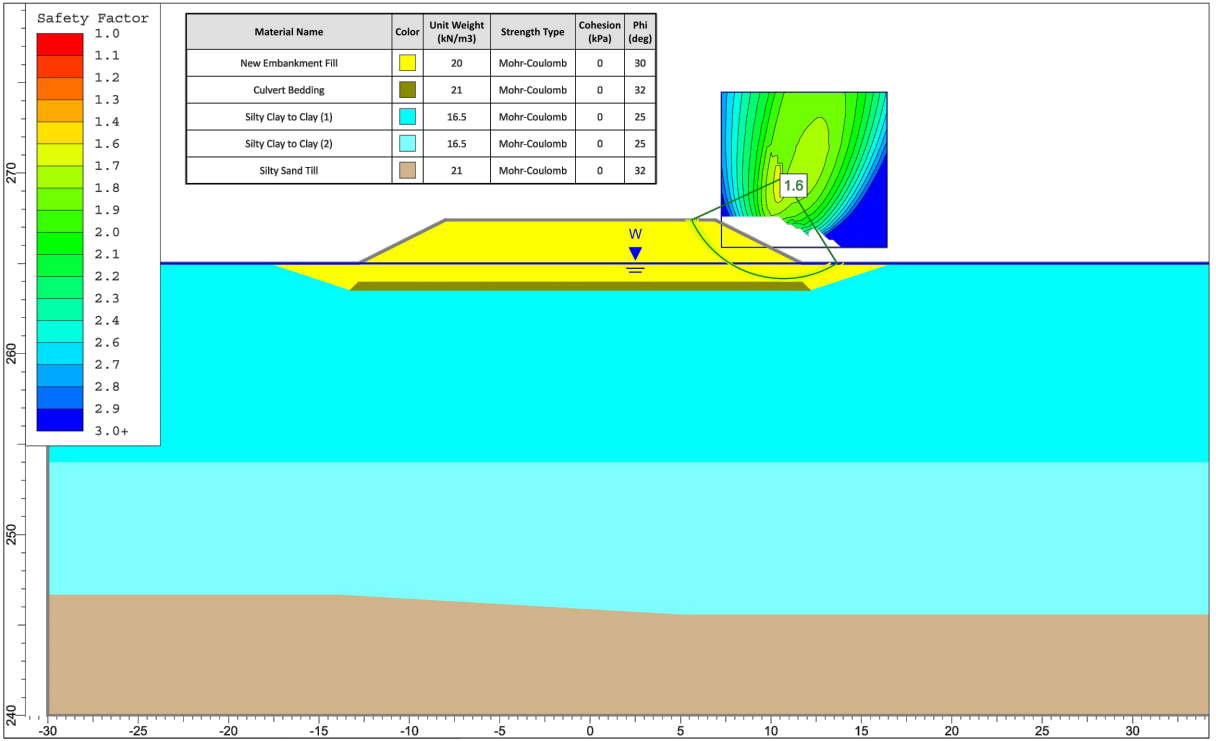
Approach Embankment Adjacent to Culvert-Total Stress Analysis



Approach Embankment Adjacent to Culvert-Total Stress Analysis



Approach Embankment Adjacent to Culvert-Effective Stress Analysis



Approach Embankment Adjacent to Culvert-Effective Stress Analysis