

**FOUNDATION INVESTIGATION AND DESIGN REPORTS  
PROPOSED CEDAR CREEK CULVERT REPLACEMENT  
HIGHWAY 631, NORTH OF HORNPAYNE, ONTARIO  
WP 5270-08-01 SITE NO. 38N-014/C  
G.W.P. 5270-08-00  
MTO GEOCRETS NO. 42F-24**

Prepared for:

**MCINTOSH PERRY CONSULTING ENGINEERS**

By:

**SPL CONSULTANTS LIMITED**

Project: 750-1001 (Cedar Creek)  
January 2013



146 Colonnade Road  
Ottawa, Ontario K2E 7Y1  
Tel: 613.228.0065 Fax: 613.228.0045

**PART A**  
**FOUNDATION INVESTIGATION REPORT**  
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## Table of Contents

### PART A: FOUNDATION INVESTIGATION REPORT

<b>1. INTRODUCTION .....</b>	<b>1</b>
<b>2. SITE DESCRIPTION.....</b>	<b>1</b>
<b>3. INVESTIGATION PROCEDURES .....</b>	<b>1</b>
3.1 Desktop Study .....	1
3.2 Field Investigation .....	2
3.3 Laboratory Testing .....	2
<b>4. SUBSURFACE CONDITIONS.....</b>	<b>2</b>
4.1 Soil Conditions .....	3
4.1.1 Asphalt .....	3
4.1.2 Granular Fill.....	3
4.1.3 Peat .....	3
4.1.4 Silty Sand and Sandy Silt Till .....	3
4.1.5 Auger Refusal .....	4
4.2 Groundwater Conditions .....	5
4.3 Summary .....	5
<b>5. CLOSURE.....</b>	<b>5</b>

## Drawings

## No.

Site Plan	1
Borehole Locations & Soil Strata	2
Grain Size Distribution	3

## Appendices

Appendix A: Borehole Logs (Record of Borehole Sheets)
Appendix B: Chemical Test Results
Appendix C: Explanation of Terms used in Report

## **1. INTRODUCTION**

SPL Consultants Limited (SPL) was retained by McIntosh Perry Consulting Engineers to conduct a foundation investigation as part of the proposed culvert replacement at Cedar Creek on Highway 631 approximately 20.5 km north of Hornpayne, Ontario.

The terms of reference (TOR) for this investigation are outlined in the Request for Quotation (RFQ) issued by the Ministry of Transportation (MTO) under Agreement No. 5010-E-0001 dated April, 2010 and SPL's subsequent proposal No. P10.06.018 dated June, 2010.

The purpose of the foundation investigation was to obtain subsurface information at the site by means of exploratory boreholes. This report presents the findings of the foundation investigation carried out at the site, as well as general comments and recommendations for the design and construction of the proposed culvert replacement.

As part of this project a geotechnical (pavement) investigation was also carried out at the site concurrent with the foundation investigation. The results of the pavement investigation are presented under separate cover.

## **2. SITE DESCRIPTION**

The site is located on Highway 631 approximately 20.5 km north of Hornpayne, Ontario (see Drawing No. 1). The existing culvert is a 1.6 m wide x 1.4 m high, triple cell, timber culvert with a fill cover of approximately 1.5 m.

The elevation of the natural ground in the general vicinity of the culvert crossing is approximately 320.5 m. The elevation of the highway (top of pavement) at the crossing is approximately 323 m (the embankment is approximately 2.5 m high at the crossing).

## **3. INVESTIGATION PROCEDURES**

The foundation investigation was carried out in June, 2011. The scope of work for this assignment included a desk study, field investigations, laboratory testing, analysis and preparation of this report.

### **3.1 Desktop Study**

Surficial geology maps of the area indicate the site is underlain by silt and sand glacial till as well as glaciolacustrine silt and clay deposits with minor sand.

Bedrock geology maps of the general area indicate the bedrock to be granitic rocks of Muscovite-Biotite and Granodiorite-Tonalite. Metasedimentary rocks of Paragneiss and Migmatites also exist in the area.

### **3.2 Field Investigation**

Field investigations were carried out on June 7 to June 9, 2011 and included drilling a total of 6 boreholes at the crossing location (BH11-1 through BH11-6). As mentioned previously, additional shallow boreholes were advanced at the same time for the geotechnical (pavement) portion of the work; the results of these boreholes are submitted with the geotechnical (pavement) investigation report under separate cover.

The boreholes were advanced using a truck-mounted drill rig supplied and operated by George Downing Estate Drilling Limited of Hawkesbury, ON. The boreholes were drilled using hollow-stem auger drilling to depths ranging from 7.1 m to 10.2 m. During drilling, sampling and in-situ testing [including Standard Penetration Testing (SPT) and Dynamic Cone Penetration Testing (DCPT)] were carried out.

Standpipe piezometers were installed in Boreholes BH11-1, BH11-4, BH11-5 and BH11-6 to allow for measurement of groundwater levels at the site. All boreholes were backfilled with Bentonite and soil cuttings and were sealed at the ground surface. All boreholes were abandoned in accordance with Ontario Regulation 903 at the end of the field program.

Borehole locations are shown in Drawing No. 2. Borehole logs are included in Appendix A of this report.

### **3.3 Laboratory Testing**

During drilling and in-situ testing, disturbed soil samples were obtained for further examination and classification. A laboratory testing program, including determination of natural water content, grain size distribution (sieve and hydrometer) and chemical analyses, was carried out on selected representative soil samples.

The results of natural water content tests are included on the relevant borehole logs in Appendix A. The results of determination of grain size distribution are summarized on the individual borehole logs and also presented in Drawing No. 3.

Chemical testing to determine sulphate content, chloride content, pH, soil resistivity and electrical conductivity was also carried out on selected soil samples obtained during drilling. The results of these tests are included in Appendix B.

## **4. SUBSURFACE CONDITIONS**

The subsurface conditions at the site are discussed in the following sections. Detailed descriptions of the soil and groundwater conditions encountered at each of the borehole locations are included in the individual borehole logs in Appendix A.

## **4.1 Soil Conditions**

### **4.1.1 Asphalt**

All boreholes drilled as part of this investigation were drilled on the shoulder of the existing highway. All boreholes encountered a layer of asphalt ranging from 40 mm to 60 mm thick. Additional investigations of the asphalt on the highway were carried out during the field program, the results of which are included in the geotechnical (pavement) report submitted separately.

### **4.1.2 Granular Fill**

The asphalt is underlain by granular fill which forms the pavement structure of the highway as well as the existing highway embankment.

The granular fill is predominantly sand and gravel with occasional cobbles and boulders. Fragments of wood and asphalt were encountered near the culvert (in Borehole BH11-1 and BH11-3). A layer of clayey silt fill was also encountered at the base of the embankment in Borehole BH11-5 some 40 m to the south of the crossing.

The consistency of the fill material (as interpreted based on SPT “N” values) typically ranged from loose to compact.

The fill material extends to depth of 1.5 m to 2.3 m below the existing road surface in the boreholes drilled as part of this investigation (2.3 m at the culvert location). This corresponds to elevation of approximately 320.6 m at the crossing location.

### **4.1.3 Peat**

A layer of peat was encountered in all boreholes beneath the road fill material, suggesting the area was not stripped prior to placement of the original fill embankment (and possibly the existing culvert structure, although this cannot be confirmed until the existing structure is removed). The peat layer varies in thickness from 0.1 m to 0.9 m along the length of investigation on the road, and from 0.5 m to 0.9 m in the immediate vicinity of the culvert (in Boreholes BH11-1 through BH11-4).

