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**Preliminary Foundation Investigation and Design
Report**
Sturgeon River Bridge
Township of Shenston, District Rainy River
Highway 11

Assignment 2016-E-0028
WP 6855-14-00
Geocres No. XX XXX
Site No. 45-105

Prepared for
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Part A - FOUNDATION INVESTIGATION REPORT

1 Introduction

TBT Engineering Limited (TBTE) has been retained by the Associated Engineering Ltd. (AE) to provide preliminary foundation investigation and design services for the replacement of Sturgeon River Bridge, Site 45-105, in Township of Shenston, Rainy River District, Ontario. It is understood the bridge will be replaced at its current location. The project is located on Highway 11, approximately 8.4 km west of Highway 71. The site coordinates are as follows:

- Latitude: 48.654788°
- Longitude: -94.027112°

The preliminary foundation investigation was completed to investigate subsurface conditions. The investigation consisted of two boreholes total, one at each abutment of the existing bridge (within 5-10 m of each abutment). This report (Part A) describes the subsurface conditions encountered during the investigation.

The MTO Foundation section has assigned Geocres No. _____ to this site.

2 Site Description

The Sturgeon River Bridge is located on Highway 11, approximately 10.5 km east of Statton and 8.4 km west of Highway 71. The existing timber bridge with concrete decking spans approximately 54 m, and is founded on timber piles. The river level was measured at elevation 329.7 m on February 2013 (provided by others), and at 329.2 m on July 25, 2017 and flows from south to north of the Highway. The existing approach embankments vary from 2:1 to 2.5:1 slopes and range in height from 2 to 3 m. The foreslope of the river embankment adjacent to the structure is approximately 7 m high with slopes varying from 2:1 to 5:1. The existing roadway embankments are at elevations 332.3 m (east) and 334.4 m (west).

Figure 2.1: Entire Bridge Looking Northerly



Figure 2.2: East Approach



Figure 2.3: West Approach



2.1 Surficial Geology

The Sturgeon River Bridge is located in an alluvial plain deposit which consists of peat, silt, clay and sand as defined by the Ontario Ministry of Natural Resources Northern Ontario Engineering Geology Terrain Study (NOEGTS), 1980, Map No. NTS 52D/NE "Stratton Area". Some alluvial deposits occur above flood level along Rainy River and consist of sand and silt in association with glaciolacustrine silt and clay. These deposits are underlain by sand and gravel of glaciofluvial origin.

With reference to the NOEGTS, the alluvial plans in the area generally have poor drainage, high water table and/or periodic flooding

Presence of clay, silt, and sand underlain by sand and gravel deposit is confirmed from field investigation.

3 Investigation Procedures

A geotechnical site investigation was undertaken on June 13 to 23, 2017. The field program consisted of one borehole at each abutment for a total of two boreholes

The borehole locations were identified in the field by TBTE personnel and service clearances were completed prior to mobilizing the drill rig to site. The boreholes were advanced using a tire mounted drill rig equipped with hollow stem augers and a cat head used to carry out Standard Penetration Testing (SPT). During the drilling operations for the boreholes, soil samples were obtained from the auger flights and using the techniques of the standard penetration test (SPT). In addition, thin walled tube samples were taken and field vane tests were completed within the clay soils.

Borehole locations were surveyed by TBTE and were based on North American Datum 1983, MTM CSRS CBNV6-1997 Zone 16 and Canadian Geodetic Vertical Datum 1928:1978 adjustment (CGVD1928;78). Control was established from published control downloaded from Ministry of Natural Resources Cosine Online Geodetic Control Service and control shown on the provided Ministry Plans. Multiple control points were used for control establishment and position verification.

A summary of the borehole location data is provided on the enclosed Borehole Location Plan and Strata Drawings, Appendix C.

All boreholes have been backfilled and/or decommissioned in accordance with O.Reg. 903.

4 Laboratory Testing

Samples which were obtained during the field investigation were subjected to routine laboratory testing. The routine testing included moisture content, Atterberg and hydrometer/grain size analysis. The results of this testing are shown on the Borehole Logs (Appendix A and on the laboratory data reports Appendix B).

5 Subsurface Conditions

Details of the subsurface conditions are provided on the borehole logs (Appendix A), and on the Soil Strata Drawings (Appendix C).

The subsurface soils at this site typically consist of asphalt on surface which is underlain by granular and clay fill. Clay underlies the fill and extends to sand in which both boreholes were terminated.

5.1 Asphalt/Concrete

A 70 - 90 mm thick layer of asphalt was encountered at the surface of both boreholes and 260 mm of concrete was encountered under asphalt at Borehole 1.

5.2 Fill

Granular fill was encountered beneath concrete and asphalt at Boreholes 1 and 2, respectively. The granular fill is encountered at elevations 335.4 and 335.6 with thicknesses of 1.1 to 0.9 m at Boreholes 1 and 2, respectively. Clay fill was identified at Borehole 1 below the granular fill and extends to depth of 2.2 m (elevation 333.5 m). An Atterberg limit tests conducted on a single sample indicates that the clay fill is of high plasticity with natural moisture content between plastic limit and liquid limit. Hydrometer testing completed on a sample indicates that material can consist of 55 % clay, 25 % silt

and 20 % sand sized particles. This material is in stiff condition with SPT “N” value of 11 blows / 0.3 m.

5.3 Clay

Clay is present underlying the fill material at both boreholes. The deposit is encountered at elevations of 333.5 and 334.8 m with thickness of 19 to 22.4 m, at Boreholes 1 and 2 respectively. The clay deposit consists of two distinct sub strata, clay with low to medium plasticity and clay of high plasticity.

5.3.1 Clay – Low to Medium Plasticity

Sandy clay to clay with some sand, with trace gravel was encountered beneath the fill at both boreholes and below the high plastic clay at Borehole 2 at a depth of 20.5 m. This stratum (above the high plastic clay) ranges in thickness from 12.6 to 15.2 m, for the strata below the high plastic clay is 2.8 m thick. Atterberg limit tests conducted on select samples indicate that material is of low to medium to high plasticity with natural moisture content between the plastic limit and liquid limit. Grain size analyses completed on a select samples indicate that the material can consist of 1 % gravel, 13 to 27 % sand, 34 to 44 % silt and 31 to 46 % clay sized particles. This material is in firm to stiff condition based on SPT “N” values ranging from 6 to 9 blows / 0.3 m. Insitu vane tests indicate the undrained shear strength of the material is more than 100 kPa. The results of these test could be influenced by the presence of silt and sand, and typically overestimates the undrained shear strength.

5.3.2 Clay – High Plasticity

A high plastic clay with thin layers of silt was encountered beneath the clay at a depth of 14.8 and 16.1 m (elevation 320.9 and 319.6 m), with thicknesses of 6.4 and 4.4 m, at Boreholes 1 and 2 respectively. Atterberg limit tests conducted on two samples indicate that material is of high plasticity with natural moisture content between the plastic and liquid limits. Grain size analysis completed on two samples indicate that material can consist of 0 % gravel, 0 % sand, 9 and 10 % silt and 90 and 91 % clay sized particles. This material is in soft to firm condition with SPT “N” values ranging from 4 to 8 blows / 0.3 m. Insitu vane tests indicate the undrained shear strength of the material is more than 100 kPa.

