



**FOUNDATION INVESTIGATION AND DESIGN REPORT**

**for**

**SHARPSAND RIVER TRIBUTARY CULVERT REPLACEMENT**

**HIGHWAY 129**

**TOWNSHIP OF STURGEON, ONTARIO**

**AGREEMENT NO. 5013-E-0040**

**G.W.P. 5222-05-00**

**SITE NO. 38S-331/C**

**WP NO. 5148-14-01**

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PML Ref.: 14TF038  
Index No.: 034FIR and 035FDR  
GEOCRES No.: 41J-99  
March 29, 2016



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## TABLE OF CONTENTS

### PART A - FOUNDATION INVESTIGATION REPORT

1. INTRODUCTION .....	1
2. SITE DESCRIPTION AND GEOLOGY .....	1
3. INVESTIGATION PROCEDURES .....	2
4. SUMMARIZED SUBSURFACE CONDITIONS.....	3
4.1 Pavement Structure and Fill .....	4
4.2 Sand and Gravel.....	5
4.3 Sand.....	5
4.4 Groundwater .....	6
5. CLOSURE .....	7

#### Appendix FIR-A – Site Photographs

Appendix FIR-B – Drawing C15-1: Borehole Locations and Soil Strata, Sharpsand River Tributary Culvert  
Explanation of Terms Used in Report  
Record of Borehole and Auger Probe Sheets: C15-1 to C15-4, AP-15-1 and AP-C15-2  
Results of Grain Size Distribution Analyses – Figures NC-GS-1 to NC-GS-3

**PART A - FOUNDATION INVESTIGATION REPORT**  
for

Sharpsand River Tributary Culvert Replacement  
Highway 129 (Site 38S-331/C)  
Township of Sturgeon, Ontario  
GWP 5222-05-00, WP No. 5148-14-01

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**1. INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for the proposed Sharpsand River Tributary culvert replacement at Sta. 19+860 on Highway 129, located approximately 22 km north of the Highway 129 / Highway 17 intersection, in the Township of Sturgeon in the Algoma District. The investigation was carried out by Peto MacCallum Ltd. (PML) for AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

**2. SITE DESCRIPTION AND GEOLOGY**

Refer to Appendix FIR-A, Photographs P1 to P6 for general views of the site. The existing Sharpsand River Tributary crossing includes a 2.5 m diameter CSP culvert and embankments some 4 m high.

The existing Sharpsand River Tributary culvert is located about 3.5 km south of another Sharpsand River Tributary crossing. The tributary stream runs south parallel to the highway and crosses the highway at the existing culvert location. A gravel-surface service road connects to Highway 129 about 100 m south of the culvert. The culvert site is surrounded by heavily treed wood lots.

The site geology described in previous GEOCREs reports carried out in the general area of the project indicates that the site lies in a deep valley located within Pre-Cambrian rock formations, which in this area consist mainly of granite and other intrusive rocks. During the last ice age, continental glaciers eroded much of the bedrock and laid out a shallow mantle of glacial debris that covered the area. The surface of these glacial till sheets have further been modified by post-glacial streams which have cut valleys into the tills and laid out alluvial soils on the surface.

There were no previous investigation reports available for the existing culvert. Also, as-built drawings were not available at the time of this investigation.



### **3. INVESTIGATION PROCEDURES**

The field work for the current foundation investigation involved four (4) boreholes and two (2) auger probes that were carried out during the period of November 2015 to January 2016 at the approximate locations shown on Drawing C15-1. Borehole 15-4 and auger probes AP-C15-1 and AP-C15-2 were advanced to depths of 0.5 to 0.8 m and terminated by refusal on a boulder field. Boreholes 15-1 to 15-3 were advanced to depths of 6.9 to 19.8 m where competent soils were encountered.

The inlet and outlet of the existing culvert are located within the heavily wooded area, surrounded by stand of trees. The boreholes were advanced using various methods at these two locations.

Borehole C15-1, at the culvert inlet, was advanced using a track-mounted D-25 drill rig that was lowered and retrieved by cranes to and from the bottom of the highway embankment.

Major tree cutting would be required to access the proposed location of Borehole C15-4, at the culvert outlet, and the crane operation was also not feasible at this location due to the required off-set. As a result, Borehole C15-4 was advanced by portable tripod. The borehole was advanced to refusal at a depth of 0.8 m. Borehole C15-4 was re-drilled at two additional locations identified on auger probes C15-4A and C15-4B. These additional test holes confirmed the wide extent of the cobble and boulder deposit.

The boreholes C15-2 and C15-3 were advanced using a truck mounted A-53 drill rig and a track-mounted D-25 drill rig equipped with continuous flight 75 mm diameter solid stem augers and NQ rotary casing, respectively. The equipment was supplied and operated by specialist drilling contractors working under the full-time supervision of a PML field supervisor.

Soil samples were obtained at selected intervals using a split-spoon sampler in accordance with the Standard Penetration Test (SPT) procedures described in the ASTM D1586, Standard Test Method for Standard Penetration Test. The drill rigs were equipped with 63.5 kg (140 lb) automatic hammers with calibrated 760 mm (30 in.) falls. The tripod method used a manual 63.5 Kg hammer. It is noted that poor sample recovery was experienced although sand traps were



employed due to the encountered cobbles and boulder in the native deposits. Standard penetration and Dynamic Cone Penetration (DCP) tests were conducted to assess the strength characteristics of the encountered cohesionless soil substrata. The results of the field tests and observations during and at completion of drilling are reported on the Record of Borehole sheets.

The groundwater conditions were assessed at the borehole locations during and at completion of drilling by observation of the groundwater levels in the open holes and the condition of the drilling rods and sampler as the samples were retrieved and by examination of the soil samples.

The boreholes were backfilled with a bentonite/cement mixture where required in accordance with the MTO guidelines and MOE Reg. 903 for borehole abandonment.

The boreholes were laid out by PML and subsequently surveyed in MTM NAD 83 northing and easting coordinates by exp Geomatics under contract to AECOM. The survey provided to AECOM was used by PML for this report.

The recovered soils were identified in the field in accordance with the MTO Soil Classification procedures. The soil samples were returned to the PML Toronto laboratory for detailed visual examination, classification and routine moisture content determination. A total of 25% of the recovered samples were tested in the PML laboratories including six (6) grain size distribution analyses and nine (9) moisture content determinations. Since only non-cohesive soils were encountered, Atterberg limits were not performed.

#### **4. SUMMARIZED SUBSURFACE CONDITIONS**

Refer to the attached Record of Borehole sheets for the details of the subsurface conditions including soil classifications, groundwater observations and inferred stratigraphy. The laboratory grain size distributions are presented in Figures NC-GS-1 to NC-GS-3. The laboratory test results are shown on the Record of Borehole sheets.

The Borehole Location Plan and the stratigraphic profile and cross-sections prepared from the current borehole data are shown on Drawing C15-1. We note that the boundaries between soil



strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The existing culvert is located within an approximately 3 m to 4 m high granular fill embankment placed over native soils. The subsurface stratigraphy below the fill and organics consisted of that was at least 6.9 m thick in compact to very dense sand and gravel at borehole C15-1 located at the culvert inlet and 3.0 m to 4.6 m thick in boreholes C15-2 and C15-3 located on the highway embankment. Below the sand and gravel a deposit of 8.3 m to 15.5 m of compact to dense sand was encountered which extended to boreholes termination depths. Cobbles and boulders were encountered throughout the native cohesionless soil deposits at the outlet of the culvert, where the Borehole C15-4 and two auger probes (AP-C15A and AP-C15B) were terminated at refusal in the cobbles and boulders layer at 0.5 to 0.8 m depths.

#### **4.1 Pavement Structure and Fill**

Shoulder pavements of 80 to 100 mm asphaltic concrete with approximately 400 mm to 420 mm of sand and gravel courses were encountered surficially in Boreholes C15-2 and C15-3 (Highway 129, SBL and NBL shoulders) which extended to 0.5 m depth, Elevations 354.1 m and 354.3 m, respectively.

A 3.8 m to 4.0 m thick layer of sand and gravel fill with organic inclusions that makes up the existing embankment was encountered below the pavement structure in Boreholes C15-2 and C15-3 and extends to Elevations 350.1 m and 350.5 m, respectively.

The SPT "N"-values ("N"-values) measured within the non-cohesive fill range from 9 blows per 300 mm of penetration to 50 blows per 70 mm of penetration, indicating a variable loose to very dense compactness. The results of grain size distribution analysis completed on a sample of this deposit are shown on Figure NC-GS-1. Moisture content determinations of the fill samples ranged between 2% and 5%.



