Terraprobe

Consulting Geotechnical & Environmental Engineering

Construction Materials Inspection & Testing

FOUNDATION INVESTIGATION AND DESIGN REPORT
TEE, LYONS AND BLACK CREEKS BRIDGE STRUCTURES
TEE CREEK NORTH BOUND BRIDGE REPLACEMENT
QUEEN ELIZABETH WAY (QEW)
MINISTRY OF TRANSPORTATION, ONTARIO
DB-2014-2036, SITE:34-67/1
GEOCRES NO. 30M3-289

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TEE CREEK NORTH BOUND BRIDGE REPLACEMENT
QUEEN ELIZABETH WAY
REGIONAL MUNICIPALITY OF NIAGARA, ONTARIO
CONTRACT NUMBER DB-2014-2036

1.0 INTRODUCTION

Terraprobe Inc. (Terraprobe) has been retained by MMM Group Limited (MMM) on behalf of Dufferin Construction, to provide foundation engineering services in support of detailed designs for the replacement of the Tee Creek North-Bound bridge.

This project is based on the Ministry of Transportation, Ontario (MTO) Design Build Minor Request for Proposals titled *"Tee Creek, Lyons and Black Creeks Bridge Structures, Central Region", Contract Number. DB-2014-2036*. The terms of reference and scope of work for the foundation engineering services are outlined in MTO's RFP.

This report presents the factual data on the subsurface conditions at the Tee Creek North Bound Bridge on the Queen Elizabeth Way (QEW), City of Niagara Falls, Regional Municipality of Niagara, Ontario.

2.0 SITE DESCRIPTION

The site (with MTM coordinates of N 4,765,575; E 336,725) is located on the QEW, approximately 525 m south of the Lyons Creek Road underpass in the City of Niagara Falls, Ontario. The key plan on the Borehole Locations and Soil Strata Drawing, (Drawing 1) provides an overview of the site location.

The existing north-bound bridge is a cast-in-place concrete T-beam structure that is approximately 30.3 m long and 11.6 m wide. The bridge consists of a 19.7 m centre span and two 5.3 m long cantilevered end spans. Construction records indicate that the bridge foundations are supported on piles.

The terrain at the bridge site and surrounding area is generally flat and the bridge spans Tee Creek which flows from west to east. The topography and contour elevations indicate that the bridge was constructed to span Tee Creek via approach embankments consisting of 3± m high side slopes and 4± m high forward slopes.

3.0 INVESTIGATION PROCEDURES

3.1.1 Current Investigation

The field work for this project was carried out from November 23 to 30, 2016 and consisted of drilling and sampling two boreholes to depths of 32.7 m and 35.1 m below ground surface. The approximate borehole locations are shown on Drawing 1.

Based on drawings provided by Terraprobe, MMM's surveyors staked out the borehole locations and supplied the borehole coordinates and geodetic elevations to Terraprobe. The actual borehole locations drilled by Terraprobe are referenced to MMM's original staked location.

The boreholes were drilled with a truck-mounted drill rig supplied and operated by a specialist drilling contractor. 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 D1586[1]. Relatively undisturbed samples of the silty clay to clayey silt soils were collected with thin wall tube samplers (Shelby Tubes) and the undrained shear strength of the soil was determined by performing in-situ field vane tests with an MTO 'N' vane.

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

Ground water conditions in the open boreholes were observed during the drilling operations. The boreholes were backfilled in accordance with current MTO procedures and Ontario Regulation 903 (as amended).

The recovered soil and rock samples were subjected to Visual Identification (VI) and select soil samples were also subjected to a laboratory testing programme consisting of natural moisture content, grain size distribution analyses and Atterberg limits determinations in accordance with MTO and/or ASTM Standards as appropriate.

3.1.2 Previous Investigation

In June and July 2013, subsurface investigations were carried out by Golder Associates Limited (Golder) of Mississauga, Ontario. Two boreholes (Boreholes 13-09 and 13-10) were drilled and the data from these investigations were used to supplement the current investigation. The boreholes were advanced to depths ranging of 30.1 m and 32.1 m below ground surface and the Record of Borehole sheets and associated laboratory test results are provided in Appendix A and B respectively. The approximate locations of these boreholes are shown on Drawing 1.

The Golder boreholes were drilled using continuous flight hollow stem auger drilling techniques. The overburden soil samples were obtained at selected intervals using a split-spoon sampler in conjunction with the Standard Penetration Test (SPT) method.

3.1.3 Borehole Locations

The borehole locations in MTM NAD83 northing and easting coordinates, the ground surface elevations referenced to geodetic datum and depths drilled are summarized in the following table.

Borehole Data

Borehole No.	MTM Coordinate System		Ground Surface Elevation (m)	Borehole Depth (m)
	Northing (m)	Easting (m)		
BH1	4 765 557.5	336 743.2	174.3	32.7
BH2	4 765 589.3	336 713.0	174.1	35.1
BH 13-09	4 765 591.0	336 717.3	174.0	32.1
BH 13-10	4 765 559.9	336 740.9	174.2	30.1

4.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

4.1 Regional Geology

The site is located between the Niagara Escarpment and Lake Erie in the physiographic region of Southern Ontario referred to as the Haldimand Clay Plain. The Haldimand Clay Plain is best described as falling into a series of parallel belts with the highest ground adjacent to the Escarpment. Generally, this region is flat and poorly drained although it includes several distinctive landforms such as dunes, cobble, clay and sand beaches, limestone pavements and back-shore wetland basins[2].

2 Chapman and Putnam, “The Physiography of South Ontario”, 3rd Edition, 1984.

The Niagara Region is underlain by a sequence of very gently south-dipping dolostones, limestones, shales and sandstones overlying Precambrian basement rock. The key elements in the bedrock geology of the region are the multiple layers of softer sedimentary limestones, shale, sandstone and dolostone.

The bedrock unit at this site is the Salina Formation of Upper Silurian Age. This unit consists essentially of grey, very finely crystalline, laminated argillaceous dolostone with grey, calcareous shale partings and gypsum veins and lenses of varying thicknesses.

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” drawings. An overall description of the stratigraphy is given in the following paragraphs.

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

In summary, the ground surface is underlain by a flexible pavement and fill soils consisting of compact sand and firm to stiff silty clay. The pavement and fill material are underlain by deposits of firm to stiff silty clay to clayey silt, loose to dense silt, loose to very dense sandy silt and a very dense layer of sand gravel containing cobble and boulder inclusions. A more detailed description of the subsurface conditions encountered in the boreholes is provided in the following sections.

4.2.1 Flexible Pavement

A flexible pavement consisting of 165 mm to 300 mm thick asphalt concrete underlain by gravelly sand to sand and gravel fill was encountered. Golder Borehole 13-10 also encountered a 300 mm thick bridge approach slab. The locations, thicknesses and base elevations of the gravelly sand to sand and gravel fill are summarized in the following table.

Pavement Granular Borehole Data

Borehole No.	Fill Thickness (mm)	Fill Base Elevation (m)
BH1	1200	172.9
BH2	1100	172.7
BH 13-09	1400	172.6
BH 13-10	600	173.1

Standard Penetration tests carried out in the gravelly sand to sand and gravel fill gave SPT N-values that range from 5 blows to 59 blows for 0.3 m of penetration indicating a loose to very dense relative density.

A grain size distribution test was carried out on a sample of the gravelly sand fill and the results are illustrated on the grain size distribution curve Figure B1, Appendix B1. The grain size distribution curve of a sample of the sand and gravel fill retrieved from Golder Borehole 13-10 is depicted on Figure B1 in Appendix B2. These results show a grain size distribution consisting of 43% and 53% gravel, 42% and

35% sand and, 12% and 15% silt and clay size soil particles. The natural water content of samples of the granular fill range from 3% to 6% by weight.

4.2.2 Fill – Sand

Sand fill material was encountered at this site. Summarized in the following table are the locations, explored depths and base elevations of the sand fill.

Sand Fill Borehole Data

Borehole No.	Sand Fill Thickness (m)	Sand Fill Depth of Deposit (m)	Sand Fill Base Elevation (m)
BH1	0.2	1.6	172.7
BH2	0.2	1.6	172.5
BH 13-10	0.4	1.5	172.7

A Standard Penetration test carried out in this deposit measured an SPT N-value of 20 blows per 0.3 m of penetration indicating a compact relative density. The moisture content of a sample of this fill is 16% by weight.

4.2.3 Fill – Silty Clay

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

Silty Clay Fill Borehole Data

Borehole No.	Silty Clay Fill Thickness (m)	Silty Clay Fill Depth of Deposit (m)	Silty Clay Fill Base Elevation (m)
BH1	2.8	4.4	169.9
BH2	1.8	3.4	170.7
BH 13-09	4.2	5.6	168.4
BH 13-10	3.0	4.5	169.7

Standard Penetration tests performed in the silty clay fill measured SPT N-values that range from 4 to 11 blows for 0.3 m of penetration indicating a firm to stiff consistency. The natural water content of samples of the silty clay fill range from 24% to 35% by weight and in Golder Borehole 13-10 a natural water content of 118% was recorded in an organic layer of the silty clay fill.

The grain size distribution curves of samples of the silty clay fill are depicted on Figure B2 in Appendix B1 and the Golder grain size distribution curves of the silty clay fill are shown on Figure B2, Appendix B2. These results show a grain size distribution consisting of 0% to 1% gravel, 2% to 4% sand, 33% to 47% silt and 49% to 62% clay size particles.

Samples of the silty clay fill were also subjected to Atterberg limits tests and the results are presented on Figure B3 in Appendix B1. The plasticity chart illustrating the Golder Atterberg limits results (Boreholes 13-09 and 13-10) is provided as Figure B3 in Appendix B2. These results indicate that the fill is an intermediate plasticity (CI) cohesive soil. The results from the Atterberg limits tests are summarized below:

Liquid Limit:	45% to 46%
Plastic Limit:	21% to 24%
Plasticity Index:	22% to 25%
Natural Moisture Content:	24% to 29%

4.2.4 Silty Clay to Clayey Silt

Silty clay to clayey silt soils were encountered at this site. Summarized below are the locations, thicknesses, depths and base elevations of these deposits.

Silty Clay to Clayey Silt Borehole Data

Borehole No.	Silty Clay to Clayey Silt Thickness (m)	Silty Clay to Clayey Silt Depth (m)	Silty Clay to Clayey Silt Base Elevation (m)
BH1	19.4	23.8	150.5
BH2	21.6	25.0	149.1
BH 13-09	3.1 9.4 4.7	8.7 18.1 24.8	165.3 155.9 149.2
BH 13-10	18.7	23.2	151.0

Standard Penetration tests carried out in the silty clay to clayey silt measured SPT N-values of 1 to 15 blows per 0.3 m of penetration and in Borehole 2 one SPT test recorded an N-value of 100 blows for less than 0.3 m of penetration. Field vane tests measured in-situ undrained shear strengths that range from 42 kPa to more than 100 kPa as depicted on the undrained shear strength versus elevation plot in Figure B4, Appendix B1. Based on these tests the silty clay to clayey silt is described as having a generally firm to stiff consistency. The sensitivity of the silty clay to clayey silt varies from 1.0 to 4.0, indicating a low sensitivity soil class (Canadian Foundation Engineering Manual [CFEM], 2006).

The Terraprobe grain size distribution plots of twelve samples of the silty clay are depicted in Figures B5 and B6 in Appendix B1. The grain size distribution plots of two samples of the silty clay to clayey silt from the Golder study (Boreholes 13-09 and 13-10) are illustrated in Figure B4 in Appendix B2. These results show a grain size distribution consisting of 0% to 7% gravel, 0% to 6% sand, 31% to 68% silt and, 32% to 68% clay sized particles. The moisture content of samples of the silty clay to clay silt varies from 18% to 37% by weight and the unit weight of a tested sample is 20 kN/m^3.

Samples of the silty clay deposit from the Terraprobe study were also subjected to Atterberg limits tests and the results are presented in Figures B7 and B8 in Appendix B1. The Atterberg limits tests of two samples of the silty clay to clayey silt from the Golder study (Boreholes 13-09 and 13-10) are plotted on the plasticity chart in Figure B6, Appendix B2. These values indicate that the silty clay to clayey silt deposit is a low to intermediate plasticity (CL-CI) cohesive soil. The Atterberg limits test results are summarized below and the results are also plotted versus elevation in Figure B9, Appendix B1.

Liquid Limit:	26% to 48 %
Plastic Limit:	16% to 22%
Plasticity Index:	10% to 27%
Natural Moisture Content:	18% to 33%

A one-dimensional consolidation test was performed on a sample of the silty clay and the results are presented in Figures B10 to B12 in Appendix B1. 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
BH1, Sample TW11	9.4/164.9	114.0	185.0	0.259	0.04	0.69

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 slightly higher than the effective overburden pressure suggesting that the silty clay deposit is slightly over-consolidated.

4.2.5 Silt

Silt deposits were encountered at this site and the locations, explored depths and base elevations of the deposits are summarized in the following table.

Silt Borehole Data

Borehole No.	Silt Thickness (m)	Silt Depth (m)	Silt Base Elevation (m)
BH1	5.5	29.3	145.0
BH2	3.7	28.7	145.4
BH 13-09	4.2	29.0	145.0
BH 13-10	-	30.1*	144.1

* Borehole termination depth.

Standard Penetration tests carried out in this deposit measured SPT N-values that range from 5 blows to 40 blows per 0.3 m of penetration indicating a loose to dense relative density. The moisture content of samples of the silt range from 19% to 24% by weight.

A Terraprobe grain size distribution curve of a silt sample is shown in Figure B13 in Appendix B1. The grain size distribution plots of two samples of the silt from the Golder study (Boreholes 13-09 and 13-10) are illustrated in Figure B7 in Appendix B2. These results show a grain size distribution consisting of 0% gravel, 0% to 5% sand, 84% to 95% silt and, 1% to 7% clay sized particles.

4.2.6 Sandy Silt

Sandy silt deposits were encountered at this site. Summarized in the following table are the locations, explored depths and base elevations of the sandy silt.

Sandy Silt Borehole Data

Borehole No.	Sandy Silt Thickness (m)	Sandy Silt Depth (m)	Sandy Silt Base Elevation (m)
BH2	3.2	31.9	142.2
BH 13-09	2.0 -	20.1 32.1	153.9 141.9

* Borehole termination depth.

Standard Penetration tests carried out in this deposit measured SPT N-values of 5 blows to 61 blows per 0.3 m of penetration indicating a loose to very dense relative density. The moisture content of samples of the sandy silt range from 20% to 21% by weight.

In the Golder study grain size distribution tests were carried out on two soil samples (Borehole 13-09) and the results are shown on the grain size distribution curves Figures B5 and B7, Appendix B2. These results show a grain size distribution consisting of 8% to 20% gravel, 25% to 27% sand, 46% to 56% silt and, 7% to 11% clay sized particles.

4.2.7 Sandy Gravel

A sandy gravel deposit containing random cobble and boulder inclusions was encountered in the Terraprobe boreholes. Summarized in the following table are the locations, explored depths and base elevations of this deposit.

Sandy Gravel Borehole Data

Borehole No.	Sandy Gravel Thickness (m)	Sandy Gravel Depth of Deposit (m)	Sandy Gravel Base Elevation (m)
BH1	-	32.7*	141.6
BH2	-	35.1*	139.0

* Borehole termination depth.

Standard Penetration tests carried out in this deposit measured SPT N-values that are more than 100 blows per 0.3 m of penetration indicating a very dense relative density. The moisture content of a sample from this stratum is 6% by weight.

4.3 Ground Water Levels

The ground water conditions were observed in the boreholes during and upon completion of drilling. Based on the ground water observations, the soil moisture contents, and the creek water level; the ground water level at this site is estimated to be at an approximate elevation of 172.5± m. The ground water level is expected to fluctuate seasonally, will be controlled by the free water level in the creek, and is expected to rise during wet periods of the year.

5.0 MISCELLANEOUS

The investigation was carried out using equipment supplied and operated by DBW Drilling Services of Toronto, Ontario. The field operations were monitored by Ms. Sepideh D-Monfared, MESc., who observed the drilling, sampling and in situ testing operations and logged the boreholes. The 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. 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

TEE CREEK NORTH BOUND BRIDGE REPLACEMENT
QUEEN ELIZABETH WAY
REGIONAL MUNICIPALITY OF NIAGARA, ONTARIO
CONTRACT NUMBER DB-2014-2036

6.0 DISCUSSION AND ENGINEERING RECOMMENDATIONS

6.1 General

This report presents interpretation of the geotechnical data in the factual report and presents geotechnical design recommendations to assist the design team to select a preferred foundation alternative for the Tee Creek North-Bound Bridge replacement.

This report was prepared in the context of a design-build contract. 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. If conditions are encountered during construction that are different than what is understood at the time this report was prepared, based on the subsurface conditions and testing described herein; Terraprobe must be consulted to update, supplement or otherwise revise these recommendations as appropriate.

The existing north-bound bridge is a cast-in-place concrete T-beam structure that is approximately 30.3 m long and 14.5 m wide. The bridge has a 19.7 m centre span and two 5.3 m long cantilevered end spans. This bridge carries the QEW north bound traffic over Tee Creek.

The replacement structure being considered is a single span integral abutment bridge with a span length of 22.8 m and a deck width of 17.05 m. The bridge will be constructed on the same alignment as the existing structure. A profile sag occurs on the bridge, requiring deck drains adjacent to the east and west barrier walls at the low point on the bridge.

6.2 Foundation Alternatives

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

- Spread footings;
- Augered Caissons (drilled shafts); and
- Driven piles.

6.2.1 Spread Footings

The firm to stiff silty clay to clayey silt deposit is unsuitable for supporting spread footings. The geotechnical resistance of this deposit is low and spread footings will also experience large time dependent consolidation settlements. There are also no advantages in founding spread footings on an engineered fill pad because the geotechnical resistance of the silty clay to clayey silt deposit remains low with increasing depth. Consequently, spread footings are not considered to be a practical foundation alternative.

6.2.2 Caissons (Drilled Shafts)

Caissons will have to be founded at depths in the order of 30± m to 32± m below ground surface, in the submerged and very dense sandy gravel layer containing cobbles and boulders. It would be difficult to seal the bottom of the liner to exclude ground water because of the high permeability of the sandy gravel layer and the presence of cobbles and boulders. Attempts at dewatering and maintaining a sufficiently dry excavation to permit cleaning, inspection and high quality construction, would be challenging and most likely impractical. Therefore, caisson foundations are not recommended for supporting the structure.

6.2.3 Driven Piles

Steel tube piles were considered but were excluded. During pile driving these "high displacement" piles will temporarily alter the pore water pressure of the silty clay to clayey silt and silt deposits, causing a substantial increase in penetration resistance and heave. Since the sandy gravel layer contains cobbles and boulders, it may also be impossible to drive "high displacement" steel tube piles to the required penetration depth to achieve the desired load carrying capacity. Alternatively, H-piles are low displacement sections that have a higher probability of being installed successfully by driving to refusal in the sandy gravel deposit.

6.2.3.1 Axial Resistance

The concentric axial factored geotechnical design resistance at ULS, the geotechnical reaction at SLS, and the estimated pile tip elevations of HP 310x110 steel piles are tabulated below. The structural resistance of the pile should also be checked by the structural designer.

Axial Resistance of HP 310x110 Driven Piles

Location	Reference Borehole	Estimated Pile Tip Elevation (m)	Founding Stratum	Factored Axial Resistance U.L.S (kN)	SLS (25 mm Settlement) (kN)*
South Abutment	BH1	143.0±	Sandy Gravel	1700	1200
North Abutment	BH2	140.0±			

6.2.3.2 Downdrag

The new bridge abutment backfill will be placed in the open forward slope area below the existing bridge where excavations will extend to underside abutment elevations of elevation 170± m. Abutment backfill placed in this area will impart an additional load to the underlying silty clay to clayey silt deposit thereby causing consolidation settlement to occur.

The construction staging requires the bridge to be constructed before the abutment backfill is placed and before consolidation settlement is complete. Therefore, downdrag loads will be imparted to the piles. An HP 310x110 pile section shall be designed for an unfactored downdrag load of 500 kN per pile.

6.2.3.3 Pile Tips

The tips of all piles should be fitted with a pile point from an approved manufacturer such as Titus Steel Company (Standard H-Point, HPP-S Series) or Associated Pile & Fitting Corp. (APF Hard Bite H-Pile Point). The use of a pile point is recommended for the following reasons:

- The piles will be penetrating into soil containing cobbles and boulders and these aggressive driving conditions require a higher level of tip protection; and

- A pile point will provide increased cutting ability to the pile section, reduce the probability of misalignment and increase the probability of achieving the desired penetration in the sandy gravel deposit.

6.2.3.4 Pile Installation

Pile installation should be carried out in accordance with OPSS 903, November 2009. Steel H-piles will be driven to practical refusal in the sandy gravel deposit. Since this deposit contains cobbles and boulders, piles may encounter effective refusal in this stratum without reaching the predicted pile tip elevations.

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 Ultimate Pile Resistance "R" must have a minimum value of 3,200 kN and be greater than the Ultimate Geotechnical Resistance (or twice the ULS Design Load). Hiley formula calculations need not be carried out until the pile has been driven below Elev. 141± m at the north abutment and Elev. 144± m at the south abutment.

The pile driving hammer must be capable of installing the piles to the depths specified in the contract document. A suitable hammer capable of delivering a rated energy of at least 60 kJ/blow, but not more than 70 kJ/blow is recommended.

6.2.3.5 Integral Abutment Considerations

The ground conditions at this site are considered suitable for an integral abutment design. The integral abutment design requires that the piles possess flexibility in the upper 3 m of the pile length. The borehole data indicates that the upper 3 m of pile will be surrounded by firm silty clay fill and the native firm silty clay deposit. Based on the consistency of these soils, lateral pile movement is not expected to be constrained.

6.2.3.6 Lateral Resistance

The lateral resistance of the piles may be calculated using a value for the coefficient of horizontal subgrade reaction (k_s) and the ultimate lateral resistance (p_{ult}) as outlined in the equations below:

	k_s	=	n_h z / D [cohesionless soils]	(kN/m^3)
	k_s	=	67 S_u/D [cohesive soils]	(kN/m^3)
	p_{ult}	=	3 γ z K_p [cohesionless soils]	(kPa)
	p_{ult}	=	9 S_u [cohesive soils]	(kPa)
where	z	=	depth of pile embedment	(m)
	D	=	pile width	(m)
	S_u	=	undrained shear strength	(kPa)
	n_h	=	coefficient of horizontal subgrade reaction	(kN/m^3)
	γ	=	unit weight	(kN/m^3)
	K_p	=	passive earth pressure coefficient	(dimensionless)

The spring constant K, for analysis of a pile segment or element of length L metres, can be obtained from the expression, K = k_s x L x D (kN/m). The ultimate lateral resistance P_{ult}, of a pile segment or element of length L metres, can be obtained from the expression, P_{ult} = p_{ult} x L x D.

The equations provided above and the soil parameters provided in the following table, may be used to analyze the interaction between a pile and the surrounding soil. The lateral pressures obtained from the analysis must not exceed the ultimate lateral resistance or the factored structural flexural resistance of the pile. A maximum horizontal passive resistance of 120 kN (ULS) is recommended for design.

Recommended Soil Parameters

Area Reference Borehole No	Applicable Elevation**	Soil Type	Bulk Unit Weight (kN/m^3)	Angle of Internal Friction (ϕ) Degrees	Undrained Shear Strength (S_u) (kPa)	Recommended n_h Value (kN/m^3)*
Tee Creek North Bound Bridge						
South Abutment BH 1 and BH 13-10	170.2 – 169.7	Fill – Silty Clay	19	0	25	–
	169.7 – 150.5	Silty Clay to Clayey Silt	20	0	60	–
	150.5 – 145.0	Silt	19	30	–	11000
	145.0 – 141.6	Sandy Gravel	21	35	–	11000
North Abutment BH 2 and BH 13-09	170.2 – 168.4	Fill – Silty Clay	19	0	25	–
	168.4 – 149.1	Silty Clay to Clayey Silt	20	0	60	–
	149.1 – 145.4	Silt	19	30	–	4400
	145.4 – 142.2	Sandy Silt	19	33	–	11000
	142.2 – 139.0	Sandy Gravel	21	35	–	11000

* Values estimated based on Table 20.3 data, Canadian Foundation Engineering Manual, 3rd edition, 1992
** Based on an underside abutment elevation of 170.2 m.

