

Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

**PRELIMINARY
FOUNDATION INVESTIGATION AND DESIGN REPORT
UNNAMED CREEK CULVERT REPLACEMENT
HIGHWAY 634
ASSIGNMENT No. 5013-E-0018
MINISTRY OF TRANSPORTATION, ONTARIO
G.W.P. No. 5379-11-00, SITE 39E-244C
GEOCRES NO. 42H-61**

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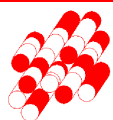
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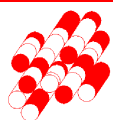
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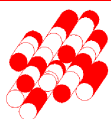
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PART A – FOUNDATION INVESTIGATION REPORT

**UNNAMED CREEK CULVERT REPLACEMENT, SITE 39E-244C
HIGHWAY 634
TOWNSHIP OF ADANAC, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5013-E-0018, G.W.P. 5379-11-00**



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 factual data on the subsurface conditions at the Unnamed Creek Culvert, Site 39E-244C on Highway 634, Township of Adanac, District of Cochrane, Ontario.

2.0 SITE DESCRIPTION

The site is located on Highway 634 (Latitude 49.553°, Longitude – 81.510°), approximately 2.3 km south of the highway's east junction with Island Falls Road in the Township of Adanac, Ontario. Island Falls is located east of the site and the main community of Smooth Rock Falls is situated approximately 35 km south of the site. The key plan on the Borehole Locations and Soil Strata Drawing, (Drawing 1) provides an overview of the site location.

The existing culvert located at Station 10+355 is a 2.4 m diameter and 30.3 m long round Structural Plate Corrugated Steel Pipe (SPCSP) with upstream and downstream invert elevations of 93.8± m. The Highway 634 embankment is approximately 6.7± m high at the culvert site with a pavement centre line elevation of 100.7± m. The watercourse flows through the culvert below Highway 634 from west to east. Vegetation at the site consists primarily of a coniferous forest with grass and shrubs.

3.0 INVESTIGATION PROCEDURES

The field work for this project was carried out between August 18 and August 22, 2014 and consisted of drilling and sampling three boreholes to depths ranging from 4.3 m to 23.8 m below ground surface. Dynamic cone penetration tests (DCPT's) were also carried out at two locations to depths of 3.9 m and 4.6 m below ground surface. The approximate borehole and DCPT 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 101 with a geodetic elevation of 100.00 m. The data from this control point was used by Terraprobe's staff to determine the ground surface elevations and local site coordinates of the boreholes. This data is summarized in the following table.

Borehole Details

Borehole No.	Local Site Coordinates		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH1	91 770.9	7 878.0	97.1	19.2
BH2	91 785.6	7 856.9	100.6	23.8
BH3	91 785.2	7 840.9	94.7	4.3
C1	91 785.2	7 839.9	94.7	4.6
C2	91 785.2	7 837.9	94.7	3.9

The boreholes were drilled with track-mounted CME 55 and portable 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. Two Dynamic Cone Penetration Tests were also performed from ground surface to refusal at distances of 1± m and 3± m west of Borehole 3. The bedrock was also cored by NQ-size diamond coring techniques in Boreholes 1 and 2. The field work was monitored 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 Boreholes 1 and 2 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 and Atterberg limits determinations in accordance with MTO and/or ASTM Standards as appropriate.

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

Surficial sediments in this area were deposited during the Late Wisconsinan glaciation. The main overburden units deposited during this time are till, debris flows, glaciofluvial and glaciolacustrine sediments. The most prominent surficial deposit found within this area is a dense, massive, impervious clay-rich till that contains rounded pebbles (Ontario Geological Survey 2001).

A compilation of studies undertaken in the general area shows that the bedrock geology is dominated largely by metasedimentary gneissic rocks. Other rock types occurring within the study area include large batholiths of granitic intrusive rocks such as granodiorite. The bedrock age ranges from Precambrian to Cenozoic (Ontario Geological Survey 2001).

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.

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

In summary, the site is generally underlain by topsoil, the highway pavement, very loose to compact sand and silty sand fill soils and soft to firm clayey silt fill. The fill soils are underlain by peat, deposits of stiff to hard clayey silt to silty clay till and dense to very dense silt. The overburden soils are further underlain by Granodiorite Schist bedrock. A more detailed description of the subsurface conditions is provided in the following sections.

4.2.1 Topsoil

A 100 mm thick layer of topsoil was encountered at this site in Borehole 3. Topsoil thickness may vary between and beyond the boreholes.

4.2.2 Flexible Pavement

Borehole 2, which was drilled through Highway 634, encountered a flexible pavement consisting of a 25 mm thick layer of asphalt concrete underlain by a 375 mm thick layer of gravelly sand fill that extends to elevation 100.2 m.

A Standard Penetration test carried out in the gravelly sand fill gave an SPT N-value of 93 blows for 0.3 m of penetration indicating a very dense relative density. The natural water content of a sample of the granular fill is 8% by weight.

4.2.3 Fill – Sand

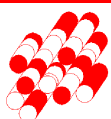
The flexible pavement in Borehole 2 is underlain by a 1.7 m thick layer of sand fill that extends to elevation 98.5 m. Standard Penetration tests carried out in the sand fill measured SPT N-values ranging from 3 to 20 blows for 0.3 m of penetration suggesting a very loose to compact relative density. The moisture content (by weight) of a sample of the sand fill is 11%.

The grain size distribution curve of a sample of the sand fill is shown on Figure B1 in Appendix B. The results show a grain size distribution consisting of 18% gravel, 75% sand and 7% silt and clay size particles.

4.2.4 Fill – Silty Sand

A 2.3 m thick layer of silty sand fill was encountered in Borehole 2 extending to a depth of 4.4 m below ground surface or to elevation 96.2 m. Standard Penetration tests performed in the silty sand fill gave SPT N-values that range from 5 to 12 blows for 0.3 m of penetration indicating a loose to compact relative density. The natural water content of samples of the silty sand fill varies from 13% to 18% by weight.

The grain size distribution curve of a sample of the silty sand fill is shown on Figure B2 in Appendix B. The results show a grain size distribution consisting of 1% gravel, 58% sand, 34% silt and 7% clay size particles.



4.2.5 Fill – Clayey Silt to Silty Clay

Clayey silt to silty clay fill was encountered across the site. 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	0.7	0.7	96.4
BH2	2.3	6.7	93.9
BH3	0.6	0.7	94.0

Standard Penetration tests performed in the clayey silt to silty clay fill gave N-values that range from 2 to 6 blows for 0.3 m of penetration indicating a soft to firm consistency. The natural water content of samples of the clayey silt to silty clay fill range from 12% to 19% by weight.

The grain size distribution plots of two samples of the clayey silt to silty clay fill are depicted on Figure B3 in Appendix B. The results show a grain size distribution consisting of 0% and 1% gravel, 14% and 31% sand, 51% and 64% silt and, 17% and 22% clay size particles.

Atterberg limits tests were also carried out on two samples of the clayey silt to silty clay fill and the results are plotted on the plasticity chart, Figure B4 in Appendix B. These values indicate that the fill is a low plasticity cohesive soil (CL-ML and CL). The results from the Atterberg limits tests are summarized below:

Liquid Limit:	18% and 24%
Plastic Limit:	13% and 16%
Plasticity Index:	5% and 8%
Natural Moisture Content:	19%

4.2.6 Peat

A layer of fibrous peat was encountered in Borehole 3 below the silty clay fill. The peat layer is 0.7 m thick and it extends to elevation 93.3 m.

A Standard Penetration test carried out in the peat layer measured a SPT N-value of 2 blows for 0.3 m of penetration. The moisture content (by weight) of a sample of the peat is 88%.

4.2.7 Clayey Silt to Silty Clay Till

Till units ranging in composition from clayey silt to silty clay were encountered. The clayey silt to silty clay till deposit is divided into upper and lower layers by an interbedded silt layer. Summarized in the following table are the locations, thicknesses, explored depths and base elevations of the clayey silt to silty clay till.

Clayey Silt to Silty Clay Till Borehole Data

Borehole No.	Clayey Silt to Silty Clay Till Thickness (m)	Clayey Silt to Silty Clay Till Depth of Deposit (m)	Clayey Silt to Silty Clay Base Elevation (m)
Upper Clayey Silt Till Layer			
BH1	3.7	4.4	92.7
BH2	2.6	9.3	91.3
BH3	2.9	4.3*	90.4
Lower Clayey Silt to Silty Clay Till Layer			
BH1	11.9	17.9	79.2
BH2	10.8	20.9	79.7

* Borehole termination depth.

Standard Penetration tests carried out in this deposit measured SPT N-values that range from 10 to more than 100 blows per 0.3 m of penetration indicating a stiff to hard consistency. The natural water content of samples retrieved from these strata range from 8% to 37% by weight.

Grain size distribution tests were carried out on six samples of the clayey silt to silty clay till and the results are illustrated in Figure B5 in Appendix B. The results show a grain size distribution consisting of 0% to 1% gravel, 5% to 17% sand, 61% to 72% silt and 20% to 27% clay size particles.

Frequent cobble and boulder inclusions were encountered in Borehole 2 at a depth of 18.9 m to 20.9 m below ground surface and NQ-size diamond coring techniques were used to extend the borehole below the cobbles and boulders. Photographs of the cobbles and boulders are provided in Figure B8 in Appendix B.

Atterberg limits tests were also carried out on six samples of the clayey silt to silty clay till and the results are plotted on the plasticity chart, Figure B6 in Appendix B. The results indicate that the till matrix generally consists of low plasticity (CL-ML and CL) clayey silt to silty clay soils. The Atterberg limits test results are summarized below.

Liquid Limit:	16% to 23%
Plastic Limit:	12% to 15%
Plasticity Index:	4% to 8%
Natural Moisture Content:	12% to 19%

4.2.8 Silt

Embedded between the clayey silt to silty clay till layers, there exists a layer of silt. Summarized below are the locations, thicknesses, depths and base elevations of the silt deposit.

Silt Borehole Data

Borehole No.	Silt Thickness (m)	Silt Depth (m)	Silt Base Elevation (m)
BH1	1.6	6.0	91.1
BH2	0.8	10.1	90.5

The N-values of Standard Penetration tests carried out in the silt deposit range from 44 to 62 blows per 0.3 m of penetration, suggesting a dense to very dense relative density and, the moisture content of two samples of this deposit are 20% and 21% by weight.

The grain size distribution curve of a sample of the silt is shown on Figure B7 in Appendix B. The results show a grain size distribution consisting of 0% gravel, 0% sand, 95% silt and 5% clay size particles.

4.2.9 Bedrock

The overburden soils are underlain by granodiorite schist bedrock. Summarized below are the depths to bedrock and the bedrock surface elevations.