## **4.2 Sand and Gravel**

A minimum of 0.5 m to 6.7 m thick deposit of sand and gravel was encountered below upper 200 mm and 300 mm thick organic layers in Boreholes C15-1 and C15-4 (culvert inlet and outlet) and extended to Elevations 351.0 and 344.3 m. The soil unit was encountered below the highway embankment fill layers in Boreholes C15-2 and C15-3 at depths of 4.5 m and 4.3 m, Elevations 350.1 m to 350.5 m, respectively. The sand and gravel deposit extends to depths of 7.5 m and 8.9 m, Elevations 347.1 m and 345.9 m, in Boreholes C15-2 and C15-3. Boreholes C15-1 and C15-4 were terminated within this deposit at 6.9 m and 0.8 m depths, Elevations 344.3 m and 351.0 m, respectively.

"N"-values measured within this deposit range from 10 blows per 300 mm of penetration and 50 blows per 50 mm of penetration, suggesting a compact to very dense compactness. The results of grain size distribution analysis completed on a sample of this deposit are shown on Figure NC-GS-2.

A Dynamic Cone Penetration test was conducted within this layer in Borehole C15-1 below Elevation 345.0 at depth of 6.2 m and the results are shown in the Record of Borehole Sheets, confirming a very dense compactness.

## **4.3 Sand**

A deposit of sand was encountered below the sand and gravel in Boreholes C15-2 and C15-3 at 7.5 m and 8.9 m depths, Elevations 347.1 m and 345.9 m. These boreholes were terminated within this deposit at 15.8 m and 19.8 m depths, Elevations 338.8 m and 335.0 m, penetrating it for about 8.3 m and 10.9 m.





The "N"-values measured within this deposit typically range from 15 blows per 300 mm of penetration to 50 blows per 50 mm of penetration, indicating a compact to very dense compactness. A single "N"-value of 8 blows was measured in Borehole C15-2 at 7.5 m depth, Elevation 347.1, suggesting a local loose compactness. The results of grain size distribution analyses of four selected samples obtained from this deposit are shown on Figure NC-GS-3.

A Dynamic Cone Penetration test was conducted within the sand layer in Borehole C15-3 below Elevation 339.0 at depth of 15.8 m and the result shown Record of Borehole Sheets, confirming the compact to dense condition of the deposit.

#### **4.4 Groundwater**

The Sharpsand River Tributary is about 2.5 to 5.0 m wide at the culvert location. The water level in the river tributary flows from north to south and was at about Elevation 350.5 m at the time of the investigation. The water level in the creek governs the water level at the site in view of the pervious native soil deposits.

During and upon completion of drilling, water was at 0.7 m to 4.5 m depths, Elevations 350.0 m to 350.5 m. No water was observed in Borehole C15-4 during or upon completion of drilling.

The groundwater level of the stream is subject to seasonal fluctuations and rainfall patterns.



## 5. CLOSURE

Mr. F. Portela carried out the field investigations under the supervision of Ms. M. Kamranzadeh, MSc, Project Supervisor, EIT and Mr. C. M. P. Nascimento, P. Eng., Project Manager. LandCore Drilling Ltd. supplied the drill equipment for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This report was prepared by Mr. M. Khorsand, BSc, EIT, Project Supervisor and reviewed by Mr. G.O. Degil, PhD, P.Eng., Senior Engineer, Geotechnical Services. Mr. C.M.P. Nascimento, P.Eng., Principal Consultant, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

A blue ink signature of Mansoor Khorsand, written in a cursive style.

Mansoor Khorsand, BSc, EIT  
Project Supervisor, Geotechnical Services



Grigory O. Degil, PhD, P.Eng.  
Senior Engineer, Geotechnical Services



Carlos M.P. Nascimento, P.Eng  
Project Manager and  
MTO Designated Principal Contact



## **APPENDIX FIR-A**

### Site Photographs



**Photograph P1:** Looking west from the centre of Highway 129 at the location of Borehole C15-3. (November 23, 2015)



**Photograph P2:** Looking north from the edge of Highway 129 southbound lane to culvert outlet. (November 22, 2015)





**Photograph P3:** Looking west from Highway 129 southbound lane shoulder to the Sharpsand Tributary culvert outlet. Note the heavily treed valley. (November 16, 2015)



**Photograph P4:** Looking north from Highway 129 southbound lane shoulder to the Sharpsand Tributary culvert outlet. (November 16, 2015)





**Photograph P5:** Looking south at the location of Borehole C15-4. The outlet of the existing CSP culvert is visible. (January 20, 2016)



**Photograph P6:** Looking east at the location of Borehole C15-1. The inlet of the existing CSP culvert is visible. (January 20, 2016)



## **APPENDIX FIR-B**

Drawing C15-1: Borehole Location Plan Sharpsand River Tributary Culvert

Explanation of Terms Used in Report

Record of Borehole Log and Auger Probe: C15-1 to C15-4, AP-15-1 and AP-C15-2

Results of Grain Size Distribution Analyses – Figures NC-GS-1 to NC-GS





## EXPLANATION OF TERMS USED IN REPORT

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

**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.

**COMPOSITION:** SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

**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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	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
F V	FIELD VANE		

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$u$	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

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /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
$S_i$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL


$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$n$	1, %	POROSITY	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$w$	1, %	WATER CONTENT	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$S_r$	%	DEGREE OF SATURATION	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$w_L$	%	LIQUID LIMIT	$D$	mm	GRAIN DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_p$	%	PLASTIC LIMIT	$D_n$	mm	n PERCENT - DIAMETER
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_s$	%	SHRINKAGE LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	$q$	m <sup>3</sup> /s	RATE OF DISCHARGE
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	$v$	m/s	DISCHARGE VELOCITY
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL				$i$	1	HYDRAULIC GRADIENT
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	$j$	kN/m <sup>3</sup>	SEEPAGE FORCE
$e$	1, %	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

**RECORD OF BOREHOLE No C15-1**

1 of 1

**METRIC**

G.W.P. 5222-05-00 LOCATION Coords: 5 181 501.9 N; 354 620.01 E ORIGINATED BY F.P.  
 DIST Algoma HWY 129 BOREHOLE TYPE 'N' Casing D25 Skid Mount COMPILED BY M.K.  
 DATUM Geodetic DATE January 20, 2016 CHECKED BY C.N.

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	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												
351.2	Ground Surface						20	40	60	80	100	20	40	60	kN/m <sup>3</sup>	GR SA SI CL				
0.0	_____ organics _____ Sand and gravel trace silt, trace clay cobbles and boulders  Compact to Brown      Wet very dense		1	SS	-	▽*	351													
			2	SS	62		350													
			3	SS	50/15cm		349													
			4	SS	57		348													
			5	SS	18		347													
			6	SS	10		346													
	very _____ dense		7	SS	50/15cm		345													
			8	SS	50/15cm															
			9	SS	50/15cm															
	Possible _____ sand and gravel																			
344.3	End of borehole																			
6.9	*      2016   02   20  ▽*      Water level observed during drilling  NOTE:      No cave-in after completion of drilling																			

**RECORD OF BOREHOLE No C15-2**

1 of 2

**METRIC**

G.W.P. 5222-05-00 LOCATION Coords: 5 181 493.3 N; 354 633.1 E ORIGINATED BY F.P.  
DIST Algoma HWY 129 BOREHOLE TYPE C.F.S.S.A. + Casing COMPILED BY M.K.  
DATUM Geodetic DATE November 21 & 22, 2015 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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											
							20 40 60 80 100											
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE											
WATER CONTENT (%)					20 40 60													
354.6 0.0	Ground Surface					▽*	354											
354.1 0.5	80mm asphalt over sand and gravel (PAVEMENT FILL)		1	SS	27													
	Sand and gravel trace silt, trace clay		2	SS	50/8cm													
	Very dense Brown Moist to loose																	
	(FILL)		3	SS	30													39 52 7 2
			4	SS	27													
			5	SS	27													
	organic inclusions																	
			6	SS	9													
350.1 4.5	Sand and gravel trace silt, trace clay cobbles		7	SS	39													75 23 1 1
	Compact to Brown/ Wet very dense grey		8	SS	57													
			9	SS	14													
347.1 7.5	Sand, trace gravel						347											
	Compact to Brown/ Wet dense grey		10	SS	8												1 97 (2)	
							346											
			11	SS	27													
							345											
			12	SS	50/5cm													
							344											
							343											
			13	SS	26													
							342											
			14	SS	19												0 99 (1)	
339.6							340											

**RECORD OF BOREHOLE No C15-2**

2 of 2

**METRIC**

G.W.P. 5222-05-00 LOCATION Coords: 5 181 493.3 N; 354 633.1 E ORIGINATED BY F.P.  
DIST Algoma HWY 129 BOREHOLE TYPE C.F.S.S.A. + Casing COMPILED BY M.K.  
DATUM Geodetic DATE November 21 & 22, 2015 CHECKED BY C.N.