### **4.1.4 Silty Sand and Sandy Silt Till**

Underlying the peat layer, the native mineral soils at the crossing location are a deposit of glacial till, which includes primarily a heterogeneous mixture of silty sand and sandy silt with trace to some gravel and trace clay. Cobbles and boulders were also encountered during drilling and should be anticipated during construction. The till deposit extended to the depth of drilling in all of the boreholes advanced at the site.

Grain size curves for several samples of the native soils are presented in Drawing No. 3, and are summarized in Table 1 below. It should be noted that these samples were obtained through SPT testing, which does not recover coarse gravel, cobble and boulder sized particles. Because of this it is expected

that the grain size distributions shown on Drawing No. 3 and in Table 1 may be finer than portions of the till deposit in the field.

**Table 1 – Results of Grain Size Analyses for Silty Sand and Sandy Silt Till<sup>1</sup>**

Borehole No.	Sample No.	Grain Size Distribution			
		% Gravel	% Sand	% Silt	% Clay
BH11-1	SS-6	10	34	50	6
BH11-2	SS-5	8	58	32	2
BH11-2	SS-6	5	30	56	9
BH11-3	SS-5	29	43	23	5
BH11-3	SS-7	3	50	40	7
BH11-4	SS-8	18	48	28	6
BH11-5	SS-5	0	4	80	16
BH11-5	SS-7	7	32	56	5
BH11-6	SS-6	7	34	51	8

The density of the till (as interpreted based on SPT “N” and DCPT resistance values) was found to range from compact to very dense, and generally increases with depth. SPT “N” values and DCPT resistance values are presented on the borehole logs included in Appendix A as well as on the Soil Strata included in Drawing No. 2.

In granular materials (particularly those which contain cobbles and boulders) SPT “N” values and DCPT values can be influenced by the size of the soil particles. Very high values often reflect the presence of cobbles and boulders, rather than a very high density of the soil matrix.

#### 4.1.5 Auger Refusal

Auger refusal was encountered in all of the boreholes drilled at the site. In the four boreholes closest to the proposed culvert the depth to refusal ranged from 7.1 m to 10.2 m below the road surface (see Table 2 below).

<sup>1</sup> Includes a layer of silty soil present above the till layer at BH11-5 (see borehole log)



## 4.2 Groundwater Conditions

Groundwater was encountered during drilling in five of the six boreholes advanced during the field investigation. Only BH11-4 remained dry during the course of drilling. Standpipe piezometers were installed in Boreholes BH11-1, BH11-4, BH11-5 and BH11-6. Stabilized water levels were read in the boreholes two days after piezometer installation. Measured water levels ranged from 320.9 m to 321.2 m elevation which is approximately coincident with the level of the creek at the time of the investigation (which would be expected due to the granular nature of the soil in the area).

It should be noted that the groundwater levels can vary and are subject to seasonal fluctuations as well as fluctuations in response to major weather events, and in particular for this site, in response to changes in the level of the creek. If construction is carried out at a time when the creek level is higher than the level in June, 2011, a corresponding increase in groundwater levels should be anticipated.

## 4.3 Summary

A summary of the soil and groundwater conditions encountered at the Cedar Creek crossing location is presented in Table 2 below.

**Table 2 – Simplified Stratigraphy and Groundwater Elevations**

Borehole No.	Ground Surface El.	Simplified Stratigraphy (Depth)			Measured Groundwater Elevation	Notes
		Granular Fill	Peat	Silty Sand and Sandy Silt Till		
BH11-1	322.9	0.0 – 2.3 m	2.3 – 3.2 m	3.2 – 8.6 m	321.0 m	Auger refusal at 8.6 m
BH11-2	322.9	0.0 – 2.3 m	2.3 – 3.1 m	3.1 – 8.4 m	--	Auger refusal at 8.4 m
BH11-3	322.9	0.0 – 2.3 m	2.3 – 2.8 m	2.8 – 7.1 m	--	Auger refusal at 7.1 m
BH11-4	322.9	0.0 – 2.3 m	2.3 – 3.1 m	3.1 – 10.2 m	320.9 m	Auger refusal at 10.2 m
BH11-5	323.1	0.0 – 2.3 m	2.3 – 2.4 m	2.4 – 8.7 m	321.2 m	Auger refusal at 8.7 m
BH11-6	322.9	0.0 – 1.5 m	1.5 – 2.3 m	2.3 – 8.7 m	321.2 m	Auger refusal at 8.7 m

## 5. CLOSURE

The field investigations were supervised by Mr. Naeem Ehsan, P.Eng. This report was prepared by Mr. Chris Hendry, P.Eng., Mr. Fanyu Zhu, P.Eng., SPL's designated MTO contact and Mr. Shaheen Ahmad, P.Eng., SPL's project quality control auditor, provided independent review and quality control of the technical aspects of this report.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

**SPL CONSULTANTS LIMITED**

Chris Hendry, M.Eng., P.Eng.