Since the piles are end bearing, their vertical resistance will not be significantly affected by the pile spacing. Pile interaction should be considered with reference to the Canadian Highway Bridge Design Code, 2006 (CHBDC 2006) Clause 6.8.9.2.

For lateral soil/pile group interaction analysis, the equation for k_s quoted in this section may be used in conjunction with appropriate reduction factors. Intermediate values of the horizontal subgrade reaction reduction factor R may be obtained by interpolation. Where a pile group is oriented perpendicular to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Perpendicular to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
4 D*	1.00
1 D*	0.50

* D is the width of the pile, and spacing is measured centre to centre.

Where a pile group is oriented parallel to the direction of loading, group action may be considered by reducing values for k_s by a reduction factor R as follows:

Pile Spacing Parallel to Direction of Loading	Horizontal Subgrade Reaction Reduction Factor, R
8 D*	1.00
6 D*	0.70
4 D*	0.40
3 D*	0.25

* D is the width of the pile, and spacing is measured centre to centre.

6.2.4 Recommended Foundation Scheme

From a geotechnical point of view, it is recommended that the new bridge be supported on steel H-pile foundations driven to effective refusal in the sandy gravel deposit. Based on the advantages, disadvantages, risks and consequences, a steel H-pile foundation scheme is reliable, allows for the design of an integral abutment bridge and has the highest probability of acceptable structural performance.

6.2.5 Design Frost Depth

Pile caps should be founded at a minimum depth of 1.2 m of earth cover below the lowest surrounding grade to provide adequate protection against frost penetration, as per OPSD 3090.101. Rock protection provides frost protection equivalent to 50% of the layer thickness and, this aspect should be considered when designing frost depths.

6.3 Lateral Earth Pressure

Earth pressures are generally calculated using the following expression:

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

where P_h = horizontal pressure on the wall (kPa);
K = lateral earth pressure coefficient;
γ = unit weight of retained soil (kN/m^3);
h = depth below top of fill where pressure is computed (m); and
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 30º to 35º the magnitude should be 12 kPa at the top of the fill decreasing linearly to 0 kPa at a depth of 1.7 m; or decreasing linearly to 0 kPa at a depth of 2.0 m for soils with an angle of internal friction that exceeds 35º.

The lateral earth pressure coefficients 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)					
	OPSS Granular A or OPSS Granular B Type II ϕ = 35°; γ = 22.8 kN/m^3		OPSS Granular B Type I ϕ = 32°; γ = 21.2 kN/m^3		Ultra Light Weight Fill ϕ = 35°; γ = 12.5 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)	0.27	0.38*	0.30	0.46*	0.27	0.38*
At rest (Restrained Wall)	0.43	-	0.47	-	0.43	-
Passive (Movement Towards Soil Mass)	3.70	-	3.30	-	3.70	-

* For wing walls.

The lateral earth pressure coefficients provided in the table above are “ultimate” values that require certain structural movements for the respective conditions to be mobilized. The values to use in design can be estimated from Figure C6.16 in the Commentary to the CHBDC, 2006.

6.4 Abutment Backfill

The backfill to the abutment walls should be carried out in accordance with OPSS 902. Granular backfill should be placed to the extents shown in OPSD 3101.150 and the granular backfill shall comply with the OPSS.PROV 1010 specifications. The design of the abutment should also incorporate a subdrain as shown in OPSD 3101.150.

All granular fill 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). Equal heights of backfill should be maintained on both sides of the structure during all stages of backfill placement, and backfilling operations should be carried out in accordance with OPSS 902. Compaction equipment including hand operated vibratory equipment shall comply with OPSS.PROV 501.

6.5 Erosion Protection

The November 2015 water level in Tee Creek is at elevation 171.0± m and this water will partially submerge and erode the forward and side slopes at the bridge abutments if the slopes are not protected. Design of an erosion protection scheme will depend on hydrologic, hydraulic and/or other concerns. We recommend using rip-rap to armour the embankment slopes with which creek water is likely to be in contact. The rip-rap should be installed in accordance with OPSS 511.

Surface water can also cause erosion beneath the rip-rap and loss of fines through the rip-rap. Therefore, a properly designed filter should be installed between the rip-rap and the embankment material.

We recommend that a qualified Hydraulics Engineer be consulted to provide inputs on the design thickness and lateral extent of rip-rap protection and to estimate 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 170± m), the OHSA soil classifications are:

- Embankment fill – Type 3 soil; and
- Silty Clay to Clayey Silt – Type 3 soil.

The side slopes of temporary excavations may be formed no steeper than 1H:1V for Type 3 soils and excavations should be carried out 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. Excavations will extend through the existing embankment fill, and the silty clay to clayey silt deposit. A suitable dewatering system that can be employed is gravity drainage and pumping from strategically placed filtered sumps.

6.8 Approach Embankments

6.8.1 Settlement

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, in Appendix C. The preconsolidation pressure (σ'_p) derived from the consolidation test data is slightly higher than the effective overburden pressure suggesting that the silty clay to clayey silt deposit is slightly over-consolidated.

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

Silty Clay to Clayey Silt Deformation Parameters

Parameter	Silty Clay to Clayey Silt
Preconsolidation Pressure (kPa.)	185
Compression Index - C_c	0.26
Recompression Index - C_r	0.04
Initial Void Ratio - e_o	0.7
Coefficient of Consolidation - C_v (m^2/s)	$3.02x10^{-7}$

The General Arrangement drawing shows that the bridge will be constructed on the same alignment and profile as the existing structure. A profile sag occurs on the bridge, requiring deck drains adjacent to the east and west barrier walls at the low point on the bridge. However, abutment backfill for the new bridge will be placed in the open forward slope area below the existing bridge. The new abutments construction will require excavations that extend approximately to the underside abutment elevations of the new bridge i.e. elevation 170± m.

The settlement analyses is guided by MTO's *Embankment Settlement Criteria for Design* Page 4 Table 1.2 which stipulates that 25 mm of post-construction settlement is allowable over a horizontal transition zone of 25 m measured from the bridge abutment.

It is estimated that the Granular A abutment backfill will induce approximately 80± mm of consolidation settlement in the footprint area of the new fill. About 55± mm of this settlement will be essentially complete in three months and after three months the remaining settlement in the 25 m transition zone will be 25 mm. Since it is impractical to wait three months for the 55± mm of settlement to be complete, the Design Builder can consider the following alternatives.

- Surcharge the top of abutment backfill (i.e. top of pavement design subgrade elevation) with a load equivalent to a 2 m vertical height of Granular A material. About 60± mm of the settlement will be essentially complete in six weeks and after the surcharge period, the remaining consolidation settlement in the 25 m transition zone will be 20± mm;
- Construct a Retained Soil System (RSS) behind the abutments and wingwalls that would allow construction of the substructure to proceed simultaneously with the settlement period. The RSS should be constructed using Granular A or free draining granular material that meets the RSS designer's performance specification. Provide a 50 mm compressible layer (such as Dow HD Styrofoam) between the RSS and concrete abutment and install a 150mm diameter subdrain near the base of the RSS. Surcharge the top of RSS with a load equivalent to a 2 m vertical height of Granular A material. About 60± mm of the settlement will be essentially complete in six weeks and after the surcharge period, the remaining consolidation settlement in the 25 m transition zone will be 20± mm; and
- Use lightweight cellular concrete (Cematrix) with a unit weight of 4.9 kN/m3, a material that is listed on the Designated Materials list of the Road Authority. This lightweight fill will induce approximately 30 mm of consolidation settlement and 15 mm of this consolidation settlement will be complete in 2 weeks. Therefore, after two weeks the remaining post construction settlement in the 25 m transition zone will be 15± mm.

Settlement monitoring is required to determine the constructing timing for paving operations. A special provision for settlement monitoring and instrumentation (including drawings) is provided as a separate document.

Embankments constructed with local earth fill or Granular A material will also settle during construction (fill compression) and, the magnitude of this settlement is expected to be about 1% and 0.5% of the fill height respectively. This settlement should be immediate in nature and essentially be complete shortly after construction is complete.

6.8.2 Stability

The global, internal and surficial stability of the embankment side slopes and forward slopes 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 Figure D1 in Appendix D. The analyses indicate that the factors of safety will be greater than the target factor of safety of 1.3, provided that the embankment is constructed at a minimum side slope and forward slope geometry of 2 Horizontal to 1 Vertical (2H:1V) 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)
New Embankment Fill	35	0	35	0	22.8
Silty Clay Fill	28	0	28	0	19
Silty Clay to Clayey Silt	0	60	28	0	20
Silt	30	0	30	0	19
Sandy Gravel	35	0	35	0	21
Sandy Silt	33	0	33	0	19
Design Factors of Safety	1.4		1.3		-

6.8.3 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 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 OPSD 208.010.

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

6.9 Temporary Protection Systems

Temporary protection systems should be designed in accordance with OPSS.PROV 539 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 supported 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 structures subject to unbalanced earth pressures are provided in the following table. The active earth pressure coefficients are based on the assumption that the ground surface behind the temporary 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
Granular A Fill	35	22.8	0.27	0.43	3.69
Existing Fill Soils	28	19	0.36	0.53	2.77
Silty Clay to Clayey Silt	28	20	0.36	0.53	2.77
Silt	30	19	0.33	0.50	3.00
Sandy Gravel	35	21	0.27	0.43	3.69
Sandy Silt	33	19	0.29	0.46	3.39

6.10 Seismic Requirements

The site is treated as lying in Seismic Zone 0. Reference to Annex A3.1 of the CHBDC 2006 indicates that the following seismic parameters (Fort Erie) should be used for design:

- Velocity Related Seismic Zone 0
- Zonal Velocity Ratio 0.05
- Acceleration Related Seismic Zone 2
- Zonal Acceleration Ratio 0.10
- 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.

6.11 Construction Concerns

During construction, the Design Builder should employ experienced geotechnical staff to observe construction activities related to foundation construction. Potential construction concerns include, but are not necessarily limited to:

- the possibility of encountering cobbles and boulders when installing the H-piles;
- a significant portion of consolidation settlement of the silty clay to clayey silt deposit below the abutment backfill must be complete before paving operations can begin; and
- pollution, siltation or disruption of environmentally sensitive areas.

7.0 CLOSURE

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

Terraprobe Inc.

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Principal, Senior Geotechnical Engineer

REGISTERED PROFESSIONAL ENGINEER
M. TANOS
07/15/2016
PROVINCE OF ONTARIO

Michael Tanos

Michael Tanos, P.Eng.
Principal, Designated MTO Contact

REFERENCES

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

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

Canadian Highway Bridge Design Code (CHBDC) and Commentary on CAN/CSA S6 06. 2006. CSA Special Publication, S6.1 06. Canadian Standard Association.

Chapman and Putnam, *"The Physiography of South Ontario",* 3rd Edition, 1984.

Ontario Division of Mines, *"Quaternary Geology of The Welland Area",* Preliminary Map P.796, 1972.

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.PROV 501	Construction Specification For Compacting.
OPSS 511	Construction Specification For Rip-Rap, Rock Protection and Granular Sheeting.
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.

Ontario Provincial Standard Drawings (OPSD)

OPSD 208.010	Benching Of Earth Slopes.
OPSD 3090.101	Foundation, Frost Penetration Depths For Southern Ontario
OPSD 3101.150	Walls Abutment Backfill, Minimum Granular Requirement

TABLE 1
COMPARISON OF FOUNDATION ALTERNATIVES

Pile Foundations	Spread Footings	Augered Caissons
Advantages: ▪ Reliable performance expected. ▪ High geotechnical resistances available by driving piles to refusal. ▪ Allows for the design of an integral abutment. ▪ Shallow excavation depth, reduced excavation volume and reduced dewatering requirements. ***Disadvantages:*** ▪ Construction concerns related to the possibility of piles being obstructed by boulders during driving.	***Advantages:*** None ***Disadvantages:*** ▪ Low geotechnical resistance of surficial soils does not permit the design of economical spread footings compared to pile foundations. ▪ Unreliable performance and high risk of performance related issues due to settlement.	***Advantages:*** ▪ Reliable performance expected. ▪ High geotechnical resistances available by founding caissons on competent soils. ▪ Allows for the design of a semi integral abutment. ***Disadvantages:*** ▪ Requires a permanent liner to maintain side wall support. ▪ Attempts at dewatering the caisson excavation and maintaining a sufficiently dry excavation to permit cleaning, inspection and high quality construction, would be challenging and most likely impractical.
Risks/Consequences ▪ Very low risk of bearing capacity failure. ▪ Very low risk that total settlement will exceed 25 mm.	***Risks/Consequences*** ▪ Moderate to high risk of bearing capacity failure and settlement related performance issues.	***Risks/Consequences*** ▪ Very low risk of bearing capacity failure. ▪ Very low risk that total settlement will exceed 25mm.

DRAWINGS

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

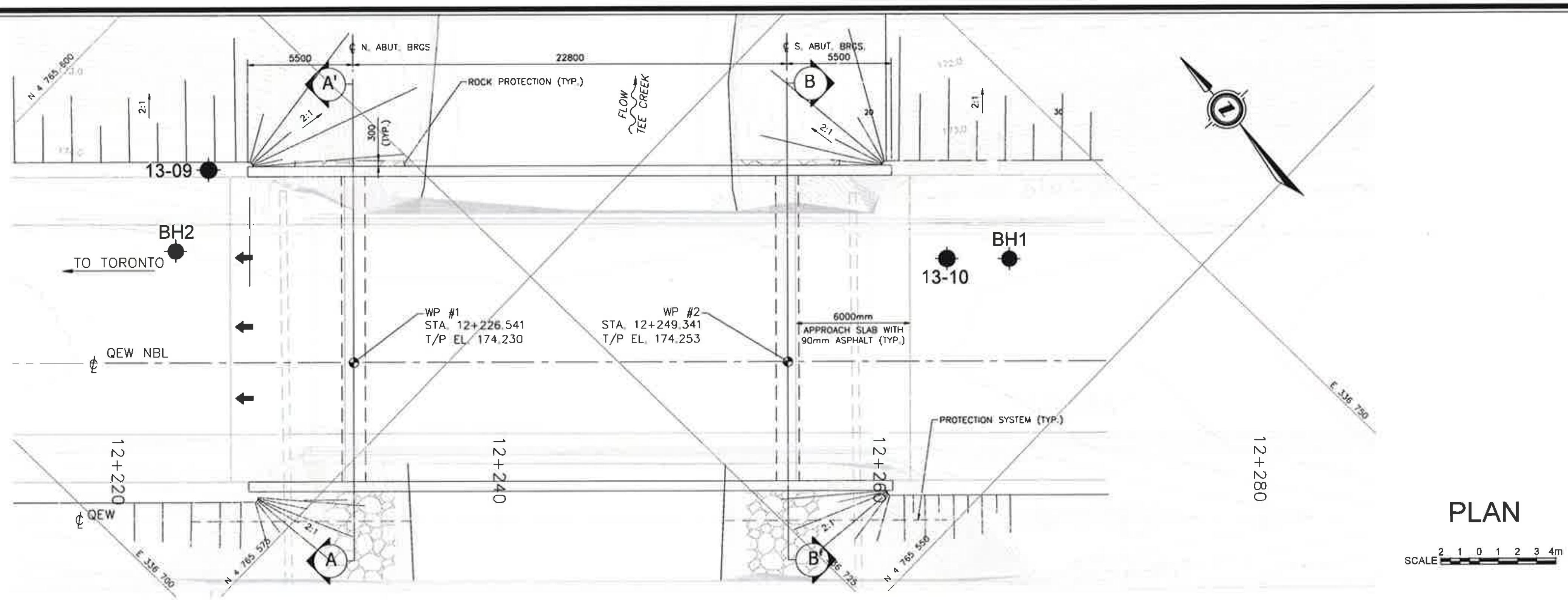
CONT DB 2014-2036
GWP No 2177-08-00

TEE CREEK BRIDGE
NORTH BOUND LANES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

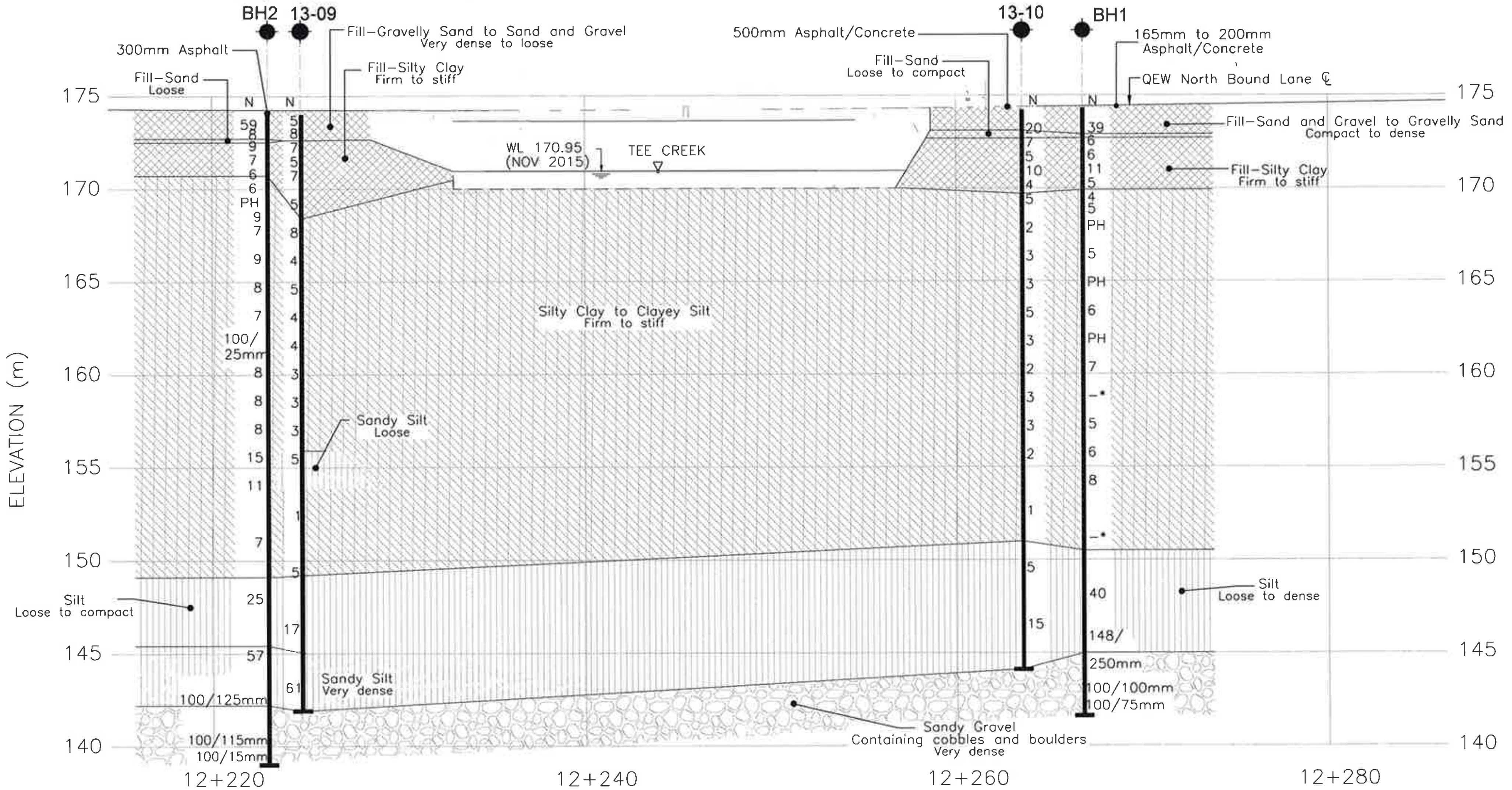
WSP | MMM GROUP

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



PLAN

SCALE 2 1 0 1 2 3 4m



℄ PROFILE OF QEW NBL

HORIZ. SCALE 2 1 0 1 2 3 4m
VERT. SCALE 2 0 2 4m

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KEY PLAN

LEGEND

●	Bore Hole
⊕	Dynamic Cone Penetration Test
◆	Bore Hole And Cone
'N'	Blows/0.3m (Std Pen Test, 475 J/blow)
CONE	Blows/0.3m (60° Cone, 475 J/blow)
	WL at Time of Investigation
	WL in Piezometer
	Piezometer
90%	Rock Quality Designation
A/R	Auger Refusal

No	ELEV	COORDINATES	
		NORTHING	EASTING
1	174.3	4 765 557.5	336 743.2
2	174.1	4 765 589.3	336 713.0
13-09	174.0	4 765 591.0	336 717.3
13-10	174.2	4 765 559.9	336 740.9

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 Limited. by email drawing file x3215095-320-001GA.dwg, 3215095-320-001XG.dwg, H3215095XB01.dwg, H3215095XB02.dwg received January 12, 2016.