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 ● QUICK TRIAXIAL    × LAB VANE												
339.6								20	40	60	80	100								
15.0	(Cont'd)	•••																		
	Sand, trace gravel	•••																		
338.8	Compact to dense	•••	15	SS	15		339													
15.8	End of borehole	•••																		
	<div>*      2015   11   22</div> <div>▽*      Water level observed          during drilling</div> <div>NOTE:   No cave-in after          completion of          drilling</div> <div>C.F.H.S.A. denotes Continuous Flight Solid Stem Augers</div>																			

**RECORD OF BOREHOLE No C15-3**

1 of 2

**METRIC**

G.W.P. 5222-05-00 LOCATION Coords: 5 181 476.1 N; 354 627.1 E ORIGINATED BY F.P.  
DIST Algoma HWY 129 BOREHOLE TYPE C.F.S.S.A. + Casing COMPILED BY M.K.  
DATUM Geodetic DATE November 23 & 24, 2015 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	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									
354.8 0.0	Ground Surface					20	40	60	80	100						
354.3 0.5	100mm asphalt over sand and gravel (PAVEMENT FILL)		1	SS	50/16cm											
	Sand and Gravel trace silt, trace clay		2	SS	50/7cm											
	Compact to Brown Moist very dense (FILL)															
			3	SS	15											
			4	SS	53											
			5	SS	43											
	organic inclusions		6	SS	12											
350.5 4.3	Sand and gravel trace silt, trace clay cobbles		7	SS	50/5cm											
	Compact to Brown/ Wet very dense grey															
			8	SS	50/15cm											
			9	SS	15											
			10	SS	35											
345.9 8.9	Sand, with gravel to trace gravel		11	SS	50/5cm											
	Compact to Brown Wet very dense															
			12	SS	50/5cm											
			13	SS	41											
			14	SS	24											

**RECORD OF BOREHOLE No C15-3**

2 of 2

**METRIC**

G.W.P. 5222-05-00 LOCATION Coords: 5 181 476.1 N; 354 627.1 E ORIGINATED BY F.P.  
DIST Algoma HWY 129 BOREHOLE TYPE C.F.S.S.A. + Casing COMPILED BY M.K.  
DATUM Geodetic DATE November 23 & 24, 2015 CHECKED BY C.N.

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										○		
								● QUICK TRIAXIAL × LAB VANE												
339.8							20	40	60	80	100	20	40	60						
339.0 15.8	Probable sand Compact to dense		15	SS	21												4 94 (2)			

# RECORD OF BOREHOLE No C15-4

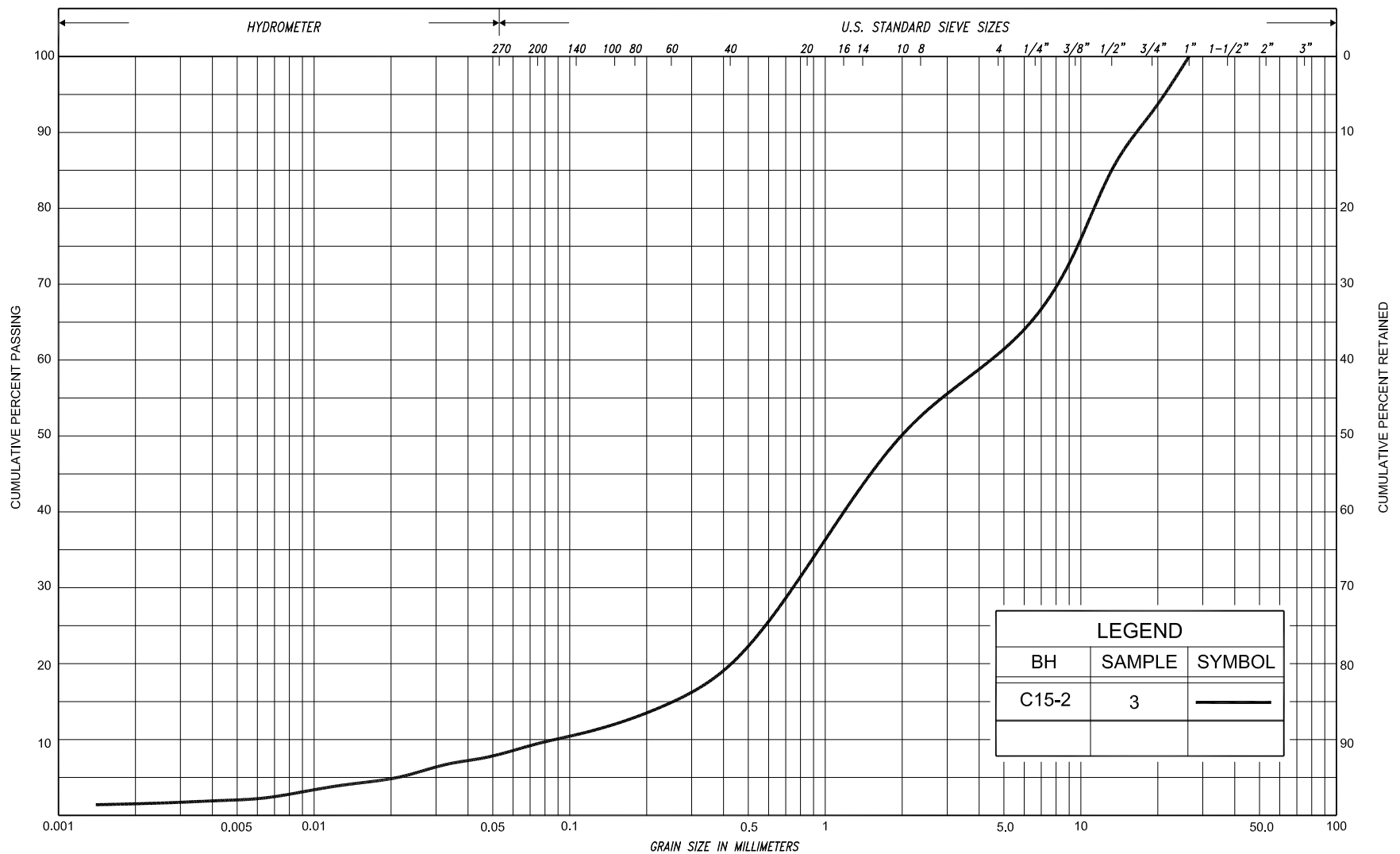
1 of 1

METRIC

G.W.P.	<u>5222-05-00</u>	LOCATION	<u>Coords: 5 181 469.3 N; 354 644.1 E</u>	ORIGINATED BY	<u>F.P.</u>
DIST	<u>Algoma</u>	HWY	<u>129</u>	BOREHOLE TYPE	<u>Tripod- 140 lb Hummer Dynamic Cone</u>
DATUM	<u>Geodetic</u>	DATE	<u>November 23, 2015</u>	CHECKED BY	<u>C.N.</u>

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT	NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20    40    60    80    100										W <sub>p</sub> W                      W <sub>L</sub>		
								SHEAR STRENGTH kPa ○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE										WATER CONTENT (%)		
351.8 0.0	Ground Surface	○																		
	_____ organics _____	○																		
	Sand and gravel	○	1	SS	7															
351.0 0.8	Loose to Brown very dense	○	2	SS	50/15cm															
	End of borehole																			
	Refusal on boulders																			
	 *    Borehole dry																			
	NOTE: No cave-in after completion of drilling															Borehole moved and re-drilled 2 times with same refusal. Refer to auger probes C15-4A and C15-4B.				





SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL		COBBLES	UNIFIED
CLAY	SAND			FINE		MEDIUM		COARSE	GRAVEL		COBBLES	M.I.T.
	SILT			V. FINE		FINE		MED.	COARSE	GRAVEL		U.S. BUREAU



## GRAIN SIZE DISTRIBUTION

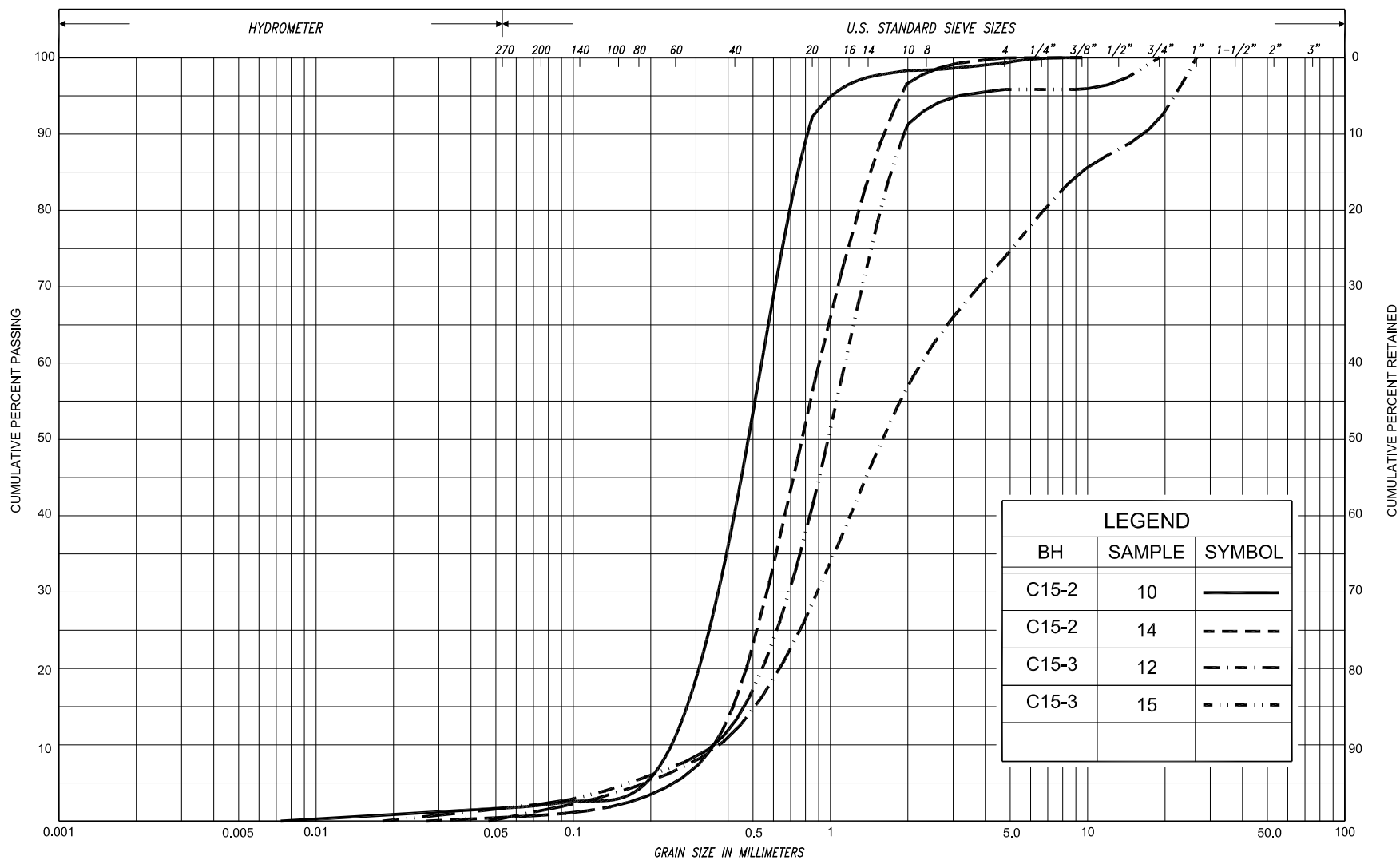
### SAND and GRAVEL (FILL)

FIG No. NC-GS-1

HWY: 129

G.W.P. No. 14TF038





SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT																
CLAY			SILT			V. FINE		FINE		MED.		COARSE		GRAVEL			U.S. BUREAU
					SAND												



## GRAIN SIZE DISTRIBUTION

SAND, trace to with gravel

FIG No. NC-GS-3

HWY: 129

Project No. 14TF038



**FOUNDATION DESIGN REPORT**

**for**

**SHARPSAND RIVER TRIBUTARY CULVERT REPLACEMENT**

**HIGHWAY 129**

**TOWNSHIP OF STURGEON, ONTARIO**

**AGREEMENT NO. 5013-E-0040**

**G.W.P. 5222-05-00**

**SITE NO. 38S-331**

**WP NO. 5148-14-01**

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Index No.: 035FDR  
GEOCRES No.: 41J-99  
March 29, 2016



## TABLE OF CONTENTS

### PART B - FOUNDATION DESIGN REPORT

6. INTRODUCTION .....	8
7. PROJECT DESCRIPTION .....	8
8. FOUNDATION RECOMMENDATIONS .....	10
8.1 Staged Construction .....	10
8.2 Geotechnical Bearing Resistances .....	11
8.3 Settlement Considerations .....	12
8.4 Subgrade Preparation and Culvert Bedding .....	12
8.5 Modulus of Subgrade Reaction .....	14
8.6 Sliding Resistance .....	14
8.7 Frost Depth .....	14
8.8 Seismic Considerations .....	15
8.9 Culvert Backfill .....	15
8.10 Headwalls and Cut-Off Wall .....	17
9. CONSTRUCTION CONSIDERATIONS .....	18
9.1 Excavation .....	18
9.2 Roadway Protection.....	19
10. Groundwater Control .....	20
11. CLOSURE .....	22

Appendix FDR-A – Sharpsand River Tributary Culvert General Arrangement  
Sharpsand River Tributary Culvert Staging Details

Appendix FDR-B – List of Standard Specifications Relevant to Report  
Non-Standard Special Provisions (NSPP's)

**PART B - FOUNDATION DESIGN REPORT**  
for  
Sharpsand River Tributary Culvert Replacement  
Highway 129 (Site No. 38S – 331/C)  
Township of Sturgeon, Ontario  
GWP 5222-05-00, WP No. 5148-14-01

---

**6. INTRODUCTION**

This Foundation Design Report is solely for the use of AECOM Canada Ltd. for the detail design of this specific project on behalf of the Ministry of Transportation and shall not be used for any other purposes or by any other parties including the construction contractor. Refer to the associated contract drawings for design requirements.

Where comments are made on construction, they are provided solely to identify aspects that could affect the design of the project. Construction contractors should make their own assessment of the factual information provided in the Foundation Investigation portion of this report for their decisions related to construction including, but not limited to, equipment selection, proposed construction methods and scheduling.

**7. PROJECT DESCRIPTION**

The existing culvert is a 2.5 m diameter Corrugated Steel Pipe founded on native soil that will be replaced with a 32.0 m long concrete culvert with an opening of 3.0 m by 2.4 m, on the same alignment. No roadway grade changes are planned at this site. PML understands that replacement with new CSP culverts is not being planned. The staging includes temporary longitudinal roadway protection along the centerline of the Highway 129 platform. The use of detours is not considered as an alternative.

The following table summarizes the foundation types considered, their advantages and disadvantages as well as relative cost and risks/consequences.



**Table 7: Advantages and Disadvantages of Culvert Alternatives**

<b>Culvert Type (Alternatives)</b>	<b>Advantages</b>	<b>Disadvantages</b>	<b>Risks/ Consequences</b>	<b>Relative Costs</b>
Precast Concrete Box Culvert	Ease of installation Less time required for construction Less complex dewatering Partial dewatering with installation in the wet is possible More tolerant to settlement than CIP options	Temporary drainage is required while the new culvert is installed along the existing alignment Transportation of culvert segments Limitation of width and height compared to other option Less sliding resistance compared to other options	Differential settlement requiring gaskets between box segments needs to be considered	Cost of temporary drainage installation, if needed, has to be considered  Lesser cost due to shorter construction time, but cost of transportation of segments has to be considered
Cast-in-Place Concrete Box Culvert	More flexibility in sizing Less transportation cost for materials than precast option CIP concrete provides higher sliding resistance than precast concrete	More complex dewatering required than precast concrete box culvert Longer culvert construction schedule than precast concrete box culvert construction Less tolerant to settlement	Differential settlement could cause cracking of concrete in the culvert base and walls	More costly than precast concrete box culvert due to longer construction time May require excavation below water level with risk of flooding into excavation Higher cost for dewatering than for concrete precast box culverts due to requirements for construction in the dry
CIP Open Footings Concrete Culvert	More flexibility in sizing Less transportation cost for materials than precast option CIP concrete provides higher sliding resistance than precast concrete	Longer culvert construction schedule than precast concrete box culvert construction More complex dewatering required than precast concrete box culvert for footing construction below water table Less tolerant to settlement than CIP concrete box culvert	Increased importance of positive dewatering at strip footings Increases risk of flooding of excavation	Higher cost for dewatering than for concrete precast box culverts due to requirements for construction in the dry  Cost of temporary drainage installation has to be considered



The selected culvert type depends on the construction staging and traffic interruption constraints, the hydraulic capacity and size of the existing and proposed culvert and other consideration. From a foundation engineering viewpoint the precast box culvert alternative is recommended since this option will be less susceptible to differential settlement and has relatively easier construction requirements than CIP options.