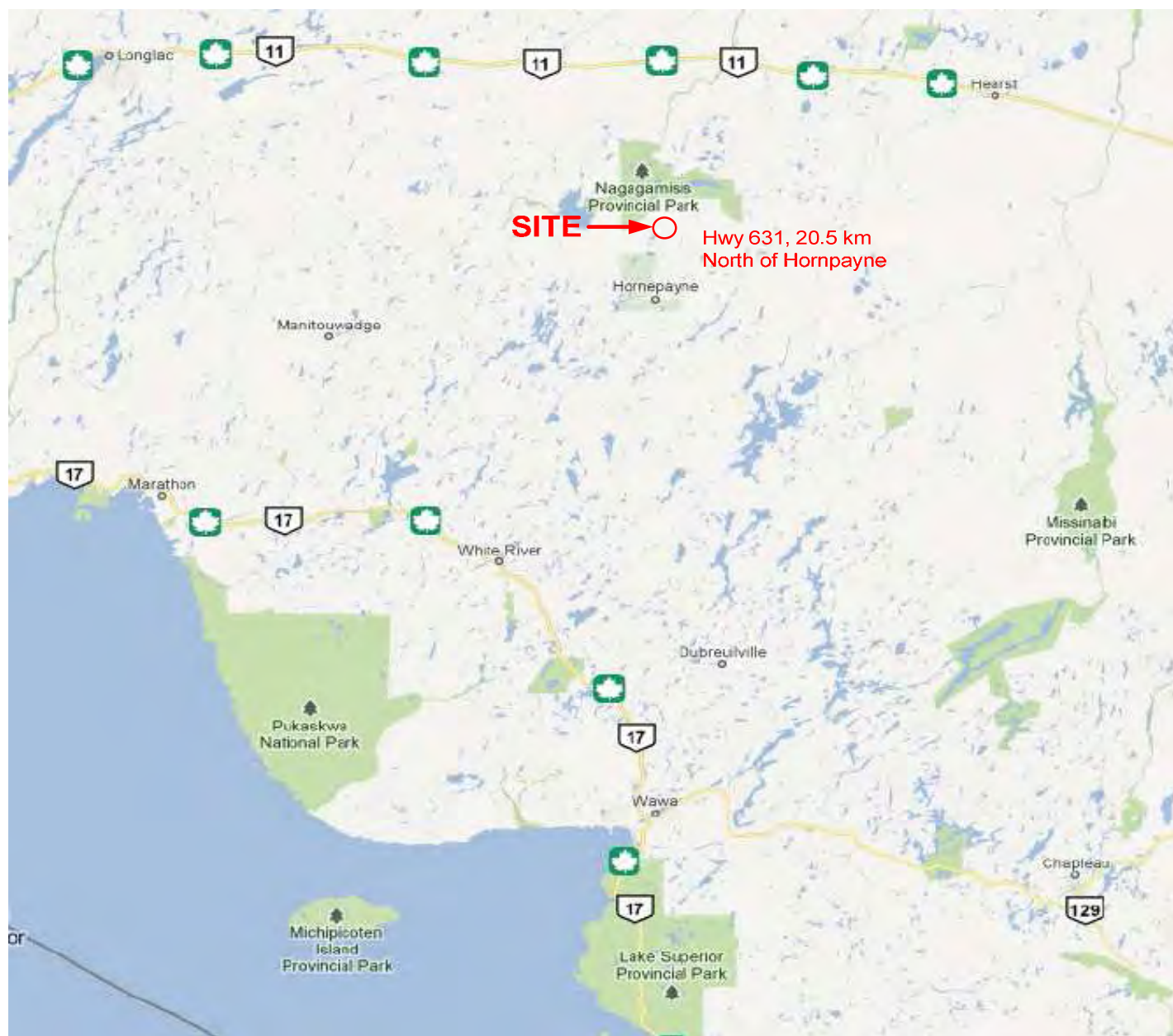



Fanyu Zhu, Ph.D., P.Eng.



Shaheen Ahmad, M.A.Sc., P.Eng.

# Drawings



Client: McIntosh Perry Consulting Engineers		Title: SITE PLAN	
Project#:	750-1001	DWG #:	1
Drawn:	NT	Approved:	CH
Date:	AUG 31-2011	Scale:	N. T. S.
Size:	Letter	Rev:	0
		 <b>SPL Consultants Limited</b> Geotechnical Environmental Materials Hydrogeology	

Project: Geotechnical Investigation - Cedar Creek Culvert - Sault Ste. Marie, Ontario



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

CONT No  
WP No 5270-08-01



CEDAR CREEK CULVERT  
HIGHWAY 631  
BORE HOLE LOCATIONS & SOIL STRATA

SHEET  
13

**SPL Consultants Limited**  
Geotechnical • Environmental • Materials • Hydrogeology



#### LEGEND

- ◆ Bore Hole
- ◆ Bore Hole & 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 July 2011
- ↓ WL in Piezometer
- ⊥ Piezometer

No	ELEVATION	STATION	OFFSET
BH11-1	322.9	17+949.9	3.7m W
BH11-2	322.9	17+962.8	3.8m W
BH11-3	322.9	17+949.6	3.0m E
BH11-4	322.9	17+962.5	3.0m E
BH11-5	323.1	17+916.0	3.0m E
BH11-6	322.9	17+996.1	3.0m E

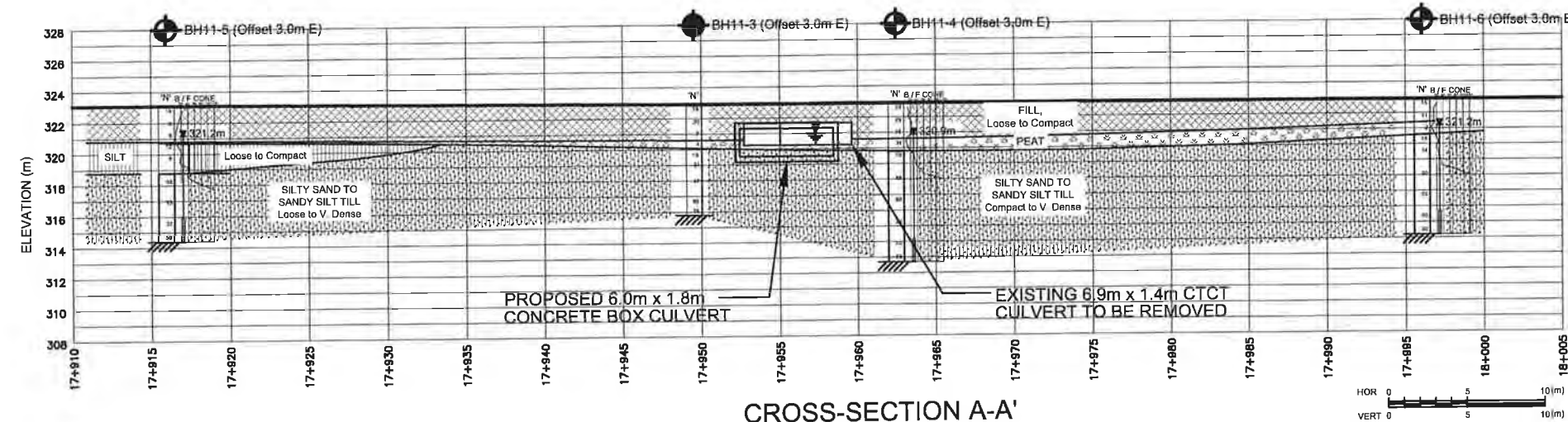
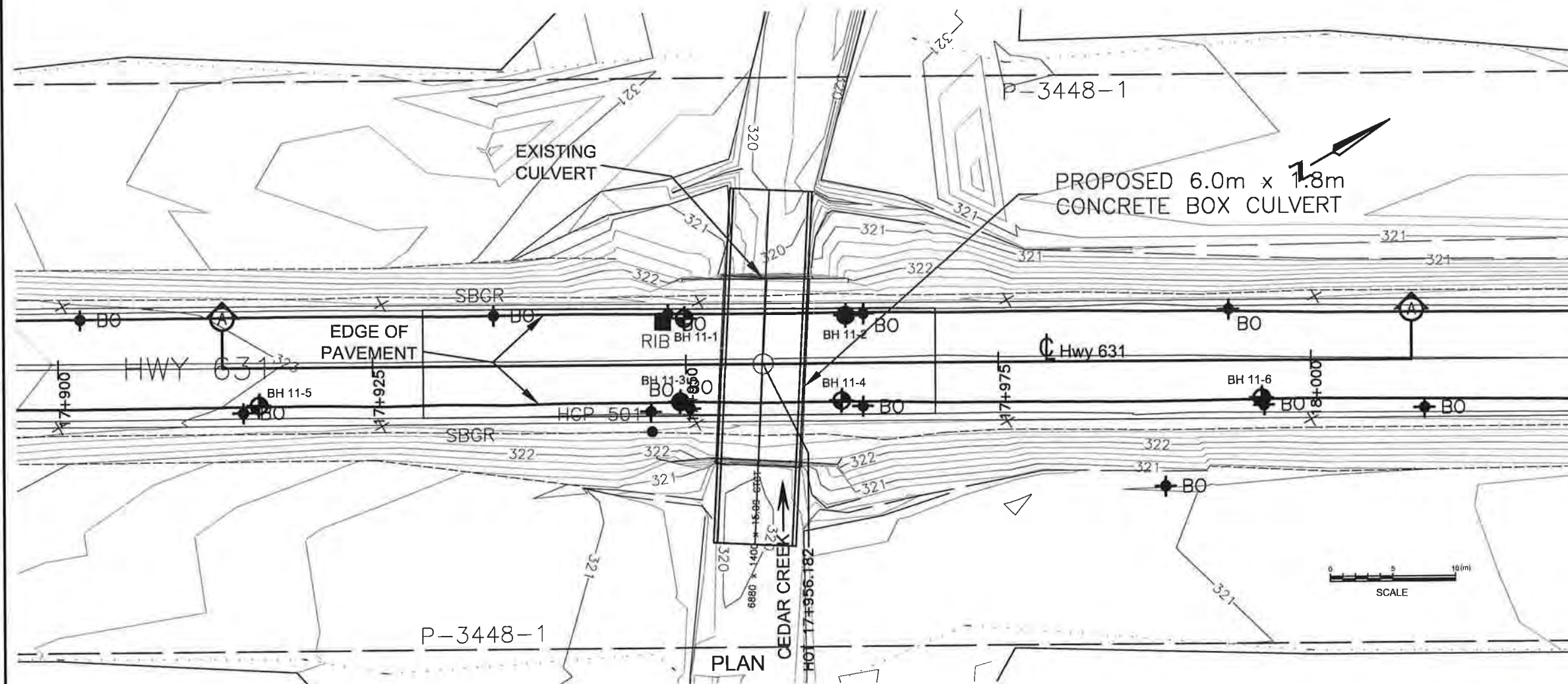
#### NOTES

