



FOUNDATION INVESTIGATION AND DESIGN REPORT

for

SHARPSAND RIVER TRIBUTARY CULVERT REPLACEMENT

HIGHWAY 129

TOWNSHIP OF RIOUX, ONTARIO

AGREEMENT NO. 5013-E-0040

G.W.P. 5222-05-00

SITE NO. 38S-373/C

WP NO. 5161-10-01

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PML Ref.: 14TF038
Index No.: 037FIR and 038FDR
GEOCRES No.: 41J-100
March 29, 2016



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PART A - FOUNDATION INVESTIGATION REPORT
for

Replacement of Sharpsand River Tributary Culvert
Highway 129 (Site No.38S-373/C)
Township of Rioux, Ontario
GWP 5222-05-00, WP No. 5161-10-01

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed Sharpsand River Tributary culvert replacement located on Highway 129 at Sta. 10+000, approximately 50.4 km north of the Highway 129/Highway 554 intersection, in the Township of Rioux 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 P4 for general views of the site. The existing Sharpsand River Tributary crossing includes a Structural Plate corrugated steel pipe (SPCSP) culvert and embankments some 4 m high. The Sharpsand River Tributary is about 5m wide at the culvert location and meanders in a relatively flat plain from east to west at the culvert location.

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.

3. INVESTIGATION PROCEDURES

The fieldwork for the current foundation investigation involved six (6) boreholes that were carried out during the period of December 2014 to November 2015. The boreholes were drilled at the approximate locations shown on Drawing SR-1 to depths of 4.0 m to 11.5 m where competent soils were encountered for the new culvert foundation.

The boreholes were advanced using various types of equipment including truck and track-mounted D-53 drill rigs equipped with continuous flight 75 to 150 mm diameter solid and hollow stem augers, respectively. Due to access constraints, the boreholes at the inlet and outlet



were advanced by washboring within a 75 mm diameter casing with portable tripod drilling equipment. The drilling 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. Standard penetration and dynamic cone penetration (DCP) tests were conducted to assess the strength characteristics of the substrata. The results of the field tests and observations during and at completion of drilling are reported on the Record of Borehole sheets. 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 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 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 ten (10) grain size distribution analyses and fourteen (14) 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 distribution charts are presented in Figures SR-GS-1 to SR-GS-4. The test results are summarized on the attached Record of Borehole sheets.



The Borehole Locations Plan and the stratigraphic profile and cross-sections prepared from the current borehole data are shown on Drawings SR-1 and SR-2. 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 CSP culvert is located under an approximately 4.5 m high granular material embankment placed over the native soils. In summary, the subsurface stratigraphy revealed in the current boreholes located at the inlet and outlet of the culvert (Boreholes SR-1 and SR-4, respectively) included 400 mm thick organic layer and 1.4 m thick sand and gravel deposit underlain by an approximately 3.6 m and 4.9 m thick non-cohesive deposit of silty sandy soils with varying gravel particle component.

The subsurface stratigraphy revealed in the boreholes SR-2, SR-3, SR-5 and SR-6 that were advanced from the shoulders of the highway embankment, generally consisted of 0.2 to 0.5 m thick pavement fill underlain by approximately 4.0 m thick granular fill. A minimum 2.1 m to 7.1 m thick non-cohesive silty sand deposit with cobbles and boulders was encountered below the fill.

4.1 Pavements

Shoulder pavements of 80 to 150 mm asphaltic concrete over approximately 320 mm to 450 mm of sand and gravel courses were encountered surficially in Boreholes SR-2, SR-3, SR-5 and SR-6 (Highway 129, SBL and NBL shoulders). The pavements extended to 0.4 to 0.6 m depths, Elevations 363.9 m to 363.7 m.

4.2 Fill

A 3.9 m to 4.1 m thick layer of non-cohesive sand and gravel fill containing cobbles and boulders was encountered below the pavement in Boreholes SR-2, SR-3, SR-5 and SR-6 and extends to Elevations 360.0 m to 359.6 m. The SPT "N"-values ("N"-values) measured within the fill range from 6 blows per 0.30 m of penetration to 50 blows per 0.10 m of penetration, suggesting a compact to very dense compactness. "N"-values of 6 and 9 blows were measured in Boreholes SR-2 and SR-3 at about 3.1 m depth, respectively Elevations 361.1 m and 361.4 m, suggesting a locally loose compactness. Figure SR-GS-1 shows the results of a grain size analysis completed on a sample of this unit.



4.3 Peat

A 400 mm thick organic layer, which was contacted surficially in Borehole SR-1 (culvert outlet) extending from Elevation 359.4 m to Elevation 359.0 m.

4.4 Sand and Gravel

A 1.4 m thick deposit of sand and gravel containing cobbles and boulders was encountered surficially in Borehole SR-4 (culvert inlet) and extended to Elevation 359.6 m.

The “N”-values measured within this deposit ranged from 19 blows per 0.30 m of penetration to 30 blows per 0.03 m of penetration, indicating a compact to very dense compactness.

4.5 Silty Sand

A minimum 2.1 m to 7.0 m thick deposit of silty sand with occasional silt and gravel zones was encountered below the 400 mm thick organic layer, which was contacted surficially in Borehole SR-1 (culvert outlet) at Elevation 359.0 m and below the sand and gravel fill and native layers in Boreholes SR-2 to SR-6 at 1.4 m to 4.5 m depths, Elevations 360.0 m to 359.6 m. All boreholes terminated within this layer at 4.0 m to 11.5 m depths, Elevations 357.6 m to 352.6 m.

“N”-values measured within this deposit varied between 10 blows per 0.30 m of penetration and 50 blows per 0.10 m of penetration, suggesting a compact to very dense compactness. Local “N”-values of 6 and 8 were measured in Borehole SR-3 and SR-4 at about Elevation 359.0 m and 357.0 m, respectively, indicating a locally loose compactness. The results of grain size analyses completed on selected samples of this deposit are shown on Figures SR-GS-2 to SR-GS-4.

4.6 Groundwater

The water level in the Sharpsand River Tributary flows from east to west and was at Elevation 360.0 m at the time of the investigation. The water level in the river governs the water level at the site in view of the relatively pervious native soils. During and upon completion of drilling, water was at 0.3 m to 4.6 m depths, Elevations 360.4 m to 359.1 m. It should be noted that the water level of the river 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 located in Toronto.

This report was prepared by Mr. Mansoor Khorsand, BSc, EIT, Project Supervisor, EIT and reviewed by Mr. Grigory 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.

Mansoor Khorsand, BSc, EIT
Project Supervisor, Geotechnical Services



Grigory 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 north at the location of Borehole SR-4. The inlet of the existing CSP culvert is visible. (January 12, 2015)



Photograph P2: Looking west from Highway 129 southbound lane shoulder. Borehole SR-1 was drilled at north-west of the culvert outlet. (January 12, 2015)



Photograph P3: Looking north from the centre of Highway 129 northbound lane shoulder at the location of Borehole SR-3. (December 4, 2014)

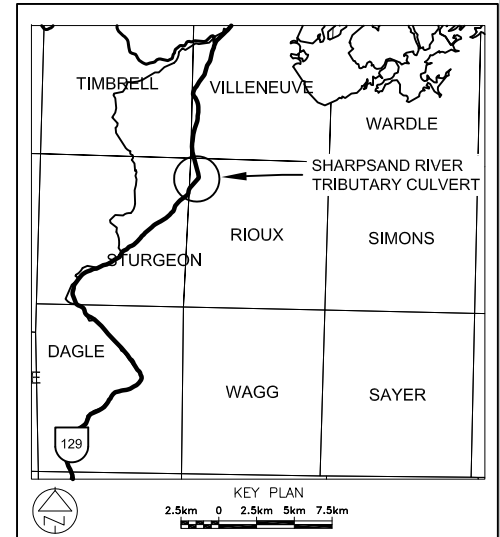
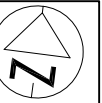