REVISIONS	DATE	BY	DESCRIPTION

HWY. QEW	PROJECT No. 1-15-0689		GEOCRES No.: 30M3-289
SUBM'D. RA	CHKD. RA	DATE: July. 2016	SITE: 34-67/1
DRAWN: KC	CHKD.	APPD: MT	DWG. 1

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

CONT DB 2014-2036

GWP No 2177-08-00

TEE CREEK BRIDGE
NORTH BOUND LANES
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

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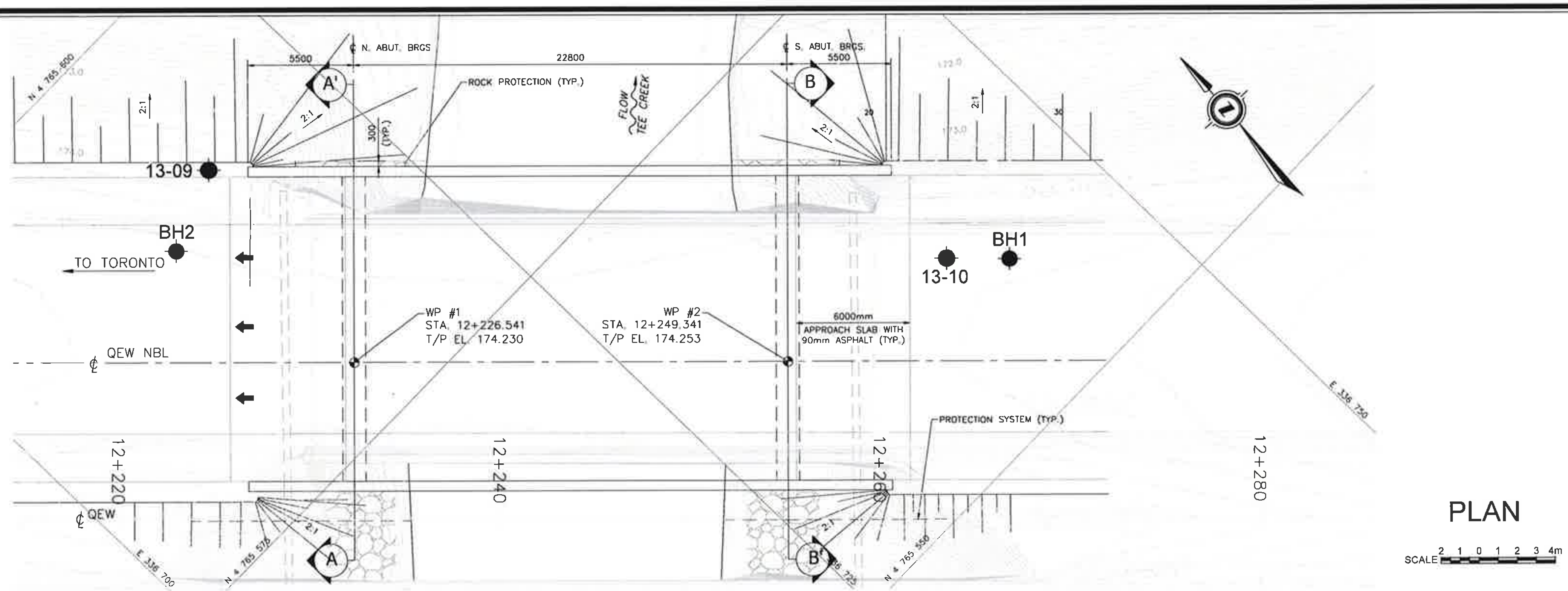
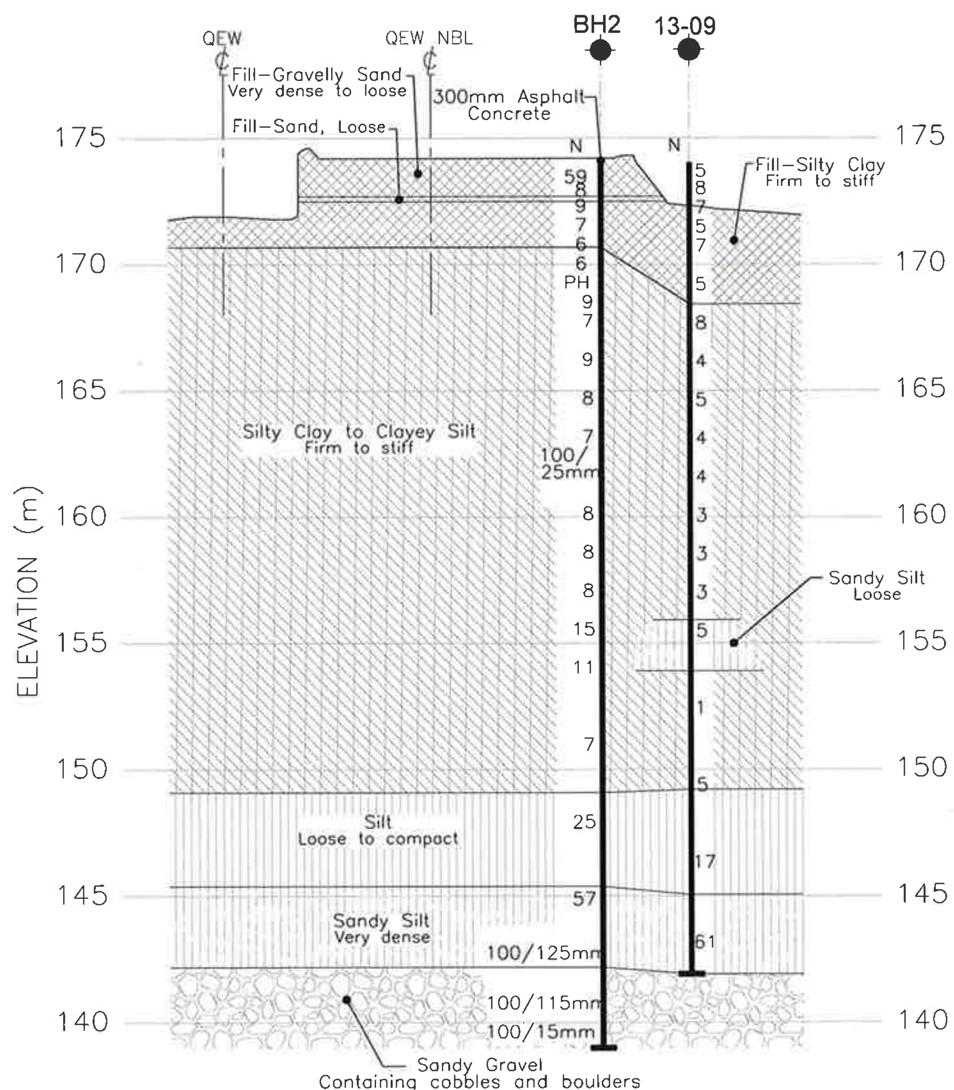
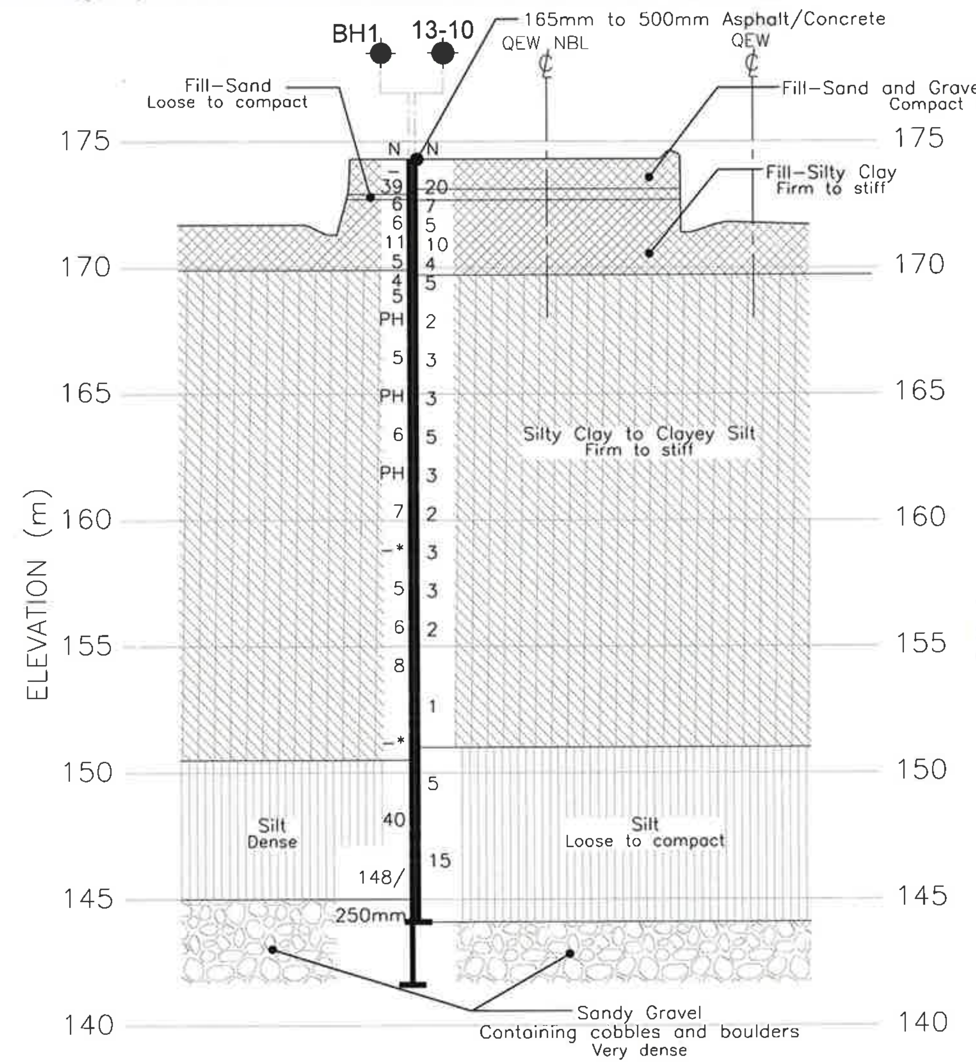


PLAN

SCALE 2 1 0 1 2 3 4m



SECTION A-A'



SECTION B-B'

HORIZ. SCALE 2 1 0 1 2 3 4m
VERT. SCALE 2 0 2 4m

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KEY PLAN

LEGEND	
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	Dynamic Cone Penetration Test
	Bore Hole And Cone
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REVISIONS	DATE	BY	DESCRIPTION

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SUBM'D. RA	CHKD. RA	DATE: July, 2016	SITE: 34-67/1
DRAWN: KC	CHKD.	APPD: MT	DWG. 2

APPENDIX A1

Record of Borehole Sheets

Terraprobe Inc.

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 ú.

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

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

PHYSICAL PROPERTIES OF SOIL

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

Ministry of Transportation Ontario

Terraprobe

RECORD OF BOREHOLE No 1

METRIC

G.W.P.	LOCATION Coords: E:336743.2 N:4765557.5	ORIGINATED BY	SD
DIST HWY QEW	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING	COMPILED BY	HA
DATUM GEODETIC	DATE 2015-11-23 - 2015-11-25	CHECKED BY	RA

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

ELEV / DEPTH (m)	DESCRIPTION
174.3	**GROUND SURFACE**
174.1 / 0.2	165mm **ASPHALTIC CONCRETE**
	FILL, gravelly sand, trace to some silt, trace clay, dense, brown, dry
172.9 / 1.4	**FILL**, sand, trace silt, loose, brown, moist
172.7 / 1.6	**FILL**, silty clay, trace sand, trace gravel, firm to stiff, brown, moist to wet
	some organics, grey
169.9 / 4.4	**SILTY CLAY**, trace sand, firm to stiff, grey, moist to wet

NUMBER	TYPE	SPT 'N' VALUE	Water content / limits	UNIT WEIGHT (kN/m³)	REMARKS & GRAIN SIZE (GR SA SI CL)
1	AS	-			
2	SS	39	○		43 42 12 3
3	SS	6			
4	SS	6			sampler wet at 2.3m
5	SS	11	LL=46		0 2 38 60
6	SS	5			
7	SS	4			
8	SS	5			0 1 47 52
9	TW	PH			
10	SS	5			
11	TW	PH		20.0	0 3 59 38
12	SS	6			0 3 58 39
13	TW	PH			
14	SS	7			1 4 50 45

Elevation scale: 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160

Field vane sensitivity values plotted: 3.9, 2.4, 2.4, 2.1, 2.7, 2.2, 2.2, 1.8, 1.4, 2.1, 2.8

Continued Next Page

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

file: 1-15-0689 tee creek rd -- bh logs.gpj

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RECORD OF BOREHOLE No 1

METRIC

G.W.P.		LOCATION	Coords: E:336743.2 N:4765557.5	ORIGINATED BY	SD
DIST	HWY QEW	BOREHOLE TYPE	HOLLOW STEM AUGERS/CASING AND WASH BORING	COMPILED BY	HA
DATUM	GEODETIC	DATE	2015-11-23 - 2015-11-25	CHECKED BY	RA

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

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued) **SILTY CLAY**, trace sand, firm to stiff, grey, moist to wet				Nov. 23, 2015 / Nov. 24, 2015
		15	SS	-*	
		16	SS	5	
		17	SS	6	2 4 55 39
		18	SS	8	
		19	SS	-*	
150.5 / 23.8	**SILT**, trace clay, trace to some sand, dense, brown, wet				
		20	SS	40	0 3 90 7
		21	SS	148 / 250mm	Nov. 24, 2015 / Nov. 25, 2015
145.0 / 29.3	**SANDY GRAVEL**, containing cobbles and boulders, very dense, grey, moist to wet				

Field vane shear strength readings (sensitivity in superscript): 1.8, 1.9, 1.9, 1.9, 1.9, 1.5, 1.9, 1.6, >

Elevation scale: 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145

Continued Next Page

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

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RECORD OF BOREHOLE No 1

METRIC

G.W.P. | LOCATION Coords: E:336743.2 N:4765557.5 | ORIGINATED BY SD

DIST | HWY QEW | BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING | COMPILED BY HA

DATUM GEODETIC | DATE 2015-11-23 - 2015-11-25 | CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE
	(continued)				
	SANDY GRAVEL, containing cobbles and boulders, very dense, grey, moist to wet	22	RC		144
		23	RC		
		24	RC		
		25	SS	100/100mm	143
					142
141.6 32.7		26	SS	100/75mm	

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

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

*Rods slipped while attempting SPT Test.

Consolidation test performed on TW11.

Borehole extended with a Tricone bit from 31.3m to 32.6m.

file: 1-15-0689 tee creek rd -- bh logs.gpj

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

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RECORD OF BOREHOLE No 2

METRIC

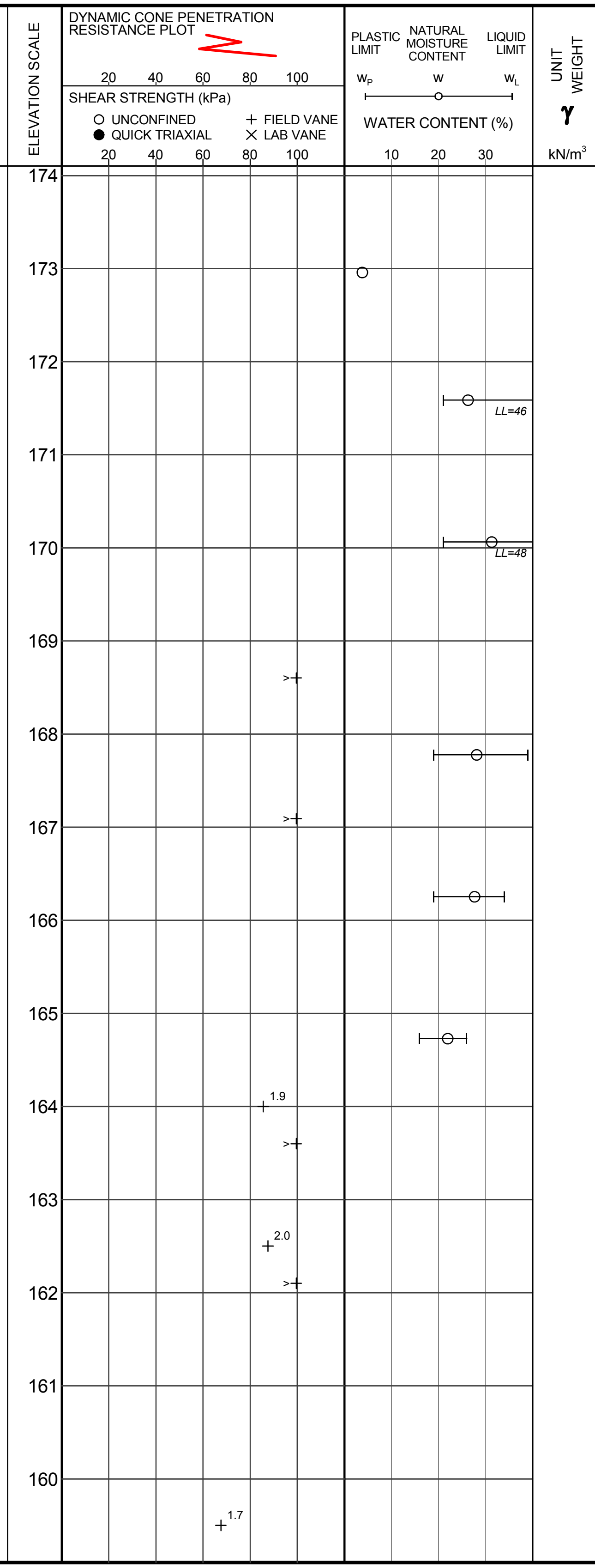
G.W.P.		ORIGINATED BY	SD
DIST	HWY QEW	COMPILED BY	HA
DATUM GEODETIC		CHECKED BY	RA
LOCATION	Coords: E:336713 N:4765589.3		
BOREHOLE TYPE	HOLLOW STEM AUGERS/CASING AND WASH BORING		
DATE	2015-11-26 - 2015-11-30		

SOIL PROFILE

ELEV DEPTH (m)	DESCRIPTION
174.1	**GROUND SURFACE**
173.8 / 0.3	300mm **ASPHALTIC CONCRETE**
172.7 / 1.4	**FILL**, gravelly sand, trace silt, very dense to loose, brown, dry
172.5 / 1.6	**FILL**, sand, trace silt, loose, brown, moist
170.7 / 3.4	**FILL**, silty clay, trace sand, trace gravel, firm to stiff, brown, moist to wet
	SILTY CLAY, trace sand, firm to stiff, grey, moist to wet

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	WATER CONTENT / LIMITS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	59		
2	SS	8		
3	SS	9		sampler wet at 1.5m
4	SS	7	LL=46	1 4 33 62
5	SS	6		
6	SS	6	LL=48	0 1 31 68
7	TW	PH		
8	SS	9		
9	SS	7		0 0 48 52
10	SS	9		0 1 56 43
11	SS	8		0 0 68 32
12	SS	7		
13	SS	100 / 25mm		Nov. 26, 2015 ----------------- Nov. 27, 2015 increased resistance to augering from 12.5m to 13.1m
14	SS	8		



UNIT WEIGHT γ kN/m³

GROUND WATER CONDITIONS

Continued Next Page

+³, ×³: Numbers refer to Sensitivity   O³% STRAIN AT FAILURE

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RECORD OF BOREHOLE No 2

METRIC

G.W.P. ______ LOCATION Coords: E:336713 N:4765589.3 ORIGINATED BY SD

DIST ______ HWY QEW BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING COMPILED BY HA

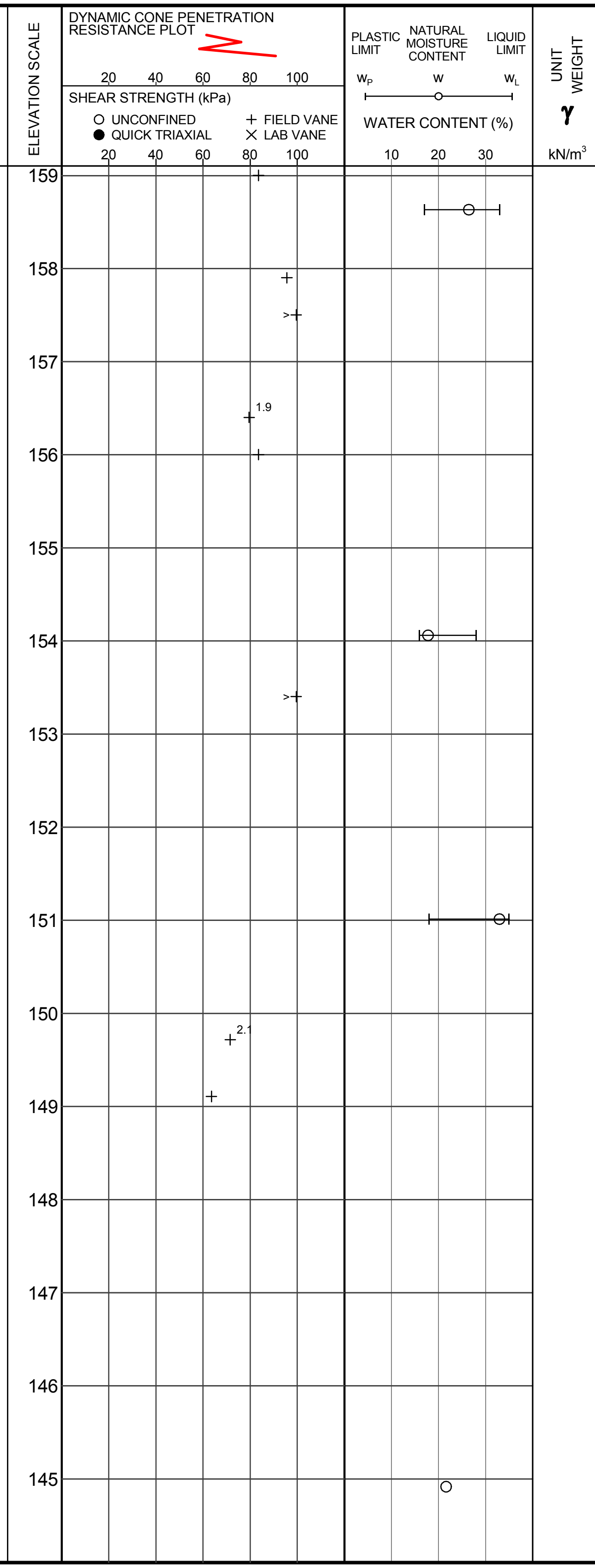
DATUM GEODETIC DATE 2015-11-26 - 2015-11-30 CHECKED BY RA

SOIL PROFILE (continued)

ELEV DEPTH (m)	DESCRIPTION
	SILTY CLAY, trace sand, firm to stiff, grey, moist to wet
149.1 / 25.0	**SILT**, trace clay, trace to some sand, compact, brown, wet
145.4 / 28.7	**SANDY SILT**, trace clay, trace to some gravel, very dense, brown, wet

SAMPLES

NUMBER	TYPE	SPT 'N' VALUE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
15	SS	8	0 5 50 45
16	SS	8	
17	SS	15	
18	SS	11	0 3 61 36
19	SS	7	0 1 47 52
20	SS	25	
21	SS	57	



Columns: GROUND WATER CONDITIONS; ELEVATION SCALE; UNIT WEIGHT γ kN/m³

Continued Next Page

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

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RECORD OF BOREHOLE No 2

METRIC

G.W.P.	LOCATION Coords: E:336713 N:4765589.3	ORIGINATED BY SD
DIST	HWY QEW	BOREHOLE TYPE HOLLOW STEM AUGERS/CASING AND WASH BORING	COMPILED BY HA
DATUM GEODETIC	DATE 2015-11-26 - 2015-11-30	CHECKED BY RA

ELEV DEPTH (m)	DESCRIPTION	NUMBER	TYPE	SPT 'N' VALUE	ELEVATION SCALE	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	(continued)						
	SANDY SILT, trace clay, trace to some gravel, very dense, brown, wet				144		
					143		
142.2 / 31.9	**SANDY GRAVEL**, containing cobbles and boulders, very dense, grey, moist to wet	22	SS	100 / 125mm	142		Nov. 27, 2015 / Nov. 30, 2015
					141		
		23	SS	100 / 115mm		~8	
					140		
139.0 / 35.1		24	SS	100/ 15mm			

END OF BOREHOLE

Borehole filled with drill water upon completion of drilling.

Borehole extended with a Tricone bit from 33.5m to 35.1m.

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

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

APPENDIX A2
Record of Borehole Sheets
Golder

LIST OF SYMBOLS

Unless otherwise stated, the symbols employed in the report are as follows:

I. GENERAL

π	3.1416
ln x,	natural logarithm of x
$\log_{10}$	x or log x, logarithm of x to base 10
g	acceleration due to gravity
t	time
FoS	factor of safety

II. STRESS AND STRAIN

γ	shear strain
Δ	change in, e.g. in stress: $\Delta\,\sigma$
ε	linear strain
ε_v	volumetric strain
η	coefficient of viscosity
υ	Poisson's ratio
σ	total stress
σ'	effective stress ($\sigma' = \sigma - u$)
σ'_{vo}	initial effective overburden stress
σ_1, σ_2, σ_3	principal stress (major, intermediate, minor)
σ_{oct}	mean stress or octahedral stress $= (\sigma_1 + \sigma_2 + \sigma_3)/3$
τ	shear stress
u	porewater pressure
E	modulus of deformation
G	shear modulus of deformation
K	bulk modulus of compressibility

III. SOIL PROPERTIES

(a) Index Properties

$\rho(\gamma)$	bulk density (bulk unit weight)*
$\rho_d(\gamma_d)$	dry density (dry unit weight)
$\rho_w(\gamma_w)$	density (unit weight) of water
$\rho_s(\gamma_s)$	density (unit weight) of solid particles
γ'	unit weight of submerged soil ($\gamma' = \gamma - \gamma_w$)
D_R	relative density (specific gravity) of solid particles ($D_R = \rho_s / \rho_w$) (formerly G_s)
e	void ratio
n	porosity
S	degree of saturation

* Density symbol is ρ. Unit weight symbol is γ where $\gamma = \rho g$ (i.e. mass density multiplied by acceleration due to gravity)

(a) Index Properties (continued)

w	water content
w_l or LL	liquid limit
w_p or PL	plastic limit
I_p or PI	plasticity index $= (w_l - w_p)$
w_s	shrinkage limit
I_L	liquidity index $= (w - w_p) / I_p$
I_C	consistency index $= (w_l - w) / I_p$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
I_D	density index $= (e_{max} - e) / (e_{max} - e_{min})$ (formerly relative density)

(b) Hydraulic Properties

h	hydraulic head or potential
q	rate of flow
v	velocity of flow
i	hydraulic gradient
k	hydraulic conductivity (coefficient of permeability)
j	seepage force per unit volume

(c) Consolidation (one-dimensional)

C_c	compression index (normally consolidated range)
C_r	recompression index (over-consolidated range)
C_s	swelling index
C_α	secondary compression index
m_v	coefficient of volume change
c_v	coefficient of consolidation (vertical direction)
c_h	coefficient of consolidation (horizontal direction)
T_v	time factor (vertical direction)
U	degree of consolidation
σ'_p	pre-consolidation stress
OCR	over-consolidation ratio $= \sigma'_p / \sigma'_{vo}$

(d) Shear Strength

τ_p, τ_r	peak and residual shear strength
ϕ'	effective angle of internal friction
δ	angle of interface friction
μ	coefficient of friction = tan δ
c′	effective cohesion
c_u, s_u	undrained shear strength ($\phi = 0$ analysis)
p	mean total stress $(\sigma_1 + \sigma_3)/2$
p′	mean effective stress $(\sigma'_1 + \sigma'_3)/2$
q	$(\sigma_1 - \sigma_3)/2$ or $(\sigma'_1 - \sigma'_3)/2$
q_u	compressive strength $(\sigma_1 - \sigma_3)$
S_t	sensitivity

Notes:
1. $\tau = c' + \sigma' \tan \phi'$
2. shear strength = (compressive strength)/2

LIST OF ABBREVIATIONS

The abbreviations commonly employed on Records of Boreholes, on figures and in the text of the report are as follows:

I. SAMPLE TYPE

AS	Auger sample
BS	Block sample
CS	Chunk sample
DS	Denison type sample
FS	Foil sample
RC	Rock core
SC	Soil core
SS	Split-spoon
ST	Slotted tube
TO	Thin-walled, open
TP	Thin-walled, piston
WS	Wash sample

II. PENETRATION RESISTANCE

Standard Penetration Resistance (SPT), N:

The number of blows by a 63.5 kg. (140 lb.) hammer dropped 760 mm (30 in.) required to drive a 50 mm (2 in.) drive open sampler for a distance of 300 mm (12 in.)