Bedrock Borehole Data

Borehole No.	Depth to Bedrock (m)	Top of Bedrock Elevation (m)
BH1	17.9	79.2
BH2	20.9	79.7

The granodiorite schist bedrock is described as unweathered, massive brownish grey rock of very high strength (estimated by chipping rock specimens with a geological hammer). Photographs of the bedrock core samples are provided in Figures B9 in Appendix B. Summarized below are the Rock Quality Designation, Rock Mass Quality, Total Core Recovery and Solid Core Recovery.

Rock Core Sample Data

Borehole No.	Rock Quality Designation (RQD)	Rock Mass Quality ²	Total Core Recovery (TCR)	Solid Core Recovery (SCR)
BH1	87%	Good	100%	87%
BH2	100%	Excellent	100%	100%

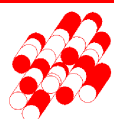
4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling and standpipe piezometers were installed in Borehole 1 and Borehole 2. The ground water levels measured in the piezometers are summarized in the following table:

Ground Water Level Data

Borehole No.	Date	Water Levels	
		Depth (m)	Elevation (m)
BH1	September 16, 2014	0.1	97.0
	October 27, 2014	0.3	96.8
BH2	September 16, 2014	2.7	97.9
	October 27, 2014	2.4	98.2

² Deere et al., 1967.



The ground water elevations in Borehole 1 and 2 are higher than the floodplain elevation of the creek indicating that a hydrostatic head exists in the lower clayey silt to silty clay till in which the piezometer screens were made. The free water level at this site is estimated to be at an approximate elevation of $94.0 \pm$ m (flood plain level) based on the soil moisture conditions and the ground surface topography.

5.0 MISCELLANEOUS

The investigation was carried out using drilling equipment supplied and operated by Landcore Drilling of Chelmsford, Ontario. The field operations were supervised by Mr. Wen Zhu and the routine laboratory testing was carried out at Terraprobe's Brampton laboratory.

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

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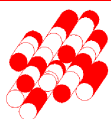


Michael Tanos, P.Eng.
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PART B – FOUNDATION DESIGN REPORT

**UNNAMED CREEK CULVERT REPLACEMENT, SITE 39E-244C
HIGHWAY 634
TOWNSHIP OF ADANAC, DISTRICT OF COCHRANE, ONTARIO
ASSIGNMENT No. 5013-E-0018, G.W.P. 5379-11-00**



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 culvert rehabilitation alternative. 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. The material choice includes steel, concrete, high density polyethylene and polyvinyl chloride. 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. Fish passage is also an important factor that affects the choice of culvert type. Some factors that are to be considered include:

- Steel and plastic culverts have the advantage of simpler and quicker construction, which is especially advantageous in remote areas. Steel also has the 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;
- Flexible pipe culverts may have an advantage over concrete box culverts in certain unfavourable foundation soil conditions; and
- Normally where weak foundation soils and/or settlement sensitive soils exist concrete box culverts or pipe culverts may provide better solutions.

The existing culvert consists of a 2.4 m diameter and 30.3 m long Structural Plate Corrugated Steel Pipe (SPCSP) with upstream and downstream invert elevations of 93.81 m and 93.79 m respectively. The maximum height of embankment fill at the site is approximately 6.7± m. Listed below are the alternative rehabilitation solutions that were considered.

- Alternative 1 – Slip-lining the existing culvert with a 2120 mm inside diameter (ID) round tunnel liner plate;
- Alternative 2 – Slip-lining the existing culvert with a 1961 mm ID high density polyethylene pipe;
- Alternative 3 – Slip-lining the existing culvert with a 1811 mm ID high density polyethylene pipe;
- Alternative 4 – Paving the culvert invert with 100 mm thick concrete; and
- Alternative 5 – Replacing the existing culvert with a single span 2.4 m x 1.8 m precast concrete box culvert while maintaining one lane of traffic with roadway protection and temporary signals; and
- Alternative 6 – Replacing the existing culvert with a single span 2.4 m x 1.8 m precast concrete box culvert while maintaining one lane of traffic with temporary detour and a temporary culvert.

6.2 Slip Lining

Rehabilitating culverts by slip lining is one of several methods available for extending the life of an existing culvert. It is often cost effective when compared to complete replacement, particularly where there are deep fills or where open cut excavations would cause extensive traffic disruptions.

At this site the embankment is approximately 6.7± m high and the roadway consists of two lanes i.e. a single lane in each direction. Therefore, either a temporary detour, or lane reduction to single lane traffic will be required to install a new culvert. Building a temporary detour or installing a temporary protection system to accommodate a lane reduction are expensive options. Additionally, both of these construction options (temporary detour and lane reduction) will cause traffic disruptions.

A thorough examination of the existing culvert is necessary to ensure that it is a candidate for rehabilitation by slip lining. The choice of material used for slip lining should be based on functionality and life cycle costs. Based on the existing site and culvert conditions, Alternatives 1, 2, 3 and 4 are feasible and practical options. Practical construction methodologies for slip lining around the timber posts within the existing culvert should be taken into consideration.

Full length grouting of the liner is recommended to affix the liner to the existing culvert thereby increasing hoop strength and, reducing the potential for joint leakage that can create future piping problems. The grout shall be suitable for low pressure pumping into the void between the liner and the existing culvert and should be compatible with the liner and culvert material. Grout pressure must be monitored to minimize the potential for liner and culvert damage from excessive grout pressure.

6.3 Foundation Alternatives

If a culvert replacement is selected as the preferred option, the upstream and downstream invert elevations of the replacement culvert will be similar to the existing culvert i.e. 93.8± m approximately. The borehole data indicates that a new culvert can be supported on spread footings designed to bear on the very stiff to hard clayey silt to silty clay till deposit that exists above and close to the proposed design invert. Therefore, alternative foundation schemes such as augered caissons and driven piles are impractical and were therefore ruled out from further consideration.

6.3.1 Geotechnical Resistances Strip Footings

The recommended highest founding depths and geotechnical resistances for strip footings (minimum footing width of 1.2 m) founded on undisturbed competent natural soils are tabulated as follows:

Footing Depths and Geotechnical Resistance For Spread Footings

Borehole Location And Number	Existing Ground Surface Elev. (m)	Bottom of Footing Level Below Existing Ground Surface (m)	Highest Founding Elevation (m)*	Factored Geotech. Resistance at ULS (kPa)	Geotech. Reaction at SLS (kPa)	Subgrade Soil
BH 1	97.1±	Below 0.7±	Below 96.4	300	200	Clayey Silt Till
BH 2	100.6±	Below 6.7±	Below 93.9	300	200	Clayey Silt Till
BH 3	94.7±	Below 1.4±	Below 93.3	300	200	Clayey Silt Till

* Structural engineer shall select a design founding elevation based on site topography, frost depth and structural dimensions.

Since the silty clay till is susceptible to disturbance when wet, it is recommended that a 75 mm thick layer of lean concrete (mud mat) be poured on the foundation bearing surfaces as soon as possible after excavation and approval.

The ULS and SLS values provided herein are 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 2006* (CHBDC 2006), Clause 6.7.3 and Clause 6.7.4. The recommended SLS values correspond to a settlement of up to 25 mm, a significant portion of which will be complete by the end of construction.

6.3.2 Ultimate Coefficient of Friction

Resistance to lateral forces/sliding resistance between the concrete footing 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' – ultimate coefficient of friction of 0.7; and
- Silty Clay till – ultimate coefficient of friction of 0.6.

6.3.3 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 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.

6.4 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.

6.5 Culvert Bedding and Backfill

Structural backfill and cover around the culvert should be placed in accordance with the limits illustrated in OPSD 803.010 (concrete culvert). 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.

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 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 silty sand) can be re-used for non-structural backfill provided they are free of organics and other deleterious material.

6.6 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.

6.7 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 93.0 m), the OHSA soil classifications are:

- Embankment fill – Type 3 soils; and
- Clayey silt to silty clay till – Type 2 soils.

The side slopes of temporary excavations may be formed no steeper than 1H:1V for Type 2 and Type 3 soils. Excavations should be carried out in accordance with OPSS 902.

6.8 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 clayey silt to silty clay till deposit. A suitable dewatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

6.9 Embankments

6.9.1 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. Two scenarios related to embankment stability were examined, namely:

- Slip Lining Option – verification that the global stability of the existing embankment is equal to or greater than a minimum target factor of safety of 1.3; and
- Culvert Replacement Option – verification that the global stability of a new embankment constructed at a minimum 2 Horizontal to 1 Vertical (2H:1V) side slope geometry will be equal to or greater than a minimum target factor of safety of 1.3.

For the purpose of embankment stability analyses, the commercially available slope stability program Slide 6.0 developed by Rocscience Inc. was used and the Morgenstern-Price and Spencer methods for stability analysis were employed. The soil parameters used for the slope stability analyses of the two scenarios outlined above and the factors of safety that were obtained are provided in the following table. These soil parameters were established from predictions/empirical correlations using SPT N-values, laboratory results and back calculation of the existing slopes, tempered with engineering judgement from our experience with similar soils in this region of Ontario.

The slope stability models depicting the corresponding factors of safety are provided in Figures C1 and C2 in Appendix C. Our analyses indicate that the factors of safety of the existing embankment will be equal to the target factor of safety of 1.3. A factor of safety of 1.3 was also obtained for a new embankment 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/m³)
Existing Embankment (Slip Lining Option)					
Sand to Silty Sand Fill	N/A*		30	0	19
Clayey Silt Fill			28	0	19
Clayey Silt Till			33	0	21
Silt			30	0	19
Clayey Silt to Silty Clay Till			33	0	21
Design Factors of Safety	-		1.3		-
New Embankment (Culvert Replacement Option)					
Embankment Fill	30	0	30	0	20
Clayey Silt Till	0	200	33	0	21
Silt	30	0	30	0	19
Clayey Silt to Silty Clay Till	0	200	33	0	21
Design Factors of Safety	1.3		1.3		-

* Total stress analysis not applicable for the existing embankment.

6.9.2 Settlement

The embankment settlement analysis (for new embankment construction) was carried out using elastic deformation moduli established from predictions/empirical correlations using undrained shear strengths, Atterberg limits and SPT N-values, tempered with engineering judgement from our experience with similar soils in this region of Ontario. Since the Highway 634 grade will remain unchanged, the additional new embankment load imparted to the underlying soils is not expected to be greater than about 15± kPa. It is estimated that this additional embankment load will induce about 10± mm of total settlement in the footprint area of the new embankment.

Embankments constructed with local 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.

6.9.3 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 SPMDD. 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 sequence of work will also 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 of 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 ³)	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
Clayey Silt to Silty Clay Till	33	21	0.29	0.46	3.39
Silt	30	19	0.33	0.50	3.00

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 c_u value of 200 kPa can be used for the clayey silt to silty clay till.

6.11 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 (Smooth Rock Falls) should be used for design:

- Velocity Related Seismic Zone 0;
- Zonal Velocity Ratio 0.05;
- Acceleration Related Seismic Zone 1;
- Zonal Acceleration Ratio 0.05; and
- Peak Horizontal Ground Acceleration 0.08 g (10% in 50 years).