The boundaries between soil strata have been established only at Bore Hole locations. Between Bore holes the boundaries are assumed from geological evidence.

REVISIONS	DATE	BY	DESCRIPTION
Nov16/13	TJC		Final Revision
Feb16/13	TJC		Revision 1

GEORES No 42F-24

HWY No 101	CHECKED CH	DATE Nov16, 2012	DIST Algoma
SUBM'D CH	CHECKED CH	APPROVED FZ	SITE 38N-014/C
DRAWN TJC	CHECKED CH		DWG 2



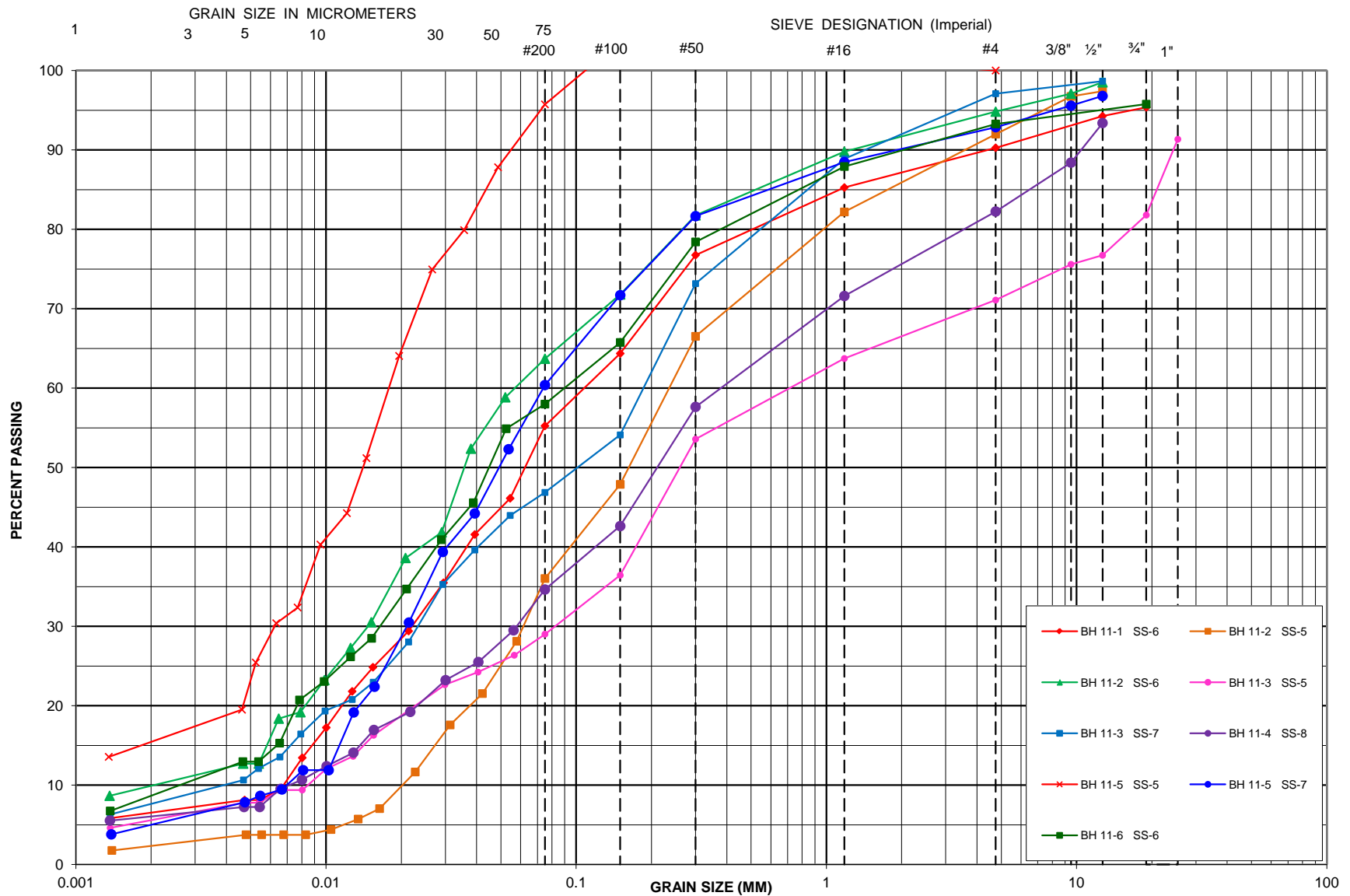
#### SOIL STRATA SYMBOLS

- GRANULAR FILL
- PEAT
- AUGER REFUSAL
- SILTY SAND TO SANDY SILT TILL
- SILT



# UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY AND SILT	SAND			GRAVEL	
	Fine	Medium	Coarse	Fine	Coarse



# Appendix A

## Borehole Logs (Record of Borehole Sheets)

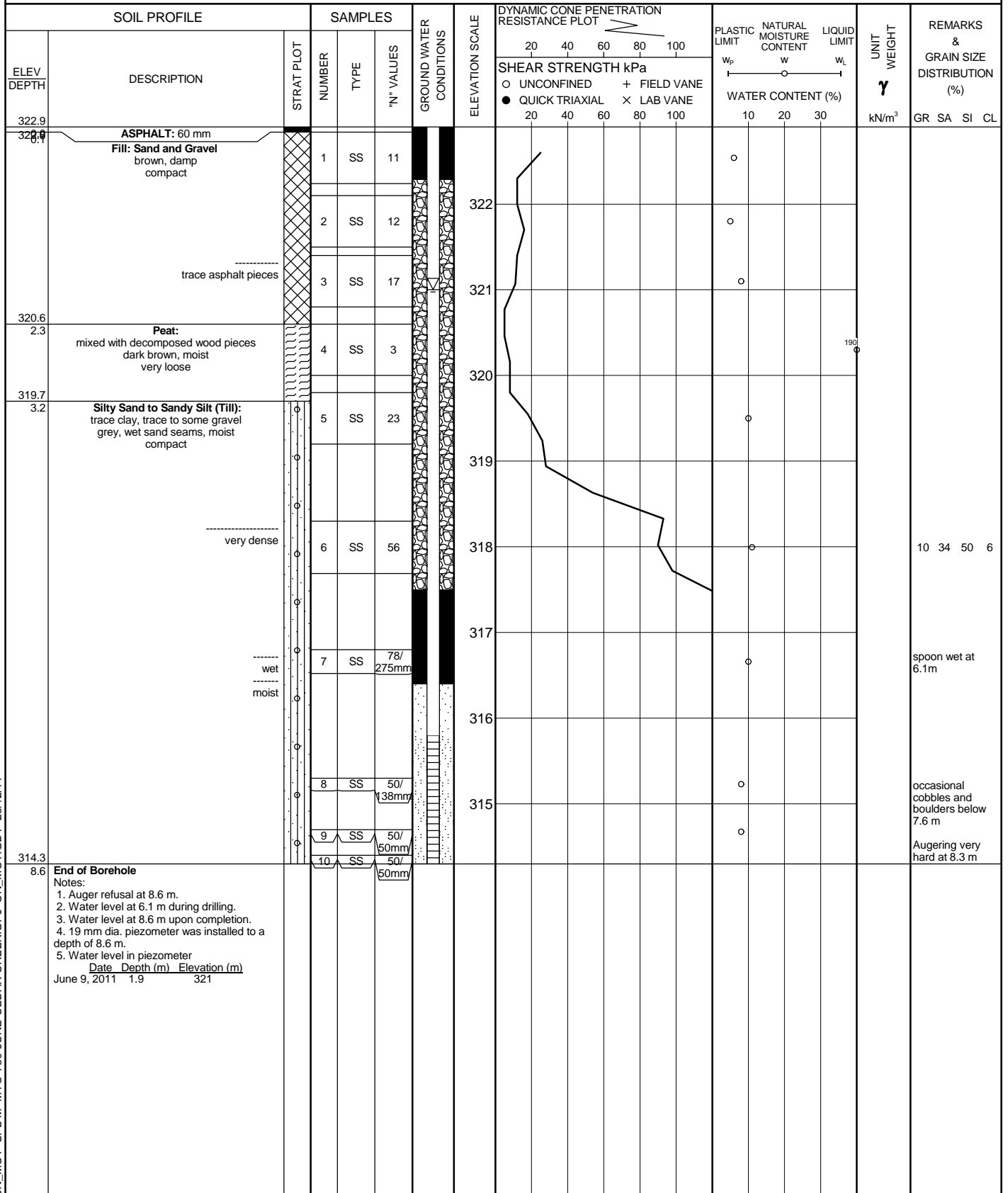


**RECORD OF BOREHOLE No BH-1**

1 OF 1

**METRIC**

W.P. 5270-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE  
DIST Algoma HWY 631 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE  
DATUM Geodetic DATE 08/06/2011 CHECKED BY CH



+<sup>3</sup>, ×<sup>3</sup>: Numbers refer to Sensitivity ○ 3% STRAIN AT FAILURE

ONL\_MOT\_SPL-M-MTO-750-JUNE-CEDAR CREEK.GPJ ONL\_MOT\_GDT 28/12/11



**RECORD OF BOREHOLE No BH-2**

1 OF 1

**METRIC**

W.P. 5270-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE  
DIST Algoma HWY 631 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE  
DATUM Geodetic DATE 09/06/2011 CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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								—		
							20	40	60	80	100									
322.9																				
320.6	ASPHALT: 50 mm Fill: Sand and Gravel brown, damp compact		1	SS	17															
			2	SS	14															
			3	SS	11															
320.6	Peat: dark brown, moist loose		4	SS	6															
319.8																				
319.8	Silty Sand to Sandy Silt (Till): trace clay, trace gravel grey, wet sand seams, moist compact		5	SS	18															

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

ONL\_MOT\_SPL-M-MTO-750-JUNE-CEDAR CREEK.GPJ ONL\_MOT\_GDT 28/12/11

**RECORD OF BOREHOLE No BH-3**

1 OF 1

**METRIC**

W.P. 5270-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE  
DIST Algoma HWY 631 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE  
DATUM Geodetic DATE 07/06/2011 CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	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											
322.9																	
322.8	<b>ASPHALT:</b> 40 mm <b>Fill: Sand and Gravel</b> trace cobble, trace asphalt pieces brown, damp compact		1	SS	18												
	trace to some gravel		2	SS	20												
	loose, decomposed wood pieces, very moist		3	SS	7												
320.6	<b>Peat:</b> dark brown, moist loose		4	SS	4										178		