Dynamic Cone Penetration Resistance; N_d:

The number of blows by a 63.5 kg (140 lb.) hammer dropped 760 mm (30 in.) to drive uncased a 50 mm (2 in.) diameter, 60º cone attached to “A” size drill rods for a distance of 300 mm (12 in.).

PH: Sampler advanced by hydraulic pressure
PM: Sampler advanced by manual pressure
WH: Sampler advanced by static weight of hammer
WR: Sampler advanced by weight of sampler and rod

Piezo-Cone Penetration Test (CPT)

A electronic cone penetrometer with a 60° conical tip and a project end area of 10 cm^2 pushed through ground at a penetration rate of 2 cm/s. Measurements of tip resistance (Q_t), porewater pressure (PWP) and friction along a sleeve are recorded electronically at 25 mm penetration intervals.

III. SOIL DESCRIPTION

(a) Non-Cohesive (Cohesionless) Soils

Density Index / Relative Density	N: Blows/300 mm or Blows/ft
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils
Consistency

	c_u, s_u: kPa	c_u, s_u: psf
Very soft	0 to 12	0 to 250
Soft	12 to 25	250 to 500
Firm	25 to 50	500 to 1,000
Stiff	50 to 100	1,000 to 2,000
Very stiff	100 to 200	2,000 to 4,000
Hard	over 200	over 4,000

IV. SOIL TESTS

w	water content
w_p	plastic limit
w_l	liquid limit
C	consolidation (oedometer) test
CHEM	chemical analysis (refer to text)
CID	consolidated isotropically drained triaxial test[1]
CIU	consolidated isotropically undrained triaxial test with porewater pressure measurement[1]
D_R	relative density (specific gravity, G_s)
DS	direct shear test
M	sieve analysis for particle size
MH	combined sieve and hydrometer (H) analysis
MPC	Modified Proctor compaction test
SPC	Standard Proctor compaction test
OC	organic content test
SO_4	concentration of water-soluble sulphates
UC	unconfined compression test
UU	unconsolidated undrained triaxial test
V	field vane (LV-laboratory vane test)
γ	unit weight

Note: 1 Tests which are anisotropically consolidated prior to shear are shown as CAD, CAU.

V. MINOR SOIL CONSTITUENTS

Per cent by Weight	Modifier	Example
0 to 5	Trace	Trace sand
5 to 12	Trace to Some (or Little)	Trace to some sand
12 to 20	Some	Some sand
20 to 30	(ey) or (y)	Sandy
over 30	And (non-cohesive (cohesionless)) or	Sand and Gravel
	With (cohesive)	Silty Clay with sand / Clayey Silt with sand

Golder Associates

RECORD OF BOREHOLE No 13-09

SHEET 1 OF 3 **METRIC**

PROJECT 12-1111-0088
G.W.P. 2177-08-00
DIST Central HWY QEW
DATUM Geodetic

LOCATION N 4765591.0 ;E 336717.3
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
DATE June 21 to 24, 2013

ORIGINATED BY SB
COMPILED BY AV
CHECKED BY MM

ELEV / DEPTH	DESCRIPTION
174.0	GROUND SURFACE
0.0	Sand and gravel, silt pocket at a depth of 1.0 m (FILL) Loose Brown Moist
172.6 / 1.4	Silty clay, trace sand, trace organics, trace rootlets (FILL) Firm Grey Wet Black staining below a depth of 4.6 m.
168.4 / 5.6	SILTY CLAY, trace sand Firm Mottled brown and grey Wet
165.3 / 8.7	CLAYEY SILT, trace sand Stiff Grey Wet

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	5	
2	SS	8	
3	SS	7	
4	SS	5	0 4 47 49
5	SS	7	
6	SS	5	
7	SS	8	
8	SS	4	
9	SS	5	
10	SS	4	
11	SS	4	
12	SS	3	0 4 56 40

ELEVATION SCALE: 173, 172, 171, 170, 169, 168, 167, 166, 165, 164, 163, 162, 161, 160

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: O UNCONFINED; + FIELD VANE; ● QUICK TRIAXIAL; × REMOULDED: 20 40 60 80 100

Field vane readings: >96 + (170); >96 + (168.5); >96 + 4, + 2, + (165.5–165); + 3, + 2 (164–163.5); + 2 (162); >96 +, 2 + (161–160.5); + 2 (159.3)

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

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

GTA-MTO 001 T:\PROJECTS\2012\12-1111-0088 (URS, VARIOUS STRUCTURE REPLACEMENT, QEW)\LOG\12-1111-0088.GPJ GAL-GTA.GDT 12/15/14

Golder Associates

RECORD OF BOREHOLE No 13-09

SHEET 2 OF 3

METRIC

PROJECT 12-1111-0088

G.W.P. 2177-08-00

LOCATION N 4765591.0 ;E 336717.3

ORIGINATED BY SB

DIST Central HWY QEW

BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers

COMPILED BY AV

DATUM Geodetic

DATE June 21 to 24, 2013

CHECKED BY MM

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace sand Stiff Grey Wet	13	SS	3	
		14	SS	3	
155.9 18.1	Sandy SILT, trace to some clay, trace to some gravel Loose Grey Wet	15	SS	5	8 25 56 11
153.9 20.1	SILTY CLAY, trace sand, trace gravel Stiff Grey Moist	16	SS	1	
		17A	SS	5	
149.2 24.8	SILT, trace to some clay, trace sand Loose to compact Brown Wet	17B			
		18	SS	17	0 5 84 11
145.0 29.0	Sandy SILT, some gravel, trace to some clay Very dense Brown Wet				

Elevation scale: 158, 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

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

Field vane sensitivity values plotted: 2, 2, 2, >96, 2, 2

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

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

Continued Next Page

$+^3, \times^3$: Numbers refer to Sensitivity

$\circ^{3\%}$ STRAIN AT FAILURE

Golder Associates

RECORD OF BOREHOLE No 13-09

SHEET 3 OF 3 — **METRIC**

PROJECT 12-1111-0088

G.W.P. 2177-08-00

LOCATION N 4765591.0 ;E 336717.3

ORIGINATED BY SB

DIST Central HWY QEW

BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers

COMPILED BY AV

DATUM Geodetic

DATE June 21 to 24, 2013

CHECKED BY MM

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

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%)	GR	SA	SI	CL
	Sandy SILT, some gravel, trace to some clay Very dense Brown Wet	19	SS	61	143	~20	20	27	46	7
141.9 32.1	END OF BOREHOLE AUGER REFUSAL NOTE: 1. Water level not noted upon completion of drilling.				142					

+ 3, × 3: Numbers refer to Sensitivity

O 3% STRAIN AT FAILURE

Golder Associates

Foundation Design

RECORD OF BOREHOLE No 13-10

SHEET 1 OF 3

METRIC

PROJECT 12-1111-0088

G.W.P. 2177-08-00

LOCATION N 4765559.9 ;E 336740.9

ORIGINATED BY SB

DIST Central HWY QEW

BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers

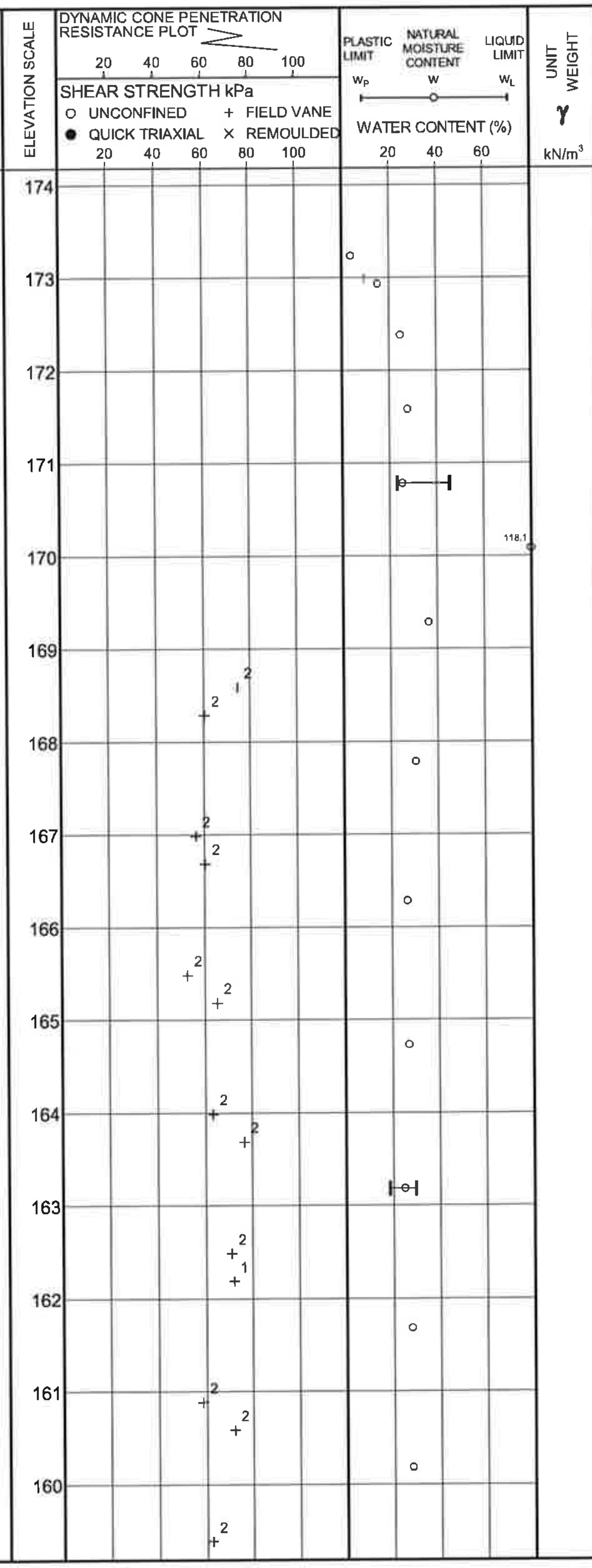
COMPILED BY AV

DATUM Geodetic

DATE July 3, 2013

CHECKED BY MM

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
174.2	GROUND SURFACE					
0.0	ASPHALT (200 mm)					
	CONCRETE (300 mm)					
173.7 / 0.5	Sand and gravel, trace silt (FILL), Compact, Brown, Moist	1A	SS	20		53 35 (12)
173.1	Sand (FILL), Compact, Reddish brown, Wet	1B				
172.7 / 1.5	Silty clay, trace to some sand, trace gravel (FILL), Firm to stiff, Brown to grey, Wet	2	SS	7		
		3	SS	5		
		4	SS	10		0 2 43 55
	- Organics and wood fragments below a depth of 3.8 m.	5	SS	4		
169.7 / 4.5	CLAYEY SILT, trace to some sand, trace to some gravel, Stiff, Grey, Wet	6	SS	5		
					∇	
		7	SS	2		
		8	SS	3		
		9	SS	3		
		10	SS	5		
		11	SS	3		
		12	SS	2		



Continued Next Page

+³, ×³: Numbers refer to Sensitivity O³% STRAIN AT FAILURE

RECORD OF BOREHOLE No 13-10

SHEET 2 OF 3 METRIC

PROJECT 12-1111-0088
G.W.P. 2177-08-00
LOCATION N 4765559.9 ;E 336740.9
ORIGINATED BY SB
DIST Central HWY QEW
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
COMPILED BY AV
DATUM Geodetic
DATE July 3, 2013
CHECKED BY MM

— CONTINUED FROM PREVIOUS PAGE —

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
	CLAYEY SILT, trace to some sand, trace to some gravel; Stiff; Grey; Wet	13	SS	3	
		14	SS	3	
		15	SS	2	7 6 54 33
		16	SS	1	
151.0 / 23.2	SILT, trace clay; Loose to compact; Brown; Wet	17	SS	5	0 0 95 5
		18	SS	15	

Elevation scale: 159, 158, 157, 156, 155, 154, 153, 152, 151, 150, 149, 148, 147, 146, 145

Field vane shear strength (kPa) / sensitivity values plotted (approx. 70–100 kPa; sensitivities 2, 2, 2, 2, 2, 2, 1, 1, 1).

Continued Next Page

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

Golder Associates

RECORD OF BOREHOLE No 13-10

SHEET 3 OF 3

METRIC

PROJECT 12-1111-0088
G.W.P. 2177-08-00
DIST Central HWY QEW
DATUM Geodetic
LOCATION N 4765559.9 ;E 336740.9
BOREHOLE TYPE 108 mm I.D. Continuous Flight Hollow Stem Augers
DATE July 3, 2013
ORIGINATED BY SB
COMPILED BY AV
CHECKED BY MM

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × REMOULDED (20 40 60 80 100)	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES						
	— CONTINUED FROM PREVIOUS PAGE —										
144.1 / 30.1	END OF BOREHOLE AUGER REFUSAL NOTE: 1. Water level inside auger at a depth of 5.8 m below ground surface (Elev. 168.4 m) upon completion of drilling.										

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

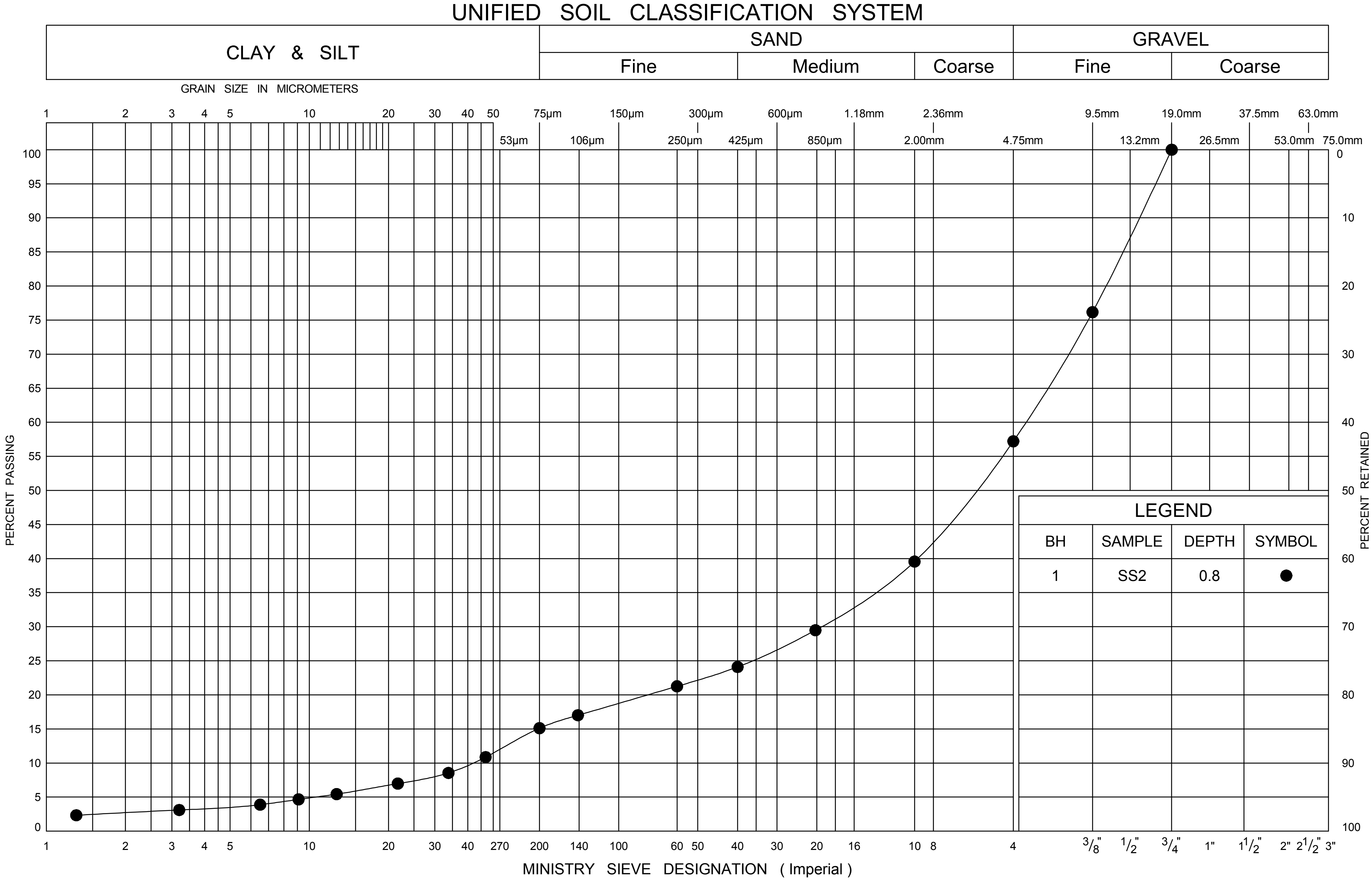
APPENDIX B1

Laboratory Test Results

Terraprobe Inc.

Terraprobe

78 12 M

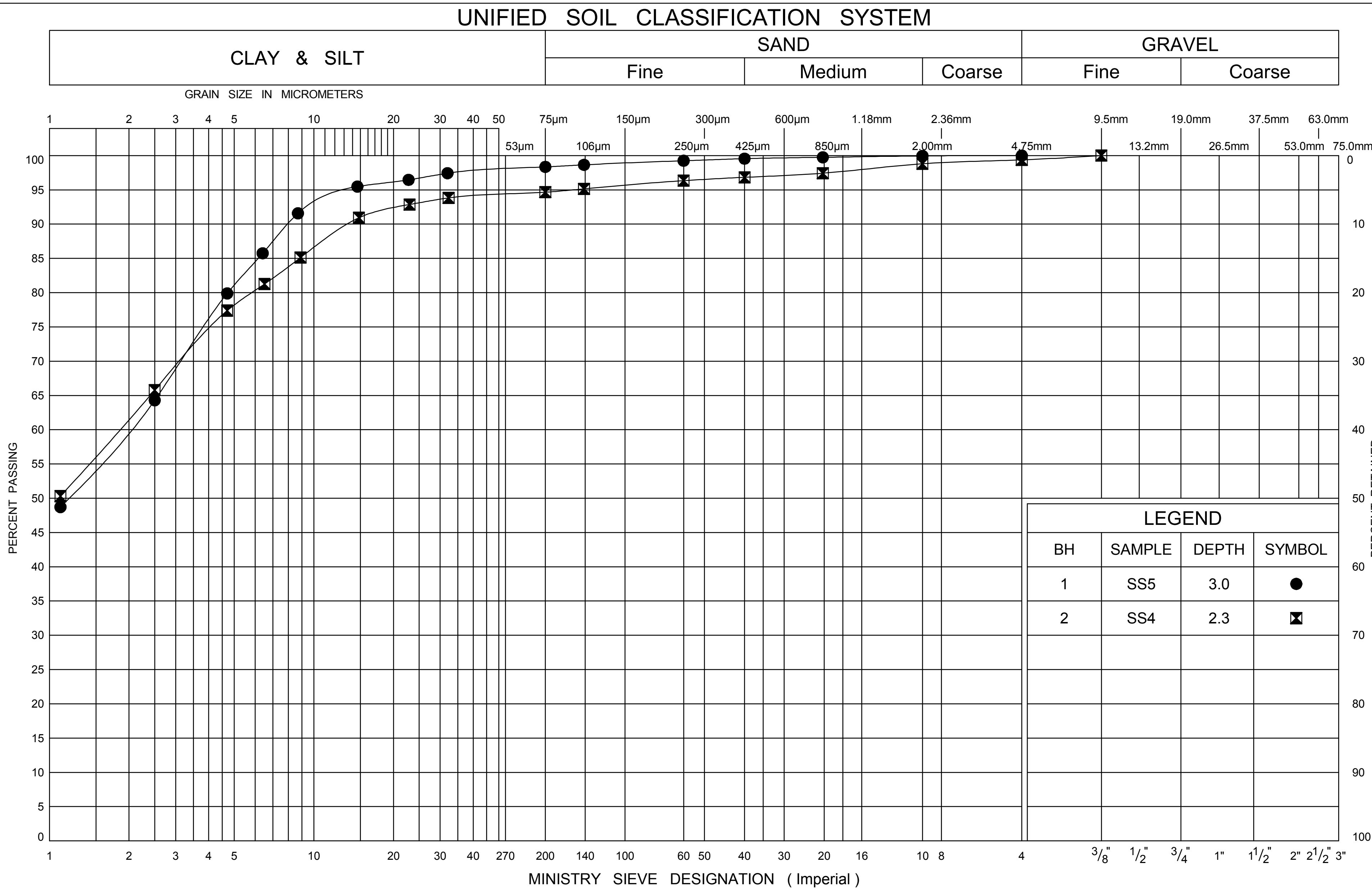


Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION FILL-GRAVELLY SAND	FIG No B1 DB 2014-2036