The soil profile type at this site has been classified as Type I 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 1.0. 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.

6.12 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
- Prepare Non Standard Special Provisions for the slip-lining option.

7.0 CLOSURE

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

Terraprobe Inc.



Rehman Abdul, P.Eng.
Principal, Senior Geotechnical Engineer



Michael Tanos, P.Eng.
Designated MTO Contact



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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.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)

OPSD 208.010	Benching Of Earth Slopes.
OPSD 803.010	Backfill And Cover For Concrete Culverts With Spans Less Than Or Equal To 3 m.
OPSD 3090.100	Foundation, Frost Penetration Depths For Northern Ontario.

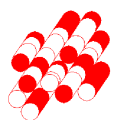
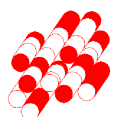
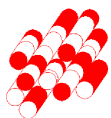


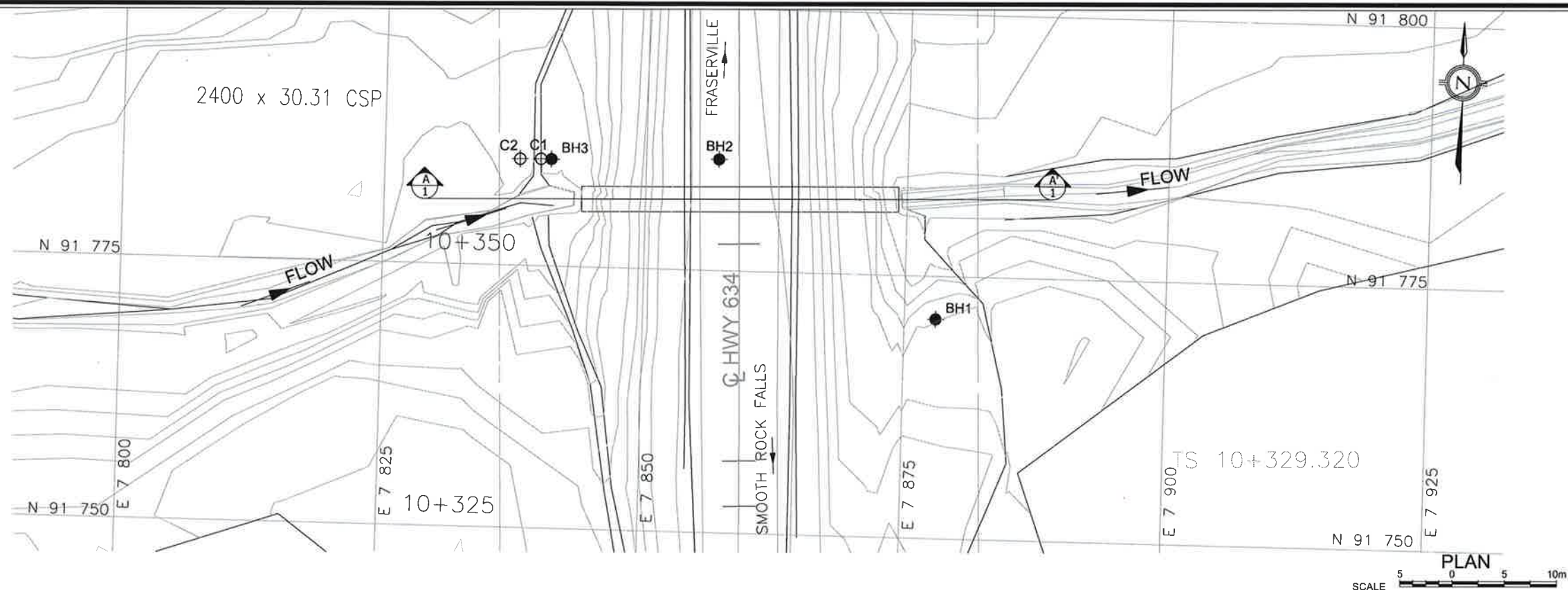
TABLE 1
COMPARISON OF CULVERT REHABILITATION ALTERNATIVES

Slip Lining	Culvert Replacement
<p>Advantages:</p> <ul style="list-style-type: none"> Reliable performance expected. Eliminates open cut excavations, temporary shoring and detours. Less costly compared to culvert replacement. Minimal user delay costs. Minimal disruption of natural environment during construction. Well tested technology. <p>Disadvantages:</p> <ul style="list-style-type: none"> Slip liner dimensions are limited to existing culvert geometry and size. Reduces cross-sectional area of existing culvert. Slip liner alignment and grade needs to be carefully controlled. Timber posts within existing culvert barrel are an impediment to the slip lining process. 	<p>Advantages:</p> <ul style="list-style-type: none"> Reliable performance expected. Precast culvert units can be used facilitating easy transportation, handling and placement. New culvert opening can be designed to accommodate current and future hydraulic conditions including adverse storm events. <p>Disadvantages:</p> <ul style="list-style-type: none"> More expensive than slip lining. Requires an open cut excavation, temporary shoring and detours. Significant user delay costs. Construction is disruptive to natural environment.
<p>Risks/Consequences</p> <ul style="list-style-type: none"> Requires careful control of grout pressures to prevent damage to the slip lining and culvert. 	<p>Risks/Consequences</p> <ul style="list-style-type: none"> Very low risk of bearing capacity failure.



DRAWING





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

GWP No 5379-11-00

HWY 634
UNNAMED CREEK CULVERT
BOREHOLE LOCATIONS AND SOIL STRATA

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Consulting Geotechnical & Environmental Engineering
Construction Materials Engineering, Inspection & Testing
11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2850



KEY PLAN

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

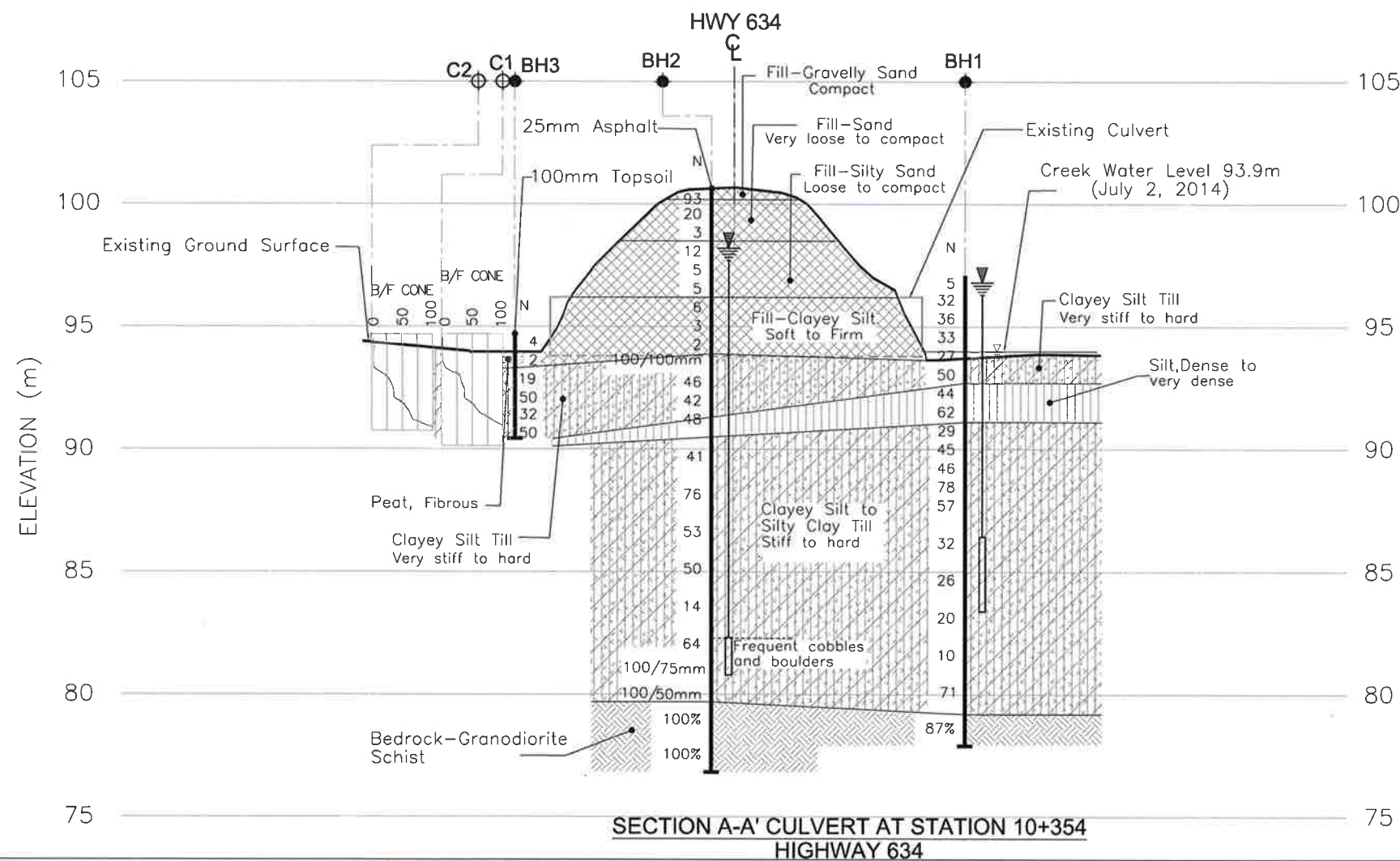
No	ELEV. (m)	LOCAL SITE COORDINATES (m)	
		NORTHING	EASTING
BH 1	97.1	91770.9	7878.0
BH 2	100.6	91785.6	7856.9
BH 3	94.7	91785.2	7840.9
C1	94.7	91785.2	7839.9
C2	94.7	91785.2	7837.9

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 GC 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 B11180634004, DTM11180634004, received September 11, 2014

REVISIONS	DATE	BY	DESCRIPTION

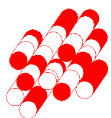
HWY: 634	PROJECT No: 11-14-4066	Geocres No: 42H-61
SUBM'D. HA	CHKD. RA	DATE: June 2016
DRAWN: KC	CHKD. RA	APPD: MT
		SITE: 39E-244
		DWG: 1



Michael Tanos

APPENDIX A

Record of Borehole Sheets



LIMITATIONS AND RISK

Procedures

The soil conditions were confirmed at the borehole locations only and conditions may vary between and beyond the boreholes. The boundaries between the various strata as shown on the logs are based on non-continuous sampling. These boundaries represent an inferred transition between the various strata, rather than a precise plane of stratigraphic change.

This investigation has been carried out using investigation techniques and engineering analysis methods consistent with those ordinarily exercised by Terraprobe and other engineering practitioners, working under similar conditions and subject to the time, financial and physical constraints applicable to this project. The discussions and recommendations that have been presented are based on the factual data obtained.