Photograph P4: Looking south from Highway 129 southbound lane shoulder. Borehole SR-5 advanced using a truck-mounted drill rig. (November 18, 2015)



APPENDIX FIR-B

Drawings SR-1 and SR-2
Borehole Locations Plan and Soil Strata at Sharpsand River Tributary Culvert
Explanation of Terms Used in Report
Record of Borehole Sheets: SR-1 to SR-6
Results of Grain Size Distribution Analyses – Figures SR-GS-1 to SR-GS-4



LEGEND			
	Borehole		
	Cone		
	Borehole and Cone		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60 Cone, 475 J/blow)		
	WL at time of investigation Nov. 2014 to Jan. 2015		

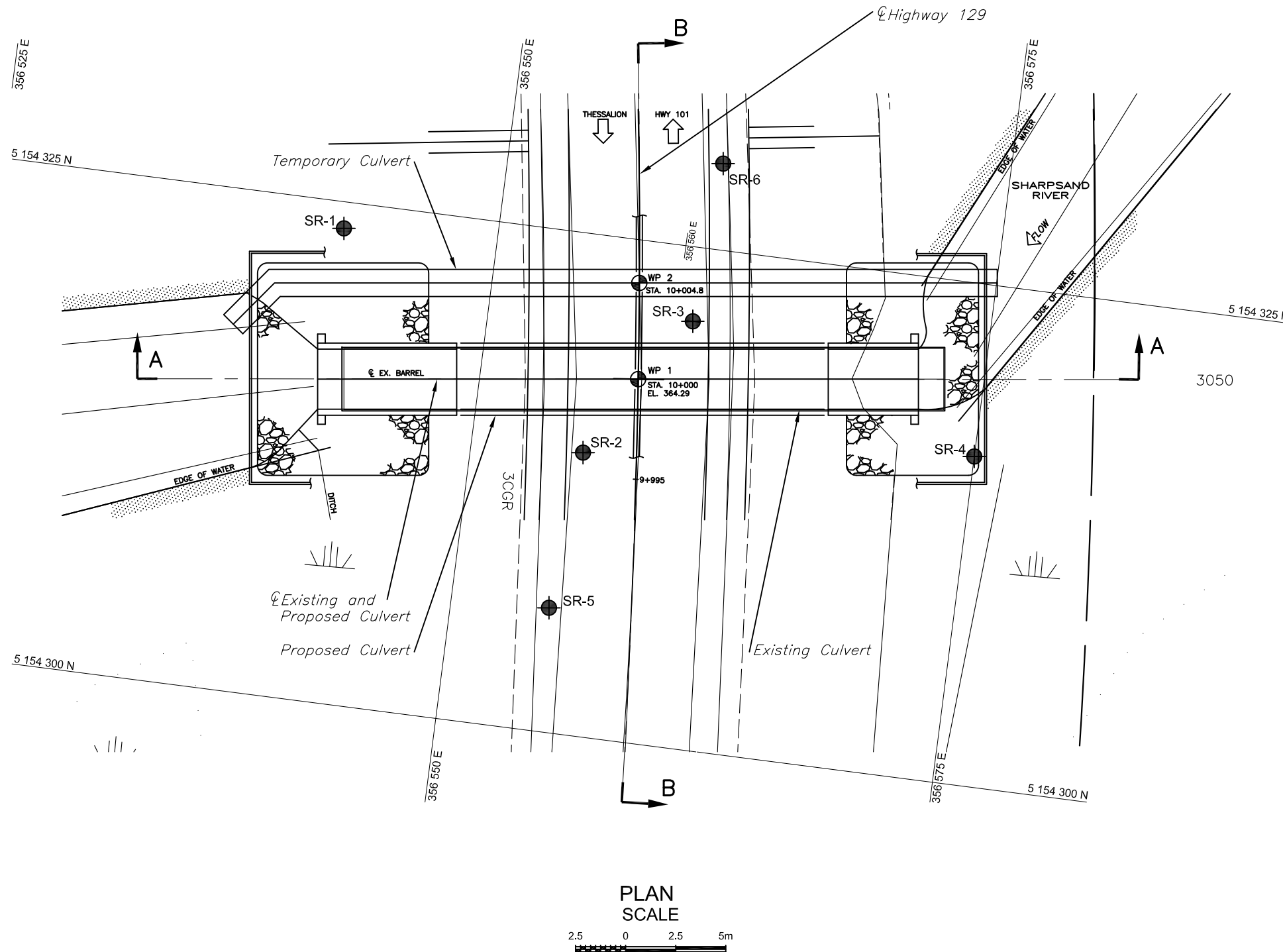
BH No	ELEVATION	NORTHINGS	EASTINGS
SR-1	359.4	5 184 323.7	356 542.3
SR-2	364.2	5 184 314.1	356 555.6
SR-3	364.5	5 184 321.3	356 560.2
SR-4	361.0	5 184 316.4	356 575.0
SR-5	364.1	5 184 306.2	356 554.9
SR-6	364.4	5 184 329.3	356 560.7

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

REVISIONS	DATE	BY	DESCRIPTION

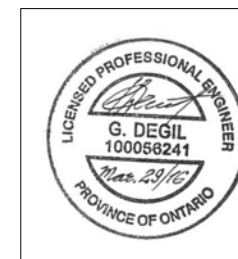
Geocres No. 41J-100

HWY No	129	DIST	ALGOMA
SUBM'D	NA	CHECKED MK	DATE MARCH 29, 2016
DRAWN	NA	CHECKED DD	APPROVED CN
		SITE	385-373/C
		DWG	SR-1



NOTES:

- DRAWINGS SR-1 AND SR-2 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
- REFER TO DRAWING SR-2 FOR PROFILE A-A AND SECTION B-B.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

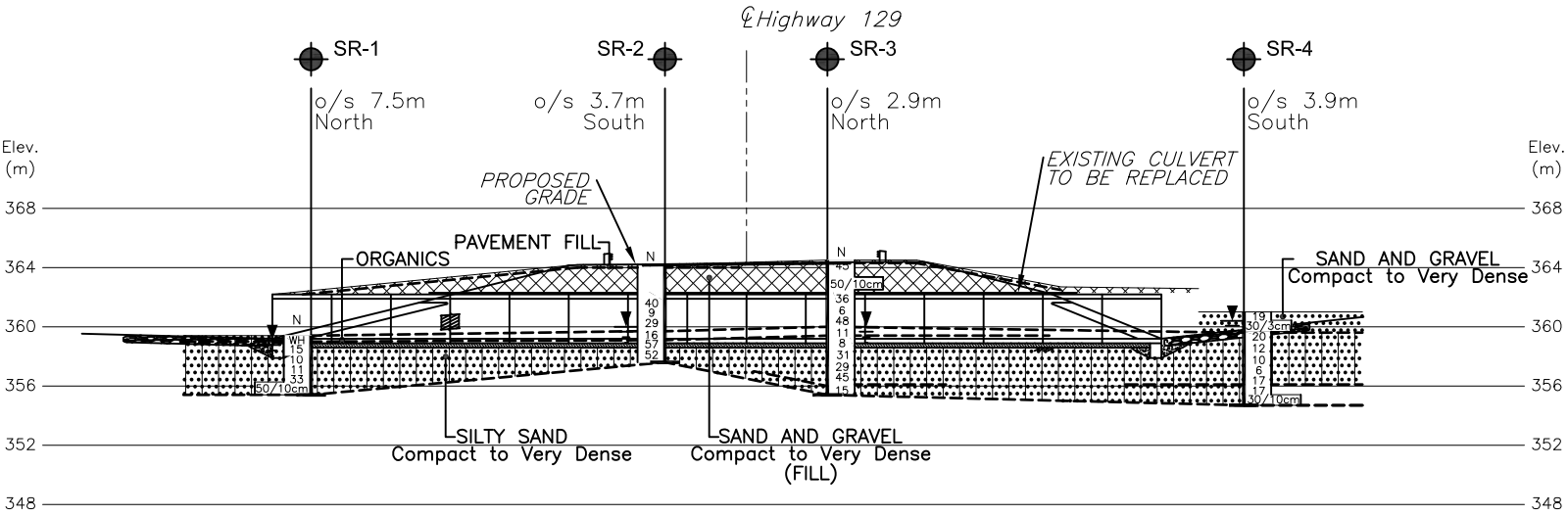
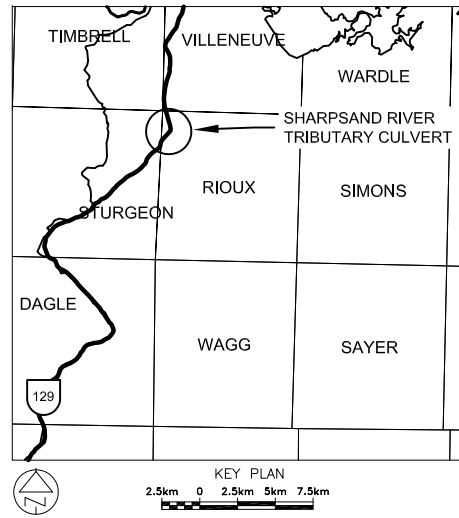