320.1	<b>Silty Sand to Sandy Silt (Till):</b> trace clay, trace gravel grey, wet sand seams, moist compact																
2.8			5	SS	19												
	dense		6	SS	31												
			7	SS	47												
	very dense		8	SS	95/ 225mm												
			9	SS	78/ 275mm												
315.8																	
7.1	<b>End of Borehole</b> Notes: 1. Auger refusal at 7.1 m on possible cobble or boulder. 2. Water level at 6.1 m during drilling. 3. Water level at 6.4 m and borehole was open upon completion.																

**RECORD OF BOREHOLE No BH-4**

1 OF 1

**METRIC**

W.P. 5270-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE  
DIST Algoma HWY 631 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE  
DATUM Geodetic DATE 07/06/2011 CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
								20 40 60 80 100						
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE						
								20 40 60 80 100						
322.9														
320.6	ASPHALT: 40 mm Fill: Sand and Gravel brown, damp compact		1	SS	23									
			2	SS	25									
			3	SS	19									
320.6														
2.3	Peat: trace rootlets dark brown, moist		4	SS	34									
319.8														
3.1	Silty Sand to Sandy Silt (Till): trace clay, trace gravel grey, wet sand seams, moist compact		5	SS	19									

**RECORD OF BOREHOLE No BH-5**

1 OF 1

**METRIC**

W.P. 5270-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE  
DIST Algoma HWY 631 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE  
DATUM Geodetic DATE 08/06/2011 CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa						
323.1								20	40	60	80	100		
322.8	ASPHALT: 40 mm Fill: Sand and Gravel brown, damp compact		1	SS	14			○ UNCONFINED	+ FIELD VANE					
								● QUICK TRIAXIAL	× LAB VANE					
322.2	Fill: Clayey Silt trace gravel, trace rootlets brown, moist stiff		2	SS	9									
0.9			3	SS	8									
320.8	Peat: dark brown, moist loose		4	SS	12									
320.7	Silt : trace clay, trace rootlets grey, moist compact		5	SS	6									
2.4	wet seams of sand and loose													
318.8	Silty Sand to Sandy Silt (Till): trace clay, trace gravel grey, moist very dense		6	SS	56									
4.3			7	SS	93									0 1 84 15
			8	SS	37									
			9	SS	50/ 60mm									
314.4	End of Borehole													Augering very hard at 8.6 m
8.7	Notes: 1. Auger refusal at 8.7 m. 2. Water was at 6.1 m during drilling. 3. Water was at 6.3 m and borehole was open upon completion. 4. 19 mm dia. piezometer was installed to a depth of 8.7 m. 5. Water level in piezometer Date Depth (m) Elevation (m) June 9, 2011 1.9 321.2													

**RECORD OF BOREHOLE No BH-6**

1 OF 1

**METRIC**

W.P. 5270-08-01 LOCATION See Borehole Location Plan ORIGINATED BY NE  
DIST Algoma HWY 631 BOREHOLE TYPE Hollow Stem Augers COMPILED BY NE  
DATUM Geodetic DATE 07/06/2011 CHECKED BY CH