It must be recognized that there are special risks whenever engineering or related disciplines are applied to identify subsurface conditions. Even a comprehensive sampling and testing programme implemented in accordance with the most stringent level of care may fail to detect certain conditions. Terraprobe has assumed for the purposes of providing design parameters and advice, that the conditions that exist between sampling points are similar to those found at the sample locations. The conditions that Terraprobe has interpreted to exist between sampling points can differ from those that actually exist.

It may not be possible to drill a sufficient number of boreholes or sample and report them in a way that would provide all the subsurface information that could affect construction costs, techniques, equipment and scheduling. Contractors bidding on or undertaking work on the project should be directed to draw their own conclusions as to how the subsurface conditions may affect them, based on their own investigations and their own interpretations of the factual investigation results, cognizant of the risks implicit in the subsurface investigation activities.

Changes In Site And Scope

It must be recognized that the passage of time, natural occurrences, and direct or indirect human intervention at or near the site have the potential to alter subsurface conditions. Groundwater levels are particularly susceptible to seasonal fluctuations.

The design advice is based on the factual data obtained from this investigation made at the site by Terraprobe and are intended for use by the owner and its retained designers in the design phase of the project. If there are changes to the project scope and development features, or there is any additional information relevant to the interpretations made of the subsurface information, the geotechnical design parameters and comments relating to constructibility issues and quality control may not be relevant or complete for the revised project. Terraprobe should be retained to review the implications of such changes with respect to the contents of this report.

This report was prepared for the express use of the Ministry of Transportation, its retained design consultants and MMM Group Limited. It is not for use by others. This report is copyright of Terraprobe Inc. and no part of this report may be reproduced by any means, in any form, without the prior written permission of Terraprobe Inc. The Ministry of Transportation, its retained design consultants and MMM Group Limited, are authorized users.

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. _____ LOCATION _____ Coords: E:7878 N:91770.9 ORIGINATED BY W.Z
 DIST _____ HWY 634 BOREHOLE TYPE CASING AND WASH BORING/NQ CORING COMPILED BY S.D
 DATUM LOCAL DATE 2014-8-20 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)					WATER CONTENT (%)				
97.1	GROUND SURFACE																
96.4	FILL, clayey silt, sandy, trace gravel, firm, brown, moist		1	SS	5												
0.7	CLAYEY SILT, trace to some sand, very stiff to hard, grey, moist (GLACIAL TILL)		2	SS	32											0 14 66 20	
			3	SS	36												
			4	SS	33											sampler wet at 2.3m	
			5	SS	27												
			6	SS	50												
92.7	SILT, trace clay, dense to very dense, grey, wet		7	SS	44											0 8 72 20	
4.4			8	SS	62												
91.1	CLAYEY SILT to SILTY CLAY, trace sand, stiff to hard, grey, moist (GLACIAL TILL)		9	SS	29											0 0 95 5	
6.0			10	SS	45												
			11	SS	46											0 5 68 27	
			12	SS	78												
			13	SS	57												
			14	SS	32												
			15	SS	26												
			16	SS	20												

Continued Next Page

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

report: mto-terraprobe soil file: 3-unknown creek culvert (39e-244) bh logs - copy rev2.gpj

RECORD OF BOREHOLE No 1

2 of 2

METRIC

G.W.P. _____ LOCATION _____ Coords: E:7878 N:91770.9 _____ ORIGINATED BY W.Z
 DIST _____ HWY 634 _____ BOREHOLE TYPE CASING AND WASH BORING/NQ CORING _____ COMPILED BY S.D
 DATUM LOCAL _____ DATE 2014-8-20 _____ 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			20 40 60 80 100					w _p			w	w _L					
								SHEAR STRENGTH (kPa)										WATER CONTENT (%)				
								○ UNCONFINED ● QUICK TRIAXIAL + FIELD VANE × LAB VANE														
	(continued)						20	40	60	80	100	10	20	30	GR	SA	SI	CL				
	CLAYEY SILT to SILTY CLAY, trace sand, stiff to hard, grey, moist (GLACIAL TILL)		17	SS	10									○								
			18	SS	71																	
														○								
79.2																						
17.9		cobble																				
	BEDROCK - GRANODIORITE SCHIST, unweathered, massive, brownish grey, very high strength (with secondary weak planes)		1	RUN	NQ													Run #1 TCR: 100% SCR: 87% RQD: 87%				
77.9																						
19.2																						

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

Piezometer installation consists of a 19mm diameter schedule 40PVC pipe with a 3.0m slotted screen.

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 16, 2014	0.1	97.0
Oct 27, 2014	0.3	96.8

METRIC[illegible]

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

RECORD OF BOREHOLE No 2

2 of 2

METRIC

G.W.P. _____ LOCATION _____ Coords: E:7856.9 N:91785.6 ORIGINATED BY W.Z
 DIST _____ HWY 634 BOREHOLE TYPE CASING AND WASH BORING/NQ CORING COMPILED BY S.D
 DATUM LOCAL DATE 2014-8-18 - 2014-8-19 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			20 40 60 80 100							PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT LIMIT		
								SHEAR STRENGTH (kPa)							w _p w w _L		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE							WATER CONTENT (%)		
								20 40 60 80 100							10 20 30		
(continued)																	
79.7 20.9	CLAYEY SILT to SILTY CLAY, trace sand, stiff to hard, grey, moist (GLACIAL TILL)		17	SS	50		85									1 17 61 21	
			18	SS	14												
			19	SS	64												
			20	RC													
				frequent cobbles and boulders			21	SS	100 / 75mm								
			22	RC													
76.8	BEDROCK - GRANODIORITE SCHIST, unweathered, massive, brownish grey, very high strength (with secondary weak planes)		23	SS	100 / 50mm		80									Run #1 TCR: 100% SCR: 100% RQD: 100%	
			1	RUN	NQ												
			2	RUN	NQ												

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

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

WATER LEVEL READINGS

Date	Water Depth (m)	Elevation (m)
Sep 16, 2014	2.7	97.9
Oct 27, 2014	2.4	98.2

report: mto-terraprobe soil file: 3- unknown creek culvert (39e-244) bh logs - copy rev2.gpj

RECORD OF BOREHOLE No 3

1 of 1

METRIC

G.W.P. _____ LOCATION _____ Coords: E:7840.9 N:91785.2 ORIGINATED BY W.Z
DIST _____ HWY 634 BOREHOLE TYPE PORTABLE EQUIPMENT AND SPT SAMPLING COMPILED BY S.D
DATUM LOCAL DATE 2014-8-21 - 2014-8-22 CHECKED BY R.A

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)							WATER CONTENT (%)			
								20 40 60 80 100										
94.7	GROUND SURFACE																	
	100mm TOPSOIL		1	SS	4									GR SA SI CL				
94.0	FILL, silty clay, some sand, trace organics, soft, brown, moist													0 14 64 22				
0.7	PEAT, fibrous, black, wet		2	SS	2													
93.3																		
1.4	CLAYEY SILT, trace to some sand, very stiff to hard, grey, moist (GLACIAL TILL)		3	SS	19													
			4	SS	50									0 17 63 20				
			5	SS	32													
			6	SS	50													
90.4														August 21, 2014 August 22, 2014				
4.3																		

END OF BOREHOLE

RECORD OF BOREHOLE No C1

1 of 1

METRIC

G.W.P. _____ LOCATION _____ Coords: E:7839.9 N:91785.2 ORIGINATED BY W.Z
 DIST _____ HWY 634 BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST COMPILED BY S.D
 DATUM LOCAL DATE 2014-8-22 CHECKED BY R.A

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE			SHEAR STRENGTH (kPa)		W _p	W	W _L		
94.7	GROUND SURFACE							20 40 60 80 100 ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL X LAB VANE		10 20 30 WATER CONTENT (%)			kN/m ³	GR SA SI CL
90.1 4.6	Refer to BH3 for inferred soil stratigraphy													

END OF BOREHOLE

RECORD OF BOREHOLE No C2

1 of 1

METRIC

G.W.P. _____ LOCATION _____ Coords: E:7837.9 N:91785.2 ORIGINATED BY W.Z
 DIST _____ HWY 634 BOREHOLE TYPE DYNAMIC CONE PENETRATION TEST COMPILED BY S.D
 DATUM LOCAL DATE 2014-8-22 CHECKED BY R.A

SOIL PROFILE		SAMPLES				GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH (m)	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	SPT 'N' VALUE								
94.7	GROUND SURFACE												
90.8	Refer to BH3 for inferred soil stratigraphy												
3.9													