GWP No 5222-05-00
WP No 5161-10-01

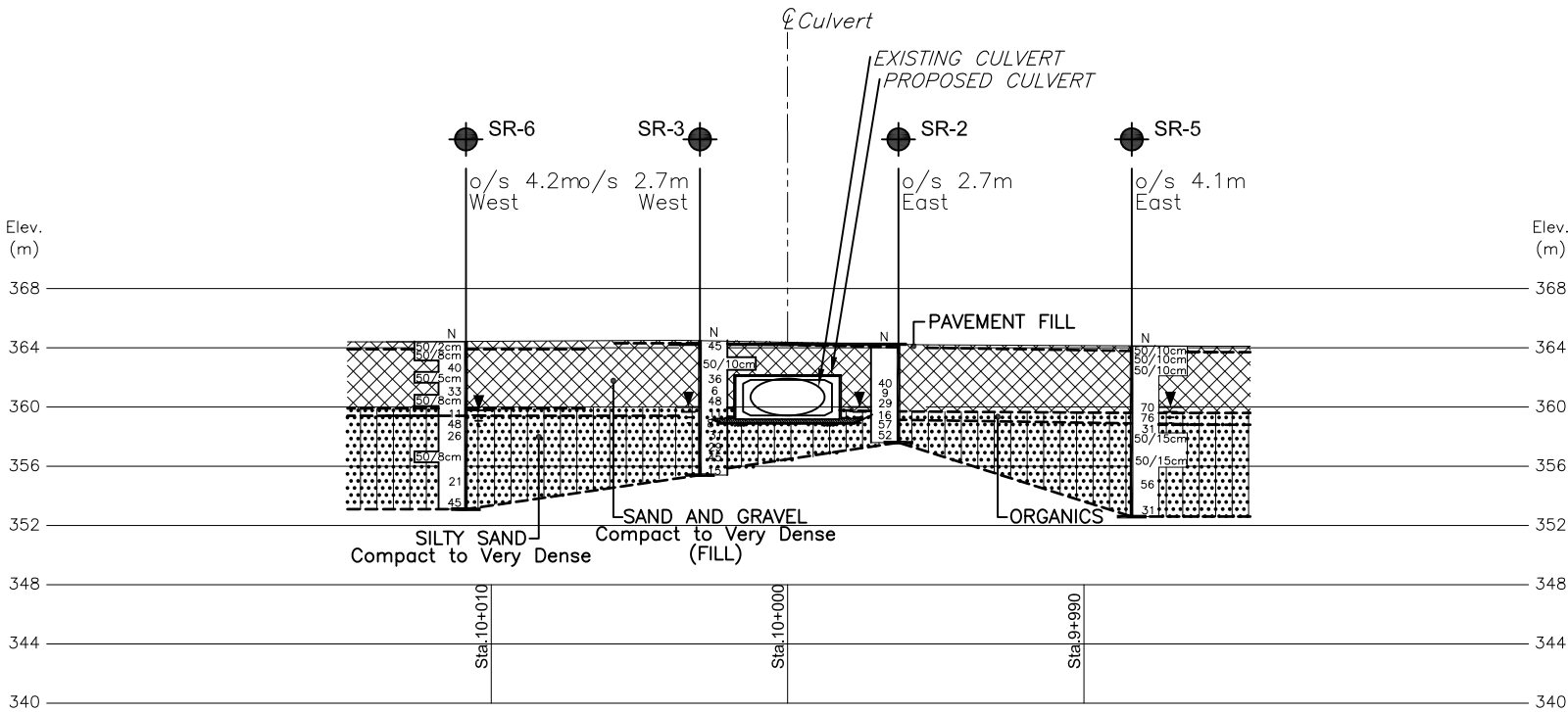
SHARPSAND RIVER TRIBUTARY CULVERT
STA. 10+100 HIGHWAY 129, RIOUX TWP
SOIL STRATA

SHEET

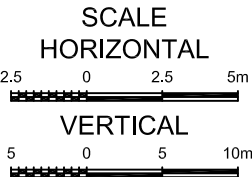
PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



PROFILE A-A ALONG CL SHARPSAND RIVER CULVERT



SECTION B-B ALONG CL HIGHWAY 129



NOTES:

- DRAWINGS SR-1 AND SR-2 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
- REFER TO DRAWING SR-1 FOR BOREHOLE LOCATION PLAN.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

LEGEND

- Borehole
- Cone
- Borehole and Cone
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL at time of investigation Nov. 2014 to Jan. 2015
- * Water level not established
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
REFER TO DWG. SR-1, FOR DETAILS			

NOTE

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



REVISIONS	DATE	BY	DESCRIPTION
Geocres No. 41J-100			
HWY No	129	CHECKED MK	DATE MARCH 29, 2016
SUBM'D	NA	CHECKED DD	APPROVED CN
DRAWN	NA	CHECKED DD	APPROVED CN
DIST	ALGOMA	SITE	385-373/C
DWG	SR-2		

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
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa ⁻¹	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m ² /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{v0}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTP		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No SR-1

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Coords: 5 184 323.7 N; 356 542.3 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Tripod + Casing (Washboring) COMPILED BY M.K.
DATUM Geodetic DATE January 12, 2015 CHECKED BY C.N.

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 _p	w	w _L
								SHEAR STRENGTH kPa												
359.4	Ground Surface																			
0.0	organics		1	SS	WH**	▽*	359													
	Silty sand occasional silt zones, occasional gravel zones																			
	Compact to Brown Wet very dense		2	SS	15		358										1 2 94 3			
	(Lacustrine)		3	SS	10															
			4	SS	11		357										6 78 16 0			
			5	SS	33		356													
355.4	cobbles		6	SS	50/10cm															
4.0	End of borehole																			
	* 2015 01 12																			
	▽* Water level observed during drilling																			
	WH** Penetration due to weight of rods and hammer																			
	NOTE: Borehole caved-in at 2.1m																			

RECORD OF BOREHOLE No SR-2

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Coords: 5 184 314.1 N; 356 555.6 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE C.F.H.S.A. + Casing COMPILED BY M.K.
DATUM Geodetic DATE December 05, 2014 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	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												
364.2 0.0	Ground Surface 100mm asphalt over sand and gravel (PAVEMENT FILL)					▽*	364													
363.7 0.5	Sand and gravel cobbles and boulders Compact to Brown dense (FILL)						363													
							362													
			1	SS	40		361													
			2	SS	9		360													
			3	SS	29															
359.7 4.5	organics Silty sand occasional silt zones, occasional gravel zones Compact to Brown Moist very dense		4	SS	16		359									59 35 (6)**				
			5	SS	57															
357.6 6.6	End of borehole		6	SS	52		358													
	* 2014 12 05 ▽* Water level observed during drilling ** Composite of samples SS5 and SS6 NOTE: No cave-in after extraction of hollow stem augers C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers																			