## **8. FOUNDATION RECOMMENDATIONS**

The invert levels of the proposed culvert are specified to be near Elevation 350.19 at the north end (inlet) and Elevation 349.75 at the south end (outlet). The proposed road grade at the proposed culvert will be about Elevation 354.8 indicating that the soil cover above the culvert will be approximately up to 2.3 m.

### **8.1 Staged Construction**

Staged construction will be required to remove the existing culvert and to install the new Sharpsand River Tributary culvert while maintaining traffic on Highway 129. AECOM provided the preliminary staged construction drawings (Sharpsand River Tributary Sta. 19+860, Sturgeon TWP. Culvert Staging Details February 2016) for the one Single Lane Traffic that is included in Appendix FDR-A. Two construction stages are identified for dewatering and traffic control for replacing and removing the existing culvert.

Stage 1:

- a) Install dewatering system and maintain flow through a temporary bypass
- b) Install traffic control measures to maintain single lane traffic on south side of highway
- c) Remove north portion of existing barrel
- d) Install north portion of culvert

Stage 2:

- e) Maintain dewatering system and flow through temporary bypass
- f) Install traffic control measures to maintain single lane traffic on north side of highway
- g) Remove south portion of existing barrel
- h) Install south portion of culvert
- i) Remove temporary bypass piping





It is considered that the proposed staged construction scheme is feasible at this site, provided that adequate roadway protection is provided. Recommendations for roadway protection are provided in Section 9.2 of this report.

## **8.2 Geotechnical Bearing Resistances**

The OPSS1010 Granular A or Granular B Type II bedding materials and the underlying cohesionless soils that are in the zone of influence below the design subgrade level are considered capable of adequately supporting the stress imposed by the replacement concrete box culvert.

The precast culvert bedding should be 300 mm thick and consist of Granular A or Granular B Type II with a maximum particle size of 37.5 mm. Alternatively, 19 mm Type II clear stone meeting OPSS.PROV 1004 physical and gradation specifications can be utilized for granular bedding and levelling course provided that this material is wrapped with filter fabric to prevent migration of fines from the native soils and ultimately potential failure of the culvert. The Granular B Type II is preferred in wet construction conditions. A leveling course approximately 100 mm thick should also be incorporated in the bedding thickness, if required.

The founding level of the culvert is approximately at Elevation 349.3 to 349.7 m allowing for 0.5 m thick culvert base concrete (assuming 200 mm) and 300 mm thick bedding below the invert levels. The recommended factored geotechnical bearing resistance at ultimate limit states (ULS) and the geotechnical reaction at serviceability limit states (SLS) for the 3.0 m wide concrete box culvert constructed on the native cohesionless soils are as follows:

**Table 8.2: Recommended Maximum Geotechnical Bearing Resistances**

<b>Culvert Section</b>	<b>Subgrade Soil Type</b>	<b>Factored Geotechnical Resistance at ULS (kPa)</b>	<b>Geotechnical Reaction at SLS (kPa)</b>
Entire Length	Typically compact to very dense cohesionless sand and gravel	500	350



The geotechnical reaction resistances were computed according with the CHBDC and the reaction at SLS normally allows for a 25 mm compression of the founding medium. A foundation embedment depth of 2.0 m and groundwater at about 0.5 m above the level of the culvert invert were assumed for computation of the geotechnical resistance.

### **8.3 Settlement Considerations**

Since it is not planned to raise the existing roadway grades and no new added loads will be imposed on the existing compact to very dense sand and gravel subgrade, it is anticipated that only negligible settlements will occur under the replacement culvert.

For design purposes, the estimated magnitude of differential settlements under the culvert is 10 to 15 mm and should be considered to occur during construction. This settlement is considered tolerable for the proposed precast box concrete culvert and should not require cambering. Construction joints should be added to accommodate the estimated 15 mm of differential settlement.

### **8.4 Subgrade Preparation and Culvert Bedding**

Excavations and preparation of the subgrade for construction of the culvert should be performed and monitored in accordance with OPSS 902 (Construction Specification for Excavation and Backfilling - Structures). All of the cobbles and boulders should be removed from the subgrade level. A site review should be conducted by qualified geotechnical personnel during preparation of the subgrade and compaction of the granular fill.

For the box culvert, it is recommended to provide a 300 mm thick granular bedding below the culvert. The bedding material should comprise Granular A or Granular B Type II material, satisfying the specifications within OPSS.PROV 1010 (Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material), compacted to 95% of the ASTM D-698 (standard Proctor) maximum dry density in conformance to OPSS 501 (Construction Specification for Compacting).



The geometry of the subgrade preparation, cover backfill and frost taper treatment for the box culverts should be carried out in accordance with OPSS 422 (Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut), MTO SP 422S01 (Precast Concrete Box Culvert) and MTOD 803.021 (Bedding and Backfill for Precast Concrete Box Culverts).

Topsoil, organics surficial material and any other deleterious soils revealed during the subgrade preparation should be excavated and cobbles and boulders also be removed prior to placement of the granular base below the box culvert, and the materials replaced with compacted Granular A or Granular B Type II. Granular B Type II should be preferred for construction under wet conditions. In addition, any boulders encountered immediately below the subgrade level should be removed to avoid point loads.

In case the construction is carried out under wet conditions and provided that the water level is below the culvert invert level, the bedding may consist of 19 mm Type II clear crushed stone according to OPSS.PROV 1004 all wrapped in a geotextile fabric to prevent loss of ground and potential settlement of the culvert. The geotextile should conform to OPSS 1860 and consist of a Class II non-woven geotextile with a filtration opening size (FOS) of 50 to 100  $\mu\text{m}$ . The filter fabric should be placed beneath and wrap over the clear stone bedding. Alternatively, the subgrade should be immediately covered with a layer of biaxial geogrid (25 by 35 mm maximum aperture / 1.2 to 2.0 kN/m minimum peak tensile strength). Granular B Type II (with a maximum particle size of 37.5 mm) should be used to backfill over the geogrid where required to bring the grade up to the bottom of the granular bedding. To obtain adequate compaction of the granular bedding material under water, the granular bedding may be placed to a level about 300 mm above the design grade to be above the estimated groundwater level and be compacted from the surface. The grade should then be lowered to the design level of the bedding by carefully excavating the excess bedding material. Under wet conditions the levelling course may not be required. It is recommended to provide the minimum 300 mm thickness of granular bedding above the geogrid.



## 8.5 Modulus of Subgrade Reaction

The estimated value of the modulus of subgrade reaction for a box culvert constructed on the recommended compacted base materials over the native soils is as follows.

**Table 8.5: Modules of Subgrade Reaction**

Soil Type	Modulus of Subgrade Reaction (MN/m <sup>3</sup> )
Granular A or Granular B Type II or 19 mm clear stone, Type II	45
Compact to dense sand and gravel	20

## 8.6 Sliding Resistance

The following parameters should be used to compute the sliding resistance of precast box culvert and precast cut-off wall foundations. The friction angles have been reduced by a factor of 0.67 for precast box culvert foundations to account for the smooth concrete base.

**Table 8.6: Sliding Resistance Parameters**

Soil Type	Foundation Friction Angle (Degrees)		Cohesion (kPa)	Unit Weight (kN/m <sup>3</sup> )
	Cast-In-Place	Precast		
Granular A or Granular B Type II or 19 mm clear stone Type II	35	23	0	22.8
				21.0
Compact to dense sand and gravel	30	20	0	19.0

The structural designer should use a factor of 0.8 for the above values of friction angle and cohesion when performing the sliding resistance check.

## 8.7 Frost Depth

The foundation frost depth for structure foundations at this site is 2.3 m, according to the OPSD 3090.100 (Foundation, Frost Penetration depths for Northern Ontario).



## **8.8 Seismic Considerations**

The reference Peak Ground Acceleration (PGA) for the project site is 0.036 for both Town of Chapleau and the City of Sault Ste. Marie, Ontario (National Building Code of Canada, 2015). The soil at this site for seismic design purposes is classified as Type D in accordance with Clause 4.4.3.2, CHBDC 2014.