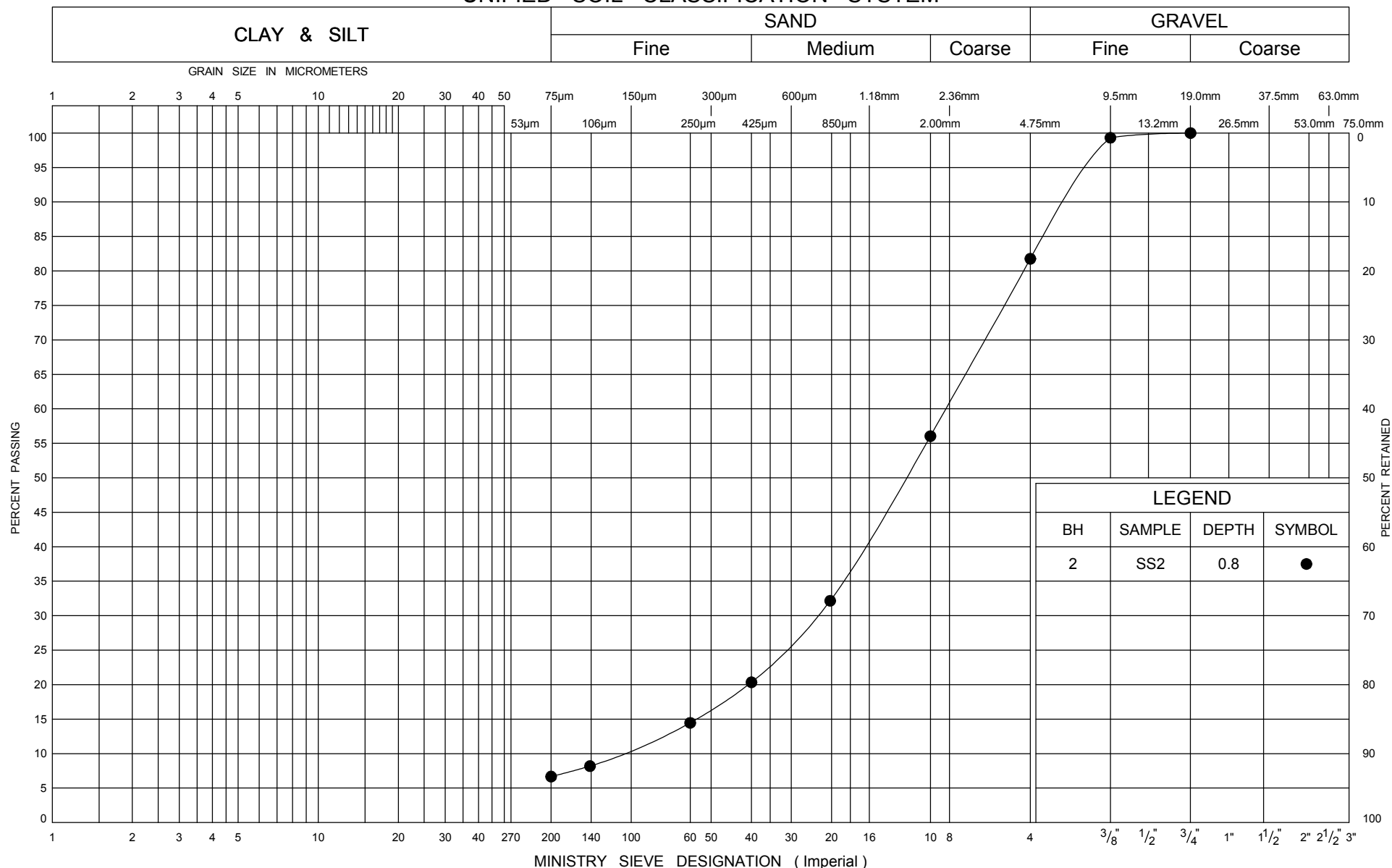
END OF BOREHOLE

APPENDIX B

Laboratory Test Results & Photographs



UNIFIED SOIL CLASSIFICATION SYSTEM



GRAIN SIZE DISTRIBUTION FILL-SAND

FIG No B1

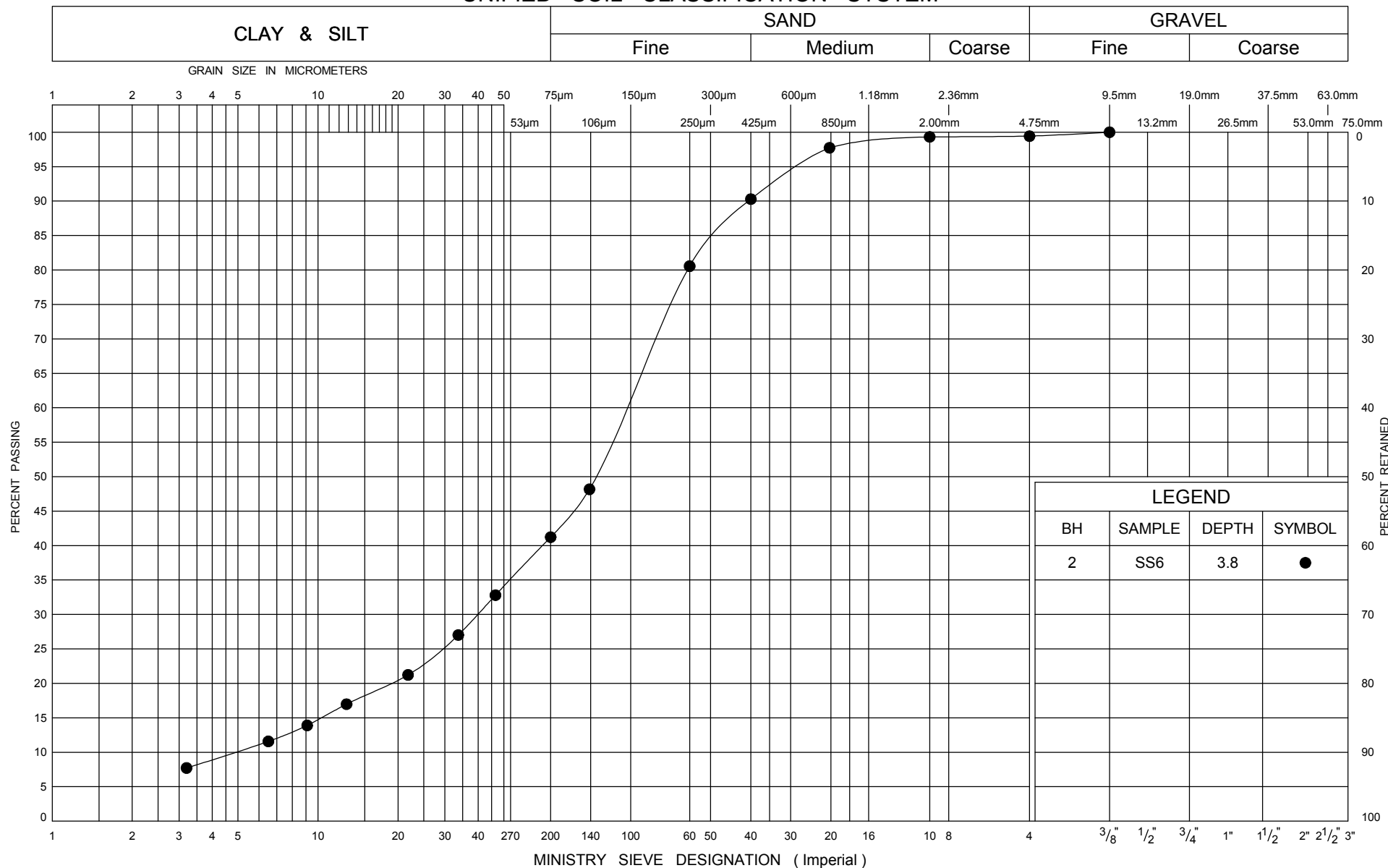
G W P 5379-11-00

Unnamed Creek Culvert (39E-244)