RECORD OF BOREHOLE No SR-3

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Coords: 5 184 321.3 N; 356 560.2 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE C.F.H.S.A. + Casing COMPILED BY M.K.
DATUM Geodetic DATE December 04, 2014 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE		
364.5	Ground Surface						20	40	60	80	100									
0.0	150mm asphalt over sand and gravel (PAVEMENT FILL)		1	SS	45	▽*														
363.9	Sand and gravel cobbles and boulders																			
0.6	Compact to Brown Moist very dense (FILL)		2	SS	50/10cm															
			3	SS	36															
			4	SS	6															
			5	SS	48															
360.0	organics		6	SS	11															
4.5	Silty sand occasional silt zones, occasional gravel zones																			
	Compact to Brown Wet dense (Lacustrine)		7	SS	8															
			8	SS	31															
			9	SS	29															
			10	SS	45															
	cobbles and boulders																			
			11	SS	15															
355.4	End of borehole																			
9.1	* 2014 12 04																			
	▽* Water level observed during drilling																			
	NOTE: No cave-in after extraction of hollow stem augers																			
	C.F.H.S.A. denotes Continous Flight Hollow Stem Augers																			

RECORD OF BOREHOLE No SR-4

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Coords: 5 184 316.4 N; 356 575.0 E ORIGINATED BY F.P.

DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY M.K.

DATUM Geodetic DATE January 12, 2015 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										○		
								● QUICK TRIAXIAL × LAB VANE												
361.0	Ground Surface						20	40	60	80	100									
0.0	Sand and gravel cobbles and boulders		1	SS	19	▽*														
	Compact to Brown Moist very dense to wet																			
			2	SS	30/3cm															
359.6																				
1.4	Silty sand occasional silt zones, occasional gravel zones		3	SS	20									○						
	Compact to Brown Wet very dense														○			7 90 (3)		
			4	SS	12															
			5	SS	10															
			6	SS	6										○					
			7	SS	17									○			20 77 (3)			
	cobbles		8	SS	17									○						
			9	SS	30/10cm															
354.7																				
6.3	End of borehole																			
	* 2015 01 12																			
	▽* Water level observed during drilling																			
	NOTE: Borehole caved-in at 2.4m																			

RECORD OF BOREHOLE No SR-5

1 of 1

METRIC

G.W.P. 5222-05-00	LOCATION	Coords: 5 184 306.2 N; 356 554.9 E	ORIGINATED BY F.P.
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DIST	Algoma	HWY	129	BOREHOLE TYPE	C.F.S.S.A. + Casing	COMPILED BY	M.K.
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DATUM Geodetic DATE November 18, 2015 CHECKED BY C.N.

[illegible]

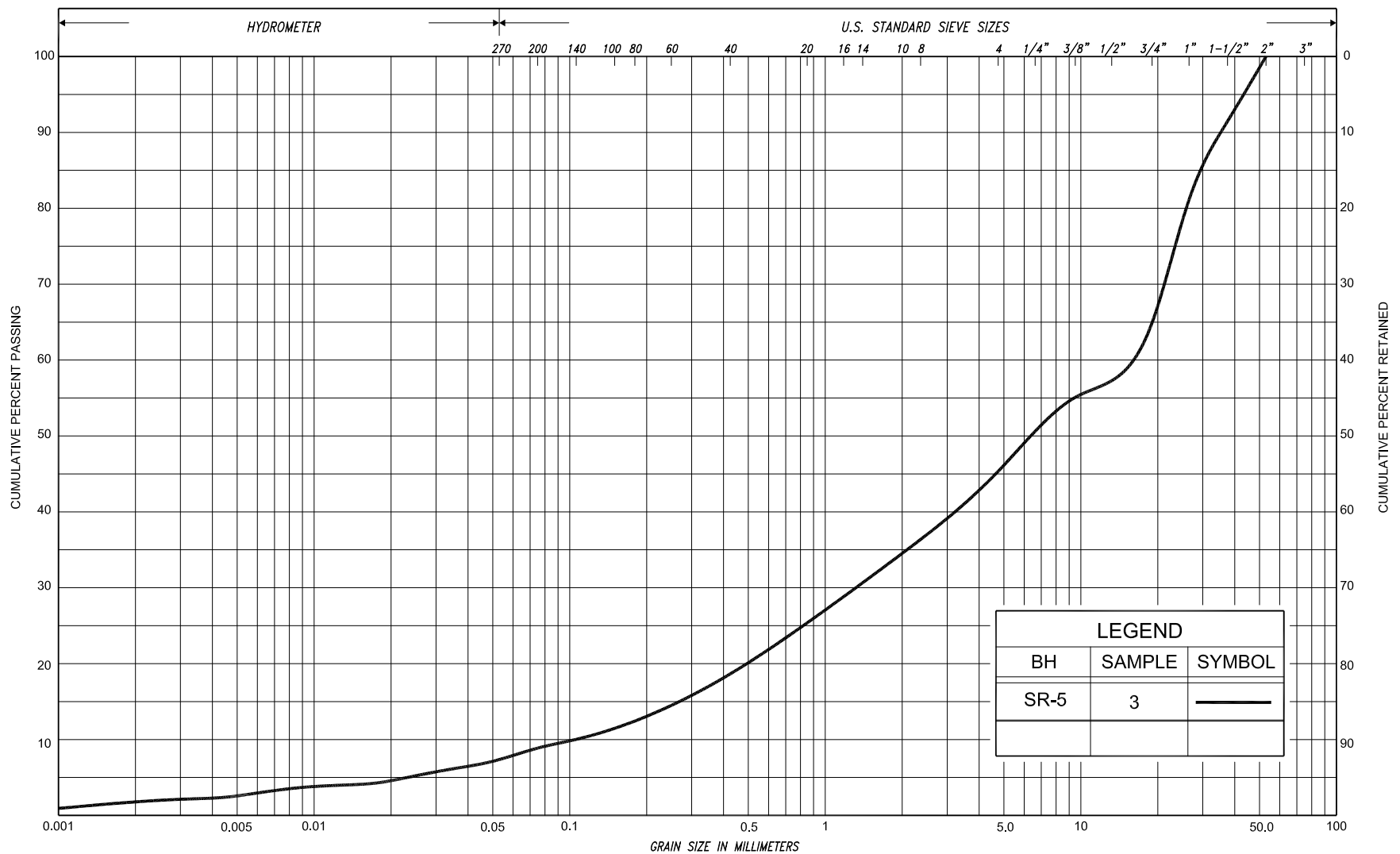
RECORD OF BOREHOLE No SR-6

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Coords: 5 184 329.3 N; 356 560.7 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 19 & 20, 2015 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT										PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	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														
364.4 0.0	Ground Surface						20	40	60	80	100										
363.9 0.5	80mm asphalt over sand and gravel (PAVEMENT FILL) Sand and gravel cobbles and boulders Compact to Brown Moist very dense (FILL)		1	SS	50/2cm	▽*	364														
			2	SS	50/8cm		363														
			3	SS	40		362														
			4	SS	50/5cm		361														
			5	SS	33		360														
			6	SS	50/8cm		359														
							358														
							357														
							356														
							355														
							354														
359.9 4.5	organics Silty sand occasional silt zones, occasional gravel zones, cobbles Compact to Brown Moist very dense (Lacustrines)		7	SS	11																
			8	SS	48																
			9	SS	26																
									</												