## **8.9 Culvert Backfill**

Backfill adjacent to the culvert should be placed in accordance with OPSS 422 (Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut), MTO SP 422S01 (Precast Concrete Box Culvert) and MTOD 803.021 (Bedding and Backfill for Precast Concrete Box Culverts).

Backfill should be brought up simultaneously on each side of the culvert and operation of heavy equipment within 1.0 times the height of the culvert (each side) should be restricted to minimize the potential for movement and/or damage of the culvert due to the lateral earth pressure induced by compaction. Refer to OPSS 902 (Construction Specification for Excavating and Backfilling - Structures) for additional comments.

The new culvert must be designed to support the stress imposed by the overlying fill as well as to resist the unbalanced lateral earth pressure and compaction pressure exerted by the backfill adjacent to the culvert walls.

The lateral earth and water pressure,  $p$  (kPa), should be computed using the equivalent fluid pressures presented in Section 6.12 of the *Canadian Highway Bridge Design Code (CHBDC)* or employing the following equation assuming a triangular pressure distribution.

$$p = K (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_p + C_s$$

Where  $p$  = lateral earth pressure (kPa)

$K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of backfill material above design water level (kN/m<sup>3</sup>)

$\gamma'$  = unit weight of submerged backfill material below design water level (kN/m<sup>3</sup>)  
=  $\gamma - \gamma_w$



$\gamma_w$  = unit weight of water  
 = 9.8 kN/m<sup>3</sup>  
 $h_1$  = depth below final grade (m), above design water level  
 $h_2$  = depth below design water level (m)  
 $q$  = any surcharge load (kPa)  
 $C_p$  = compaction pressure (refer to clause 6.12.3 of *CHBDC*)  
 $C_s$  = earth pressure induced by seismic events, kPa (refer to clause 4.6.5 of *CHBDC*)  
 Where  $\phi$  = angle of internal friction of retained soil (35° for Granular A or B Type II)  
 $\delta$  = angle of friction between soil and wall (23.5° for Granular A or B Type II)

The seismic site coefficient for the conditions at this site was provided in Section 8.8 of this report.

The following parameters are recommended for design:

**Table 8.9: Geotechnical Earth Pressures for Lateral Resistances**

Parameter	Granular A or Granular B Type II	Excavated Material (*)
Angle of Internal Friction, degrees	35	30
Unit Weight, kN/m <sup>3</sup>	22.8	19.0
Coefficient of Active Earth Pressure ( $K_a$ )	0.27	0.33
Coefficient of Earth Pressure At Rest ( $K_o$ )	0.43	0.50
Coefficient of Passive Earth Pressure ( $K_p$ )	3.69	3.0

(\*) Assumes that excavated materials used for backfill are inorganic cohesionless soils

The design should consider both the maximum water level in the stream and the stabilized groundwater level condition. The maximum stream water level (HWL) will be dictated by flood flow conditions and should be defined by the project hydrological engineer.

The coefficient of earth pressure at rest should be employed to design rigid and unyielding walls.



### **8.10 Headwalls and Cut-Off Wall**

Inlet and outlet protection should be constructed in general accordance with OPSS 511 (Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting), OPSS.PROV 1004 (Material Specification for Aggregates - Miscellaneous) and OPSD 810.010 (General Rip-Rap Layout Sewer and Culvert Outlets) to prevent erosion adjacent to the culvert as well as scour.

The inlet cut off protection may consist of vertical cut-offs and structural head walls. The outlet protection may consist of structural head walls.

For the design of headwalls and cut-off wall, the previous recommendations and geotechnical parameters for culvert foundations and backfill should be utilized for the design of the foundations. Backfill adjacent to the headwall and cut-off wall should be placed in accordance with OPSD 3121.150 (Walls, Retaining, Backfill, Minimum Granular Requirement). The wall founding levels should match those of the culvert where the walls are designed integral with the culvert structure. For walls designed separately from the culvert structure, the founding levels should be established with minimum 2.1 m of earth cover for adequate frost protection.

Vertical cut-off walls might be constructed of pre-cast concrete and extend vertically from the invert of the culvert to a minimum depth below the culvert invert equivalent to the height to the high water level above the culvert invert and also horizontally across the width of the culvert and backfill zone. Headwalls could be constructed of CIP concrete and extend vertically from the high water level (HWL) to the foundation depth and horizontally across the top of the culvert and a minimum distance on each side of the culvert equivalent to the height of the HWL above the culvert invert. However, slope protection and horizontal apron inlet and outlet protection could be considered instead of structural headwalls. The rock protection should provide for horizontal apron inlet and outlet protections in conformance to OPSS 511 (Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting) with a minimum dimension of 0.3 m and a minimum thickness of 0.5 m and should extend to a minimum of 0.3 m above the culvert obvert level.



The design of the culvert walls should be checked for sliding resistance using the geotechnical parameters provided previously in Table 8.9 for cast-in-place concrete foundations.

A weeping tile system and/or weep holes should be installed to minimise the build-up of hydrostatic pressure behind the culvert walls. The weeping tiles should be surrounded by a properly designed granular filter or non-woven Class II geotextile with an FOS of 75-150  $\mu\text{m}$  according to OPSS 1860 (Material Specification for Geotextiles) placed to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade.

## **9. CONSTRUCTION CONSIDERATIONS**

### **9.1 Excavation**

Surface water should be diverted away from open excavations and all excavations should be carried out in accordance with the Occupational Health and Safety Act and Regulations for Construction Projects. Stockpiles of excavated material should be set back from the edge of the excavation by a distance at least equal to the excavation depth.

Excavation to the anticipated founding level of the proposed Sharpsand River Tributary replacement culvert is expected to extend through the pavement structure into cohesionless fill material and continued into a native sand and gravel deposit. Subject to adequate groundwater control, excavation of the soils should be feasible using conventional equipment. All excavations should be conducted in accordance with OPSS 902 (Excavation and Backfilling of Structures).

The existing fill material and compact native soils are classified as Type 3 soil and the dense to very dense sand and gravel is classified as Type 2 soil, according to the Occupational Health and Safety Act (Ont. Reg. 213/91) classification system. Temporary unsupported excavations within the Types 3 soils above the groundwater level should not be steeper than 1 horizontal to 1 vertical (1H:1V). Where composite soil types exist, the excavation slopes should be cut to the requirements of the soil type with the highest number that is present in the slope according to OHSAA. Below the water table, cut slopes should be shaped at 3H:1V or flatter.





The associated NSSP - Variable Mixed Fill at Embankments provided in Appendix FDR-B should be included in the contract documents to advise the Contractor of potentially challenging conditions for excavation and for installation of shoring as cobbles, boulders may be encountered within the ground.

Assuming that the excavation are kept free of water during construction, 2H:1V side slopes are anticipated to be temporarily stable based on the condition of the subgrade and embankment fill material.

## **9.2 Roadway Protection**

An adequate temporary support system should be provided at all locations where space limitations prevent construction of sufficiently shallow slopes or where required to provide protection to existing buried services, roadways or other existing facilities. Any existing services present within the road right-of-way were likely constructed using open cut methods and the service trenches are anticipated to be backfilled with variable fill materials. Alternatively, temporary shoring systems may be used to support the sidewalls of the excavations in these areas.

Temporary roadway protection, where required, should be constructed in accordance with OPSS 539 (Temporary Protection Systems). A minimum performance level of 2, according to OPSS 539 will be assumed. The design should consider both the maximum water level and the stabilised groundwater level condition. Refer to Section 8.9 for recommended soil parameters for temporary roadway protection/shoring design. The Contractor should be responsible for the selection, detailed design and performance of the roadway protection scheme.

Trench and excavation slopes should be inspected as required, particularly following periods of heavy rainfall or snow melting. Stockpiling or surcharging near excavations should not be permitted as this may induce excavation slope or basal failures.

Alternative roadway protection schemes such as sheet piling or anchored soldier piles and lagging should be considered. Typically, sheet piling can be used to reduce loss of native soils below the water table. Soldier piles and lagging are generally considered suitable for applications above



groundwater table in cohesive materials but can be used for cohesionless fill materials provided that adequate care to prevent loss of ground from behind the shoring is implemented by the Contractor.

The following table presents an overview assessment of the advantages and disadvantages, including relative costs and risk/consequences of the roadway protection system alternatives from the foundation perspectives at the Sharpsand River Tributary culvert site.