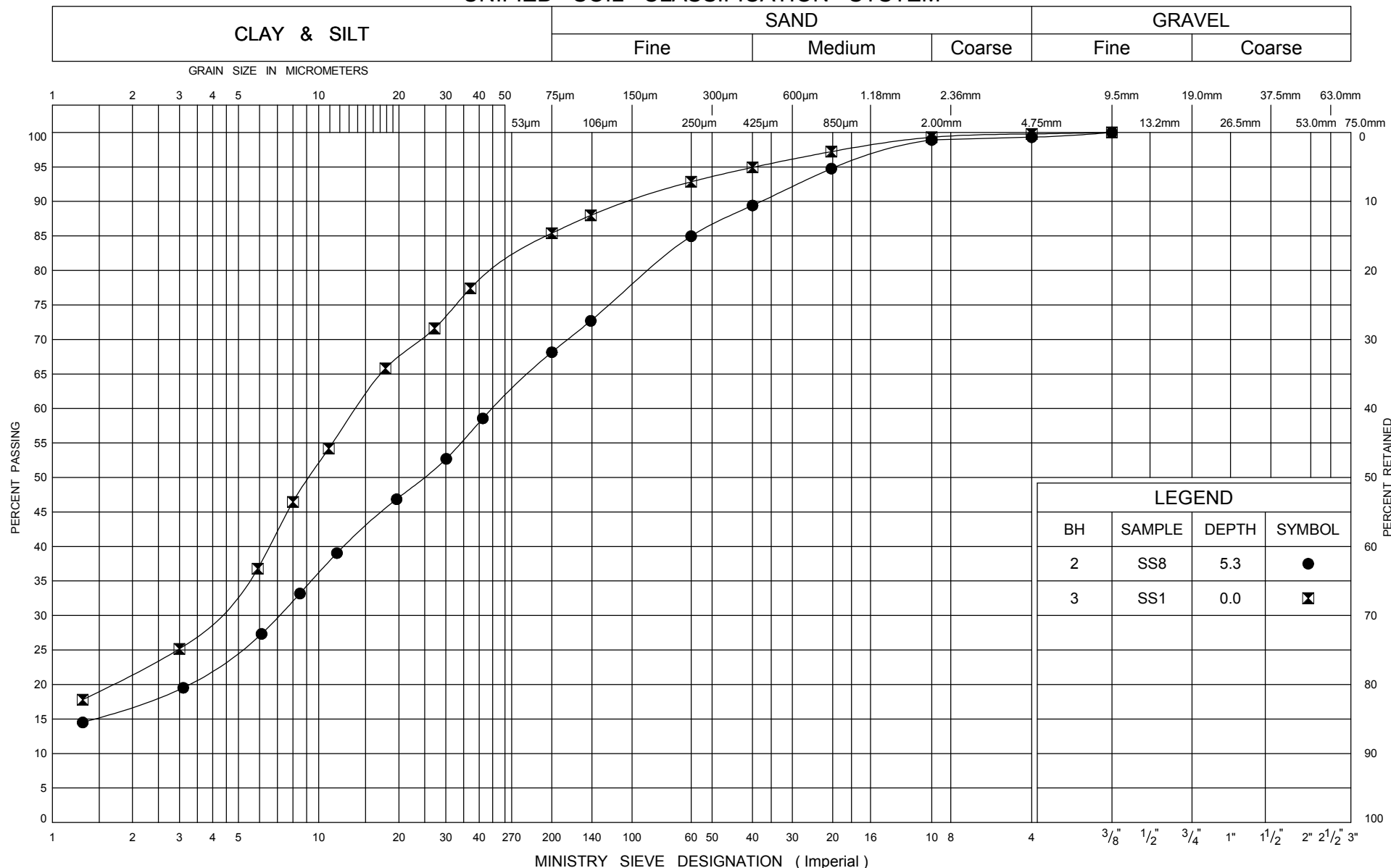


Ministry of
Transportation

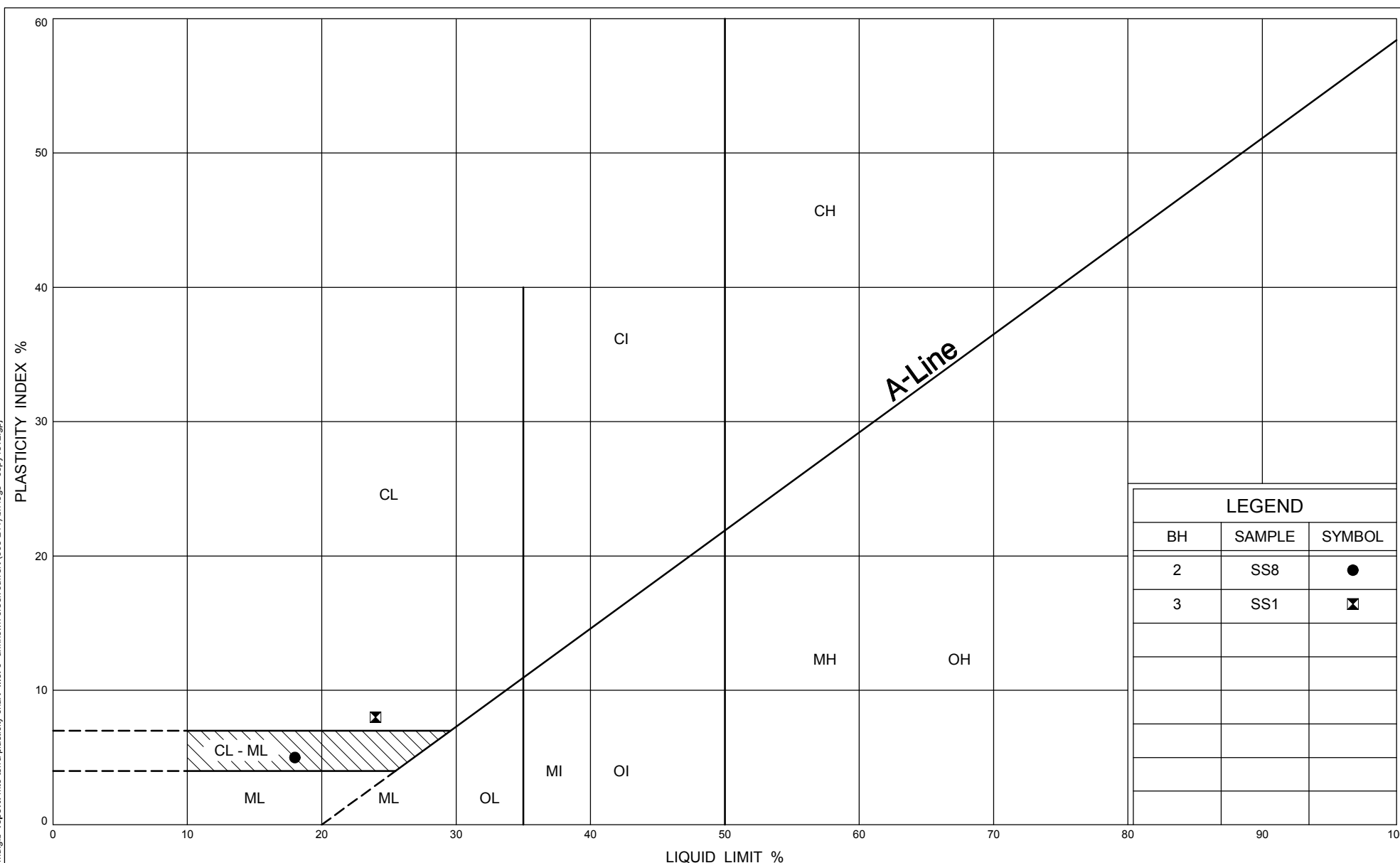
UNIFIED SOIL CLASSIFICATION SYSTEM



UNIFIED SOIL CLASSIFICATION SYSTEM



library: library - terraprobe.gint - md.glb report: mto-terra-plasticity-chart file: 3-unknown-creek-culvert (39E-244) bh-logs - copy rev2.gpj



Ministry of
Transportation

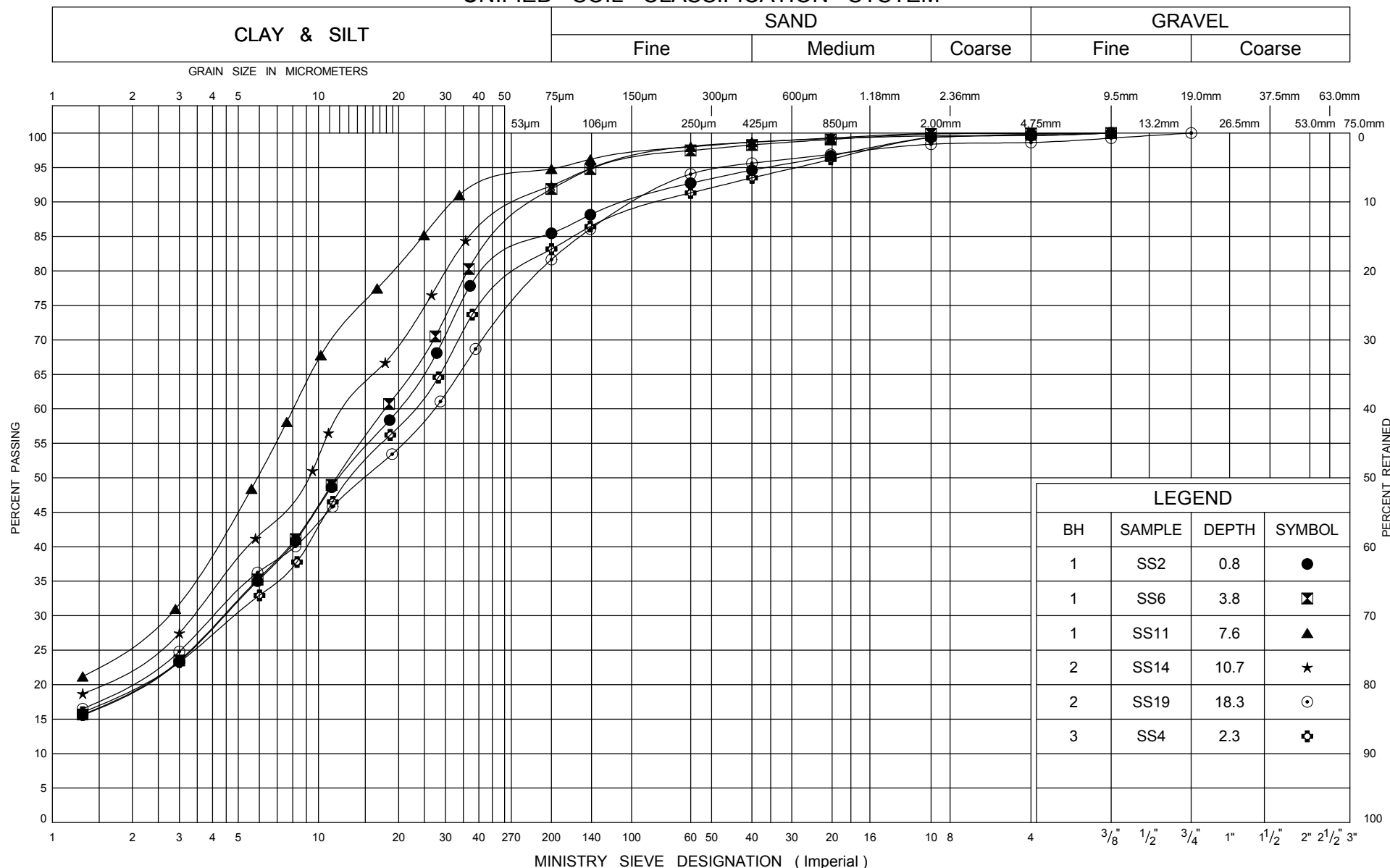
PLASTICITY CHART FILL-CLAYEY SILT TO SILTY CLAY

FIG No B4

G W P 5379-11-00

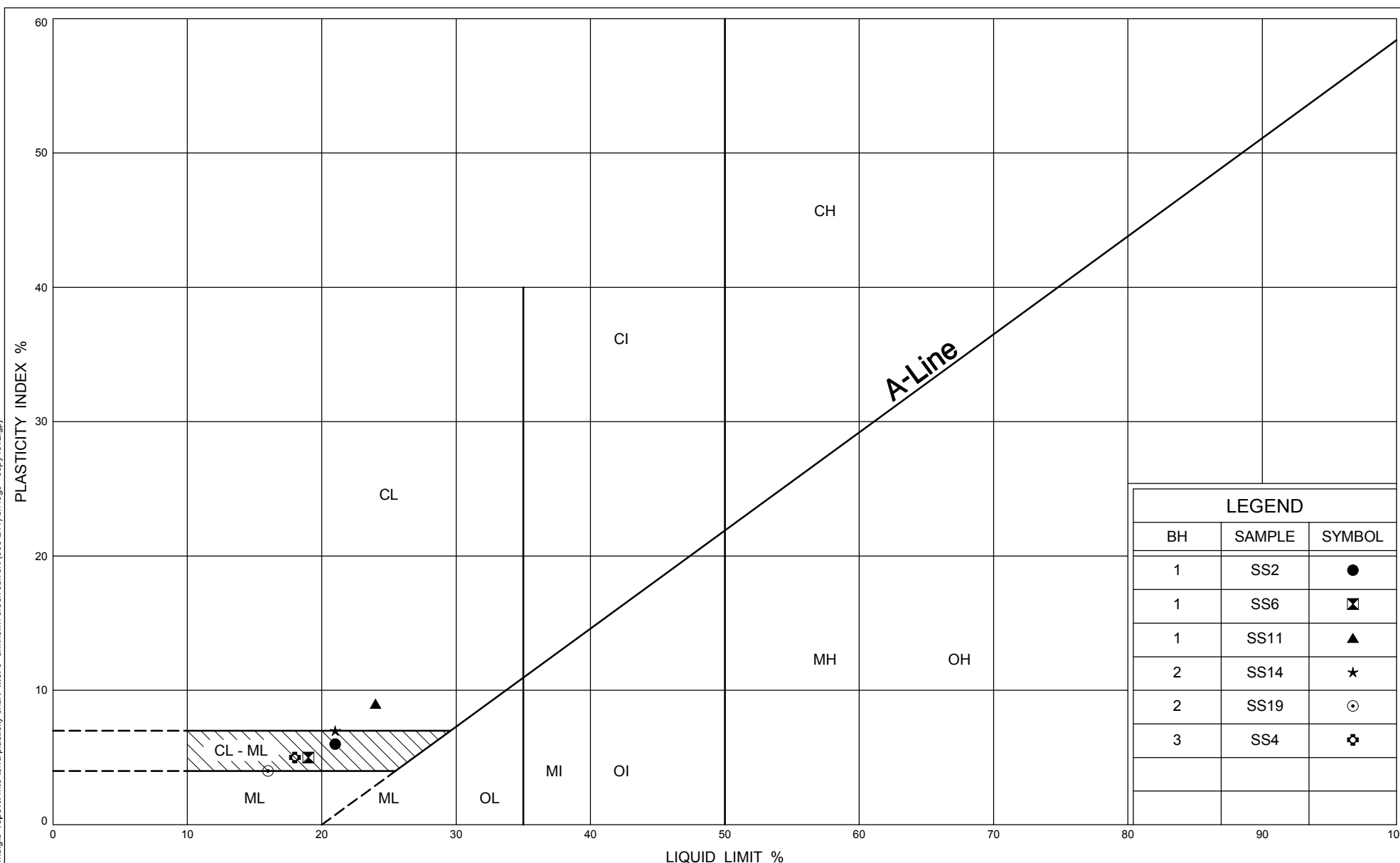
Unnamed Creek Culvert (39E-244)

UNIFIED SOIL CLASSIFICATION SYSTEM



LEGEND			
BH	SAMPLE	DEPTH	SYMBOL
1	SS2	0.8	●
1	SS6	3.8	⊠
1	SS11	7.6	▲
2	SS14	10.7	★
2	SS19	18.3	⊙
3	SS4	2.3	⊕

library: library - terraprobe.gint - md.glb report: mto-terra-plasticity-chart file: 3-unknown-creek-culvert (39E-244) bh logs - copy rev2.gpj