5.4 Silt

A layer of sandy silt to trace sand with some to trace gravel was encountered at Borehole 1, beneath the high plastic clay. The silt is encountered at a depth of 21.2 m (elevation 314.5 m) with a thickness of 2.8 m. Two samples were selected for grain size analyses and indicate that the material can consist of 0 to 11 % gravel, 05 to 32 % sand, and 57 to 95 % silt/clay sized particles. The silt is in a very loose to loose condition with SPT "N" values of 1 and 9 blows / 0.3 m.

5.5 Sand

Sand and silt with trace gravel to gravelly silty sand is encountered beneath the silt at Borehole 1 and beneath the low plastic clay at Borehole 2. Both boreholes were terminated within this material at depths of 30 and 29.5 m. Grain size analyses was completed on three samples indicate that material can consist of 0 to 21 % gravel, 57 to 86 % sand, and 14 to 40 % silt/clay sized particles. Occasional cobbles were also noted with within this stratum. The sand is in loose to dense condition with SPT "N" values ranging from 7 to 39 blows / 0.3 m.

5.6 Auger Refusal/Termination

Borehole 1 was terminated at 30 m depth (elevation 305.7 m) and, Borehole 2 met auger and SPT refusal (100 blows per 0.3 m) at 29.5 m depth (elevation 306.2 m) both within the sand stratum. Sampling of the refusal material using coring techniques was not required as part of preliminary investigation.

6 Groundwater

The groundwater levels were read upon completion of drilling. Measured groundwater levels have been provided below. The river level was reported at be at elevation 329.7 m on February 2013 (provided by others) and measured at an elevation of 329.2 on July 25, 2017. Groundwater levels may vary from season to season and from the effects of heavy precipitation events.

Table 5.5: Groundwater Levels

Location	Surface Elevation (m)	Groundwater Level (elev. m)
		Completion of Drilling
1	335.6	330.7
2	335.7	330.5

7 Miscellaneous

Laboratory testing was carried out at the TBT Engineering laboratory in Thunder Bay. The drill equipment for this investigation was operated by TBT Engineering Limited. The field operations were supervised by Alan Finke. Laboratory testing was supervised by T. Fummerton C.E.T. This report was prepared by Steven Seller, P.Eng, and reviewed by W. Hurley, P.Eng (TBTE designated principal contact identified for MTO Foundation Engineering projects).

Part B - FOUNDATION DESIGN RECOMMENDATIONS

8 Introduction

TBT Engineering Limited (TBTE) has been retained by the Associated Engineering Ltd. (AE) to provide preliminary foundation investigation and design services for the replacement of Sturgeon River Bridge, Site 45-105, in Township of Shenston, Rainy River District, Ontario. It is understood the bridge will be replaced at its current location. The project is located on Highway 11, approximately 8.4 km west of Highway 71. The site coordinates are as follows:

- Latitude: 48.654788°
- Longitude: -94.027112°

The foundation investigations as described in Part A, were completed to investigate subsurface conditions at this site. The investigations consisted of the advancement of two boreholes, and laboratory testing. The Part A report describes the subsurface conditions encountered during the preliminary investigation.

The foundation soils at this site generally consists of embankment fill overlying clay which further overlies sand.

The purpose of this section of the report (Part B) is to provide preliminary approach embankment design and structure foundation recommendations for the replacement at Sturgeon River. These are based on the conditions encountered at the borehole locations, TBTE's interpretation of the subsurface conditions at the site and analyses of embankment stability.

9 Structure Foundation

Two foundation systems have been considered for the proposed crossing. Design recommendations for viable foundation systems are presented below based on the subsurface conditions encountered on site.

Unless noted otherwise, foundation design parameters are given for static, vertically and concentrically loaded foundations in compression.

9.1 Initial Review of Foundation Options

Three options for the proposed crossing were reviewed from a foundations perspective and are presented below. Options reviewed address structures on footings, structures on end bearing piles, and friction piles.

Table 9.1: Foundation Options

Option		Advantages	Disadvantages	Comments
Foundation Type	Footings	<ul style="list-style-type: none"> - footings can be founded on granular pads to increase ULS and SLS values over the native clay - relatively low cost of construction 	<ul style="list-style-type: none"> - highly compressible foundation soils will lead to low SLS values and long term settlements - dewatering required for construction in the dry - deep excavation required to provide frost protection - if existing piles are left in place, they may influence the performance of the footings 	Not Recommended
	End Bearing Piles	<ul style="list-style-type: none"> - highest capacities can be achieved - excavation below water level may be reduced or eliminated - longer spans may be considered to minimize construction within the existing channel - suitable for integral and semi integral abutment design 	<ul style="list-style-type: none"> - potential driving issues due to presence of cobbles and/or boulders within sand deposit - dewatering may be required for construction in the dry depending on pile cap elevation and location - relatively high cost of construction, specialized 	Recommended
	Friction Piles	<ul style="list-style-type: none"> - capacities higher than footing can be achieved - excavation below water level may be reduced or eliminated - longer spans may be considered to minimize construction within the existing channel - suitable for integral and semi integral abutment design 	<ul style="list-style-type: none"> - presence of cobbles and/or boulders within sand deposit - dewatering may be required for construction in the dry depending on pile cap elevation and location - relatively high cost of construction, specialized - lower capacity than end bearing piles - numerous piles required - potential long term settlement 	Not Recommended

9.2 End Bearing Driven Piles

Driven piles for the bridge foundation can be considered. The use of end bearing piled foundations must consider the type, quality and variation of the refusal material. The following pile recommendations are subject to the following conditions:

- Pile type is a HP 310 x 110
- Steel graded to 350 MPa
- For piles located near/adjacent to the river the underside of the pile cap has been assumed to be at the approximate elevation of 326 m. This is based on

constructing the pile cap at the estimated frost depth, measured from the bottom of the water body.

- For piles located within the embankment the underside of the pile cap has been assumed to be at the approximate elevation of 330.0 to 332.1 m, for west and east embankments respectively. This is based on constructing the pile cap at the estimated frost depth, measured from the top of the roadway.
- Piles for the east and west abutment have been assumed to bear within the auger/spoon refusal material. The provided resistances are based on static analysis. Higher resistances may be provided once refusal material is confirmed.

For preliminary design purposes, the pile capacities indicated below are appropriate for either piles within the embankment or adjacent to the river.

Table 9-2 - Pile Design Capacities for Piles Driven to Refusal Material

Abutment	Pile Designation	ULS Factored Geotechnical Axial Resistance	SLS Geotechnical Resistance for 25 and 50 mm of Settlement
East & West	HP 310x110	1800 kN	Does not govern

For estimation quantity purposes only, the pile tip elevations of 304 m for the east and west abutment may be used, until further data is obtained. Preliminary pile lengths of 22 to 28 m can be estimated based on the assumed pile cap elevation.