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)
								○ UNCONFINED	+	×						
322.9	ASPHALT: 50 mm						20	40	60	80	100					
320.9	Fill: Sand and Gravel brown, damp compact		1	SS	15											
321.4			2	SS	11											
321.5	Peat: mixed with decomposed wood dark brown, moist very loose		3	SS	2											
320.6	Silty Sand to Sandy Silt (Till): trace clay, trace gravel grey, moist compact		4	SS	18											
			5	SS	14											
			6	SS	90											
			7	SS	50/ 125mm											
			8	SS	50/ 125mm											
			9	SS	50/ 50mm											
314.2	End of Borehole															
8.7	Notes: 1. Auger refusal at 8.7 m. 2. Water level at 6.1 m during drilling. 3. Water level at 8.5 m and borehole was open upon completion. 4. 19 mm dia. piezometer was installed to a depth of 8.7 m. 5. Water level in piezometer Date    Depth (m)    Elevation (m) June 9, 2011    1.7    321.2															

# Appendix B

## Chemical Test Results

Results relate only to the parameters tested on the samples submitted

Results relate only to the parameters tested on the samples submitted.



# Appendix C

## Explanation of Terms used in Report

## 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 63.5 kg, 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 N.

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 STRUCUTRAL 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

**JOINT 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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICALL PROPERTIES OF SOIL

$m_v$	$\text{kPa}^{-1}$	COEFFICIENT OF VOLUME CHANGE
$c_c$	1	COMPRESSION INDEX
$c_e$	1	SWELLING INDEX
$c_a$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	$\text{m}^2/\text{s}$	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\Phi$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\Phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $c_u / \tau_r$

## PHYSICAL PROPERTIES OF SOIL

$P_s$	$\text{kg}/\text{m}^3$	DENSITY OF SOLID PARTICLES	e	1, %	VOID RATIO	$e_{\min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{\max} - e}{e_{\max} - e_{\min}}$
$P_w$	$\text{kg}/\text{m}^3$	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
$\gamma_w$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF WATER	$s_r$	%	DEGREE OF SATURATION	$D_n$	mm	N PERCENT – DIAMETER
P	$\text{kg}/\text{m}^3$	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	$\text{kg}/\text{m}^3$	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	q	$\text{m}^3/\text{s}$	RATE OF DISCHARGE
$\gamma_d$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $(W_L - W_p)$	v	m/s	DISCHARGE VELOCITY
$\rho_{\text{sat}}$	$\text{kg}/\text{m}^3$	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $(W - W_p) / I_p$	i	1	HYDAULIC GRADIENT
$\gamma_{\text{sat}}$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SATURATED SOIL	$I_c$	1	CONSISTENCY INDEX = $(W_L - W) / I_p$	k	m/s	HYDRAULIC CONDUCTIVITY
$P'$	$\text{kg}/\text{m}^3$	DENSITY OF SUBMERED SOIL	$e_{\max}$	1, %	VOID RATIO IN LOOSEST STATE	j	$\text{kN}/\text{m}^3$	SEEPAGE FORCE
$\gamma'$	$\text{kN}/\text{m}^3$	UNIT WEIGHT OF SUBMERGED SOIL						

**PART B**  
**FOUNDATION DESIGN REPORT**  
**PROPOSED CEDAR CREEK CULVERT REPLACEMENT**  
**HIGHWAY 631, NORTH OF HORNPAYNE, ONTARIO**  
**WP 5270-08-01 SITE NO. 38N-014/C**  
**G.W.P. 5270-08-00**  
**MTO GEOCRES NO. 42F-24**

Prepared for:

**MCINTOSH PERRY CONSULTING ENGINEERS**

By:

**SPL CONSULTANTS LIMITED**

Project: 750-1001 (Cedar Creek)  
January 2013



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## Table of Contents

### PART B: FOUNDATION DESIGN REPORT

<b>6. DISCUSSION AND RECOMMENDATIONS .....</b>	<b>7</b>
6.1 General .....	7
6.2 Frost Protection .....	7
6.3 Seismic Performance.....	7
6.4 Foundation Design .....	8
6.4.1 Foundation Options .....	8
6.4.2 Bearing Resistance .....	8
6.4.3 Sliding Resistance .....	8
6.5 Bedding, Cover and Backfill.....	8
6.6 Earth Pressures and Backfilling .....	9
6.7 Embankment Widening .....	11
6.8 Erosion Protection .....	11
6.9 Construction Considerations.....	12
6.10 Corrosion and Cement Type .....	13
<b>7. CLOSURE .....</b>	<b>14</b>
<b>8. REFERENCES .....</b>	<b>15</b>

## **6. DISCUSSION AND RECOMMENDATIONS**

### **6.1 General**

The proposed new culvert is a 6.0 m wide by 1.8 m high (interior dimensions), closed bottom, concrete box culvert. The base of the new culvert will be installed at approximately 319 m elevation (which is slightly lower than the existing culvert). The current design does not include any change to the final embankment height or width.

The subsurface conditions encountered in the boreholes drilled at the site include a layer of granular fill approximately 2.3 m thick at the culvert location. The granular fill is underlain by a layer of saturated peat which is approximately 0.5 m to 0.9 m thick at the culvert crossing. The fill and peat are underlain by a deposit of native glacial till which is a compact to dense mix of predominantly silty sand and sandy silt with a trace to some gravel and occasional cobbles and boulders. The glacial till extended to the depth of drilling in all six of the boreholes. Auger refusal was met in all six of the boreholes at depths ranging from 7.1 m to 10.2 m below the existing ground surface.

Four piezometers were installed at the site. The stabilized groundwater levels were found to be at 320.9 m to 321.2 m elevation, which is roughly coincident with the level of Cedar Creek at the time of investigation.

Based on the borehole information and the proposed culvert location, the new culvert and bedding will be founded below the existing peat layer (the base of which is at approximately 320 m elevation in the vicinity of the culvert). The culvert will therefore be founded on compact to very dense silty sand and sandy silt till. This layer is expected to be adequate to support the proposed culvert.

### **6.2 Frost Protection**

The depth of frost penetration for the Cedar Creek site is 2.4 m. The existing embankment fill material within the frost depth is predominantly sand and gravel fill and would be considered to have a low susceptibility to frost heaving. As such, frost tapers are not required for the new construction.

### **6.3 Seismic Performance**

The site is located in an area of relatively low seismic activity. The Peak Horizontal Ground Acceleration (PHA) for an earthquake with a 10% probability of exceedance in 50 years (475 year return period event) is 0.011 g. Based on the Canadian Highway Bridge Design Code (CHBDC) this corresponds to a Seismic Performance Zone 1 (assuming the crossing would be classified as an Emergency Route Bridge), and Zonal Acceleration Ratio of  $A = 0$  (CHBDC Section 4.4).

For the purposes of assessing the effects of site conditions under seismic conditions, the site may be assumed to be Soil Profile Type II, which corresponds to a Site Coefficient  $S = 1.2$  (CHBDC Section 4.4.6).

## **6.4 Foundation Design**

### **6.4.1 Foundation Options**

MTO has selected a closed-bottom, pre-cast concrete box culvert as the preferred replacement option. The sub-surface conditions at the site are considered to be adequate for the founding of the preferred replacement structure (pre-cast box culvert) on normal foundations (granular bedding placed over native granular soils or existing granular fill).