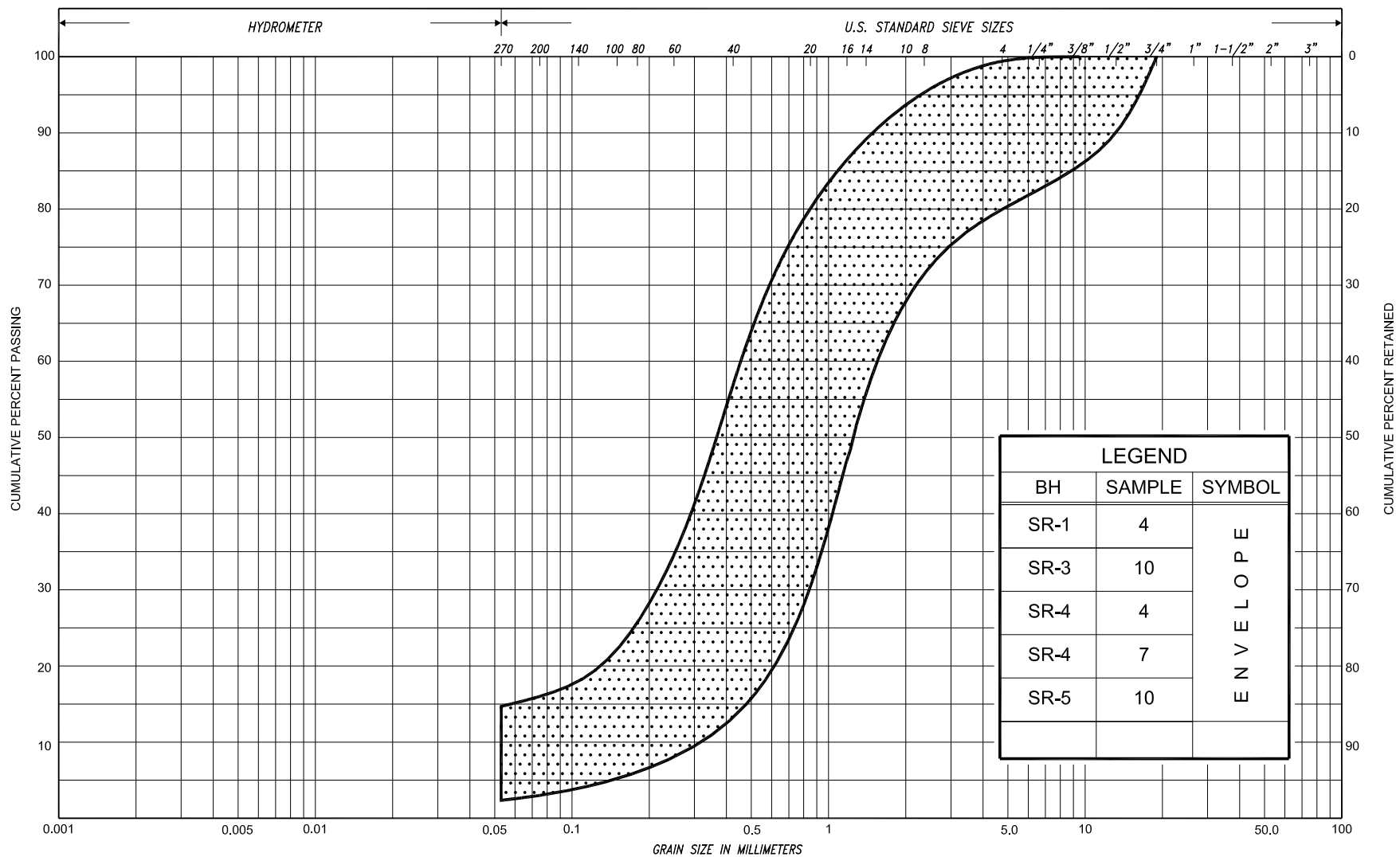
LEGEND		
BH	SAMPLE	SYMBOL
SR-5	3	—

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



GRAIN SIZE DISTRIBUTION SAND and GRAVEL (FILL)

FIG No.	SR-GS-1
HWY:	129
G.W.P. No.	14TF038



SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL				COB BLES	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												