Piles driven to practical refusal should be driven in accordance with OPSS 903. Care must be taken to ensure piles are not overstressed during driving. The installed depth of the piles may vary. The contractor must be prepared to drive piles of varying length.

The piles should be equipped with a driving shoe to prevent damage to the pile tip. The driving shoe can be a standard Titus pile point, or equivalent, if it is not available points as indicated in OPSD 3000.100 can be used. The behaviour of the piles should be monitored during driving for any signs indicative of pile damage, walking or skipping. It should be noted that cobbles and boulders were encountered within both boreholes.

10 Embankments

It is understood that the vertical alignment will not be altered. The proposed approach embankments, and the river embankment should be constructed using Granular B Type I, II and/or rock fill for various features.

10.1 Settlement

It is understood that the existing approach embankments will not be raised, nor will additional foundation loading will be applied; therefore no appreciable settlements are expected. If the approach embankments are extended (the approach spans are replaced with embankments), or foundation loads are applied to the compressible native soils settlement will occur.

For discussion purposes if a new approach embankment is constructed to an average height of 4 m over a length of 15 m (to simulate the replacement of the east approach spans with an embankment). The results of this analysis indicate that maximum settlements are in the order of 300 mm, anticipated post construction. Settlement analysis has been based on empirical correlations from the results of the laboratory testing available. Settlement induced by a new embankment will also create downdrag forces on a pile foundation, thereby reducing the structural capacities of the piles.

10.2 Geotechnical Model

Stability modeling was completed out using Slope/W software and limit equilibrium analysis using the Morgenstern-Price method.

Stability analyses have been completed to investigate potential configurations for the proposed river fore slope embankment for the proposed bridge. The design was based on providing a minimum calculated factor of safety (FoS) of 1.54 (resistance factor of 0.65) for final configuration. The resistance factor has been referenced from the 2014 Canadian Highway Bridge Code for embankment fills. A uniformly distributed traffic load of 12 kPa over the traversable lane(s) was applied in all cases.

Soil properties established for the embankment and foundation soils are presented below. The values presented for the native soils are based on empirical correlations. Complex testing during the detailed investigation should be completed to determine the strength parameters of the native soils.

Table 10-1: Stability Analyses Soil Properties

Soil	Effective Stress Strength Properties		Unit Weight γ (kN/m ³)
	Effective Angle of Internal Friction, ϕ' (degrees)	Effective Cohesion Intercept, C' (kPa)	
Native Sands	30	0	21
Native Silt	28	0	20
Fill - Clay	29	0	19
Fill - Granular	30	0	20
Clay – High Plastic	22	0	19
Clay – Low/Medium Plastic	27	0	19

10.3 Embankment Stability

The following assumptions were made for the analysis:

- Assumed water levels based on river survey and moisture contents.

The following recommendations have been derived from the analysis:

- Permanent cut slopes through existing native soil shall be constructed at 6(H):1(V), or flatter.
- Permanent cut slopes through existing native soil incorporating a 1 m thick granular sheeting can be constructed at 4(H):1(V), or flatter.

To provide steeper fore slopes of the river embankment alternatives presented below can be considered during detailed design:

- Excavation and replacement of the native clay with granular fill.
- Use of a shear key at the base of the excavation.

11 Estimated Frost Depth and Frost Protection

Based on OPSD 3090.100 Foundation Frost Penetration Depths for Northern Ontario; the estimated frost depth penetration within the expected embankment fill is 2.3 m. The embankment soils anticipated within the frost depth are considered to be of low frost susceptibility (MTO Pavement Design and Rehabilitation Manual).

12 Scour Protection

Where appropriate, foundation elements should be provided with sufficient scour protection in the event of elevated creek levels. Scour protection should be designed in accordance with Section 1.9.5 of the 2014 Canadian Highway Design Bridge Code (CHBDC).

13 Seismic Considerations

Seismic analysis for the bridge will not be required based on the following rationale as per the 2014 Canadian Highway Design Bridge Code (CHBDC). In accordance with Section 4.4.3.1 spectral ground acceleration data for the site was obtained from www.earthquakescanada.nrcan.gc.ca. In accordance with Section 4.4.4, Table 4.10 and assuming the bridge has a Seismic Importance Category of "Major-route and other bridges", the site is classified as Seismic Performance Category 1. As per Section 4.4.5.1, no seismic analyses are required for structures located in Seismic Performance Category 1.

14 Potential Construction Issues

Major construction difficulties are not foreseen at this site. Issues which may require consideration include:

- Cobbles and boulders are encountered at both boreholes.
- Safe excavation slopes.
- Dewatering efforts (if required) will increase in complexity with the depth of excavation required and river level at time of construction.
- Removal/cut off of existing piles.
- Avoidance of existing piles during driving of new piles.

15 Scope of Detailed Investigation and Future Considerations

The following items should be considered for detailed foundation design of the proposed bridge replacement.

- Borehole investigation should consist of:

- Two boreholes at each proposed abutment location, both extending to refusal, with a minimum of one borehole extending 3 m into the refusal material. Sampling of bedrock is required if encountered.
- One borehole for each approach embankment (within 20 m of the abutment, extending 3 m into competent material. No sampling of bedrock is required.
- One borehole at each proposed temporary bridge foundation location, extending into the refusal material. Sampling of bedrock is required if encountered.
- One borehole at each potential widening location (to facilitate temporary bridge), extending 3 m into refusal material. No sampling of bedrock is required.
- Two boreholes at each potential cofferdam location, if required for construction. Sampling of bedrock is required if encountered.
- Installation of piezometers for stabilized groundwater levels within the clay.
- Complex laboratory testing should be completed to further refine settlement analysis and slope stability modeling
 - One consolidation test, drained direct shear test, two undrained direct shear test on the medium plastic clay.
 - One consolidation test, drained direct shear test, two undrained direct shear test on the high plastic clay.

16 Limitations

Conclusions and recommendations presented in this report are based on the information determined at a limited number of test hole locations. Subsurface and groundwater conditions between and beyond these locations may differ from those encountered. Conditions may become apparent during construction that were not detected and could not be anticipated at the time of the site investigation.

The comments given in this report on potential construction problems and possible methods of construction are intended only for the guidance of the designer.

Groundwater levels indicated are based on the information described within the report. The presence of all conditions that could affect the type and scope of dewatering procedures which may be considered cannot readily be determined from boreholes. These include local and seasonal fluctuations of the groundwater level, changes in soil conditions between test locations, thin and/or discontinuous layers of highly permeable soils, etc.

The information contained within this report in no way reflects any environmental aspect of the site or soil.

17 Closure

We trust the above addresses your project requirements at this time. Should you have any questions or comments, please do not hesitate to contact us at your convenience.