file: 1-15-0689 tee creek rd -- bh logs.gpj

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
1	SS5	3.0	●
2	SS4	2.3	⧗

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Ontario

GRAIN SIZE DISTRIBUTION

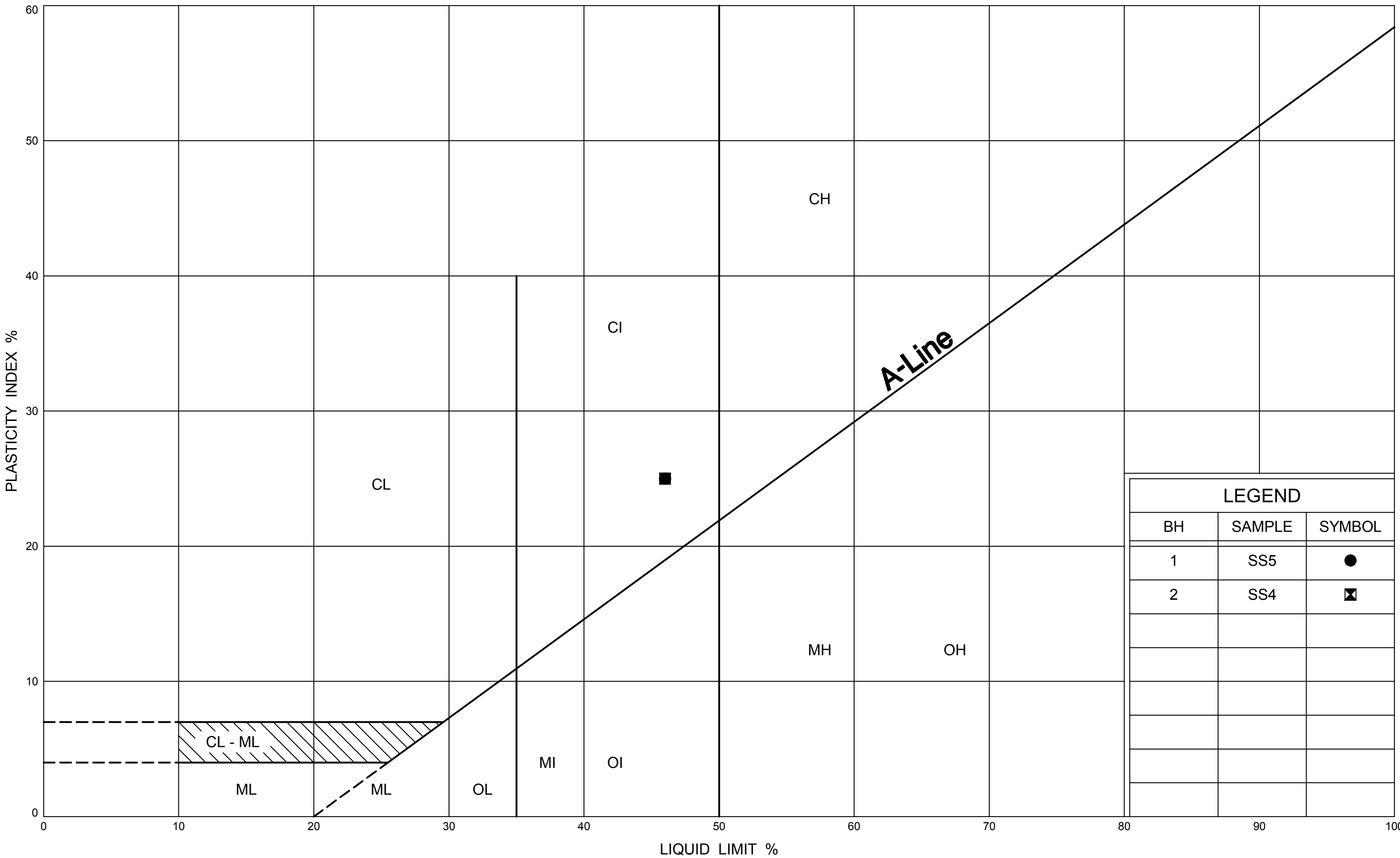
FILL-SILTY CLAY

FIG No B2

DB 2014-2036

file: 1-15-0689 tee creek rd -- bh logs.gpj

Terraprobe



Ministry of Transportation
Ontario

PLASTICITY CHART

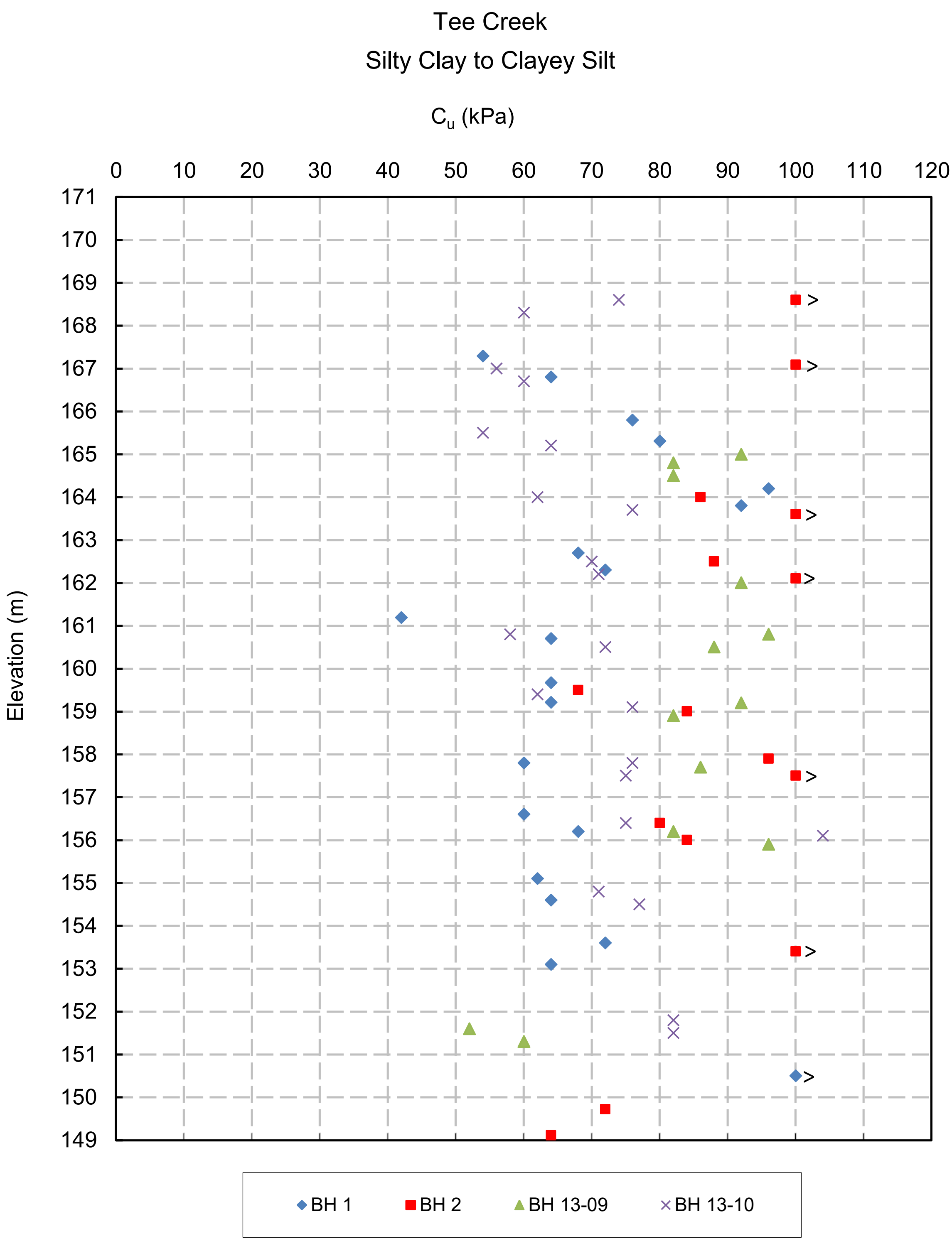
FILL-SILTY CLAY

FIG No B3

DB 2014-2036

file: 1-15-0689 tee creek rd -- bh logs.gpj

UNDRAINED SHEAR STRENGTH	FIGURE B4



Project No. : 1-15-0689

Date : February, 2016

Terraprobe Inc.

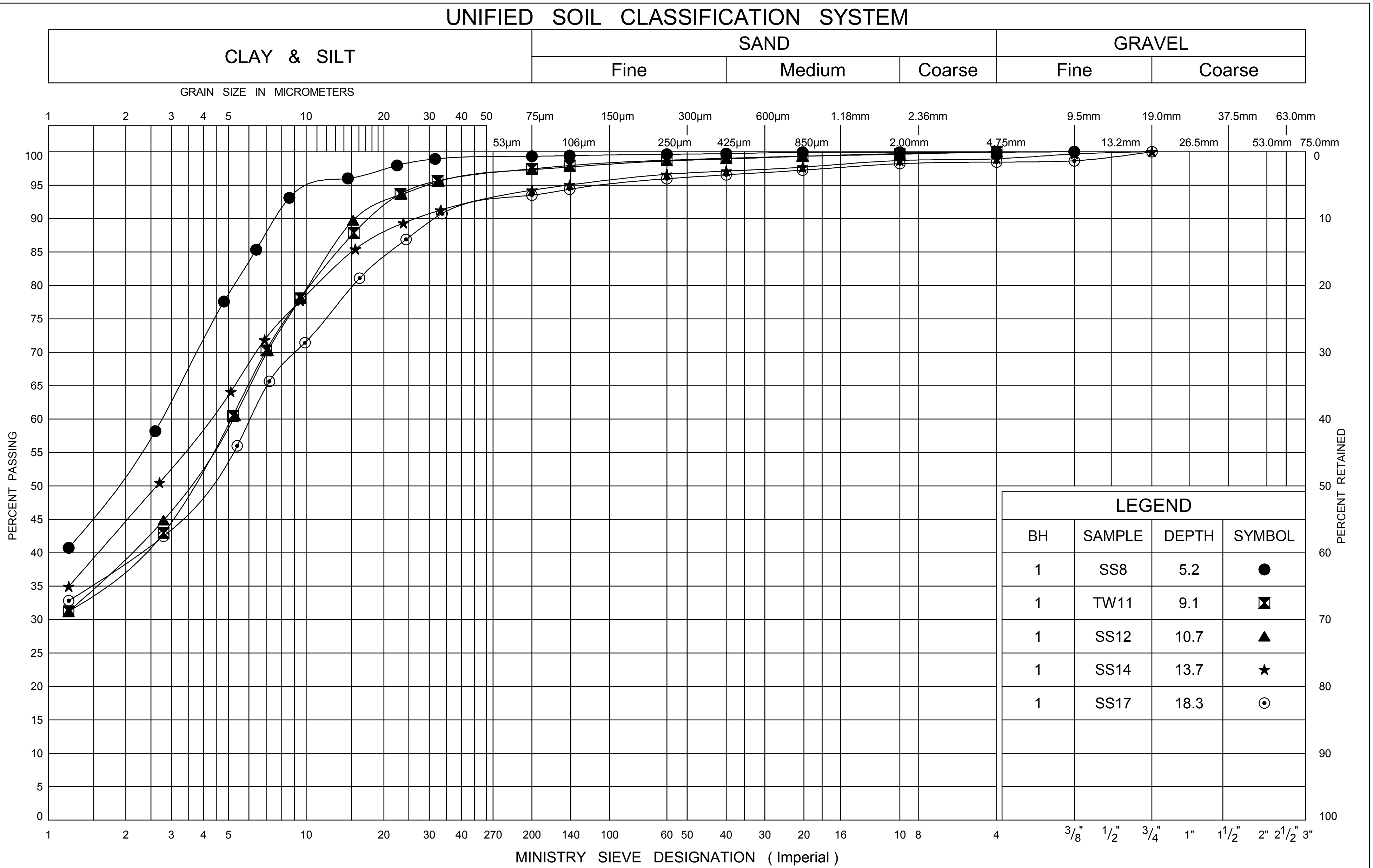
Prepared by : SD

Checked by : RA

C:\Users\rabdul\Documents\Reviewed Reports\Tee Lyons Black Creek\e0-P'c-Cc-Cr-Cu - TEE Creek RA.xlsx

Terraprobe

78 12 M



LEGEND			
BH	SAMPLE	DEPTH	SYMBOL
1	SS8	5.2	●
1	TW11	9.1	⧗
1	SS12	10.7	▲
1	SS14	13.7	★
1	SS17	18.3	⊙

Ministry of Transportation

Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY

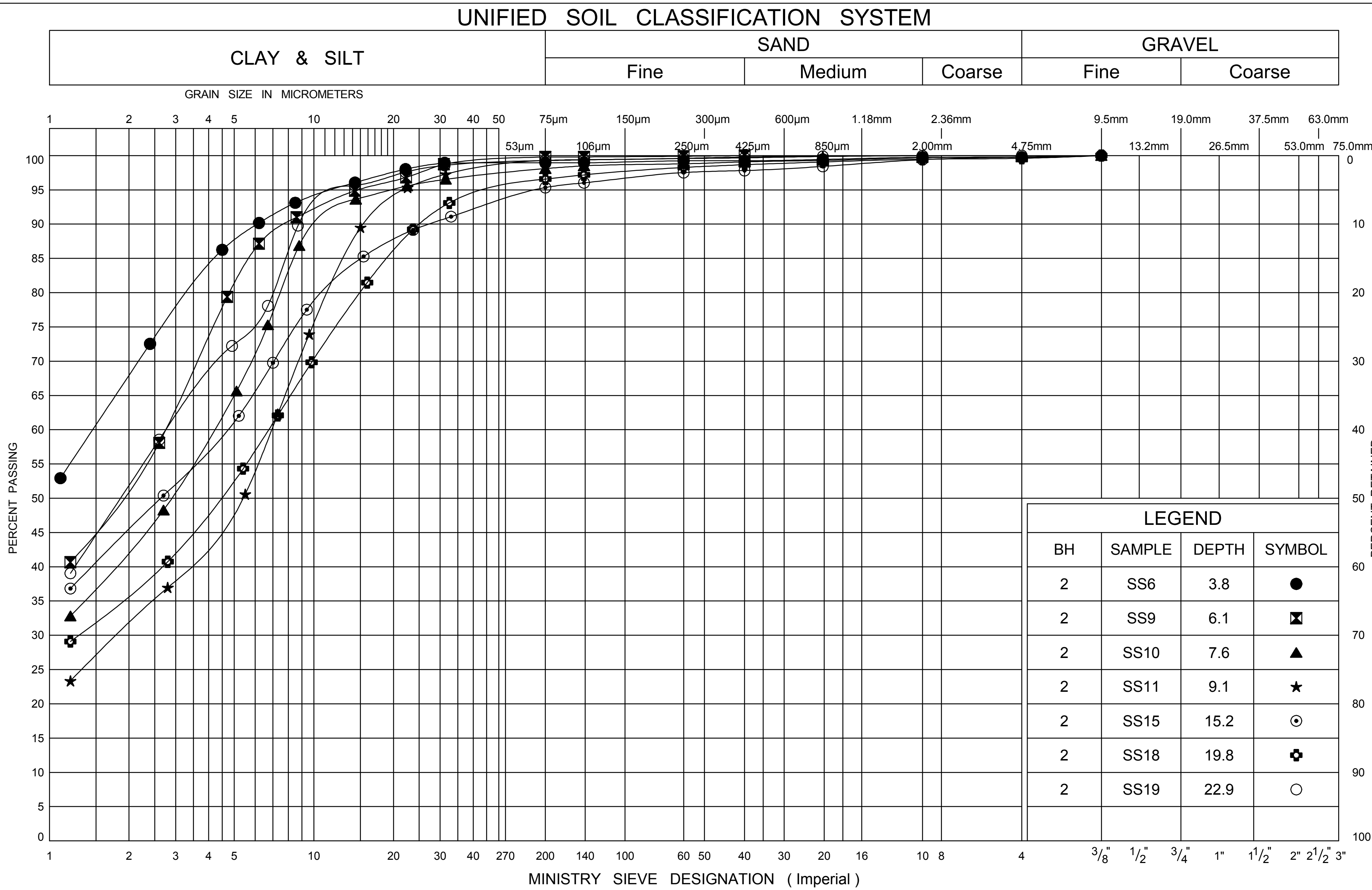
FIG No B5

DB 2014-2036

file: 1-15-0689 tee creek rd -- bh logs.gpj

Terraprobe

78 12 M



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
2	SS6	3.8	●
2	SS9	6.1	⊠
2	SS10	7.6	▲
2	SS11	9.1	★
2	SS15	15.2	⊙
2	SS18	19.8	✤
2	SS19	22.9	○

Ministry of Transportation

Ontario

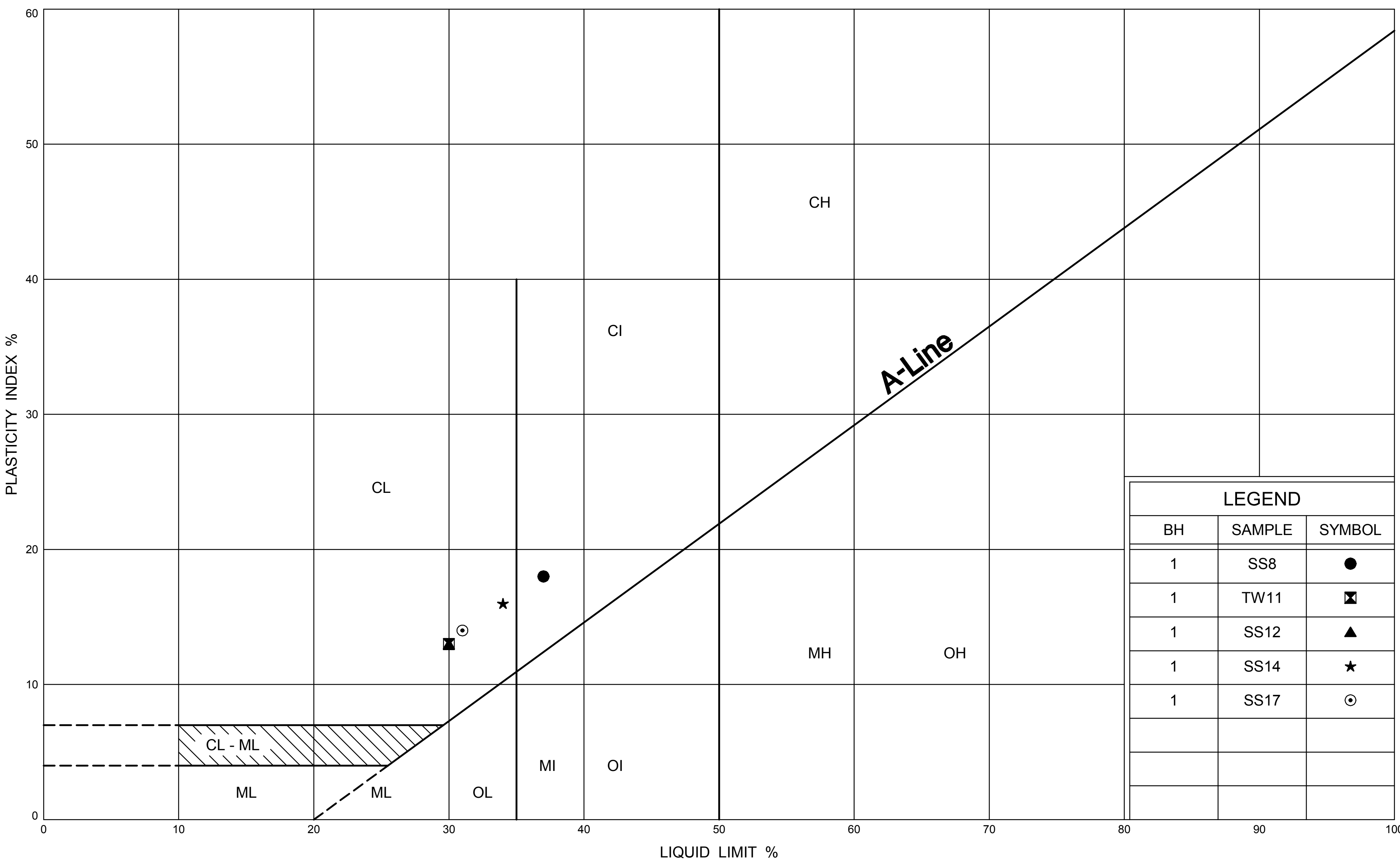
GRAIN SIZE DISTRIBUTION

SILTY CLAY

FIG No B6

DB 2014-2036

Terraprobe



Ministry of Transportation
Ontario

PLASTICITY CHART

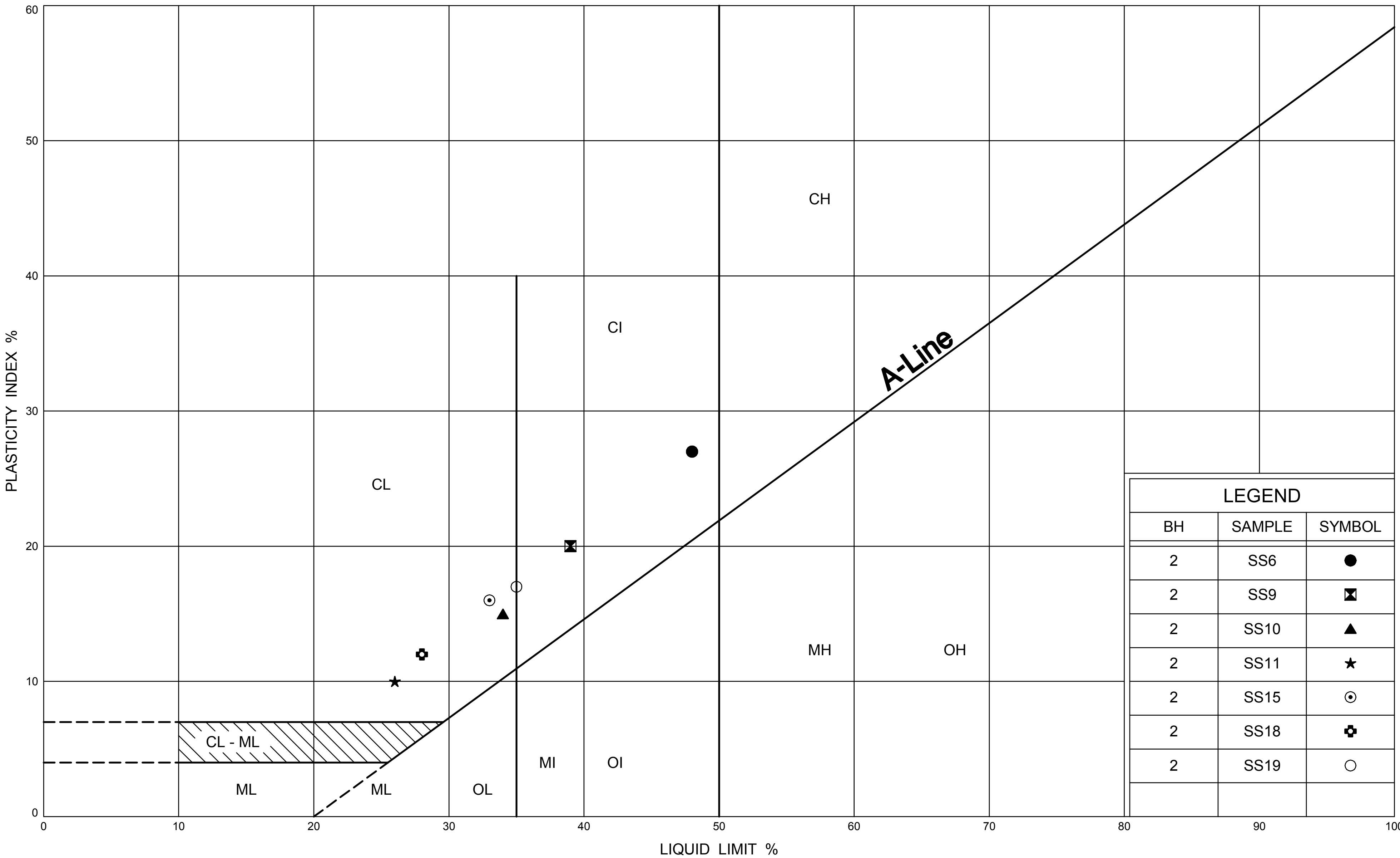
SILTY CLAY

FIG No B7

DB 2014-2036

file: 1-15-0689 tee creek rd -- bh logs.gpj

Terraprobe



LEGEND		
BH	SAMPLE	SYMBOL
2	SS6	●
2	SS9	⧗
2	SS10	▲
2	SS11	★
2	SS15	⊙
2	SS18	✿
2	SS19	○

Ministry of Transportation
Ontario

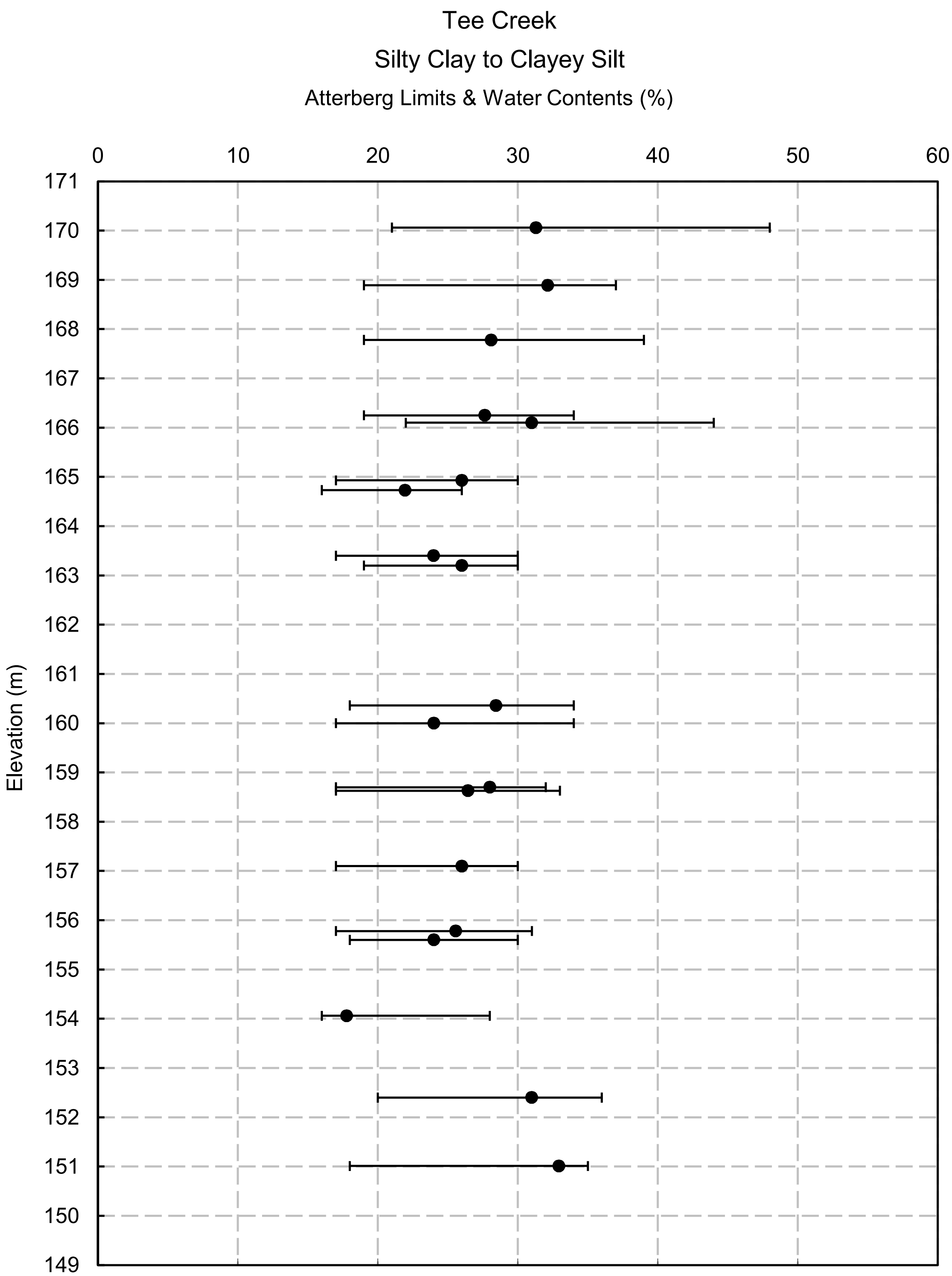
PLASTICITY CHART
SILTY CLAY

FIG No B8

DB 2014-2036

file: 1-15-0689 tee creek rd -- bh logs.gpj

ATTERBERG LIMITS AND WATER CONTENTS	FIGURE B9



C:\Users\rabdul\Documents\Reviewed Reports\Tee Lyons Black Creek\e0-P'c-Cc-Cr-Cu - TEE Creek RA.xlsx

Project No. : 1-15-0689
Date : February, 2016

Terraprobe Inc.