**Table 9.2: Alternative Roadway Protection Comparison System**

Alternatives	Advantages	Disadvantages
Sheet piles	<ul style="list-style-type: none"> <li>• Sheet piles will be interlocked, therefore, loss of native soils will be negligible</li> <li>• Suitable for high water table conditions</li> <li>• Suitable to drive for varying subsoil profile, if required</li> <li>• Low risk of soil loss</li> </ul>	<ul style="list-style-type: none"> <li>• Higher cost</li> <li>• May require soil anchors/rakers for lateral support</li> <li>• Larger construction equipment is required than for soldier piles</li> </ul>
Soldier piles and lagging	<ul style="list-style-type: none"> <li>• Lower cost</li> <li>• Smaller construction equipment is required than for sheet piles</li> </ul>	<ul style="list-style-type: none"> <li>• Requires care to avoid excessive settlement that may occur due to loss of cohesionless soils</li> <li>• Unsuitable with high water table</li> <li>• High risk of soil loss</li> </ul>

Based on the above table, sheet piles are considered to be adequate due to the presence of cohesionless fill materials with a high water table and consequently, high risk of soil loss leading to excessive movement. This site condition implies that the roadway protection should not consist of a soldier piles and lagging at this site, unless care is taken during and after installation to avoid loss of soil between and behind the lagging boards.

## **10. GROUNDWATER CONTROL**

The stabilized groundwater level is expected to be consistent with the water level in the Sharpsand River Tributary, near Elevation 350.5 about 1.0 m above the inferred culvert subgrade level. It will be necessary to implement measures to temporarily lower groundwater table and to permit construction in-the-dry. The groundwater level should be lowered to a minimum of 0.5 m below the proposed founding levels to allow construction in the dry.



It is considered that a dewatering system based on dam and pumping techniques will generally be sufficient. A dewatering system should be placed at the each end of the construction areas. Depending on the culvert type being installed, water level conditions at the time of construction and the construction techniques, a cofferdam may be required. The contractor is responsible for the selection, performance and detailed design of the dewatering system including the detail design of any cofferdam. Any seepage water that may enter during the excavation from the excavation slope may be handled by sump pumping techniques. The dewatering system should be designed to conform the requirement of OPSS 517 (Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation) and OPSS 518 (Construction Specification for Control of Water from Dewatering Operations) in addition to the NSSP provided in Appendix FDR-B.

In accordance with the Ontario Water Resources Act, Water Taking and Transfer Regulation 387/04, a Permit to Take Water (PTTW) from the Ministry of Environment is required if the dewatering discharge is greater than 50,000 L/day. The expected daily flows at the culvert location should be assessed to determine if this permit will be necessary. It may be prudent to obtain the PTTW to avoid delays should the PTTW become necessary during construction.

From the Foundations standpoint, the requirement for a permit to take water (PTTW) will depend on the water tightness of the contractor's selected type of dewatering system. The PTTW requirement will also depend on the groundwater levels at the time of construction since these are subject to seasonal fluctuations and precipitation patterns. A PTTW may be required to address other engineering facets, such as those of Hydrology engineering.

For general reference only, we note that a PTTW will likely be required at this site considering that the estimated volume of water to remove or transfer to lower the groundwater approximately 1.5 m below the recorded level (noted above) to obtain stable subgrade conditions for construction in-the-dry is some 150,000 liters. This estimated volume assumes construction inside two suitably sized cofferdams to contain the new culvert sections during construction staging.

Groundwater levels are subject to seasonal fluctuations and precipitation patterns.



## 11. CLOSURE

The Preliminary Foundation Design portion of this report was prepared by Mr. M. Khorsand, BSc, EIT, and reviewed by Mr. G.O. Degil, PhD, P.Eng., Senior Engineer, Geotechnical Services. The report was independently reviewed by Mr. C.M.P. Nascimento, P.Eng., Principal Consultant.

Yours very truly,

Peto MacCallum Ltd.

A handwritten signature in blue ink, appearing to read "Mansoor", is located below the "Yours very truly," text.

Mansoor Khorsand, BSc, EIT  
Project Supervisor, Geotechnical Services



Grigory O. Degil, PhD, P.Eng.  
Senior Engineer, Geotechnical Services



Carlos M.P. Nascimento, P.Eng  
Principal Consultant  
Manager, MTO Foundation Services

MKH/DD/CN.mkh-jk



## **APPENDIX FDR-A**

Sharpsand River Tributary Culvert General Arrangement

Sharpsand River Tributary Culvert Staging Details



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

CONT No 2016-5146  
WP No 5148-14-01

SHARPSAND RIVER TRIBUTARY  
STA. 19+860, STURGEON TWP.  
CULVERT STAGING DETAILS



SHEET  
24

AECOM

NOTES :

1. FOR GENERAL NOTES SEE DRAWING 1.
2. THE LAYOUT AND DETAILS OF THE CONSTRUCTION STAGING ARE SCHEMATIC AND SHOWN FOR REFERENCE PURPOSES ONLY. THE CONTRACTOR SHALL DESIGN AND CONSTRUCT THE TRAFFIC/CONSTRUCTION STAGING TO SUIT THE FULL REQUIREMENTS OF THE WORK.
3. INSTALL TOP OF PROTECTION SYSTEM 50mm BELOW EXISTING GRADE TO PERMIT FULL TRAFFIC LANES AT END OF WORKING DAY.
4. THE CONTRACTOR SHALL PLAN THE CONSTRUCTION SEQUENCE AND CULVERT ARRANGEMENT TO ACCOMMODATE THE SKEW OF THE PROTECTION SYSTEM. THIS MAY INCLUDE THE REMOVAL AND MODIFICATIONS OF THE BOTTOM PORTION OF THE PROTECTION SYSTEM.
5. ROADWAY PROTECTION TO BE DESIGNED BY THE CONTRACTOR TO PERFORMANCE LEVEL 2.
6. EXTEND PROTECTION SYSTEM WITH LOCAL STEEL PLATES AND ATTACHMENTS ABOVE GRADE ADJACENT TO EXCAVATION, TO LATERALLY RESTRICT MOVEMENT AT THE TEMPORARY CONCRETE BARRIER. MODIFY FOR EACH STAGE.
7. THE CONTRACTOR SHALL DESIGN AND INSTALL VERTICAL SUPPORTS IN BARREL TO MAINTAIN STRUCTURAL INTEGRITY OF CUT BARREL. FIELD MEASURE MEMBERS FOR TIGHT FIT, AND FIT TIMBER WEDGES TO FIRMLY SECURE.
8. ARRANGEMENT OF THE DEWATERING SYSTEM CONCEPT IS NOT SHOWN IN THIS DRAWING, SEE DRAWING 5.
9. FOR USE OF SKEWED BOX UNITS SEE NOTE 5, DRAWING 4.
10. FOR ROAD WORKS SEE HIGHWAY ENGINEERING DRAWINGS.