Yours truly,
For TBT ENGINEERING

Steven Seller, P.Eng
Project Engineer

Wayne Hurley, P.Eng.
Principal Contact for MTO Foundations

APPENDIX A

Borehole Logs

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_f FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS \bar{N} .

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J 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

S S SPLIT SPOON	T P THINWALL PISTON
W S WASH SAMPLE	O S OSTERBERG SAMPLE
S T SLOTTED TUBE SAMPLE	R C ROCK CORE
B S BLOCK SAMPLE	P H T W ADVANCED HYDRAULICALLY
C S CHUNK SAMPLE	P M T W ADVANCED MANUALLY
T W THINWALL OPEN	F S FOIL SAMPLE

STRESS AND STRAIN

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

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	l	COMPRESSION INDEX
C_s	l	SWELLING INDEX
C_a	l	RATE OF SECONDARY CONSOLIDATION
C_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	l	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
T_s	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
T_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_r	l	SENSITIVITY = $\frac{c_v}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

RECORD OF BOREHOLE No BH-01

1 OF 3

METRIC

W.P. 6855-14-00 LOCATION N:5391184; E:229168 MTM Zone:16 ORIGINATED BY A.F.
 DIST NWR HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.C.
 DATUM Geodetic DATE 2017.06.13 - 2017.06.13 LATITUDE 48.6545778 LONGITUDE -94.0266457 CHECKED BY S.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)	
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL							× LAB VANE
								20	40	60							80
335.6															GR SA SI CL		
335.0	ASPHALT - 70 mm														Water level @		
335.3	CONCRETE - 260 mm														4.9 m two hours		
335.2	FILL - SAND & GRAVEL		1	AS											after completion		
0.5	FILL - SAND - some gravel, occasional cobbles, brown																
334.5			2	SS	5		335										
1.1	FILL - CLAY - some sand, grey/brown, firm to stiff																
			3	SS	11		334								0 20 37 43		
333.4																	
2.2	CLAY - some sand, trace gravel, grey, firm to very stiff		4	SS	8		333								1 13 44 42		
			5	SS	9												
							332										
			6	TW			331										
			7	TW			330										
	- sand seam, 100 mm thick		8	SS	6		329										
			9	SS	6		328								1 19 34 46		
			10	TW			327										
							326										

Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

PP=Pocket Penetrometer (Kg/cm²)

-DRAFT-

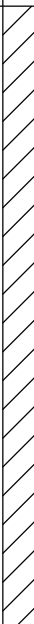
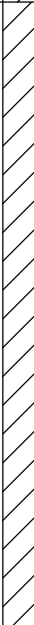
ONTARIO MTO MOD DRAFT 17-153 MTO STURGEON RIVER.GPJ ONTARIO MTO.GDT 3/8/17

RECORD OF BOREHOLE No BH-01

2 OF 3

METRIC

W.P. 6855-14-00 LOCATION N:5391184; E:229168 MTM Zone:16 ORIGINATED BY A.F.
 DIST NWR HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.C.
 DATUM Geodetic DATE 2017.06.13 - 2017.06.13 LATITUDE 48.6545778 LONGITUDE -94.0266457 CHECKED BY S.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)			
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
								20 40 60 80 100	20 40 60										
	CLAY - some sand, trace gravel, grey, firm to very stiff <i>(continued)</i>		11	SS	8		325												
								324											
				12	SS	6		323											
	----- - imbedded sand & gravel						322												
			13	SS	6		321												
320.8	CLAY - grey, firm to soft		14	TW			320												
14.8																			
			15	SS	4		319												
			16	SS	4		317												
	----- - layered						316												
			17	SS	5		315												

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Continued Next Page

+³, ×³: Numbers refer to
Sensitivity

○ 3% STRAIN AT FAILURE

PP=Pocket Penetrometer (Kg/cm²)

RECORD OF BOREHOLE No BH-01

3 OF 3

METRIC

W.P. 6855-14-00 LOCATION N:5391184; E:229168 MTM Zone:16 ORIGINATED BY A.F.
 DIST NWR HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.C.
 DATUM Geodetic DATE 2017.06.13 - 2017.06.13 LATITUDE 48.6545778 LONGITUDE -94.0266457 CHECKED BY S.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
					20 40 60 80 100					20 40 60							
314.4	CLAY - grey, firm to soft <i>(continued)</i>						314										0 5 (95)
21.2	SILT - layered, trace to some gravel, trace sand to Sandy, trace clay, grey, very loose to loose		18	SS	1												Non-Plastic
							313										
			19	SS	9												11 32 (57)
	----- - occasional cobbles						312										
311.6																	
24.0	SAND - Gravely, Silty, occasional cobbles, compact to dense		20	SS	25		311										
							310										
			21	SS	22												21 57 (22)
							309										
			22	SS	33		308										
			23	GRAB			307										
			24	SS	39		306										
	- DCPT																
305.6																	
30.0	End of Borehole @ 30.0 m.																

-DRAFT-

ONTARIO MTO MOD DRAFT 17-153 MTO STURGEON RIVER.GPJ ONTARIO MTO.GDT 3/8/17

RECORD OF BOREHOLE No BH-02

1 OF 3

METRIC

W.P. 6855-14-00 LOCATION N:5391218; E:229113 MTM Zone:16 ORIGINATED BY A.F.
 DIST NWR HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.C.
 DATUM Geodetic DATE 2017.06.23 - 2017.06.23 LATITUDE 48.654877 LONGITUDE -94.0274055 CHECKED BY S.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								
								○ UNCONFINED + FIELD VANE								
								● QUICK TRIAXIAL × LAB VANE								
							WATER CONTENT (%)				kN/m ³					
335.7							20	40	60	80	100					GR SA SI CL
335.6	ASPHALT - 90 mm		1	AS												Water level @
335.4	FILL - SAND & GRAVEL		2	AS												5.2 m two hours
0.3	FILL - SAND - Silty, some gravel, brown															after completion
334.8																
0.9	CLAY - Sandy to some sand, trace gravel, firm to very stiff, grey		3	AS	7											
			4	SS	8											1 27 41 31
			5	SS	6											
			6	TW												

Continued Next Page

+³, ×³: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PP=Pocket Penetrometer (Kg/cm²)




-DRAFT-

RECORD OF BOREHOLE No BH-02

2 OF 3

METRIC

W.P. 6855-14-00 LOCATION N:5391218; E:229113 MTM Zone:16 ORIGINATED BY A.F.
 DIST NWR HWY 11 BOREHOLE TYPE Hollow Stem Auger COMPILED BY S.C.
 DATUM Geodetic DATE 2017.06.23 - 2017.06.23 LATITUDE 48.654877 LONGITUDE -94.0274055 CHECKED BY S.S.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE									
	CLAY - Sandy to some sand, trace gravel, firm to very stiff, grey <i>(continued)</i>		11	TW														
				12	SS	7												
				13	SS	7												
				14	TW													
319.6 16.1	CLAY - grey, firm																	
				15	SS	8												
				16	TW													
315.2 20.5	CLAY - trace sand, grey, very soft																	