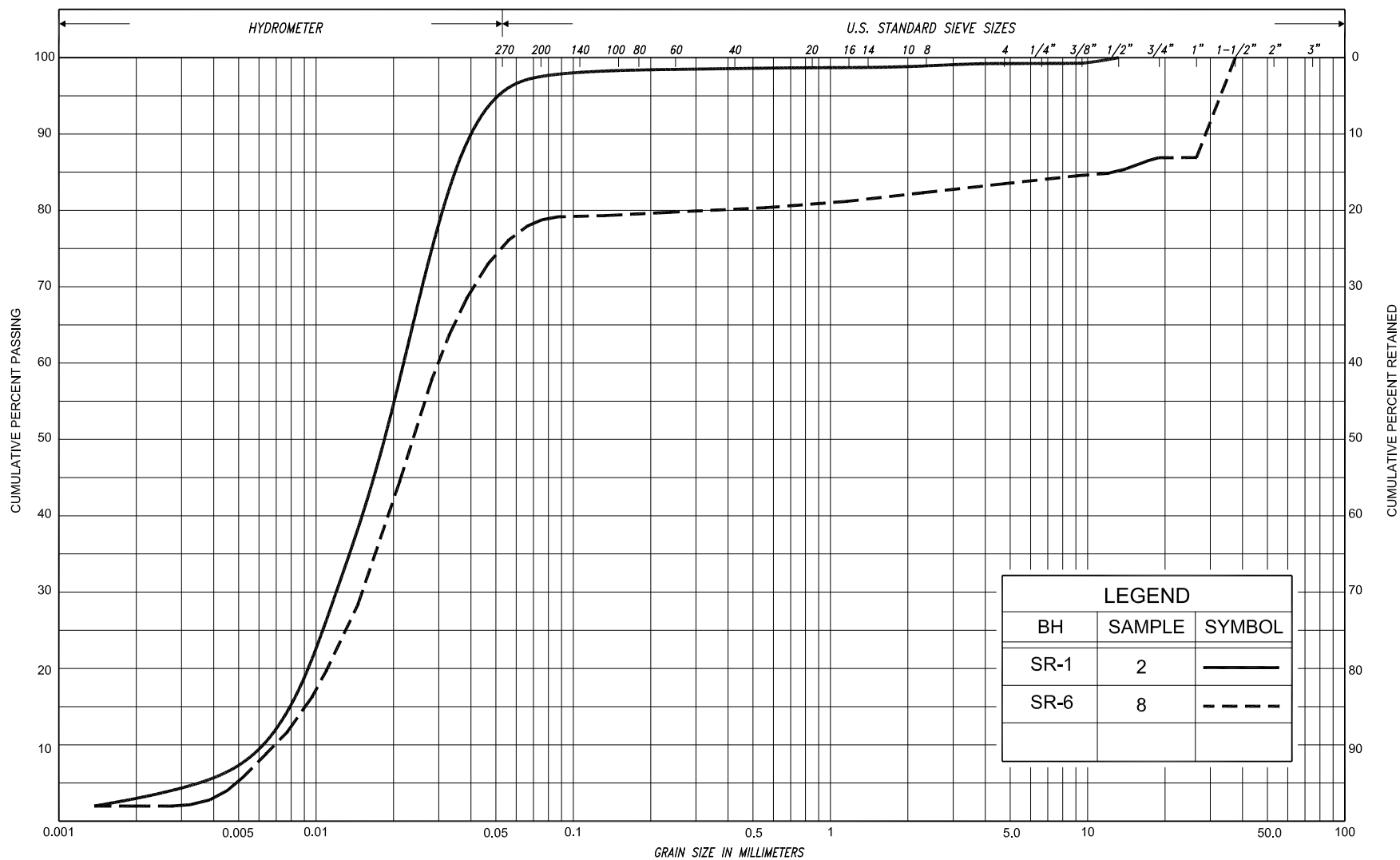
GRAIN SIZE DISTRIBUTION

SILTY SAND

FIG No. SR-GS-2

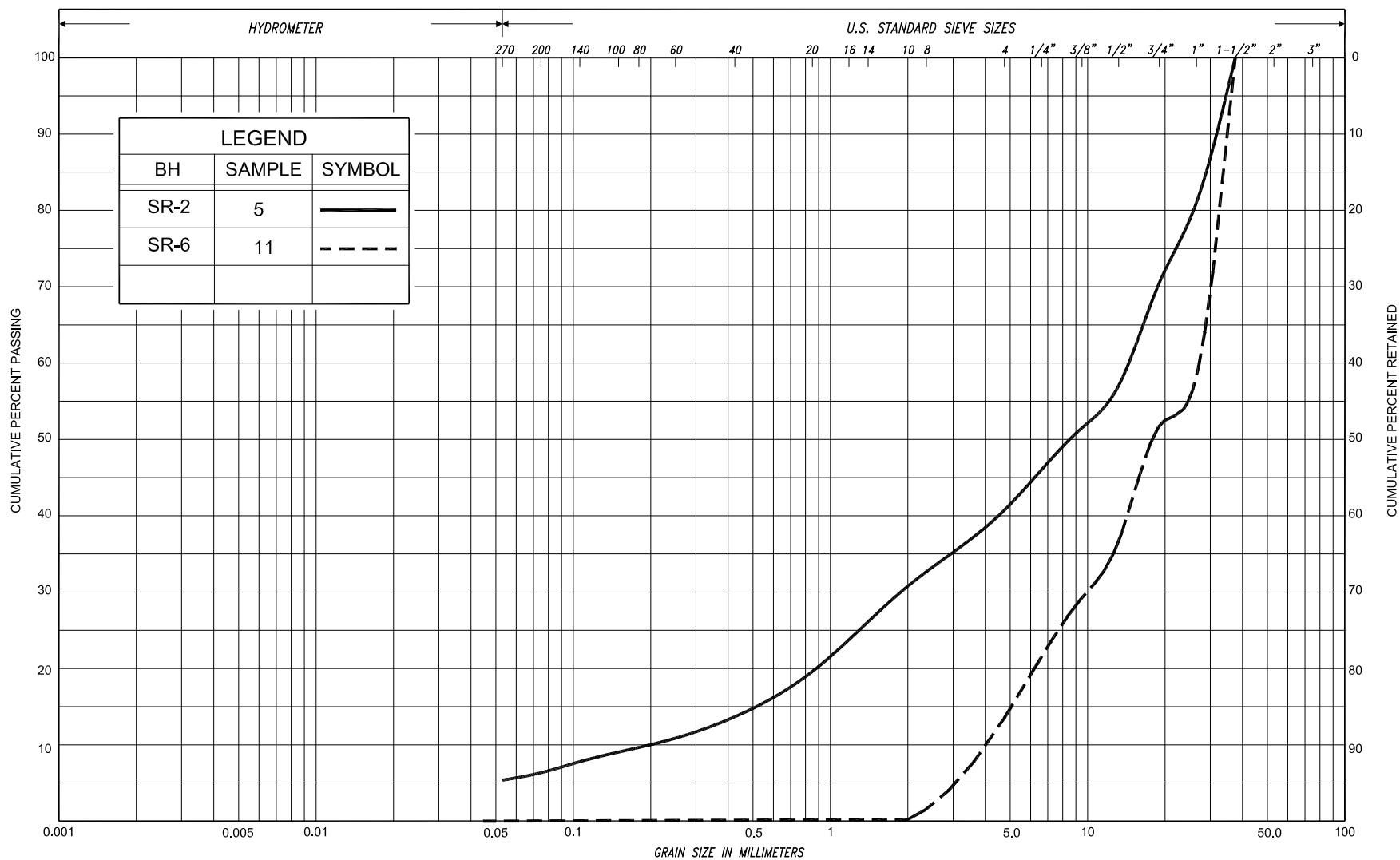
HWY: 129

G.W.P. No. 14TF038



LEGEND		
BH	SAMPLE	SYMBOL
SR-1	2	————
SR-6	8	- - - - -

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												



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



GRAIN SIZE DISTRIBUTION

OCCASIONAL GRAVEL ZONES

FIG No. SR-GS-4

HWY: 129

Project No. 14TF038



FOUNDATION DESIGN REPORT

for

SHARPSAND RIVER TRIBUTARY CULVERT REPLACEMENT

HIGHWAY 129

TOWNSHIP OF RIOUX, ONTARIO

AGREEMENT NO. 5013-E-0040

G.W.P. 5222-05-00

SITE NO. 38S-373/C

WP NO. 5161-10-01

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March 29, 2016



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PART B - FOUNDATION DESIGN REPORT
for
Replacement of Sharpsand River Tributary Culvert
Highway 129 (Site No.38S-373/C)
Township of Rioux, Ontario
GWP 5222-05-00, WP No. 5161-10-01

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 length and span of the existing and replacement culverts are approximately as 30.1 x 3.05 m and 30.0 x 3.0 m, respectively and the new culvert is to be constructed at lower invert levels. It is proposed that the existing SPCSP culvert be replaced with a new precast concrete box culvert. Alternatively, a cast-in-place concrete culvert may be considered. 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.

In summary, the subsurface stratigraphy at the proposed culvert inlet and outlet generally consisted of 80 to 150 mm asphalt concrete underlain by 320 to 450 mm granular base course over a 3.9 to 4.1 m thick sand and gravel fill layer that contains cobbles and boulders. The sand and gravel fill layer overlays a 1.4 m deposit of native sand and gravel which also contains cobbles and boulders. Below the sand and gravel, a deposit of silty sand a minimum 2.1 to 7.0 m thick was encountered.



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

Table 7: Advantages and Disadvantages of Culvert Alternatives

Culvert Type (Alternatives)	Advantages	Disadvantages	Risks/ Consequences	Relative Costs
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 options. 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 Footing 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 considerations. 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 operation required than CIP options.

8. FOUNDATION RECOMMENDATIONS

The invert levels of the proposed culvert are specified to be near Elevation 359.29 m at the east end (inlet) and Elevation 359.19 m at the west end (outlet). The proposed road grade at the proposed culvert will be about Elevation 364.29 m indicating that the soil cover above the culvert will be approximately up to 2.4 m thick.

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 the Highway 129. AECOM provided the preliminary staged construction drawings (Sharpsand River Tributary Culvert Sta. 10+000, Rioux TWP. Culvert Staging Details February 2016) for the one Single Lane Traffic that is included in Appendix B. Two construction stages were identified for dewatering and traffic control for replacing and removing the existing culvert.

Stage 1:

- a) Install dewatering system and maintain flow through temporary bypass
- b) Install traffic control measures to maintain single lane traffic on west side of highway
- c) Remove east portion of existing structure and footing
- d) Construct east 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 east side of highway
- g) Remove west portion of existing structure
- h) Install west portion of culvert
- i) Divert flow through culvert
- j) Remove temporary bypass piping and supports



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 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 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 diameter clear stone can be utilized for granular bedding and levelling course provided that this material is wrapped with filter fabric to prevent both migration of fines from the native soils and ultimately potential failure of the culvert. The clear stone should meet the physical properties and gradation requirements of 19 mm Type 2 Clear Stone in OPSS.PROV 1004. The Granular B Type II is preferred in wet construction conditions. A leveling course up to approximately 100 mm thick may also be incorporated in the bedding thickness.

The culvert founding levels will be at approximate Elevations 357.7 to 357.8 m allowing for the bedding and a 200 mm culvert base concrete thickness of approximately 0.5 m. 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 Resistances

Culvert Section	Subgrade soil type	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)
Entire Length	Typically compact to very dense cohesionless silty sand	250	150

The geotechnical resistances were computed according with the CHBDC and the reaction at SLS normally allows for a 25 mm compression of the founding medium. In addition, the cohesionless soil settlements under the proposed culvert discussed in the following section should be



considered. A foundation embedment depth of 2.0 m and groundwater at about 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 silty sand 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. This order of settlement magnitude is considered tolerable for the proposed precast box concrete culvert or for an alternative cast-in-place culvert. Construction joints should be added to accommodate the estimated 15 mm of differential settlement if required.