Prepared by : SD
Checked by : RA

CONSOLIDATION TEST SUMMARY

FIGURE B10

SAMPLE IDENTIFICATION

Borehole No. :	1	Sample No. :	TW11
		Sample Depth (m) :	9.1 - 9.6

TEST CONDITIONS

Test Type :	Laboratory Standard	Date Started :	15-Dec-15
Load Duration (hr) :	24	Date Completed :	24-Dec-15

SAMPLE DIMENSIONS AND PROPERTIES INITIAL

Sample Height (mm) :	19.04	Unit Weight (kN/m^3) :	20.00
Sample Diameter (mm) :	63.44	Dry Unit Weight (kN/m^3) :	15.88
Area (cm^2) :	31.61	Specific Gravity :	2.73
Volume (cm^3) :	60.18	Solid Height (mm) :	11.27
Water Content (%) :	25.8%	Volume of Solids (cm^3) :	35.63
Wet Mass (g) :	122.58	Volume of Voids (cm^3) :	24.56
Dry Mass (g) :	97.40	Degree of Saturation (%) :	102.53

TEST COMPUTATIONS

Stress (kPa)	Initial Height (mm)	Final Height (mm)	Void Ratio	t_{90} (min)	C_v (cm^2/s)	m_v (m^2/kN)	k (cm/s)
1.5653	19.04	19.04	0.689				
35.828	19.04	18.87	0.674	7.56	1.67E-03	2.67E-04	4.36E-08
70.091	18.87	18.71	0.660	6.25	1.98E-03	2.44E-04	4.75E-08
138.62	18.71	18.46	0.638	4.00	3.02E-03	1.93E-04	5.73E-08
275.67	18.46	17.99	0.596	4.41	2.61E-03	1.87E-04	4.79E-08
549.77	17.99	17.32	0.536	4.00	2.67E-03	1.36E-04	3.56E-08
1098.0	17.32	16.55	0.468	5.06	1.93E-03	8.09E-05	1.53E-08
2194.4	16.55	15.80	0.402	5.06	1.76E-03	4.13E-05	7.13E-09
275.7	15.80	16.05	0.424				
70.091	16.05	16.30	0.446				
18.697	16.30	16.58	0.471				

SAMPLE DIMENSIONS AND PROPERTIES FINAL

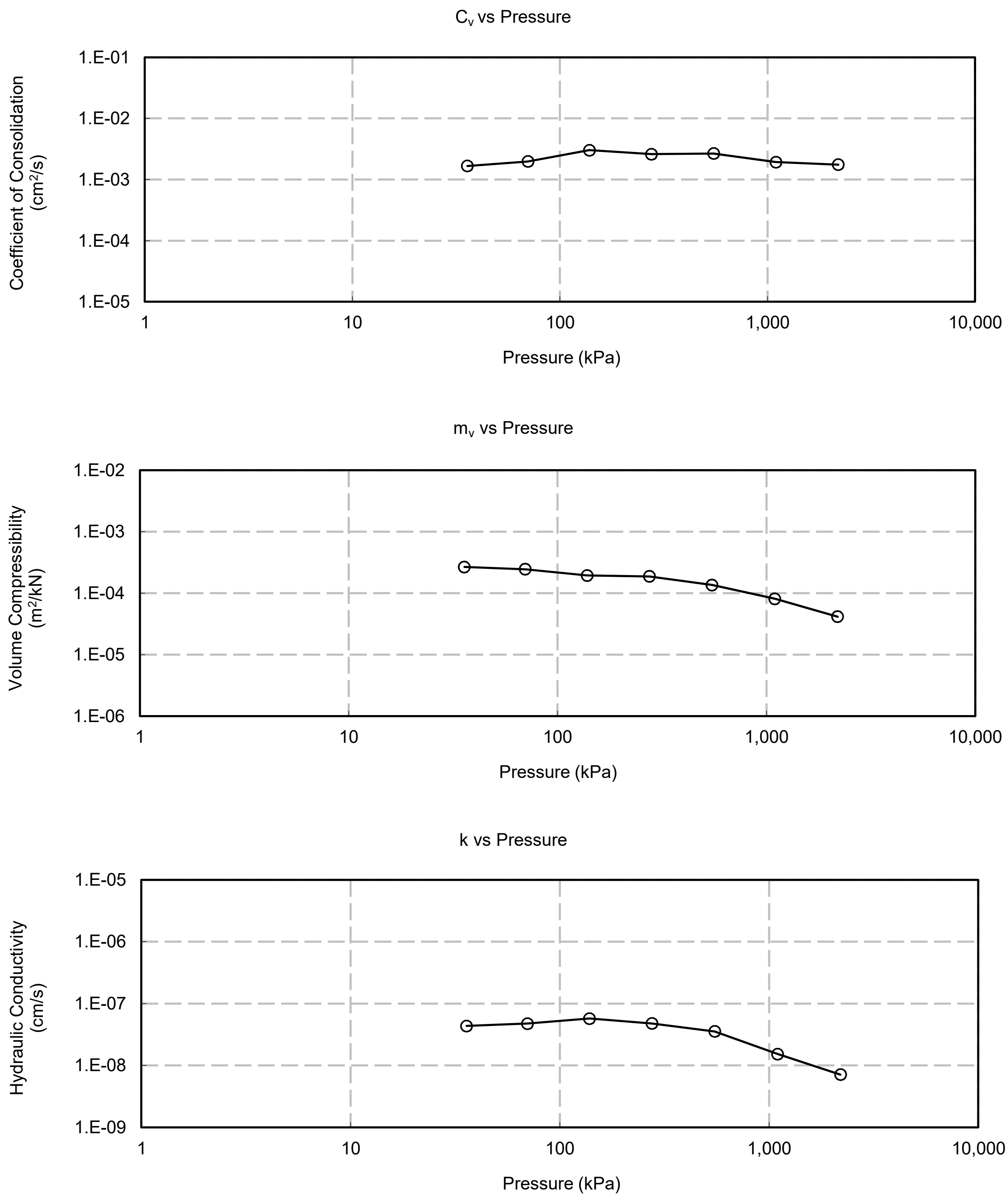
Sample Height (mm) :	16.58	Unit Weight (kN/m^3) :	21.79
Sample Diameter (mm) :	63.35	Dry Unit Weight (kN/m^3) :	18.03
Area (cm^2) :	31.52	Specific Gravity :	2.73
Volume (cm^3) :	52.26	Solid Height (mm) :	11.27
Water Content (%) :	20.9%	Volume of Solids (cm^3) :	35.14
Wet Mass (g) :	116.11	Volume of Voids (cm^3) :	17.12
Dry Mass (g) :	96.07		

Project No. :	1-15-0689	Prepared By :	SD
Date :	February 2016	Checked By :	RA

Terraprobe Inc.

CONSOLIDATION TEST	FIGURE B11

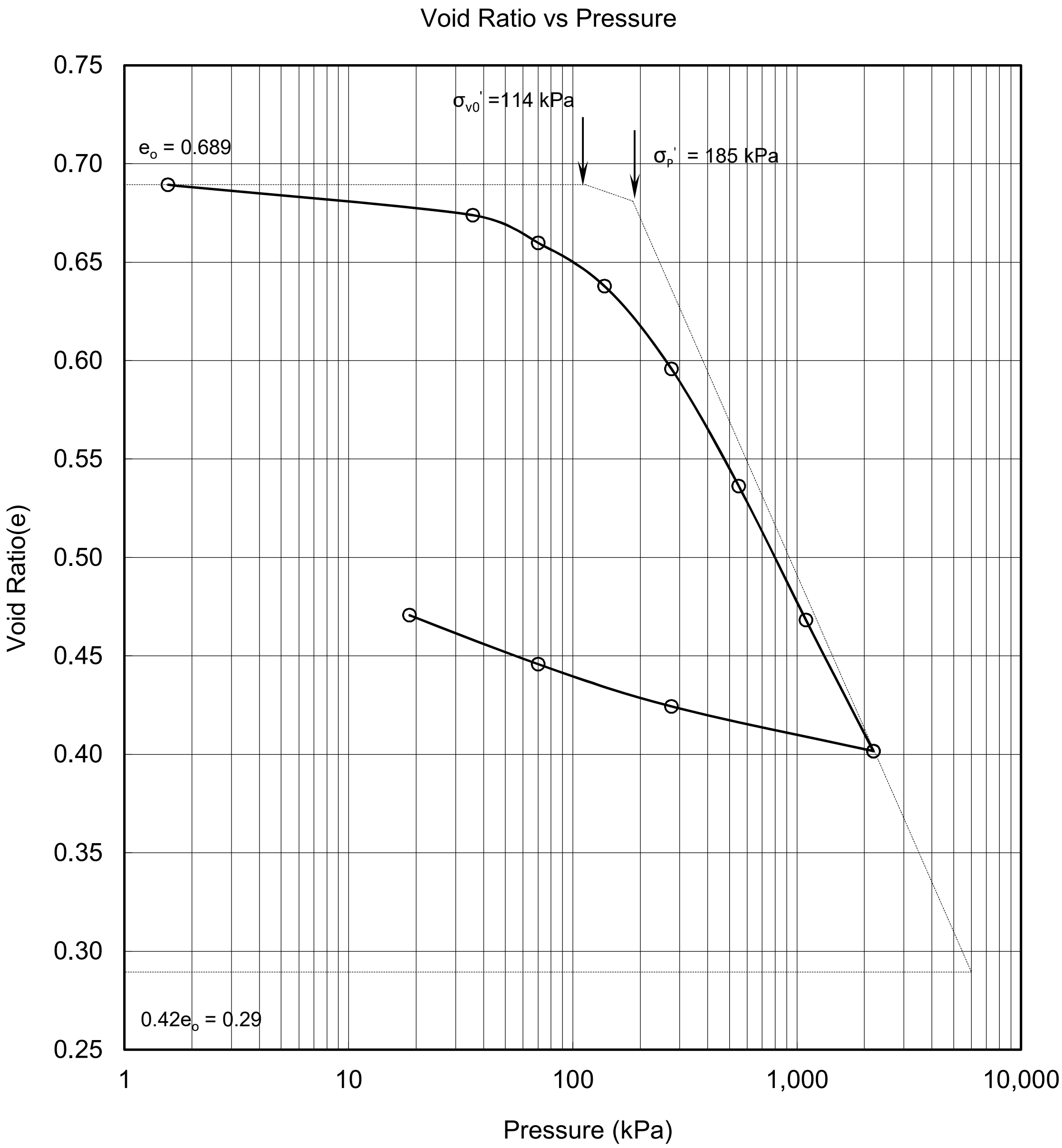
Site: Tee Creek North Bound
Sample # : BH1 TW11



Project No. :	1-15-0689	Terraprobe Inc.	Prepared By :	SD
Date :	February 2016		Checked By :	RA

CONSOLIDATION TEST	FIGURE B12

Site: Tee Creek North Bound
Sample # : BH1 TW11



Soil Type : Silty Clay

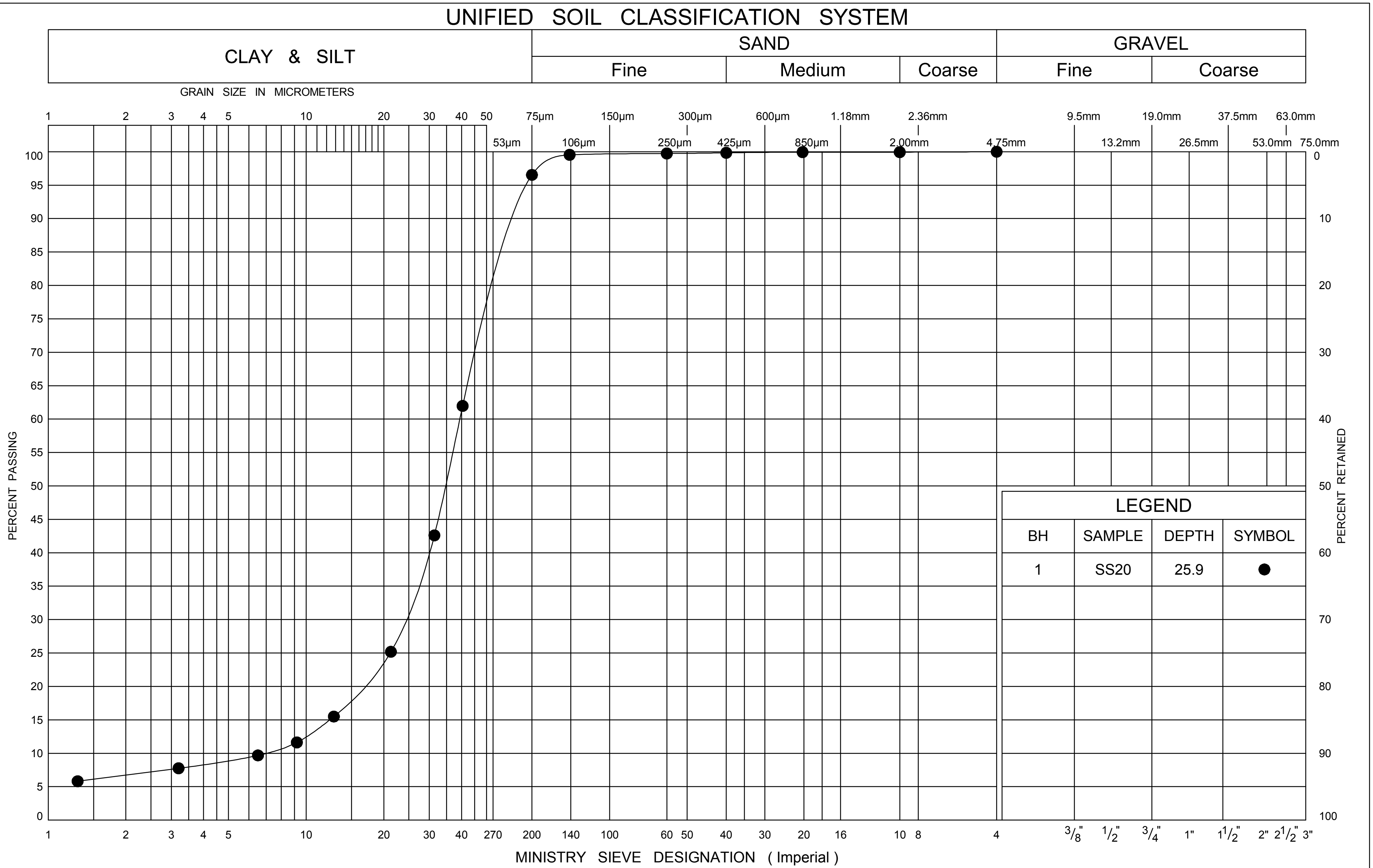
e_o =	0.69	ω_L =	30%	σ_{v0}' =	114.0 kPa
ω =	26%	ω_P =	17%	σ_P' =	185.0 kPa
γ =	20.0 kN/m^3	PI =	13%	C_c =	0.259
G_s =	2.73			C_r =	0.040

Project No. : 1-15-0689
Date : February 2016

Terraprobe Inc.

Prepared By : SD
Checked By : RA

C:\Users\rabdul\Documents\Reviewed Reports\Tee Lyons Black Creek\Consolidation Results - TEE Creek.xlsx



LEGEND

BH	SAMPLE	DEPTH	SYMBOL
1	SS20	25.9	●

Ministry of Transportation
Ontario

GRAIN SIZE DISTRIBUTION
SILT

FIG No B13

DB 2014-2036

APPENDIX B2

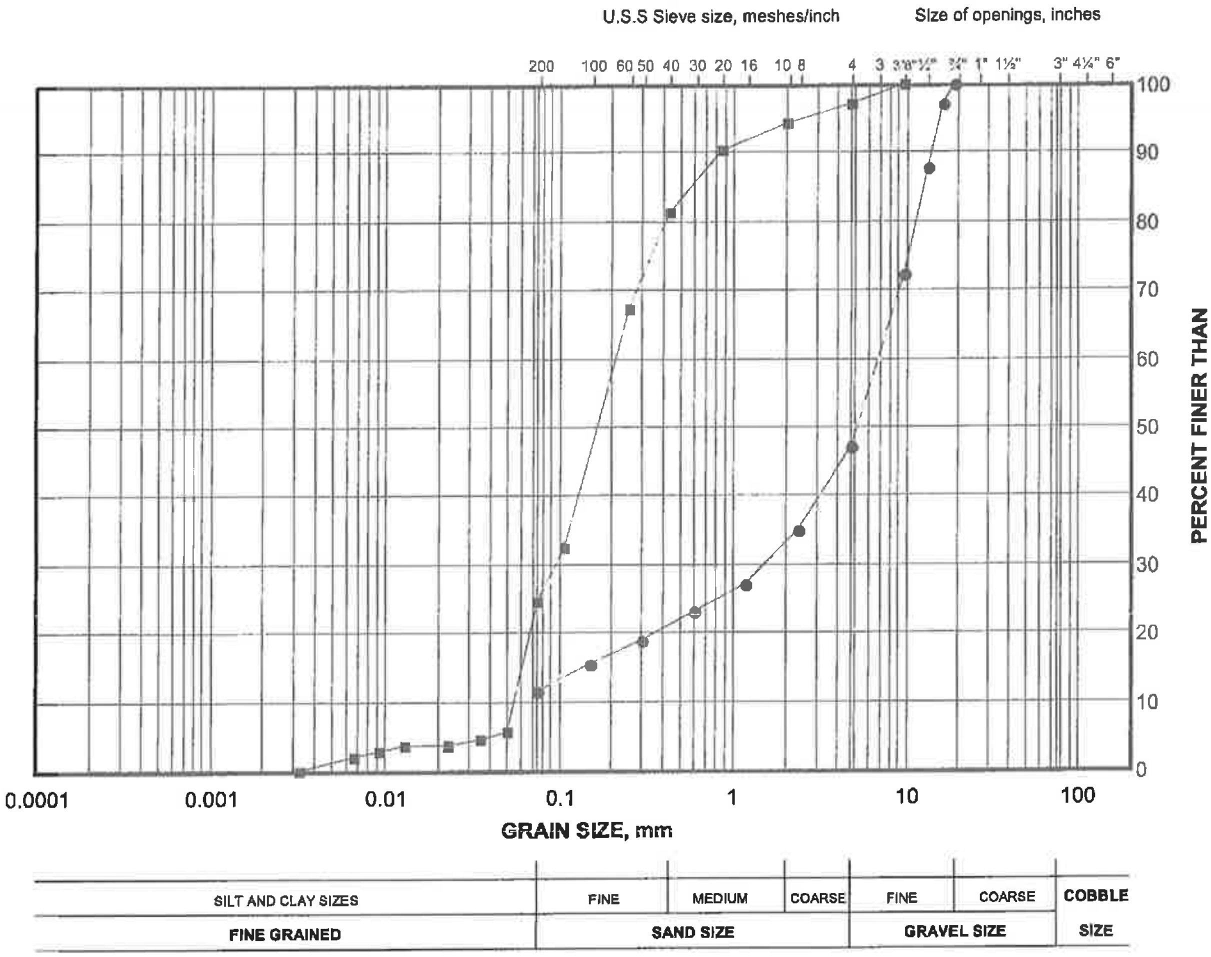
Laboratory Test Results

Golder

GRAIN SIZE DISTRIBUTION

Silty Sand to Sand and Gravel Fill

FIGURE B1



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-10	1A	173.3
■	13-07	2	173.1