-DRAFT-

Continued Next Page

+ 3, × 3: Numbers refer to Sensitivity

○ 3% STRAIN AT FAILURE

PP=Pocket Penetrometer (Kg/cm²)

ONTARIO MTO MOD DRAFT 17-153 MTO STURGEON RIVER.GPJ ONTARIO MTO.GDT 3/8/17

METRIC

-DRAFT-

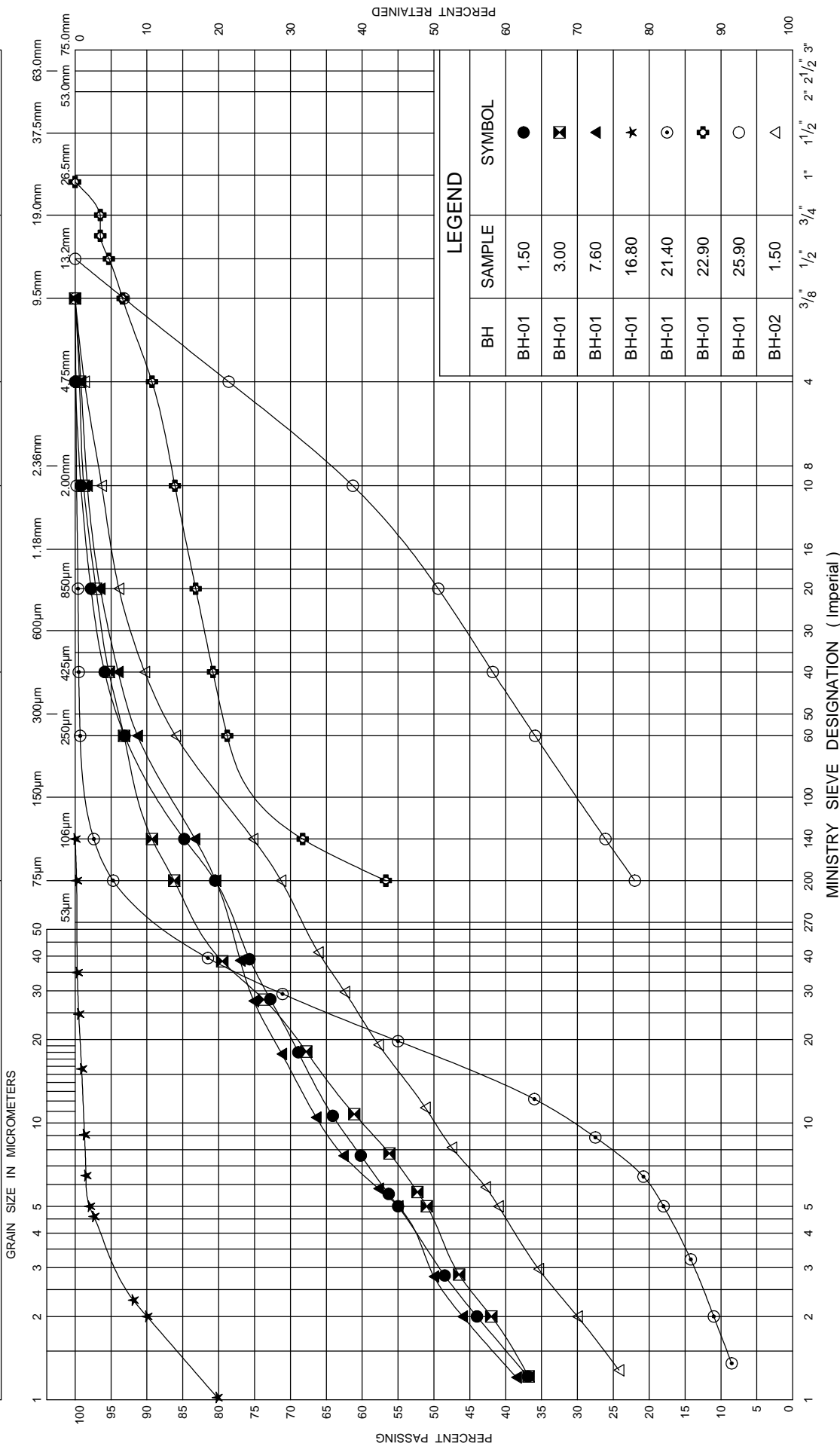
+³, ×³: Numbers refer to Sensitivity ○³% STRAIN AT FAILURE PP=Pocket Penetrometer (Kg/cm²)