8.4 Subgrade Preparation and Culvert Bedding

Excavations and preparation of the subgrade for construction of the culverts should be performed and monitored in accordance with OPSS 902 (Construction Specification for Excavation and Backfilling - Structures). All 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, the bedding may consist of 19 mm clear crushed stone 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 μm . 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 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 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 Frost Tapers

To provide a gradual transition between the existing roadway areas underlain by locally frost susceptible earth fill material and the non-frost susceptible backfill above and adjacent to the structure within the zone of seasonal frost penetration, granular frost tapers should be provided on both sides of the culvert (refer to OPSD 803.031). The granular frost tapers should be constructed using OPSS Granular B Type II and should extend from 2/3 metres below the finished roadway surface to the underside of the pavement subbase layer for the roadway at 5 horizontal to 1 vertical (5H:1V), or flatter. A steeper frost taper could be considered (such as, 2H:1V) however, for this case, the degree of differential frost heaving between the existing locally frost susceptible earth fill material and the non-frost susceptible backfill will be greater than if a flatter taper is used.



The granular material in the frost tapers should be placed in maximum 200 mm thick lifts and compacted to at least 95% of the standard Proctor maximum dry density value in accordance with OPSS 501.

8.6 Modulus of Subgrade Reaction

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

Table 8.6: Recommended Modulus of Subgrade Reaction

Soil Type	Modulus Of Subgrade Reaction (MN/m ³)
Granular A or Granular B Type II or 19 mm clear stone	45
Compact to dense silty sand	15

8.7 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 8.7: Sliding Resistance Parameters

Soil Type	Foundation Friction Angle (degrees)		Cohesion (kPa)	Unit Weight (kN/m ³)
	Cast-In-Place	Precast		
Granular A or Granular B Type II or 19 mm clear stone	35	23	0	22.8
Compact silty sand	30	20	0	20.0

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.



8.8 Foundation Frost Depth

The foundation frost depth for structure foundations at this site is 2.3 m, according to the OPSD 3090.101 (Foundation, Frost Penetration depths for Northern Ontario).

8.9 Seismic Considerations

The reference Peak Ground Acceleration (PGA) for the project site is 0.036 based on the Town of Chapleau and also on 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.

8.10 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 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

γ = unit weight of backfill material above design water level (kN/m³)

γ' = unit weight of submerged backfill material below design water level (kN/m³)

$$= \gamma - \gamma_w$$

γ_w = unit weight of water

$$= 9.8 \text{ kN/m}^3$$

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 ϕ = angle of internal friction of retained soil (35° for Granular A or B Type II)

δ = 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.

The following parameters are recommended for design:

Table 8.10: Recommended Geotechnical Parameters

Geotechnical Parameter	Granular A or Granular B Type II	Excavated Material (*)
Angle of Internal Friction, degrees	35	30
Unit Weight, kN/m ³	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 materials

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.



8.11 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 protective measures noted in the OPSD 800 series to deal with erosion (inlet/outlet treatment, headwalls, cut-off walls etc.) are considered to be appropriate.

The inlet cut off protection may consist of vertical cut-offs and structural head walls. The outlet protection may consist of structural head walls. The backfill should consist of OPSS Granular A or Granular B Type II. The cut-off walls should extend laterally to protect the granular backfill material and to a depth at least equal to the fluctuation of the water level at the culvert location to prevent flow below the culvert that could erode the granular base/bedding material. The requirements of CHBDC clauses 1.9.5.6 and 1.9.11.6.5 should be applied.

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-offs 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 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 vertical cut-off wall and structural headwall. The rock protection should provide for horizontal apron inlet and outlet protection 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 walls should be checked for sliding resistance using the geotechnical parameters provided previously in Section 8.6 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 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 µm 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 native silty sand 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 soil are classified as Type 3 soil and the dense to very dense silty sand is classified as Type 2 soil, according to the Occupational Health and Safety Act (Ont. Reg.213/91) classification system. For the protection of workers entering the excavations, 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 OHSA. 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: Comparison of shoring Alternatives for Roadway Protection

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

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 recorded water level in the Sharpsand River Tributary, near Elevation 360.5 m, that is about 1.5 m above the inferred culvert subgrade level. It will be necessary to implement measures to temporarily lower groundwater table and to permit construction. 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 dewatering system based on dam and pumping techniques will generally be sufficient. A dewatering system should be placed at each end of the construction areas. Depending on the culvert type being installed and construction techniques, a cofferdam may be



required due to the relatively pervious nature of the subsoil. 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 in the excavation from the excavation slope may be handled by sump pumping techniques. The dewatering system should be designed to conform to 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 2.0 m below the recorded level (noted above) to obtain stable subgrade conditions for construction in-the-dry is some 200,000 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. Grigory Degil, MSc, 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', with a stylized flourish.

Mansoor Khorsand, BSc, EIT
Project Supervisor, Geotechnical Services



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



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

MK/DD/CN.mk-jk



APPENDIX FDR-A

Sharpsand River TRIBUTARY Culvert General Arrangement
Sharpsand River TRIBUTARY Culvert Staging Details

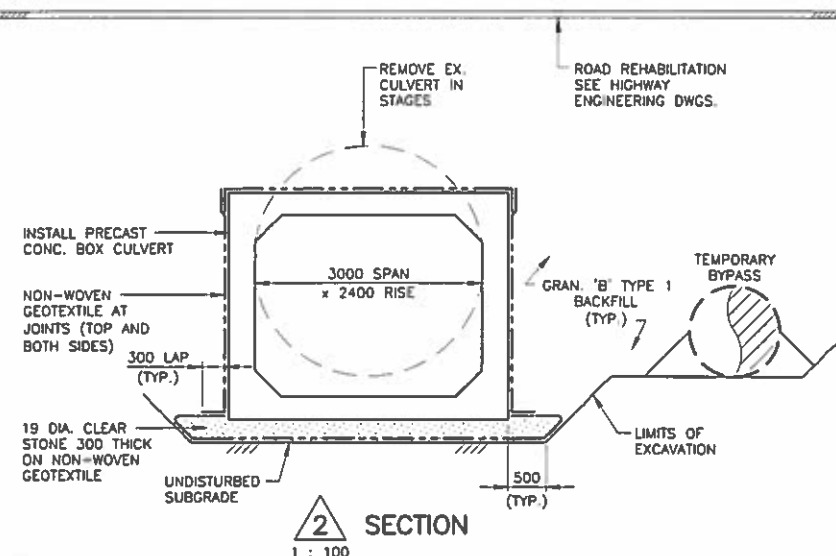
SHEET
35

LIST OF DRAWINGS :

- 1 GENERAL ARRANGEMENT
- 2 BOREHOLE LOCATIONS AND SOIL STRATA
- 3 BOREHOLE LOCATIONS AND SOIL STRATA
- 4 CULVERT STAGING DETAILS
- 5 PRECAST CONCRETE DETAILS
- 6 CONCEPTUAL FLOW MANAGEMENT
- 7 MISCELLANEOUS DETAILS

DRAWING NOT TO BE SCALED
50 mm ON ORIGINAL DRAWING

DATE: 3/1/2016 B.M. ELEVATION 362.302
HWY 129 STA 9+927.429
N&W IN ROOT, PINE
O/S 19.749 LT



REVISIONS				
	DATE	BY	DESCRIPTION	
	DESIGN J.P.	CHK G.M.	CODE CHBDC 2014 LOAD CL - 625-ONTDATE MAR 2016	
	DEBMAN J.	CHK J.B.	GUT. 3RE-373/C EXTRACT VEC-EIF INWC	

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

CONT No 2016-5142
WP No 5161-10-01



SHARPSAND RIVER CULVERT
STA. 10+000, RIOUX TWP.
CULVERT STAGING DETAILS

SHEET
38

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. ROADWAY PROTECTION TO BE DESIGNED BY THE CONTRACTOR TO PERFORMANCE LEVEL 2.
5. 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.
6. ARRANGEMENT OF THE DEWATERING SYSTEM CONCEPT IS NOT SHOWN IN THIS DRAWING, SEE DRAWING 5.
7. FOR ROAD WORKS SEE HIGHWAY ENGINEERING DRAWINGS.