Ministry of
Transportation

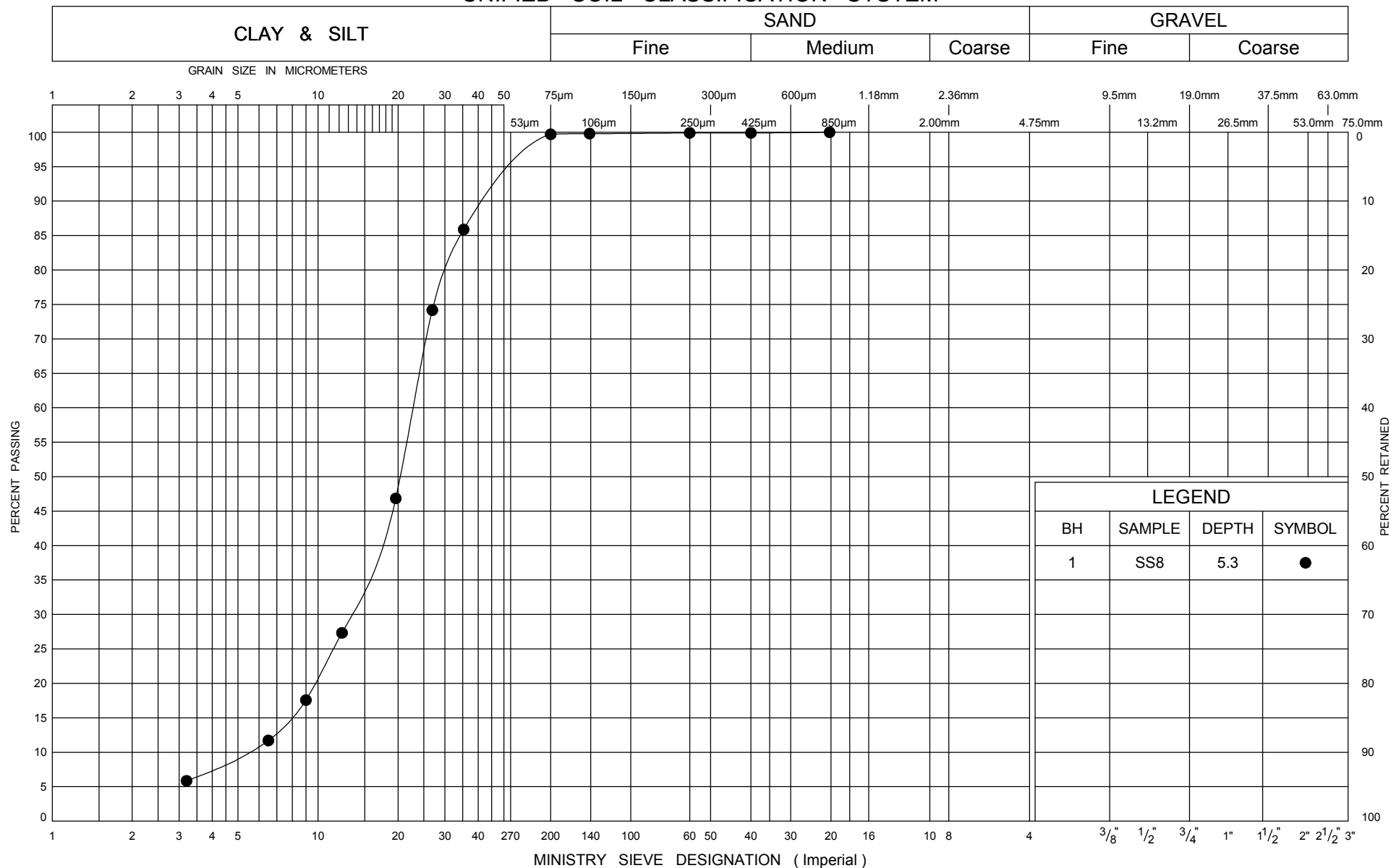
PLASTICITY CHART CLAYEY SILT TO SILTY CLAY TILL

FIG No B6

G W P 5379-11-00

Unnamed Creek Culvert (39E-244)

UNIFIED SOIL CLASSIFICATION SYSTEM



PHOTOGRAPH OF COBBLES AND BOULDERS

FIGURE B8

UNNAMED CREEK CULVERT (Site 39E-244)



C:\Users\monfared\Desktop\11-14-4066\Unnamed 244\Spread Sheets\le0-Pc-Cc-Cr-Cu.xls

Project No. : 11-14-4066
Date : June, 2015



Terraprobe Inc.

Prepared by : SD
Checked by : RA

UNNAMED CREEK CULVERT (Site 39E-244)