APPENDIX B

Laboratory Test Data

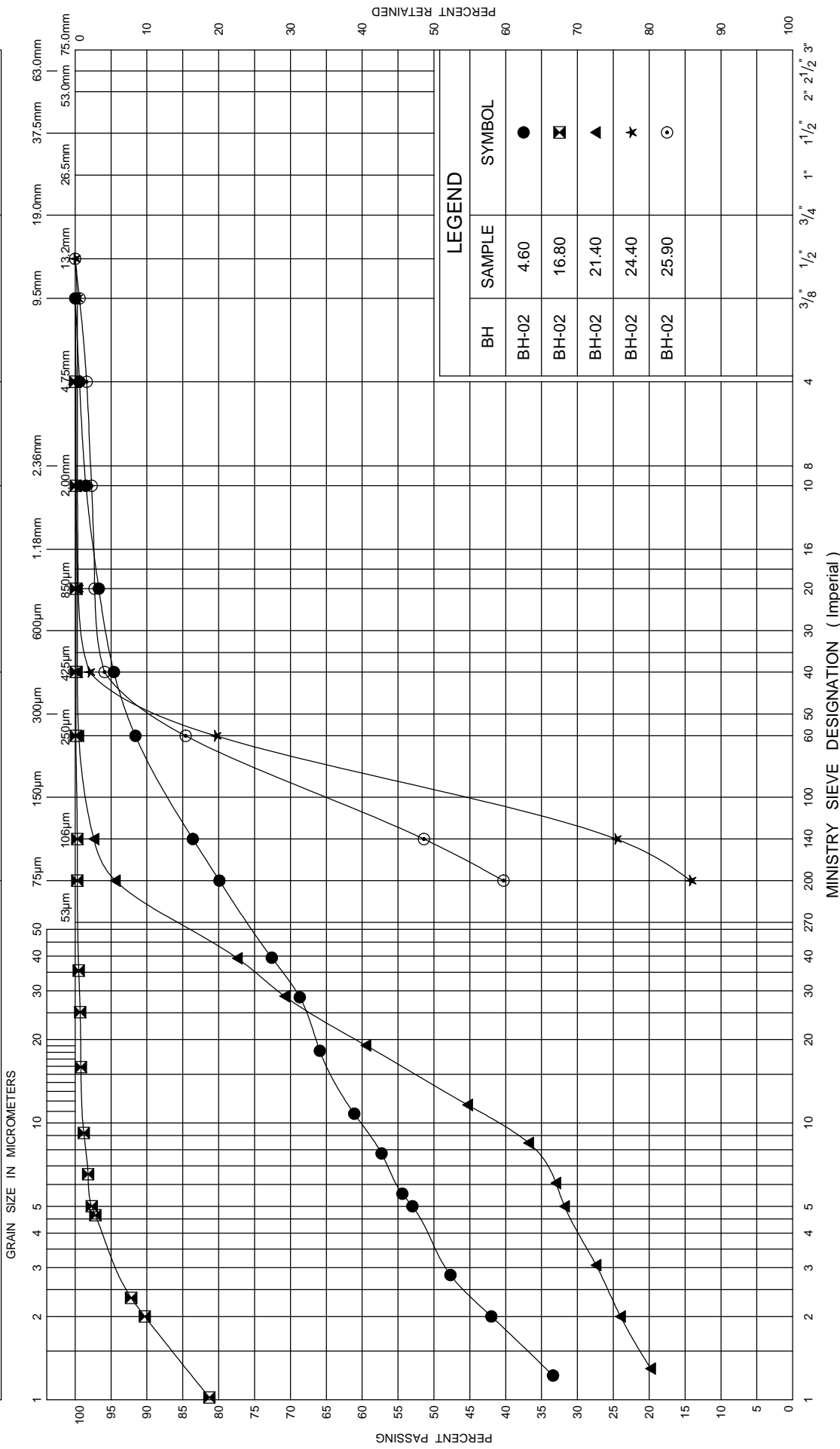
UNIFIED SOIL CLASSIFICATION SYSTEM

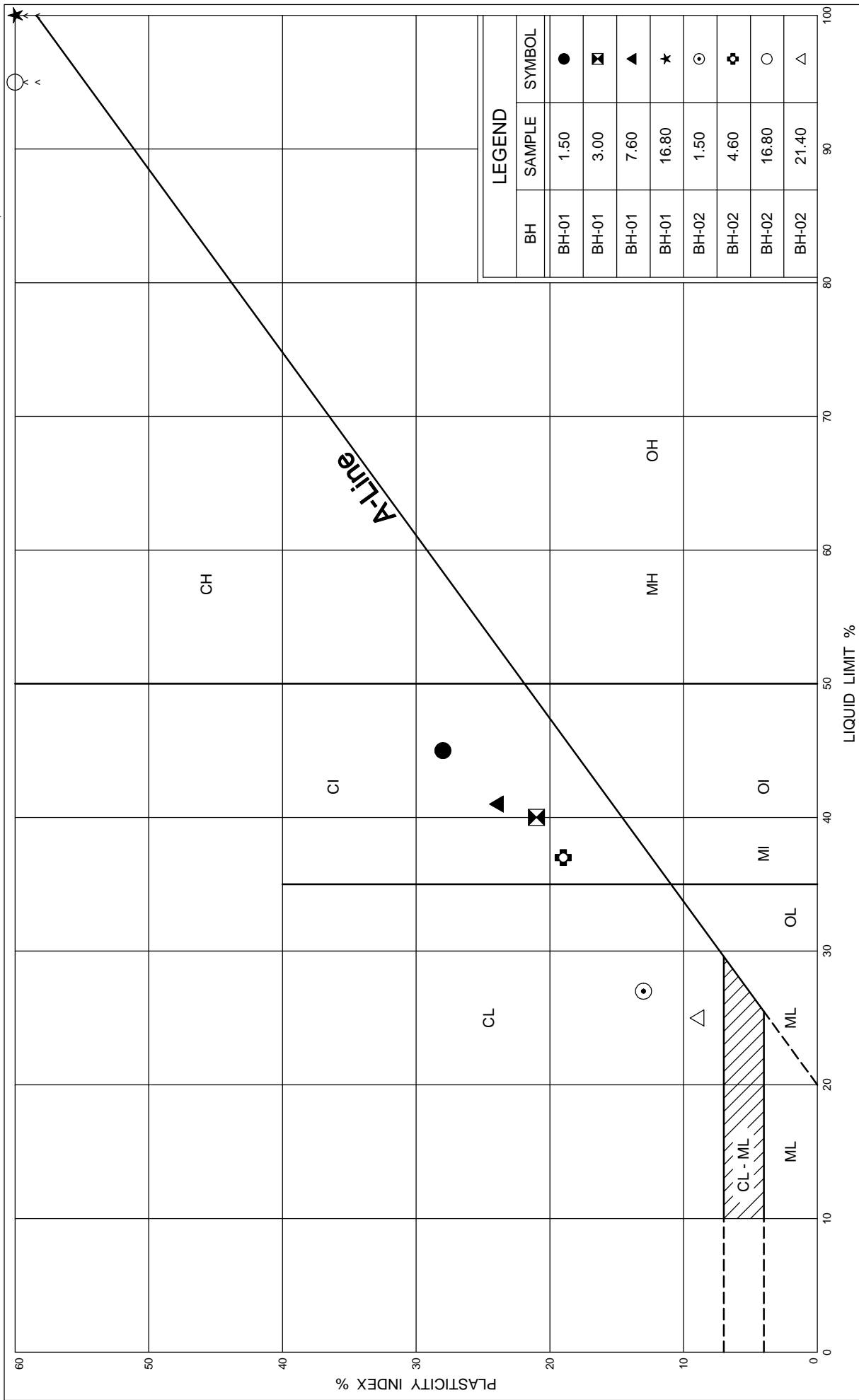
CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse



UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse





PLASTICITY CHART

FIG No

W P 6855-14-00

Sturgeon River Bridge

APPENDIX C
Borehole Locations and Soil Strata Drawing

-DRAFT-

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

GEOCREs No. XXX-XXX
CONT No. 2017-xx
GWP No. 6855-14-00



STURGEON RIVER BRIDGE
HIGHWAY 11
STA 18+440 to 18+520
BOREHOLE LOCATIONS AND SOIL STRATA

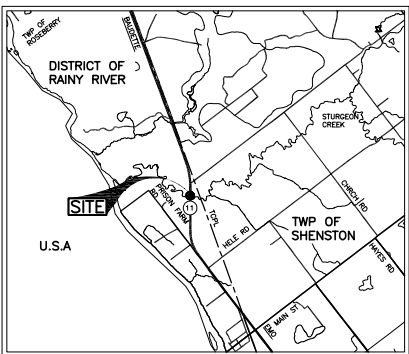
SHEET



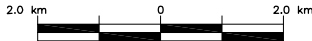
Ministry of Transportation
Northwestern Region