Project Number: 12-1111-0088

Checked By: ______

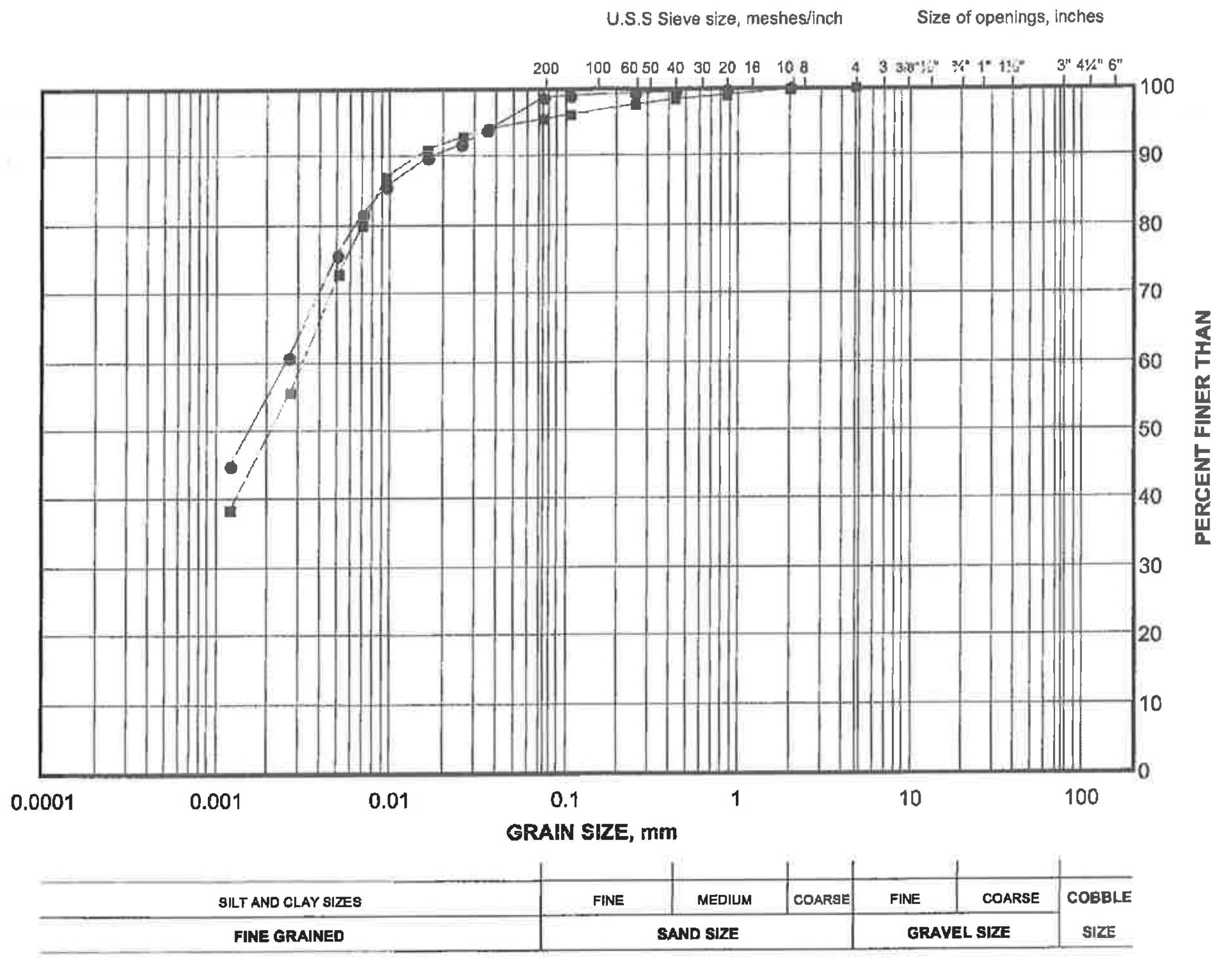
Golder Associates

Date: 30-Oct-13

GRAIN SIZE DISTRIBUTION

Silty Clay Fill

FIGURE B2



LEGEND

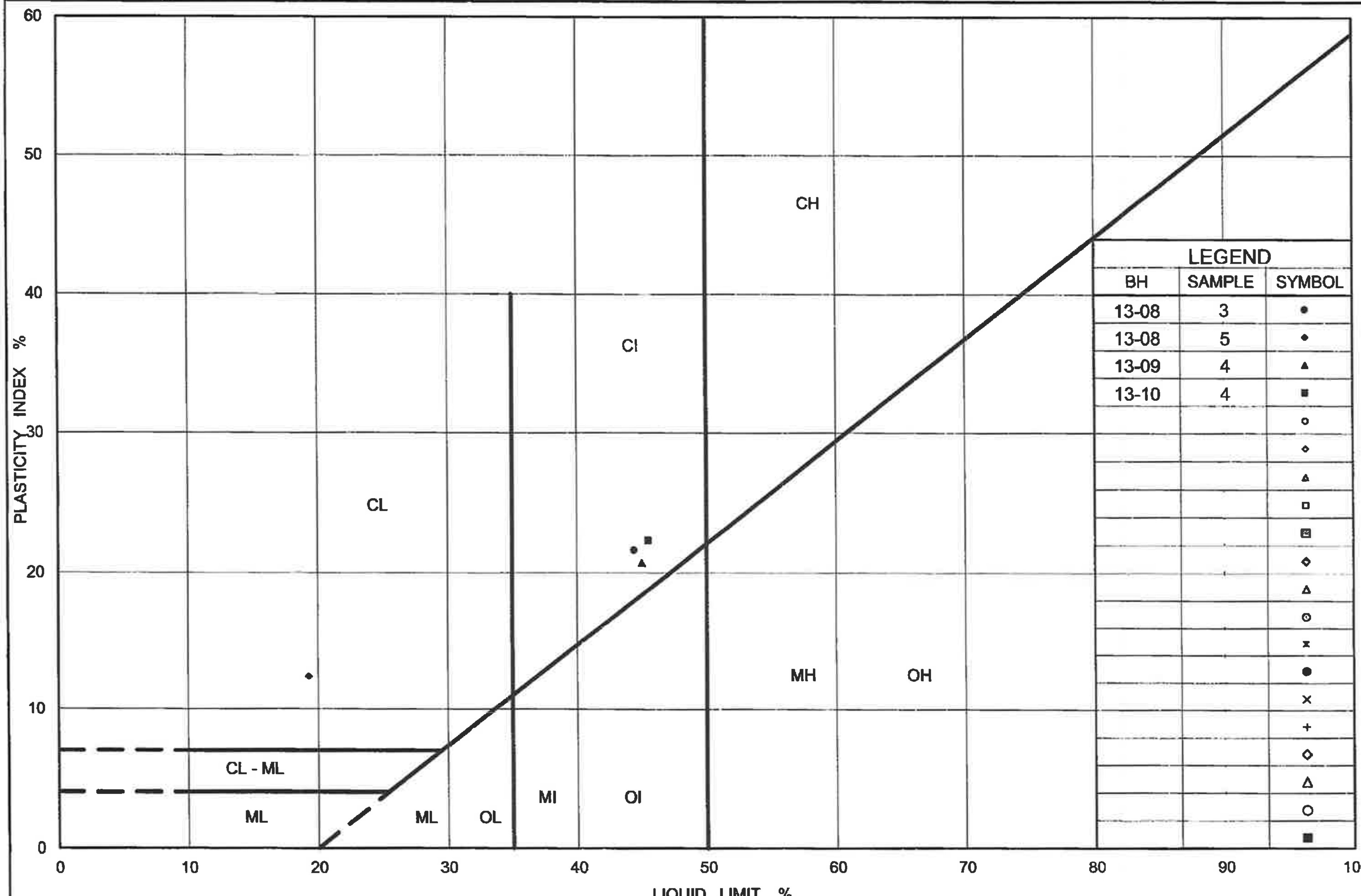
SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-10	4	170.8
■	13-09	4	171.4

Project Number: 12-1111-0088

Checked By: ______________

Golder Associates

Date: 30-Oct-13



Ministry of Transportation

Ontario

PLASTICITY CHART

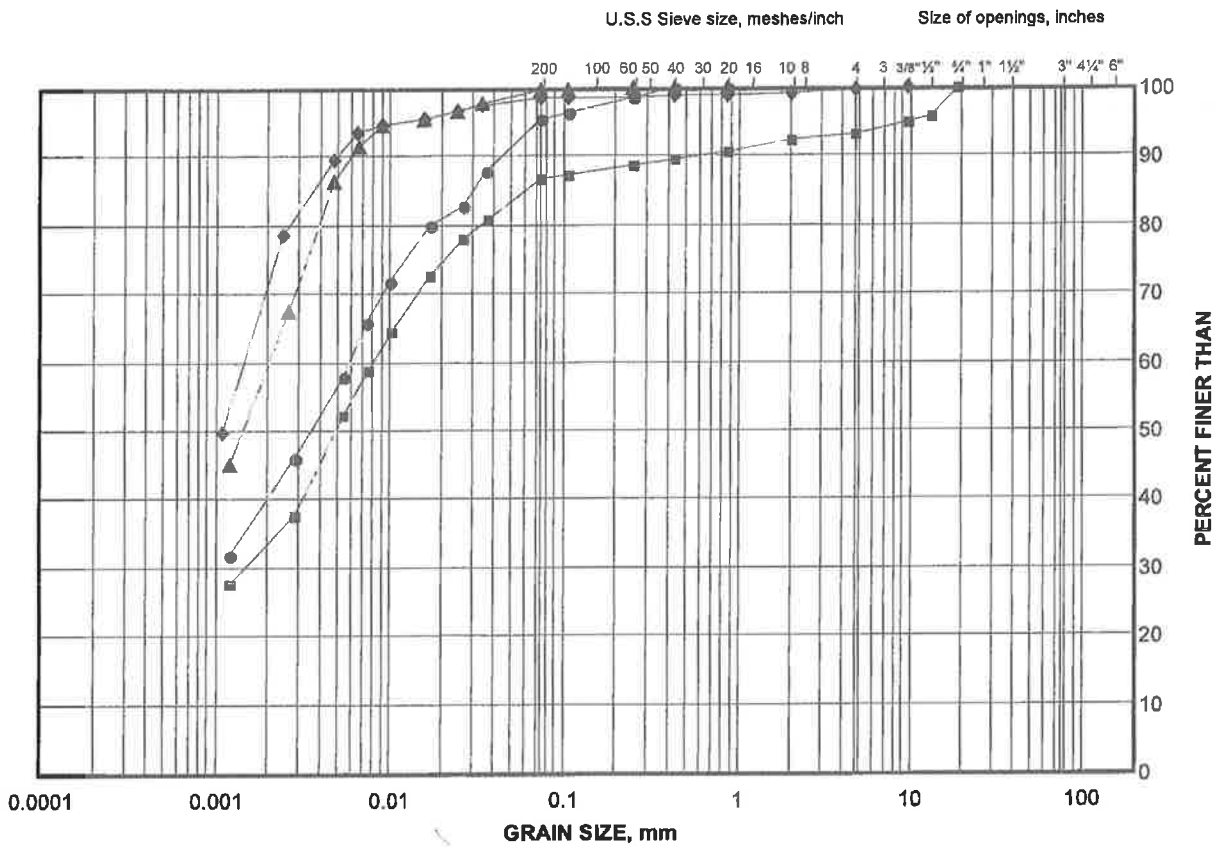
Clayey Silt to Silty Clay Fill

Figure No. B3

Project No. 12-1111-0088

Checked By:

GRAIN SIZE DISTRIBUTION Clayey Silt to Clay	FIGURE B4



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-09	12	160.0
■	13-10	15	155.6
◆	13-07	6	169.3
▲	13-08	7	169.3

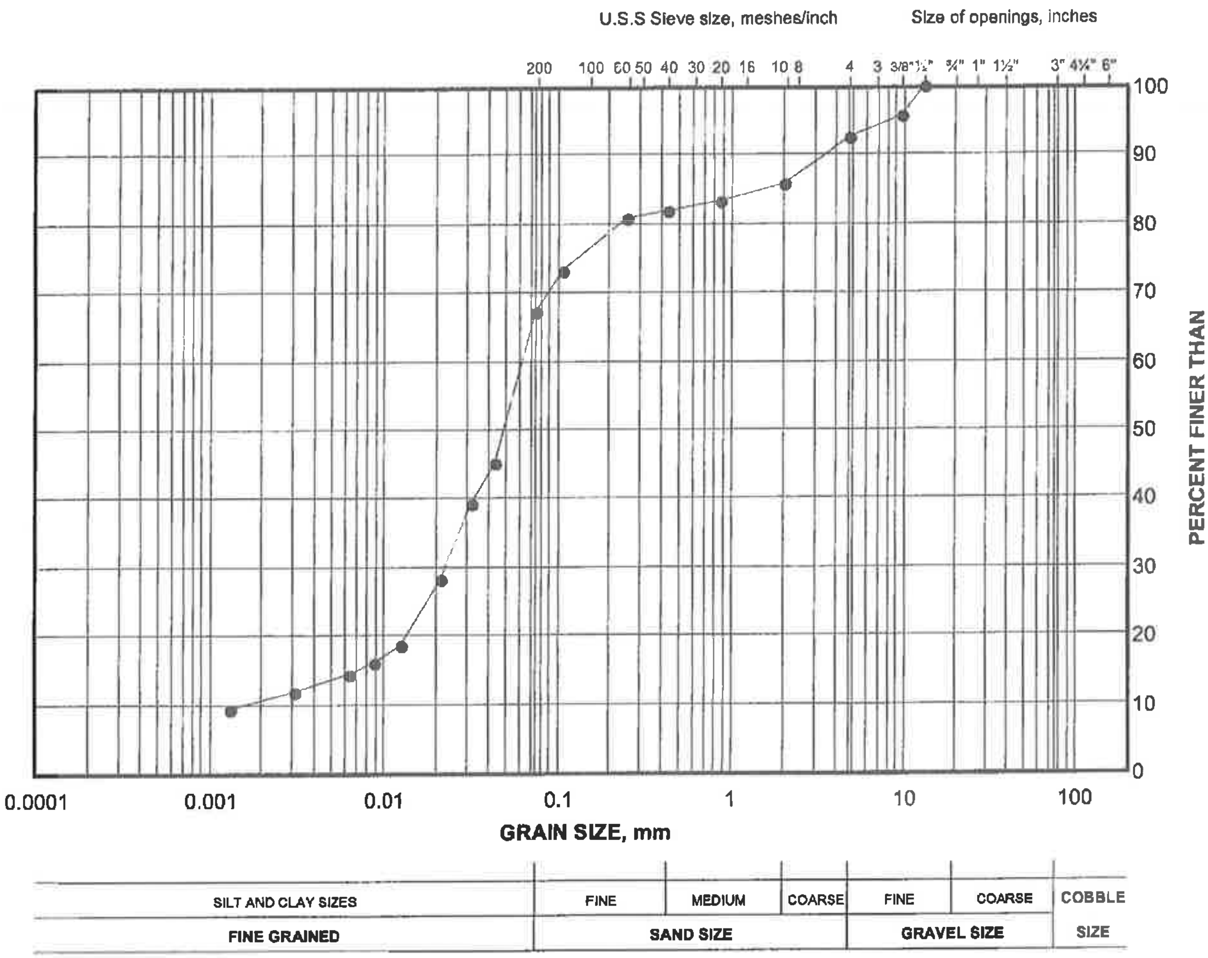
Project Number: 12-1111-0088

Checked By: ________

Golder Associates

Date: 30-Oct-13

GRAIN SIZE DISTRIBUTION Sandy Silt	FIGURE B5



LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-09	15	155.4

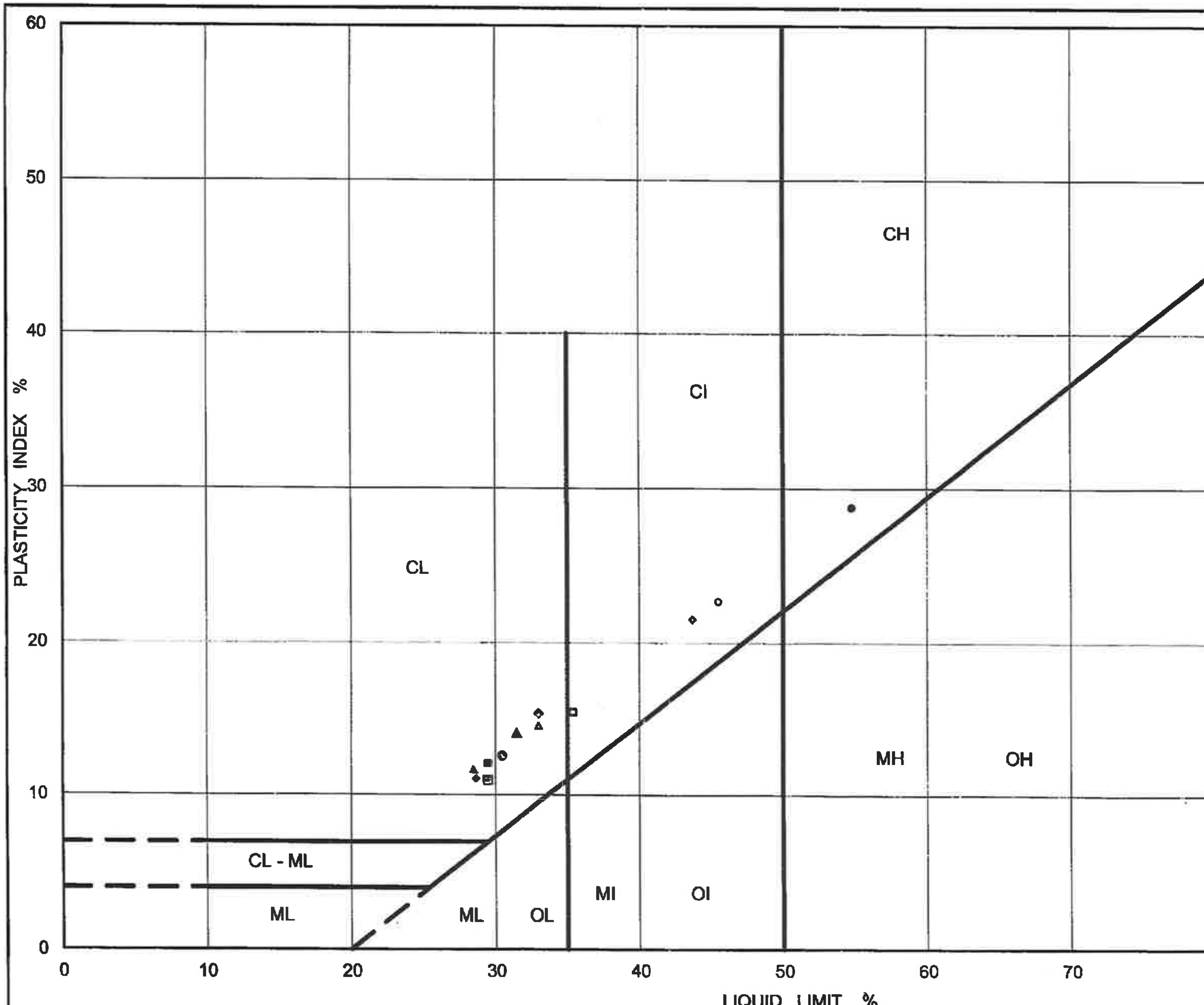
Project Number: 12-1111-0088

Checked By: ____________

Golder Associates

Date: 30-Oct-13

Oct 75, FF-S-21



LEGEND

BH	SAMPLE	SYMBOL
13-07	6	●
13-07	9	◆
13-07	10	▲
13-07	15	■
13-08	7	○
13-09	8	◇
13-09	12	△
13-09	16	□
13-10	10	⊡
13-10	13	◇
13-10	14	△
13-10	15	⊘
		×
		●
		×
		+
		◇
		△
		○
		■

Ministry of Transportation

Ontario

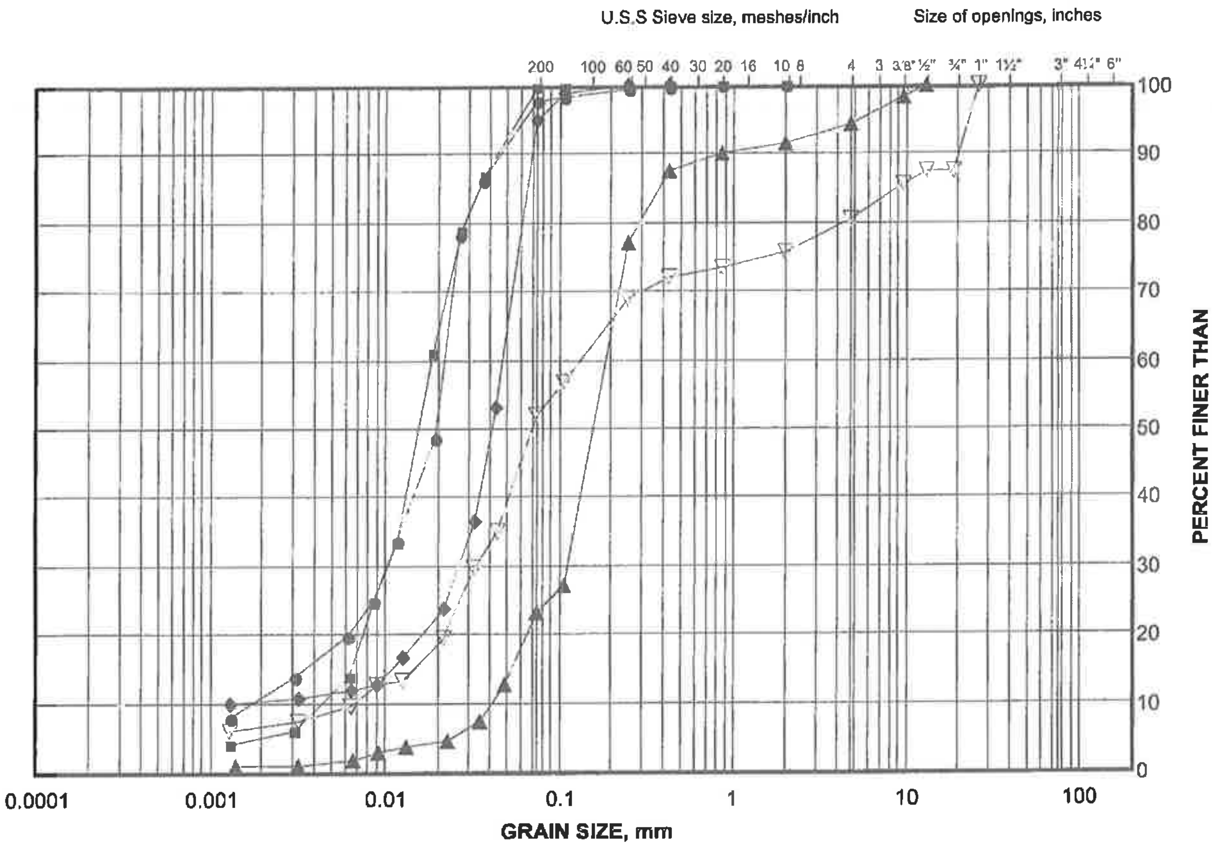
PLASTICITY CHART
Clayey Silt to Clay

Figure No. B6

Project No. 12-1111-0088

Checked By: [signature]

GRAIN SIZE DISTRIBUTION Silt to Silty Sand	FIGURE B7



SILT AND CLAY SIZES	FINE	MEDIUM	COARSE	FINE	COARSE	COBBLE
FINE GRAINED	SAND SIZE			GRAVEL SIZE		SIZE

LEGEND

SYMBOL	BOREHOLE	SAMPLE	ELEVATION(m)
●	13-07	17	149.5
■	13-10	17	149.5
◆	13-09	18	146.3
▲	13-07	19	143.4
▽	13-09	19	143.2

Project Number: 12-1111-0088

Checked By: ____________

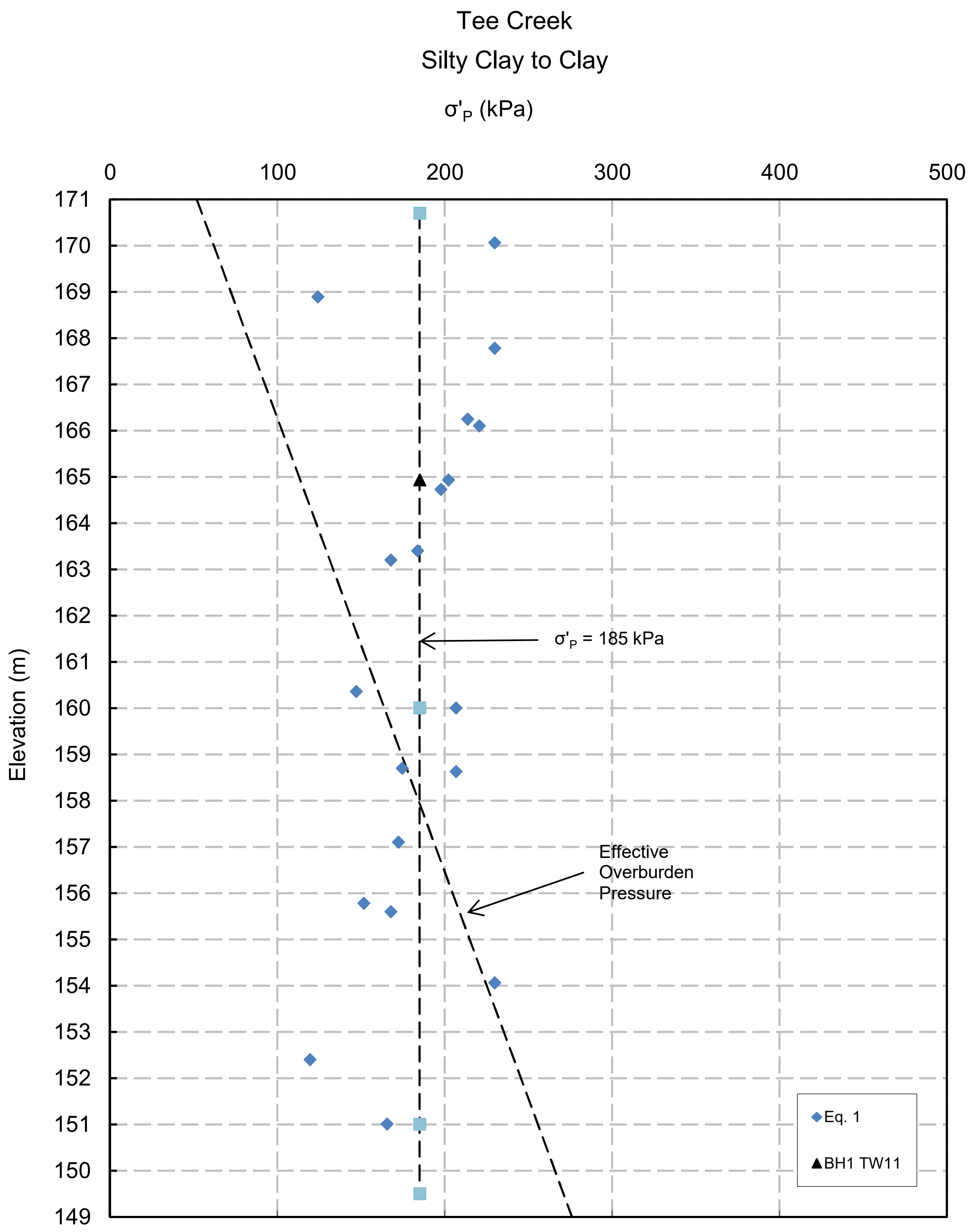
Golder Associates

Date: 30-Oct-13

APPENDIX C

Soil Design Parameters

PREDICTED AND MEASURED PRECONSOLIDATION STRESSES	FIGURE C1



Eq. 1 $\sigma'_P = C_u / (0.11 + 0.0037 * IP)$ Chandler (1988)