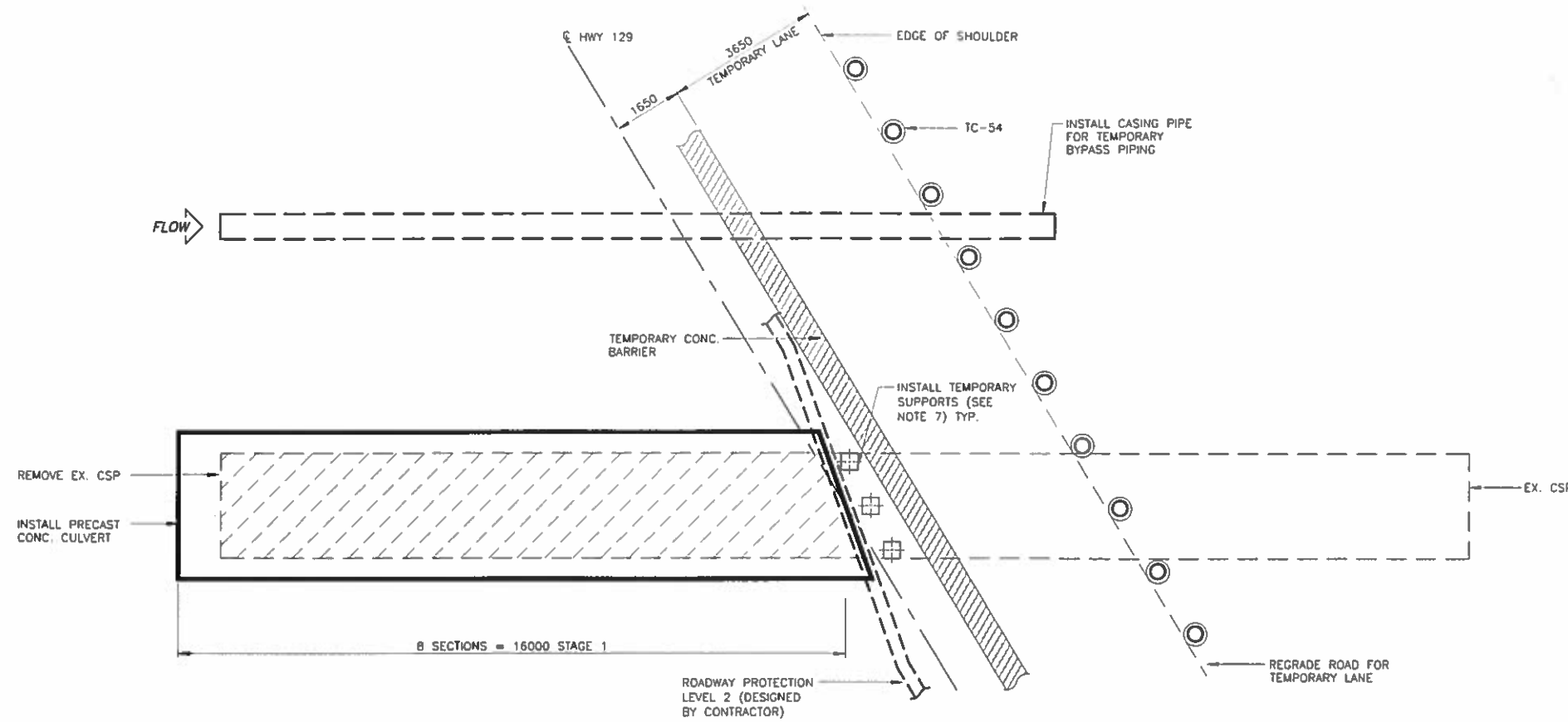
SEQUENCE OF WORK :

STAGE 1 :

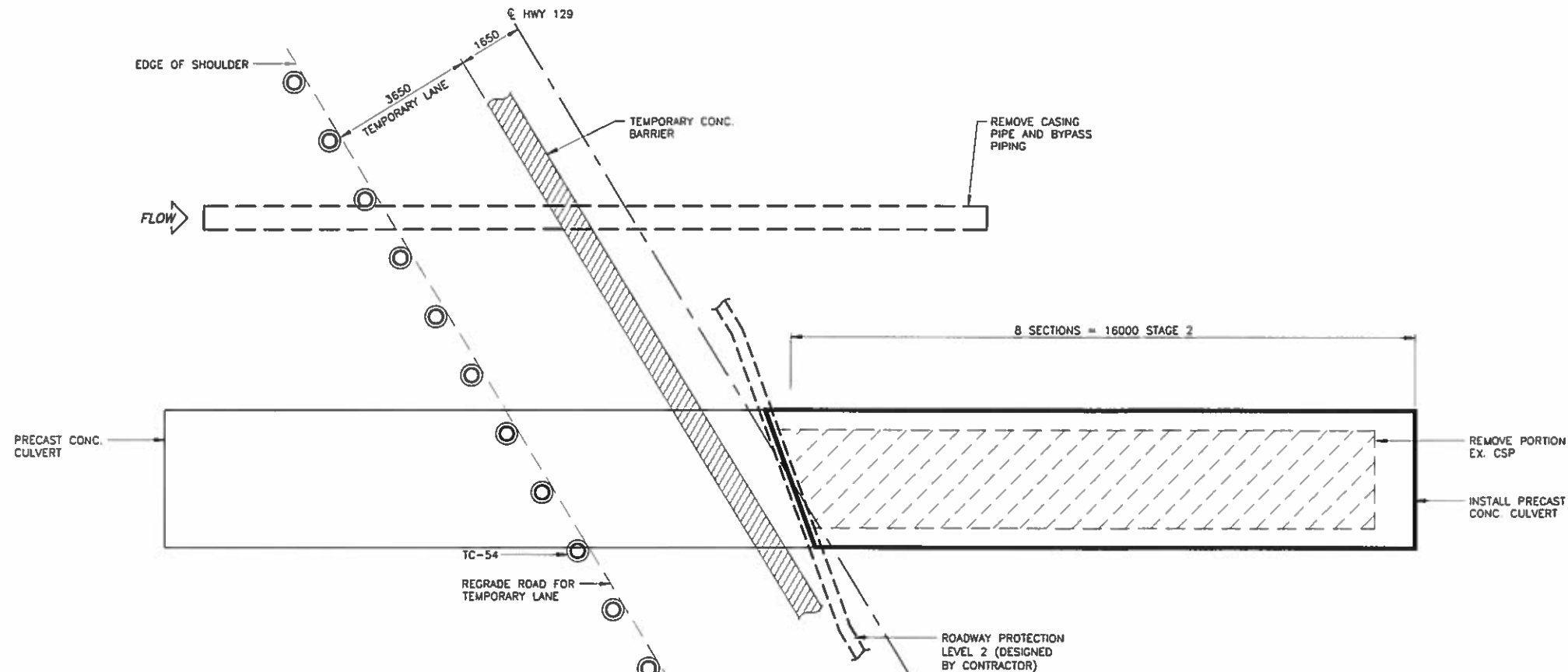
- INSTALL DEWATERING SYSTEM AND MAINTAIN FLOW THROUGH TEMPORARY BYPASS.
- INSTALL TRAFFIC CONTROL MEASURES TO MAINTAIN SINGLE LANE TRAFFIC ON SOUTH SIDE OF HIGHWAY.
- REMOVE NORTH PORTION OF EXISTING BARREL.
- INSTALL NORTH PORTION OF CULVERT.
- COMPLETE ALL OTHER WORK DETAILED AND SPECIFIED IN THE CONTRACT DOCUMENTS.

STAGE 2 :

- MAINTAIN DEWATERING SYSTEM AND FLOW THROUGH TEMPORARY BYPASS.
- INSTALL TRAFFIC CONTROL MEASURES TO MAINTAIN SINGLE LANE TRAFFIC ON NORTH SIDE OF HIGHWAY.
- REMOVE SOUTH PORTION OF EXISTING BARREL.
- INSTALL SOUTH PORTION OF CULVERT.
- REMOVE TEMPORARY BYPASS PIPING.
- COMPLETE ALL OTHER WORK DETAILED AND SPECIFIED IN THE CONTRACT DOCUMENTS. REMOVE DEWATERING SYSTEM.



PLAN - STAGE 1  
1 : 150



PLAN - STAGE 2  
1 : 150

DRAWING NOT TO BE SCALED  
50 mm ON ORIGINAL DRAWING

REVISIONS		DATE	BY	DESCRIPTION
DESIGN	JP	CHK	GM	CODE CHBDC 2014 LOADCL-625-ONT
DRAWN	TG	CHK	JP	SITE 385-331/C STRUCT SCHEME DWG 3



## **APPENDIX FDR-B**

List of Standard Specifications Relevant to Report  
Non-Standard Special Provisions (NSPP's)





### LIST OF STANDARD SPECIFICATIONS RELEVANT TO REPORT

DOCUMENT	TITLE
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS 518	Construction Specification for Control of Water from Dewatering Operations
OPSS 539	Temporary Protection Systems
OPSS 902	Excavation and Backfilling of Structures
OPSS.PROV 1010	Material Specification for Aggregates - Base, Subbase, Select Subgrade, and Backfill Material
OPSS.PROV 1004	Material Specification for Aggregates - Miscellaneous
OPSS1860	Material Specification for Geotextiles
OPSD 810.010	General Rip-Rap Layout Sewer and Culvert Outlets
OPSD 3090.100	Foundation, Frost Penetration depths for Northern Ontario
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement



### **NON-STANDARD SPECIAL PROVISIONS (NSSP)**

#### **NSSP – Surface Water Control and Dewatering (Addition to OPSS 518)**

The Contractor shall take measures for necessary surface water diversions and drainage and to lower the prevailing groundwater level a minimum of 0.5 m below the base of the excavations for work in-the-dry in overburden and to the bedrock surface for work in-the-dry in bedrock, if applicable.

In view of the relatively pervious subsoil conditions encountered at this site, the dewatering design and the implementation should prevent unsafe conditions, such as sloughing and boiling under unbalanced groundwater conditions. Although the Contractor shall be responsible for designing and implementing measures for surface water control and dewatering, the Contractor is advised that damming of the drain and diversion of the flow through pumping through temporary conduits to accommodate construction staging will probably be required at this site.