TBT ENGINEERING
CONSULTING GROUP



KEY PLAN



SOIL STRATA SYMBOLS			
	FILL		SILT
	CLAY		SAND
	SAND & SILT		

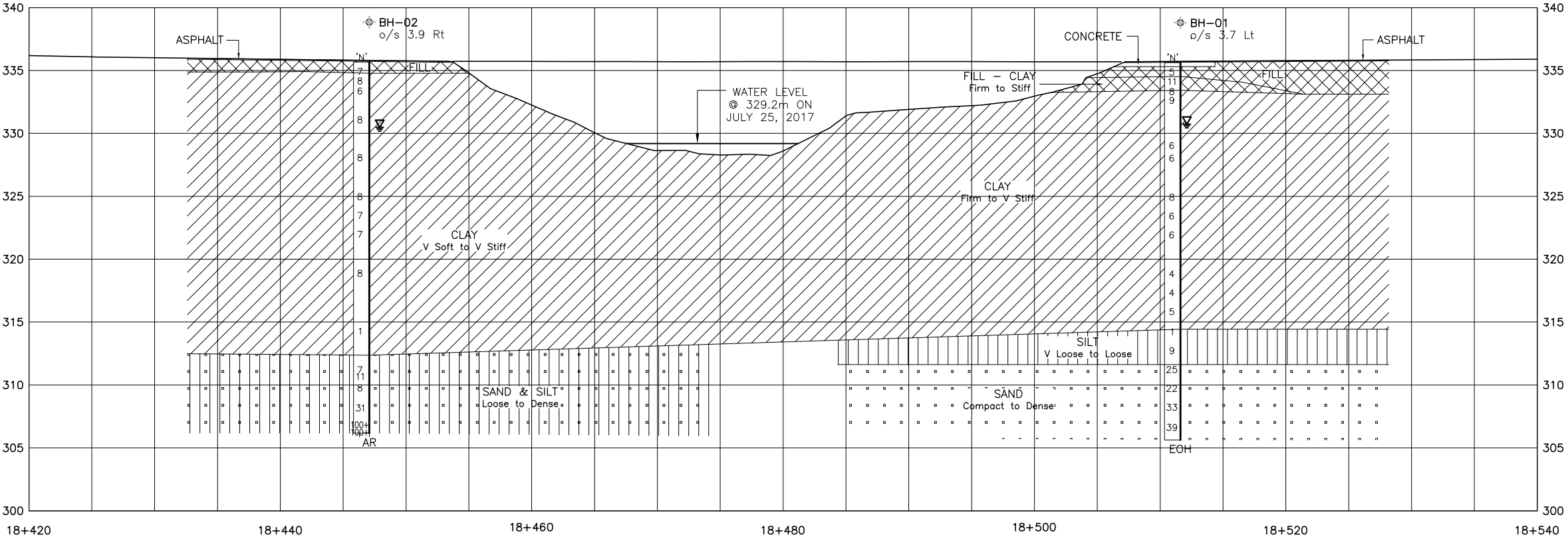
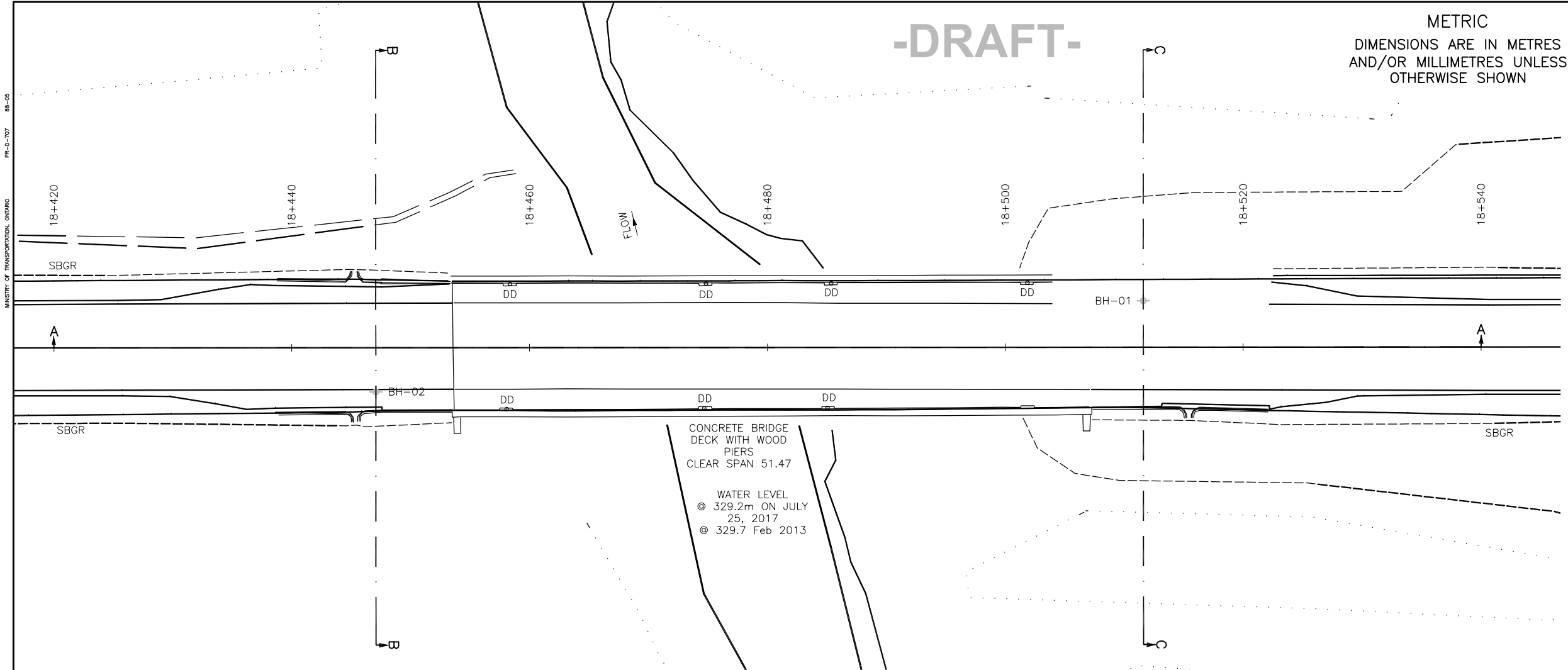
LEGEND			
	Borehole		
	Std Pen Test (Blows/0.3m)		
	Water Level		
	Water Level 2hrs after Completion		
	End of Hole		
	Auger Refusal		
	Steel Beam Guide Rail		
	Deck Drain		
No	ELEVATION	CO-ORDINATES (MTM)	
		NORTH	EAST
BH-01	335.6	16 5 391 184	229 168
BH-02	335.7	16 5 391 218	229 113