Project No. : 1-15-0689

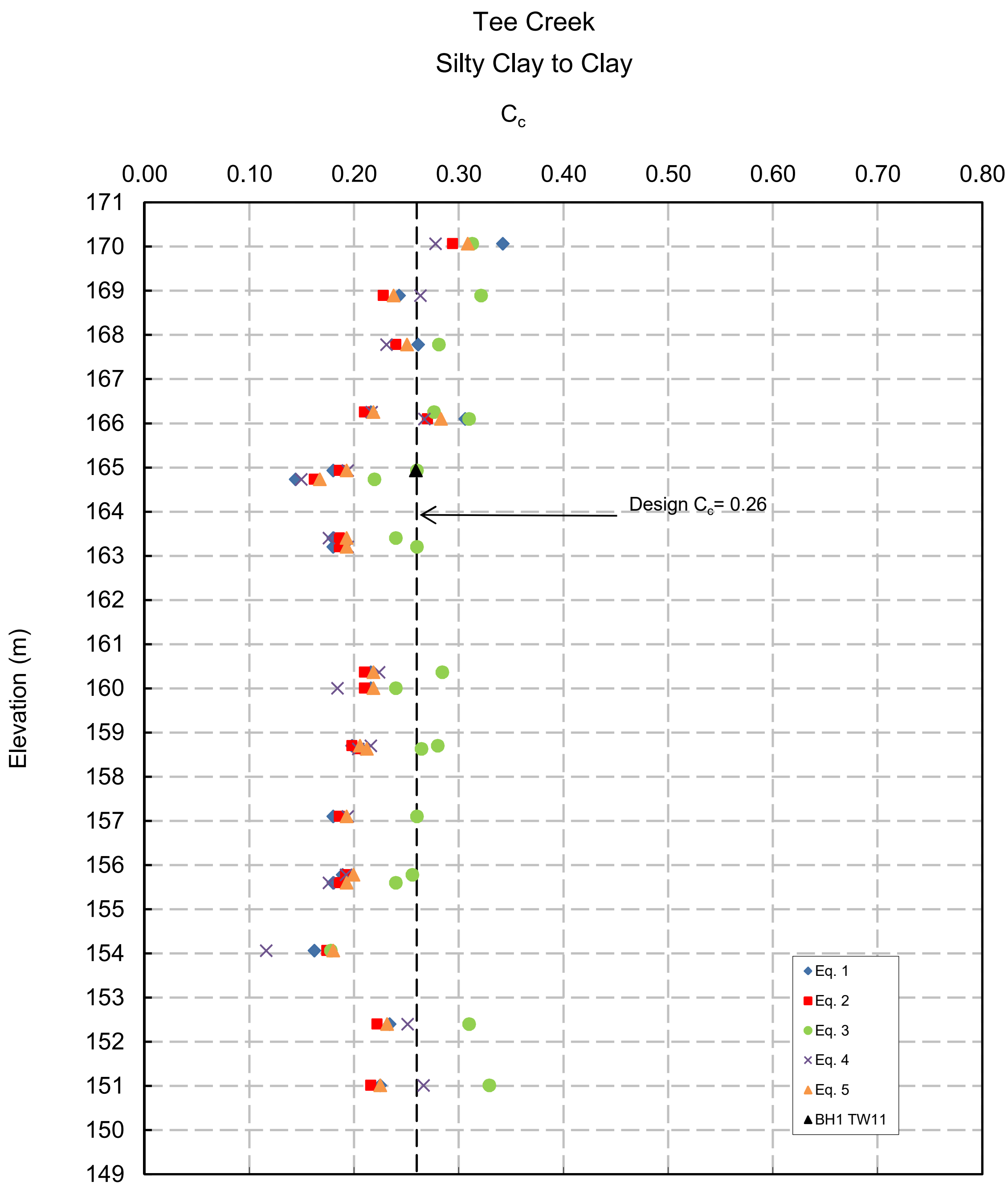
Date : February, 2016

Terraprobe Inc.

Prepared by : SD

Checked by : RA

PREDICTED AND MEASURED COMPRESSION INDICES	FIGURE C2



Eq. 1	$C_c = 0.009 * (LL - 10)$	Terzaghi & Peck (1967)
Eq. 2	$C_c = 0.006 * (LL + 1)$	Lav & Ansal (2001)
Eq. 3	$C_c = 0.01 * \omega$	Osterberg (1972)
Eq. 4	$C_c = 0.009 * \omega + 0.002 * LL - 0.1$	Azzouz et al. (1976)
Eq. 5	$C_c = 0.002343 * LL * Gs$	Nagaraj & Murty (1985)

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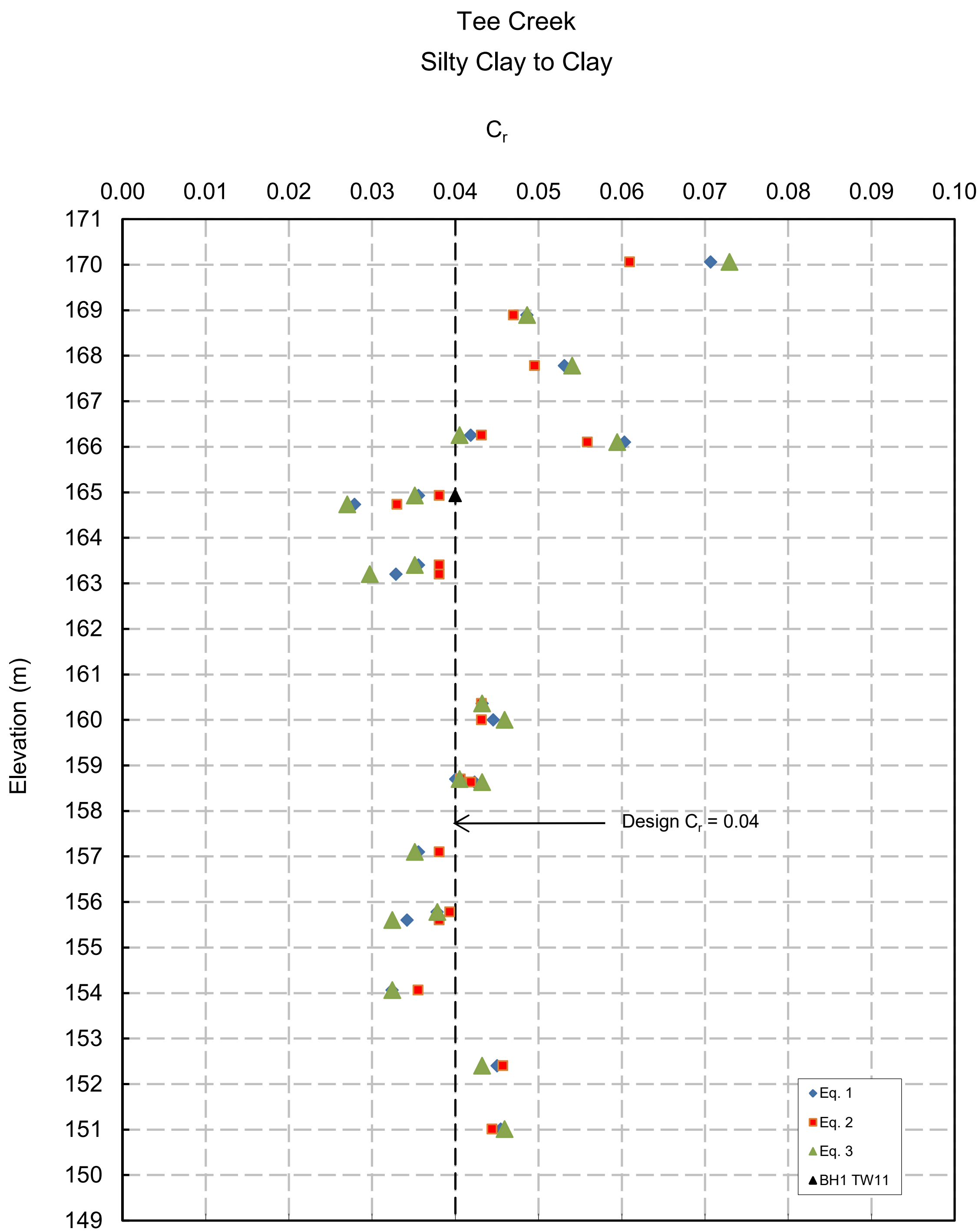
PREDICTED AND MEASURED RECOMPRESSION INDICES	FIGURE C3



Eq. 1	$C_r = C_c / 5 \sim Cc / 10$	Das (1993)
Eq. 2	$C_r = 0.000463{*}LL{*}G_s$	Nagaraj & Murty (1985)
Eq. 3	$C_r = I_p / 370$	Kulhawy & Mayne (1990)

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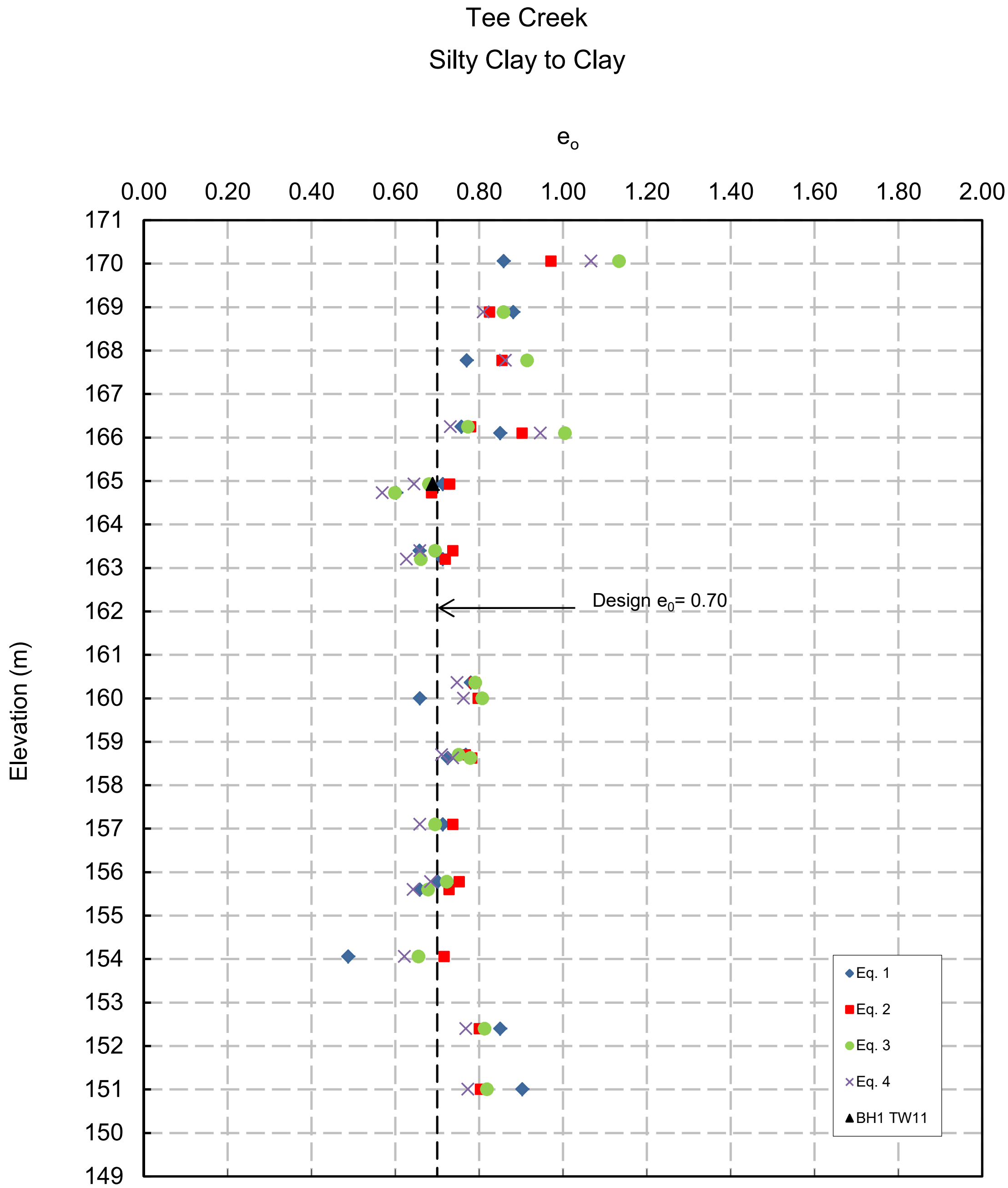
C:\Users\rabdul\Documents\Reviewed Reports\Tee Lyons Black Creek\e0-P'c-Cc-Cr-Cu - TEE Creek RA.xlsx

PREDICTED AND MEASURED VOID RATIOS	FIGURE C4



Eq. 1	$e_o = w * G_s$	when saturated
Eq. 2	$e_o = C_c / 0.75 + 0.50$	derived from Sowers (1970)
Eq. 3	$e_o = (C_c + 0.10) / 0.40$	derived from Lav & Ansal (2001)
Eq. 4	$e_o = (C_c - 0.256) / 0.43 + 0.84$	derived from Cozzolino (1961)

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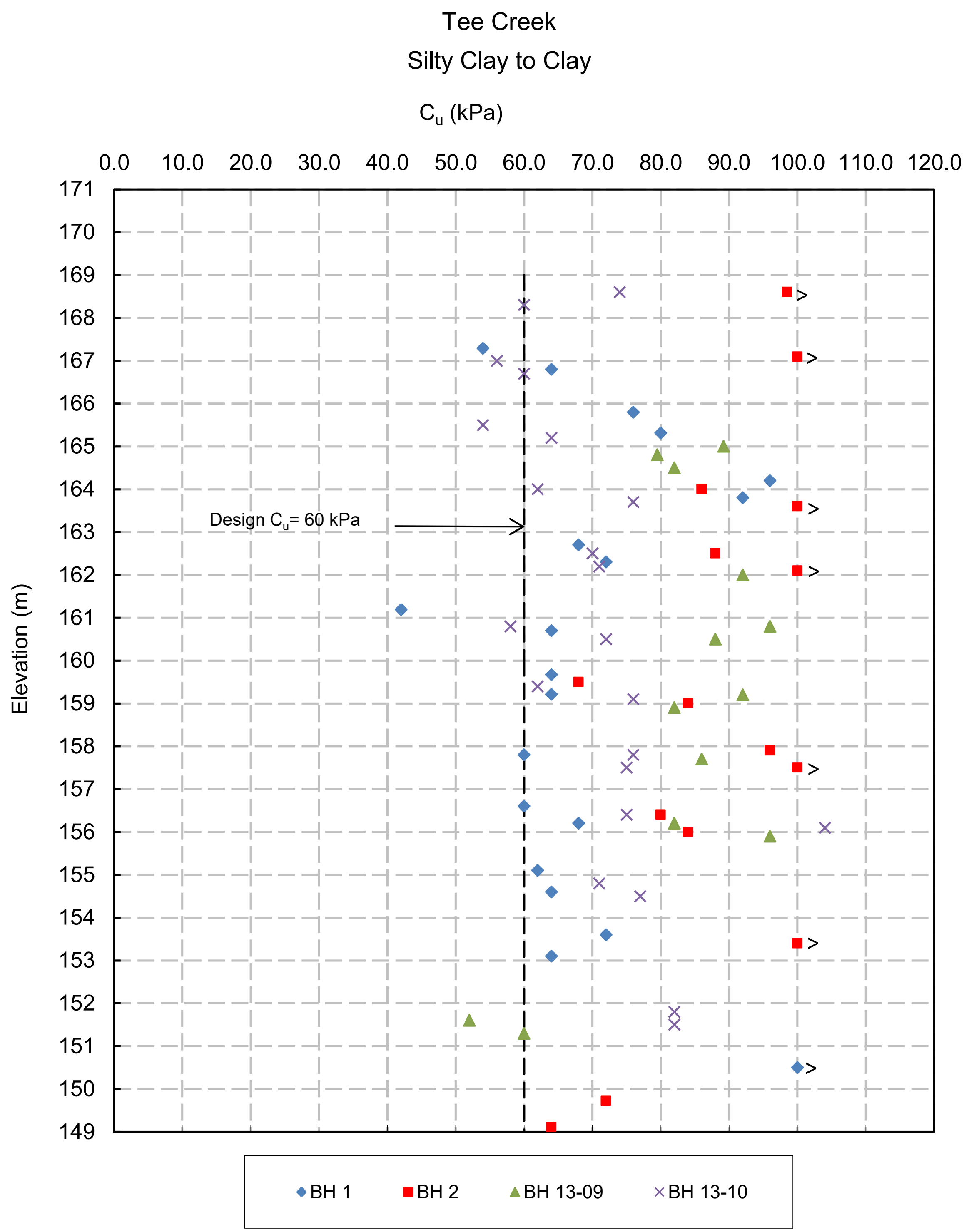
UNDRAINED SHEAR STRENGTH	FIGURE C5



Field vane shear strengths were corrected based on Bjerrum, (1972) for I_P>20

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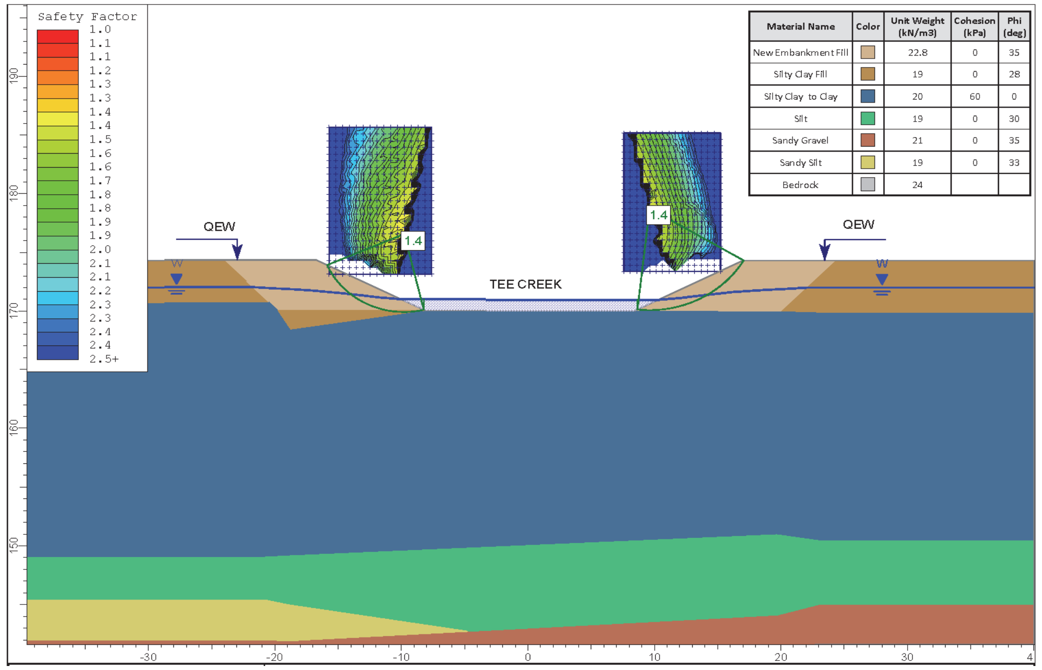
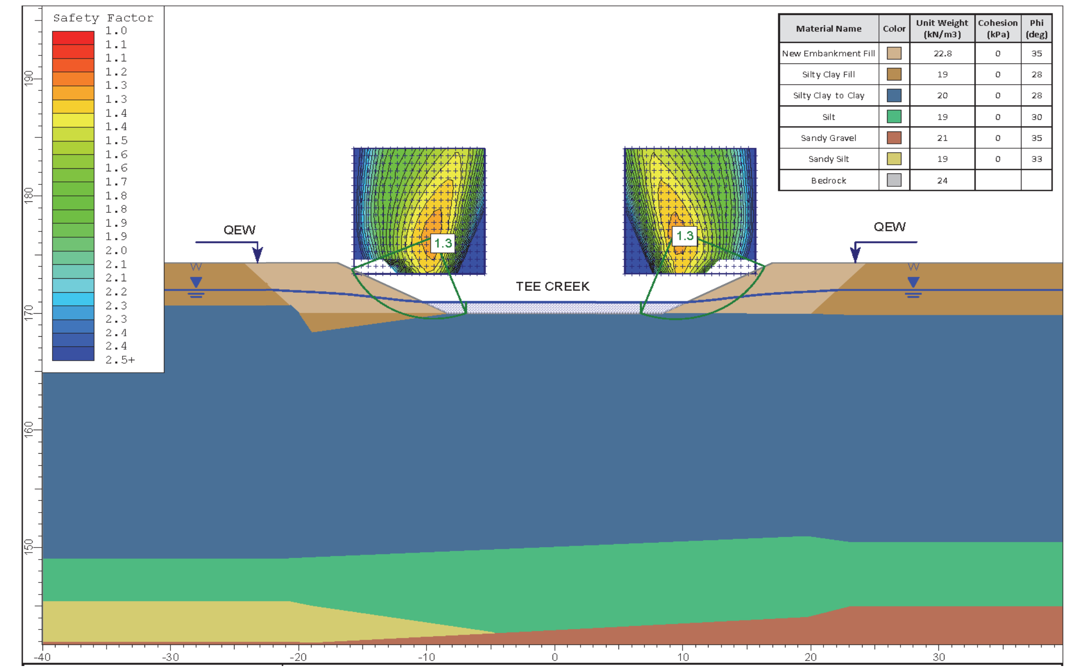
APPENDIX D

Slope Stability Models and Results



Tee Creek North-Bound Structure
Forward Slope Stability (Total Stress Analysis)



Tee Creek North-Bound Structure
Forward Slope Stability (Effective Stress Analysis)

Terraprobe Inc. Consulting Geotechnical & Environmental Engineering Construction Materials, Inspection & Testing 11 Indell Lane - Brampton Ontario L6T 3Y3 (905) 796-2650	TEE CREEK BRIDGE REPLACEMENT			
	DATE: February 12, 2016			
	SUBM'D:	SD	CHKD:	RA
	Project No:	1-15-0689	Figure:	D1