Deep foundations are technically feasible, but are not required as conventional shallow foundations will provide sufficient bearing resistance and settlement performance for the proposed culvert.

### **6.4.2 Bearing Resistance**

The new culvert and bedding should be founded on the native silty sand and sandy silt till. It is expected that the peat will be removed as part of the excavation for the new culvert, however, in the event localized areas of peat are encountered at or below the founding depth they should be removed and replaced with compacted granular fill (OPSS 1010 Granular A or B).

For the new 6 m wide culvert, which will be founded at approximately 319 m elevation, the unfactored geotechnical bearing resistance at Ultimate Limit State (ULS) can be taken as 600 kPa. A resistance factor of 0.5 should be applied to this value, yielding a factored bearing resistance of 300 kPa at ULS. This value is for a concentrically loaded foundation. Eccentric loads (if present) should be accounted for by considering an effective bearing area as outlined in Section 6.7 of the CHBDC.

The geotechnical resistance at the Serviceability Limit State (SLS) can be taken as 200 kPa.

Provided that the subgrade is not disturbed during construction the total and differential settlements associated with the above SLS resistance values are expected to be less than 25 mm and 20 mm, respectively. It is expected that for this level of settlement the new culvert will not require a camber.

### **6.4.3 Sliding Resistance**

For the purposes of evaluating the sliding resistance (Section 6.7.5 of the CHBDC) of either the native soils or the granular fill below the foundation the effective cohesion,  $c'$ , should be assumed to be zero. The effective friction angle ( $\phi'$ ) for the silty, sandy till soils may be assumed to be  $30^\circ$ . These values are unfactored values. A resistance factor of 0.8 should be applied to the resulting resistance to obtain the factored sliding resistance as per the CHBDC.

## **6.5 Bedding, Cover and Backfill**

Bedding, Cover and Backfill details for the new culverts should be generally as per MTOD 803.021.

Bedding for the new culvert may consist of either:

- 500 mm of compacted Granular A or Granular B Type II; or
- 300 mm of compacted Granular A or Granular B Type II placed over a lean concrete working slab.

If constructed properly, either bedding treatment is considered adequate from a foundations perspective.

A 75 mm levelling course of additional Granular A or fine aggregate should also be provided between the bedding and the culvert. In order to minimize the potential for piping and undermining of the culvert foundations the bedding and levelling course should be wrapped in a non-woven geotextile which meets the requirements of OPSS 1860.

Cover for the new culverts should be a minimum of 300 mm thick as per MTOD 803.021 and may include either Granular A or Granular B with a maximum particle size of 150 mm (as per OPSS 902).

Granular Backfill may consist of either imported Granular A or B material, or salvageable portions of the existing soils (Granular A and B is preferred for fills below the water table as well as immediately below the pavement structure). Portions of the fill which forms the embankment may meet the requirements of OPSS 1010 for Granular B Type I. Other portions of the existing fill embankment may meet the requirements of OPSS 1010 for SSM. The excavated soils should be reviewed as excavated and suitable portions may be stockpiled for re-use as backfill (if a cost-effective stockpile location is available). Material from below the water table, as well as the native silty till, is unlikely to be suitable for re-use as backfill and there will be a net import of granular fill required for construction. All backfill should be free from frozen or other deleterious material and should have a maximum particle size of 150 mm as per OPSS 902.

All bedding, cover and backfill should be placed in accordance with OPSS 206, and compacted in accordance with OPSS 501 and OPSS 902.

Heavy vibratory equipment should not be used behind the culvert and retaining walls within the restricted zone as outlined in OPSS 501 and OPSS 902.

## **6.6 Earth Pressures and Backfilling**

Computation of earth pressures acting against culvert walls and retaining structures should be in accordance with the Canadian Highway Bridge Design Code (CHBDC). For design purposes, the following properties can be assumed for the backfill:

### **Compacted Granular 'A' or Granular 'B' Type II**

Angle of Internal Friction ( $\phi$ ) = 35 degrees (unfactored)

Unit Weight = 22 kN/m<sup>3</sup>

Coefficients of Lateral Earth Pressure:

Earth Pressure Coefficient	Level Backfill	Sloping Backfill 3H:1V	Sloping Backfill 2H:1V
$K_a$	0.27	0.34	0.40
$K_b$	0.35	0.44	0.50
$K_0$	0.43	0.56	0.62
$K^*$	0.45	0.60	0.66

### Compacted Granular 'B' Type I

Angle of Internal Friction ( $\phi$ ) = 32 degrees (unfactored)

Unit Weight = 21 kN/m<sup>3</sup>

Coefficients of Lateral Earth Pressure:

Earth Pressure Coefficient	Level Backfill	Sloping Backfill 3H:1V	Sloping Backfill 2H:1V
$K_a$	0.30	0.38	0.47
$K_b$	0.38	0.48	0.57
$K_0$	0.47	0.61	0.69
$K^*$	0.51	0.67	0.76

Notes:

$K_a$  is the coefficient of active earth pressure;

$K_b$  is the coefficient of active earth pressure for an unrestrained structure including compaction efforts;

$K_0$  is the coefficient of earth pressure at rest;

$K^*$  is the coefficient of earth pressure at rest for a fully restrained structure including compaction efforts.

The above values assume that the backfill behind the structure is free-draining granular fill, and that proper drainage is provided. Water pressures should also be accounted for in areas below the water table.

The appropriate earth pressure coefficient for design will depend upon whether the retaining structure is restrained or some movement can occur such that the active earth pressure state can develop. The effect of compaction should also be taken into account when selecting the appropriate earth pressure coefficients. The use of heavy vibratory equipment behind the culvert and retaining walls should be limited as per current MTO practice.

According to the method outlined in the CHBDC and Commentaries Section 4.6.4, for a Zonal Acceleration Ratio of  $A = 0$  the earth pressure under the design seismic event is equal to the earth



pressure under static conditions (the horizontal seismic coefficient,  $k_h$  is 0.5 or 1.5 times the Zonal Acceleration Ratio, and for the very small design earthquake  $A = 0$ ).

## **6.7 Embankment Widening**

It is understood that the existing roadway embankment may be widened to facilitate a detour around the construction site on the east side, and that this widening would likely be on the order of 2.5 m to 3 m in height. Based on the conditions encountered in the boreholes, foundation failures are not anticipated for the proposed embankment widening with normal (2H:1V or flatter) slopes, assuming that all organic or unsuitable materials are removed prior to placing the embankment fills. In particular this will require removal of the peat layer which is present at the toe of the embankment.

The layer of peat should be removed and the approved embankment subgrade should be proofrolled. The construction of the new embankment widening may require temporary dewatering or groundwater control as discussed in Section 6.9 below where the base of the embankment is below the water table.

The sides of the existing embankment should be benched prior to placing fill material for the embankment widening, as per OPSD 208.01; benching is required when existing slopes are steeper than 3H:1V. Fill material should be placed and compacted as per OPSS 206 and 501. Borrow material should consist of select suitable inorganic earth, free of objectionable inclusions such as cobbles, boulders, frozen materials, organic soils, etc. The existing fill material is expected to be suitable for this purpose provided that any material from below the water table is properly dried prior to placement and compaction. Borrow material for the proposed embankment should be approved prior to installation from both a geotechnical and environmental standpoint.

All embankment construction (including review of exposed subgrade, approval of fill materials, etc.) should be carried out under the review and supervision of a qualified person.