-NOTE-

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

REVISIONS

DD/MM/YY	BY	REVISION	DESCRIPTION
DESIGN	CHK	CODE	XXXXX-XX
DRAWN	TB	CHK	SS
		SITE	45-105
		DATE	20170802
		DWG	1



SECTION A - A



DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING

-DRAFT-

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

GEOCREST No. XXX-XXX

CONT No. 2017-xx


GWP No. 6855-14-00

STURGEON RIVER BRIDGE


HIGHWAY 11

STA 18+440 to 18+520

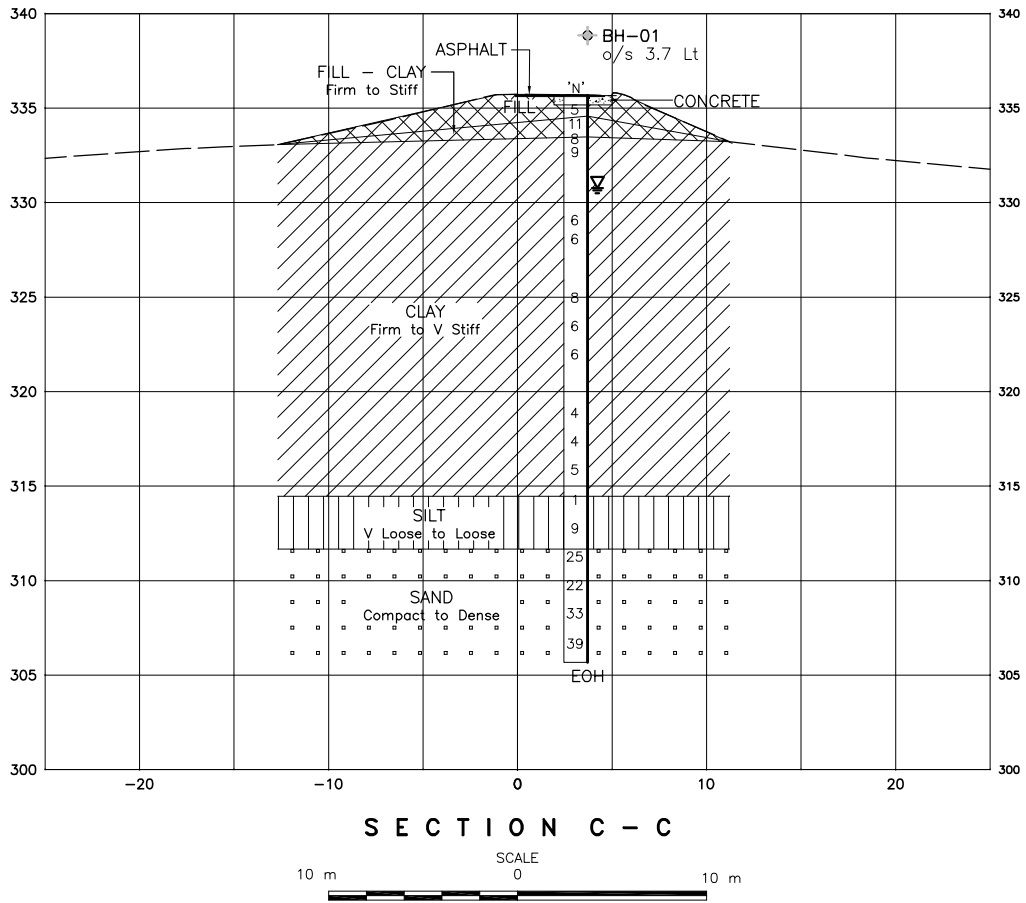
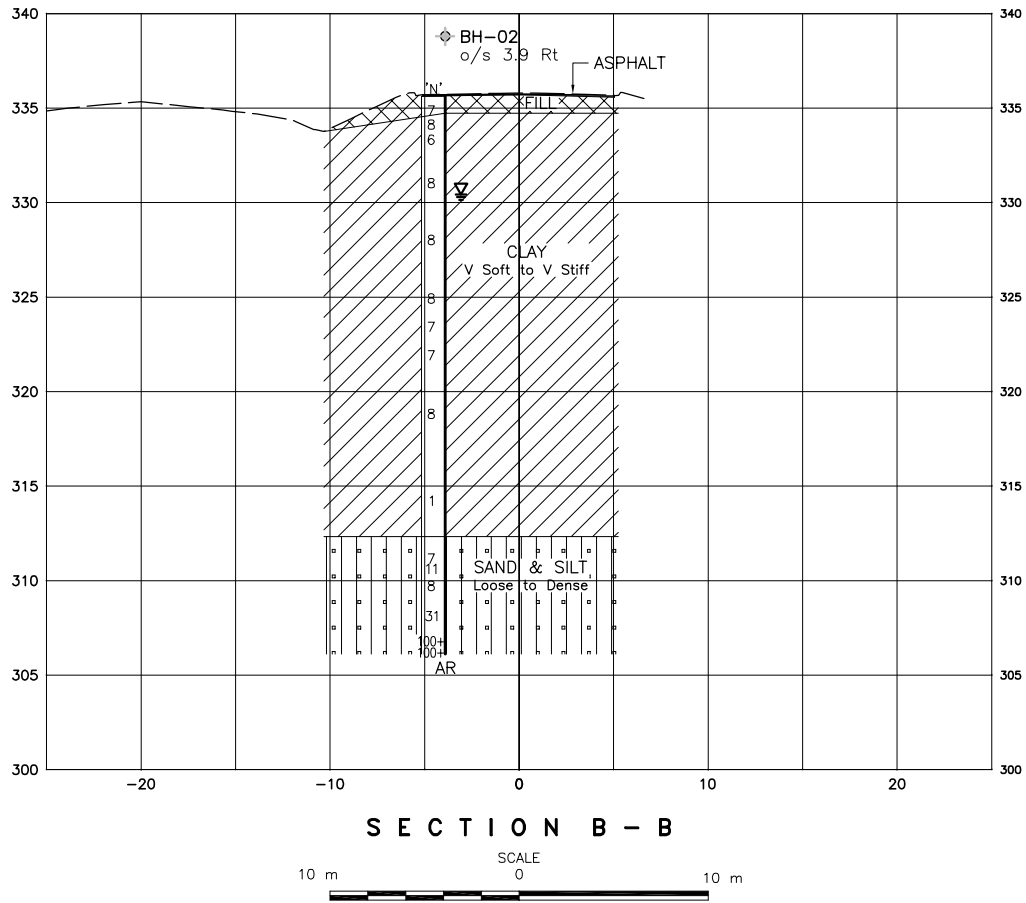
BOREHOLE LOCATIONS AND SOIL STRATA






 Ontario




Ministry of Transportation
Northwestern Region

 TBT ENGINEERING

CONSULTING GROUP



SOIL STRATA SYMBOLS	
	FILL
	CLAY
	SILT
	SAND
	SAND & SILT

LEGEND			
	Borehole		
'N'	Std Pen Test (Blows/0.3m)		
	Water Level		
	Water Level 2hrs after Completion		
EOH	End of Hole		
AR	Auger Refusal		
SBGR	Steel Beam Guide Rail		
DD	Deck Drain		
No	ELEVATION	CO-ORDINATES (MTM)	
		NORTH	EAST
BH-01	335.6	16 5 391 184	229 168
BH-02	335.7	16 5 391 218	229 113

-NOTE-
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS		REVISION		DESCRIPTION	
DD/MM/YY	BY	CHK	SS	CODE	DATE
DESIGN	CHK	CODE	SS	LOAD XX-XX-XX	20170802
DRAWN	TB	CHK	SS	SITE	DWG

DRAWING NOT TO BE SCALED
100mm ON ORIGINAL DRAWING