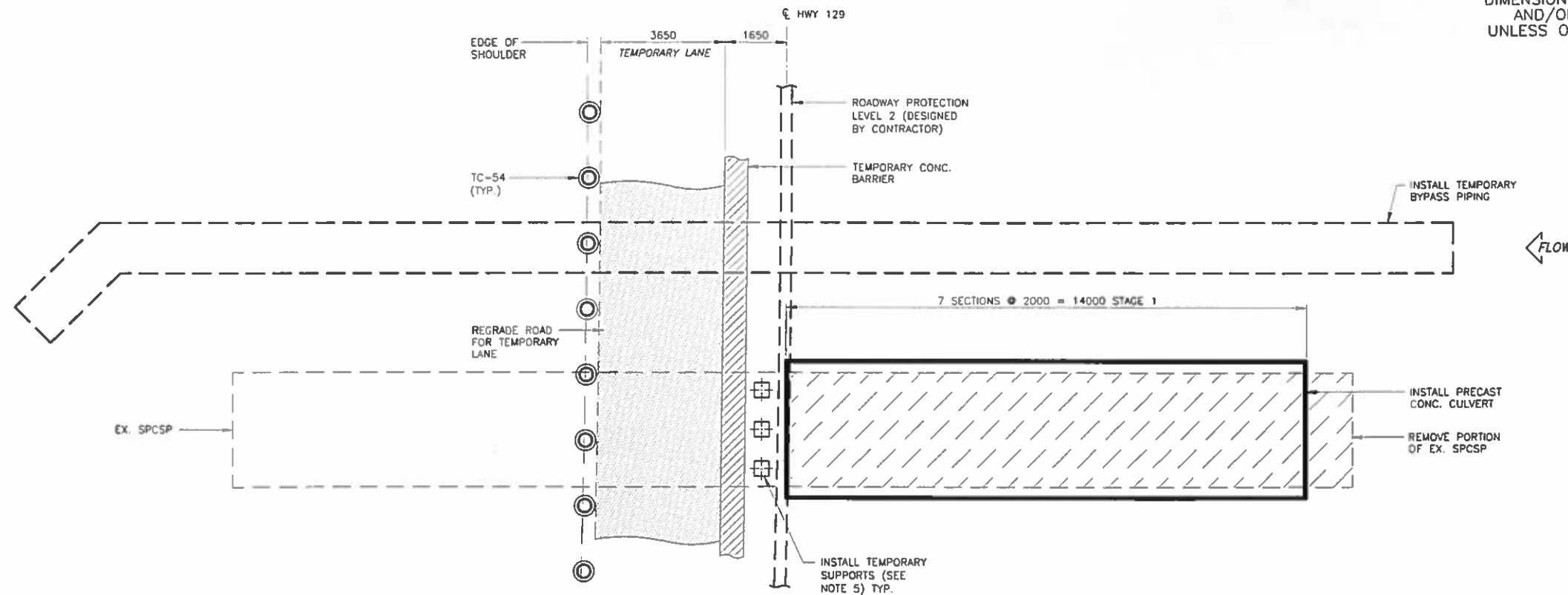
SEQUENCE OF WORK :

STAGE 1 :

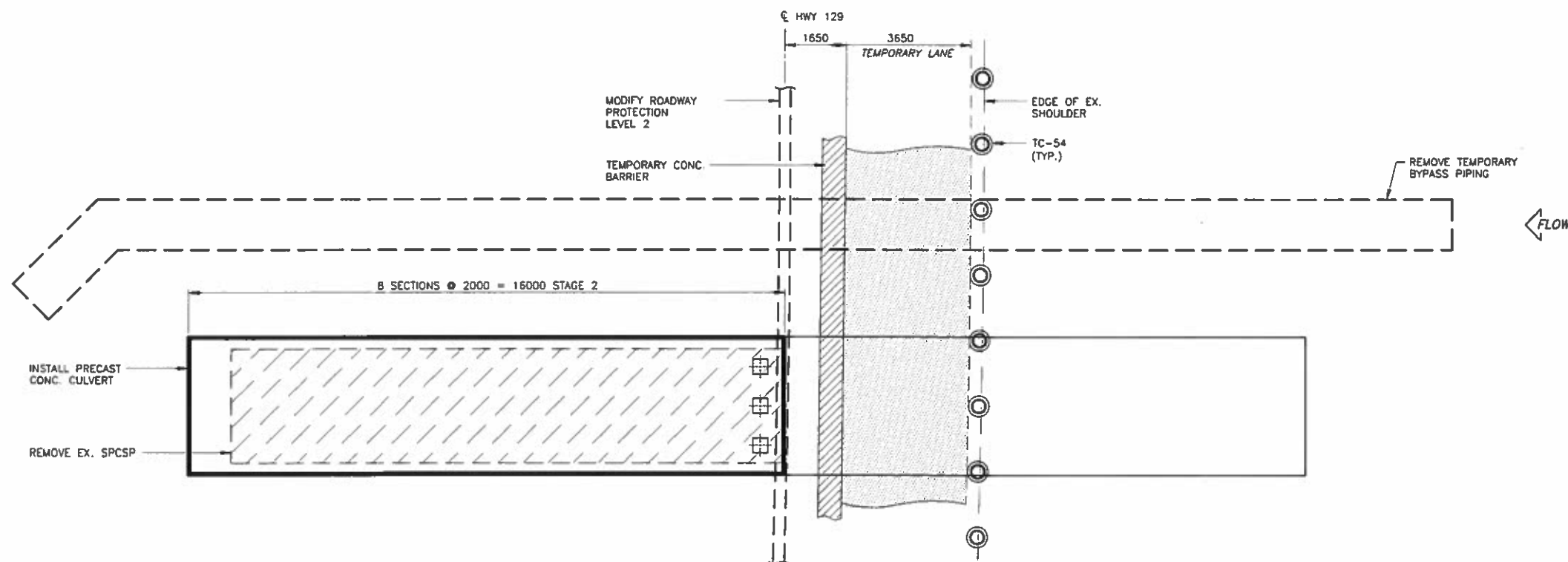
- INSTALL DEWATERING SYSTEM AND MAINTAIN FLOW THROUGH TEMPORARY BYPASS.
- INSTALL TRAFFIC CONTROL MEASURES TO MAINTAIN SINGLE LANE TRAFFIC ON WEST SIDE OF HIGHWAY.
- REMOVE EAST PORTION OF EXISTING STRUCTURE AND FOOTINGS.
- CONSTRUCT EAST PORTION OF CULVERT.
- COMPLETE ALL OTHER WORK DETAILED AND SPECIFIED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

STAGE 2 :

- MAINTAIN DEWATERING SYSTEM AND FLOW THROUGH TEMPORARY BYPASS.
- INSTALL TRAFFIC CONTROL MEASURES TO MAINTAIN SINGLE LANE TRAFFIC ON EAST SIDE OF HIGHWAY.
- REMOVE WEST PORTION OF EXISTING STRUCTURE.
- INSTALL WEST PORTION OF CULVERT.
- DIVERT FLOW THROUGH CULVERT.
- REMOVE TEMPORARY BYPASS PIPING AND SUPPORTS.
- ALL OTHER WORK DETAILED AND SPECIFIED IN THE CONTRACT DOCUMENTS.



PLAN - STAGE 1
1 : 150



PLAN - STAGE 2
1 : 150
CONSTRUCTION STAGING

DRAWING NOT TO BE SCALED
50 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	J.P.	CHK G.M.	CODE CHBDC 2014 (LOAD CL-625-ONT) DATE MAR 2016
DRAWN	T.G.	CHK J.P.	SITE 385-373/C STRUCT SCHEME DWG 4



APPENDIX FDR-B

List of Standard Specifications Relevant to Report
Non-Standard Special Provisions (NSSP)



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 3090.101	Foundation, Frost Penetration depths for Southern 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 also advised that damming of the drain and diversion of the flow by pumping through temporary conduits for construction staging will likely be required at this site.

NSSP – Installation of Shoring Of Roadway Protection (Addition to OPSS 539)

The Contractor is advised that cobbles and/or boulders may be encountered during the installation of shoring elements and during excavation of the embankment. The Contractor shall select and use the appropriate methods for shoring installation and excavations to account for such possible obstructions.