C:\Users\monfared\Desktop\11-14-4066\Unnamed 244\Spread Sheets\0-Pc-Cc-Cr-Cu.xls

Project No. : 11-14-4066
Date : June, 2015

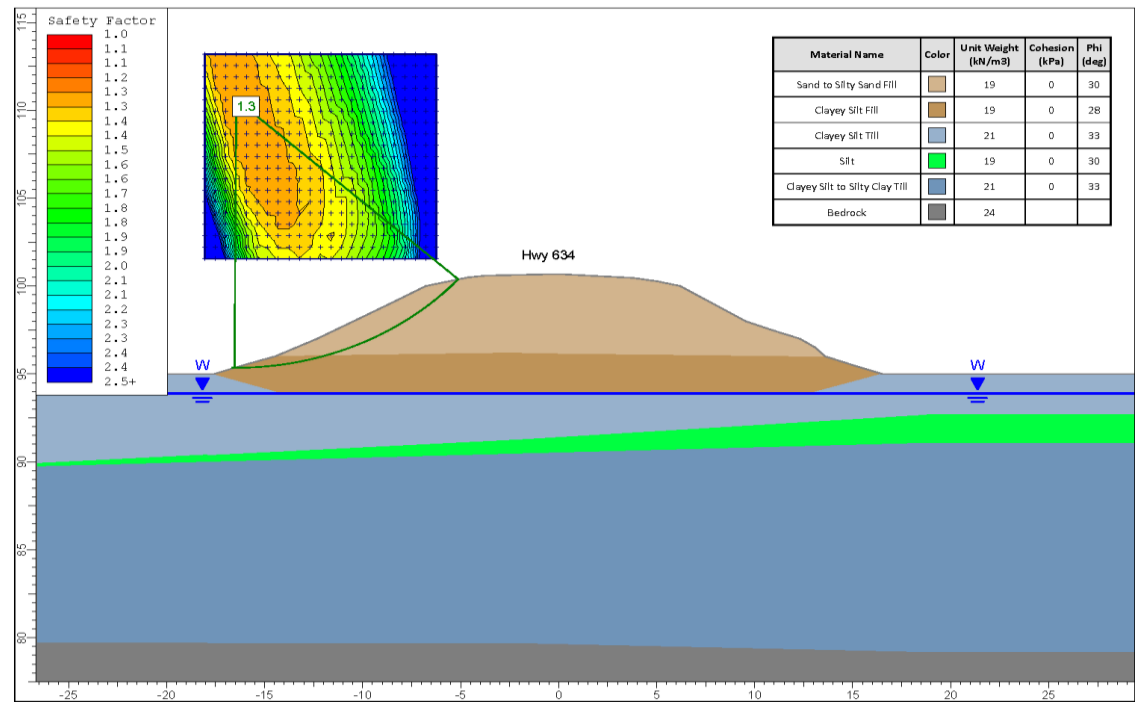


Prepared by : SD
Checked by : RA

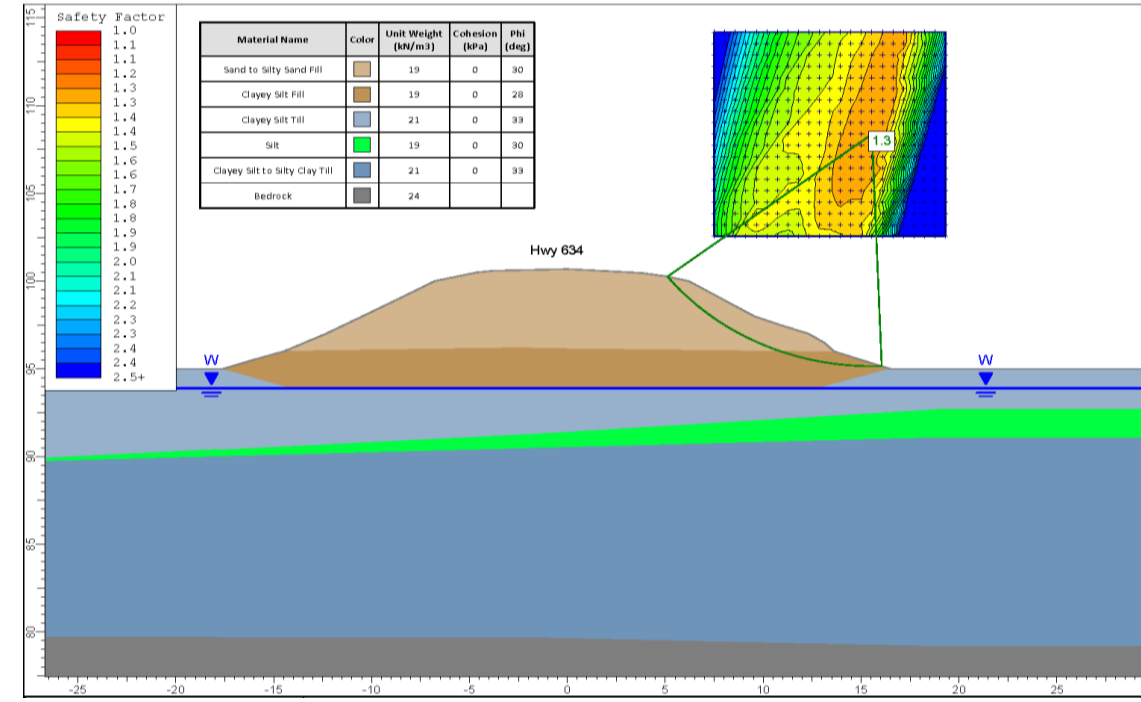
APPENDIX C

Slope Stability Models & Results




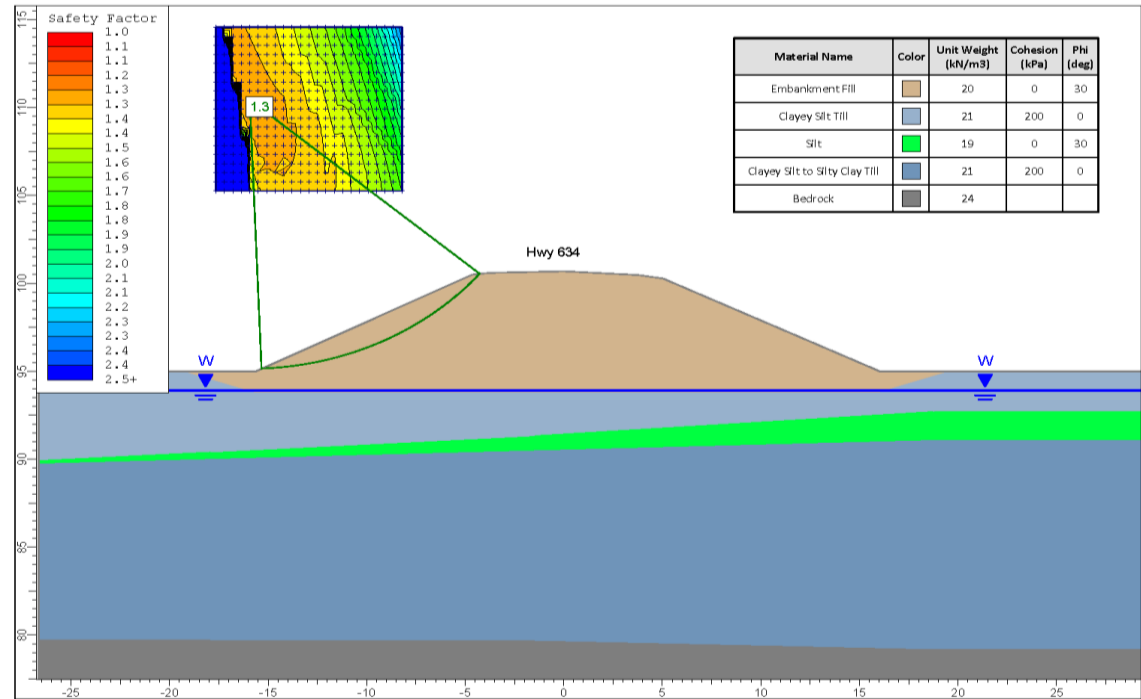


Slip Lining Option - Existing Embankment - Effective Stress Analysis
Left of Hwy 634 Centre Line

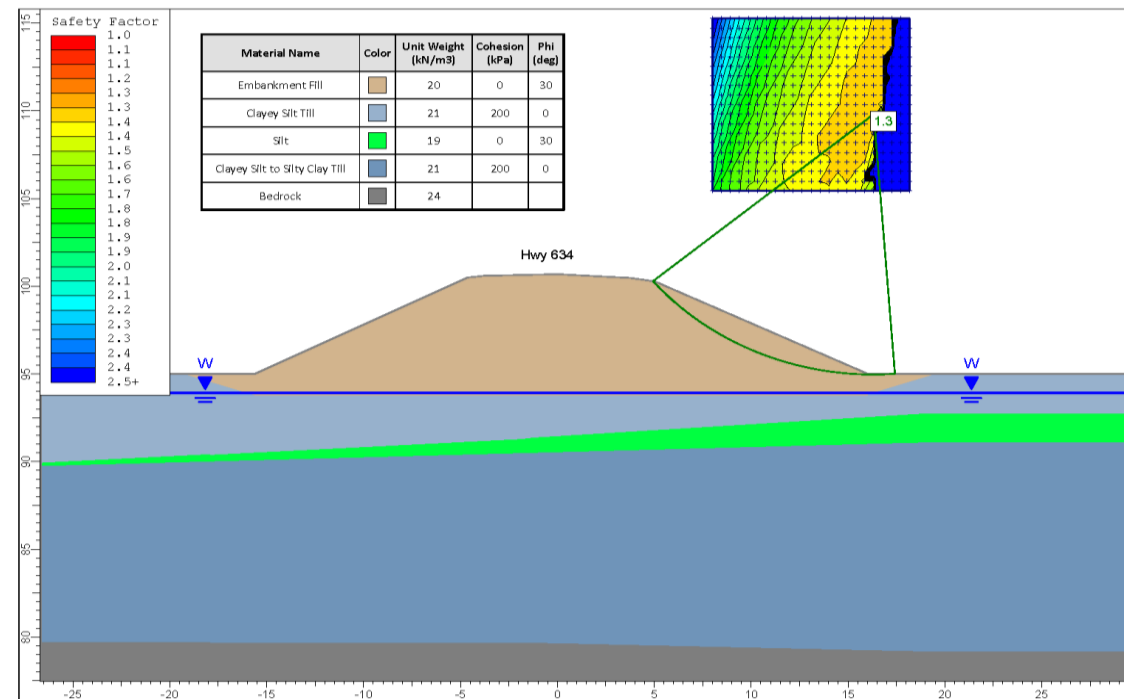


Slip Lining Option - Existing Embankment - Effective Stress Analysis
Right of Hwy 634 Centre Line

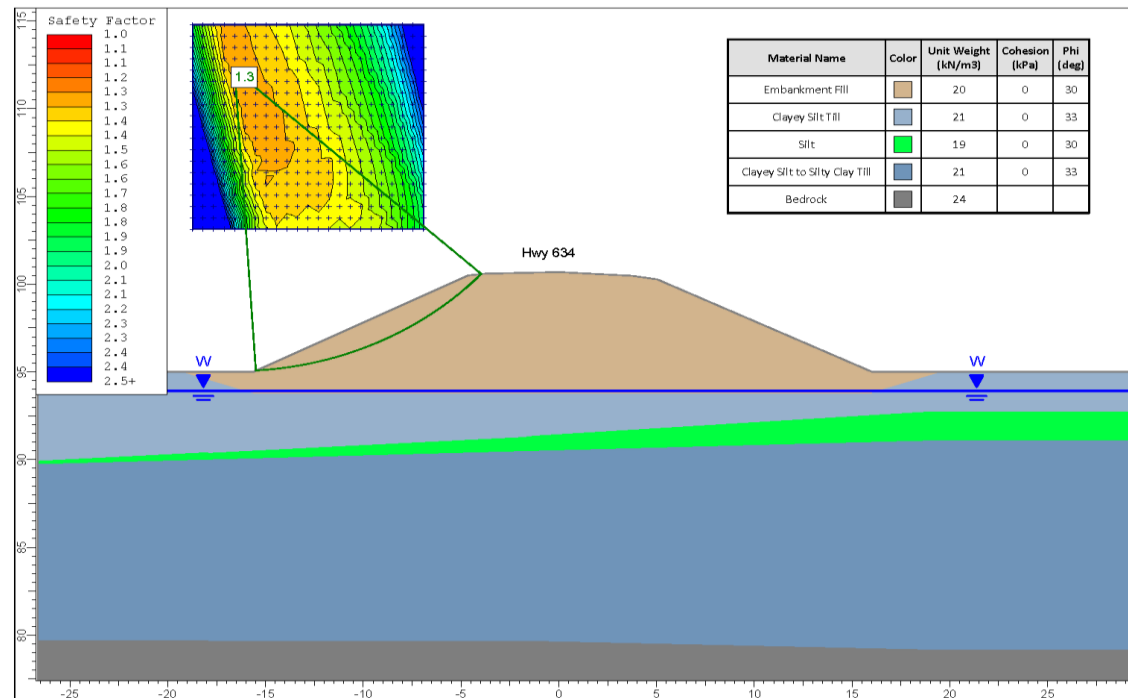
 Terraprobe Inc. Consulting Geotechnical & Environmental Engineering Construction Materials, Inspection & Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	HWY 634 UNNAMED CREEK CULVERT , SITE 39E-244			
	G.W.P. 5379-11-00		DATE: JUNE, 2016	
	SUBM'D: SD	CHKD: RA	APPD: MT	
	Project No: 11-14-4066		Figure: C1	



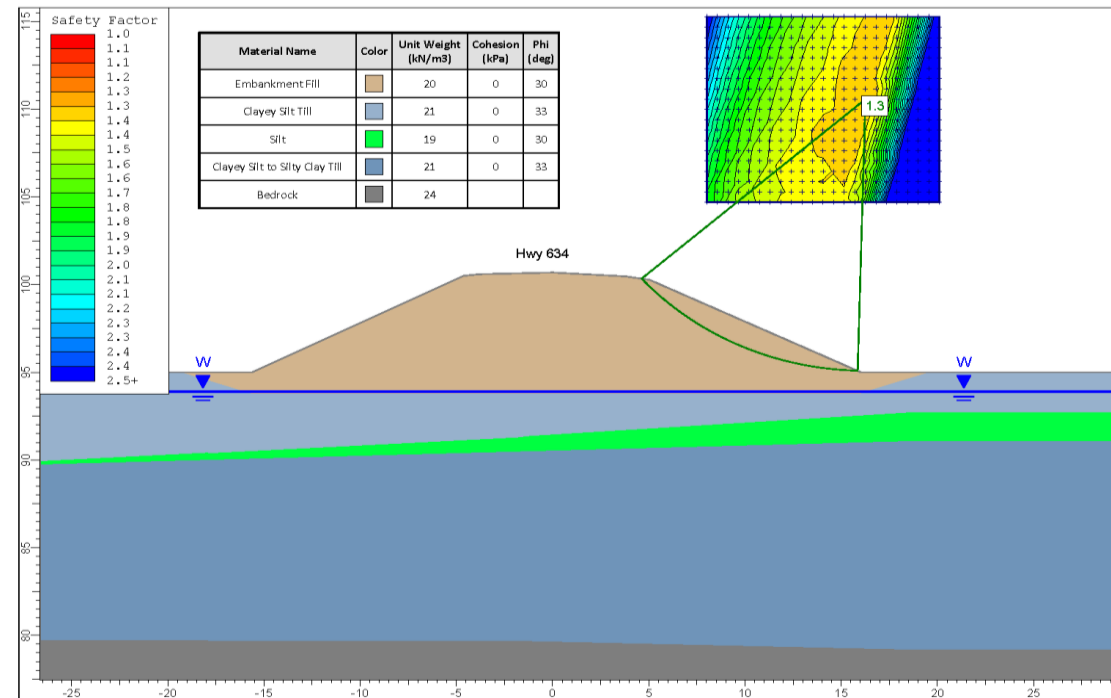
Culvert Replacement Option - New Embankment - Total Stress Analysis
Left of Hwy 634 Centre Line




Culvert Replacement Option - New Embankment - Total Stress Analysis
Right of Hwy 634 Centre Line



Culvert Replacement Option - New Embankment - Effective Stress Analysis
Left of Hwy 634 Centre Line



Culvert Replacement Option - New Embankment - Effective Stress Analysis
Right of Hwy 634 Centre Line

	HWY 634			
	UNNAMED CREEK CULVERT, SITE 39E-244			
	G.W.P. 5379-11-00		DATE: JUNE, 2016	
	SUBM'D: SD	CHKD: RA	APPD: MT	
	Project No: 11-14-4066		Figure: C2	