## **6.8 Erosion Protection**

The native soils at the site are expected to be susceptible to erosion. Erosion and scour protection (such as rip rap treatment similar to OPSD 810.919) will be required at the culvert inlet and outlet. The sizing of the erosion protection should be carried out by a specialist who is familiar with the site hydraulics and the findings of this investigation.

The current design includes upstream and downstream cut-off walls on the new culvert. These walls should extend to below the base of the bedding and levelling course to prevent flow of water below the walls through the permeable bedding layer. It is also recommended that the bedding be enclosed in a non-woven geotextile (OPSS 1860) in order to reduce the potential for piping and erosion of the culvert bedding.

## **6.9 Construction Considerations**

### Construction Dewatering

The groundwater level at the site was found to be approximately equal to the level of the water in the Creek at the time of the investigation. The groundwater level is expected to be sensitive to changes in the water level in the creek, and for this reason it is recommended that, if possible, construction be carried out in a dry period when the creek would be expected to be at its lowest level.

The culvert replacement will require excavations below the water table, and even with these measures, dewatering will be required to stabilize the native soils, to maintain a dry working area and to minimize disturbance of the foundation soils during construction. Depending upon the creek level and groundwater conditions at the time of construction, closely spaced filtered sumps may be used for excavations which extend only a short distance below the groundwater table (say 0.5 m or so). The creek level was at approximately 321 m elevation at the time of the investigation. Excavation is required to an elevation of approximately 318.5 (or deeper if over-excavation and replacement of the peat is necessary).

At this depth the excavation will likely require an active dewatering system including well points and/or deep wells. In addition to an above-ground diversion (coffer dam and diversion of the existing water course around the site), an underground impervious barrier (such as a sheet pile wall; the choice of protection systems and groundwater control will ultimately be the responsibility of the contractor) may also need to be constructed to control groundwater flows into the excavation.

### Temporary Excavations

All excavations should be carried out in accordance with the most recent Occupational Health and Safety Act (OHSA). Part III of Ontario Regulation 213/91 deals with excavations. In addition, the following Ontario Provincial Standard Specifications (OPSS) also deal with temporary excavations:

OPSS 539 – Construction Specification for Temporary Protection Systems

OPSS 902 – Construction Specification for Excavating and Backfilling - Structures

The soils at the site include granular fill in the pavement structure and embankment, underlain by compact to dense native granular soil. A layer of organic soil rests between native mineral soils and the fill material. Both granular fill and granular native soils can be classified as Type 3 soil above and below water table. The peat layer present between the fill and the native soils should be classified as Type 4 soils, and may require significant temporary support where it is exposed in the excavation.

Temporary excavations above the water table are likely feasible using sloped excavations in the granular fill. Excavations below the water table will require some form of protection system and temporary support. It is also noted that the staging will require excavation in close proximity to the travelled lanes of the highway, which will preclude the use of sloped excavations in some areas (as there is not sufficient space).

If temporary shoring is required it would generally consist of soldier piles and timber lagging or interlocking sheet piles. It should be noted that cobbles and boulders were encountered during the investigation (both in the fill material and in the native till) and refusal was met in all boreholes at depths ranging from 7 m to 10 m below the existing road surface in all of the boreholes drilled at the site). These factors should be considered when selecting shoring systems and installation methods.

### Foundation Excavations

The bearing capacities provided in Section 6.4 above assume that the subgrade is not excessively disturbed during construction. Given the fact that the foundations for the proposed structure will be below the groundwater table in silty soils, it will require careful construction control to achieve this condition. Installation and operation of an adequate dewatering system, as discussed above, will be critical to the construction of the foundations.

A lean concrete working slab (mud slab) on foundation bearing surfaces can also be included in the design (see Section 6.5 above). If used, the working slab should be placed immediately after excavation and inspection (before placement of bedding and levelling layers) in order to minimize foundation disturbance. If excavation conditions are found to be better than anticipated then the requirement for the lean concrete mud slab may be waved at the discretion of the CA and QVE. All excavated surfaces should be kept free of frost, water, etc., during the course of construction.

All excavated surfaces should be inspected prior to foundation construction by a qualified individual who is familiar with the findings of this investigation and the design and construction of similar structures.

### **6.10 Corrosion and Cement Type**

Two soil samples were submitted to EXOVA Accutest for testing related to soil corrosivity and potential exposure of concrete elements to sulphate attack. The results of these tests are included in Appendix B.

The test results indicate that the sulphate content of the native soils is relatively low, and sulphate-resistant Portland cement is not required.

The test results (soil resistivity and pH) indicate that there is a low to moderate potential for corrosion of buried steel elements. Appropriate care should be taken in designing/selecting the corrosion protection system for any buried steel structures.

## 7. CLOSURE

The field investigations were supervised by Mr. Naeem Ehsan, P.Eng. This report was prepared by Mr. Chris Hendry, P.Eng. Mr. Fanyu Zhu, P.Eng., SPL's designated MTO contact and Mr. Shaheen Ahmad, P.Eng., SPL's project quality control auditor, provided independent review and quality control of the technical aspects of this report.

We trust that the information contained in this report is satisfactory. Should you have any questions, please do not hesitate to contact this office.

### SPL CONSULTANTS LIMITED

Chris Hendry, M.Eng., P.Eng.



Fanyu Zhu, Ph.D., P.Eng.



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## 8. REFERENCES

The following section provides a general list of references, as well as a list of Ontario Provincial Standard Specifications which are expected to be relevant to the Foundations portion of the proposed work.

### General References

CAN/CSA-S6-06 Canadian Highway Bridge Design Code, 2011

Canadian Foundation Engineering Manual, 2006. 4<sup>th</sup> Edition. Canadian Geotechnical Society

### Relevant Ontario Provincial Standard Specifications

OPSS NO.	TITLE
128	Supply of Pre-Qualified Materials and Products
182	Environmental Protection for Construction in Waterbodies and on Waterbody Banks.
201	Clearing, Close Cut Clearing, Grubbing, and Removal of Surface and Piled Boulders
206	Grading
401	Trenching, Backfilling, and Compacting
404	Support Systems
501	Compacting
504	Preservation, Protection and Reconstruction of Existing Facilities
506	Dust Suppressants
510	Removals
511	Rip-Rap, Rock Protection, and Granular Sheeting
514	Trenching, Backfilling, and Compacting
518	Control of Water from Dewatering Operations
539	Temporary Protection Systems
805	Temporary Erosion and Sediment Control Measures
902	Excavating and Backfilling – Structures
1001	Aggregates - General
1010	Aggregates – Base, Subbase, Select Subgrade, and Backfill Material
1860	Geotextiles

### Relevant CDED Special Provisions

Provision No.	Title
100S60	Amendment to MTO General Conditions of Contract, April 2010 – use of unlicensed vehicles...
104S04	Amendment to OPSS 401, November 2010
105S21	Amendment to OPSS 501, November 2010
110S13	Amendment to OPSS 1010, April 2004
199S55	Record Drawings for Structures and Foundations
511S01	Rip Rap
539S02	Protection System – Amendment to OPSS 512, April 2011
805F01	Light-Duty Sediment Barriers, etc.

### Relevant OPSD's

OPSD No.	Title
810.010	Rip-Rap Treatment for Sewer and Culvert Inlets
810.020	Rip-Rap Treatment for Ditch Inlets
3090.100	Foundation, Frost Penetration Depths for Northern Ontario

### Relevant MTOD's

MTOD No.	Title
803.021	Bedding and Backfill for Precast Concrete Box Culverts