



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for**

**POULIN CREEK CULVERT REPLACEMENT
HIGHWAY 129
TOWNSHIP OF DAOUST, ALGOMA DISTRICT, ONTARIO
ASSIGNMENT NO. 5013-E-0040
G.W.P. 5222-05-00
SITE NO. 46-326/C
WP NO. 5230-05-01**

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PML Ref.: 14TF038
Index No.: 075FIR and 076 FDR
GEOCRES No.: 410-18
September 21, 2016



PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT

for

POULIN CREEK CULVERT REPLACEMENT

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TOWNSHIP OF DAOUST, ALGOMA DISTRICT, ONTARIO

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Borehole Location Plan and Soil Strata, Poulin Culvert Replacement

Explanation of Terms Used in Report

Record of Borehole Sheets – PCN1 to PCN4

Results of Grain Size Distribution Analysis – Figure PCN-GS-1

PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT

For
Replacement of Poulin Creek Culvert
Highway 129 (Site No.46-326/C)
Township of Daoust, Ontario
GWP 5222-05-00, WP No. 5230-05-01

1. INTRODUCTION

This report presents the factual findings obtained from the geotechnical investigation carried out for the proposed replacement of culvert located at the crossing of Poulin Creek and Highway 129 (Sta. 10+000) in the Township of Daoust, Algoma District, Ontario.

The fieldwork was carried out from December 16, 2014 to January 6 and 16, 2015. The purpose of the investigation was to explore the subsurface conditions expected to influence the preliminary design of the Poulin Creek culvert replacement and to aid the designer in selecting the suitable type of replacement structure.

AECOM Canada Ltd (AECOM) has retained Peto MacCallum Ltd. (PML) on behalf of the Ministry of Transportation Ontario (MTO) to provide preliminary foundation engineering services for the replacement of seven culverts on Highway 129. The scope of this project involves providing subsurface information for the preliminary design of the proposed Poulin Creek culvert replacement.

2. SITE DESCRIPTION

The proposed replacement culvert is located approximately 12.1 km north of the intersection of Highway 129 and Highway 667. The topography of the project area is generally flat, except for the highway embankments. Poulin Creek flows from Olympic Lake on the west side to Nemgos Lake on the east side of the Highway 129. Generally, the site surrounding the culvert is covered with bushes and grass. The existing grade of Highway 129 at the crossing of culvert is at about El. 444.07 and the road is approximately 4.0 m higher than the surrounding area. The surrounding area is generally heavily wooded except for the floodplain of the creek.



The existing culvert is a 3.5 m span and 21 m long corrugated steel pipe structure with a fill height of 2.1 m above the crown. This culvert was constructed in 1975 and the road accommodates two lanes of vehicular traffic. The inlet and outlet of the culvert were snow covered during the fieldwork and the conditions of the embankment or culvert could not be assessed. Refer to Photographs PCN1 to PCN4 provided in Appendix A, for general conditions of the site.

3. FIELD INVESTIGATION PROCEDURES

The staff of PML visited the site on December 16, 2014 and January 6 and 16, 2015 to mark out the borehole locations. The underground services at the borehole locations were cleared by the respective utility companies. Public and private utility authorities were informed and all the utility clearance documents were obtained before the commencement of drilling work.

The location of boreholes in the field were established by portable GPS device. Subsequently, exp Geomatics under contract to AECOM carried out the survey of the borehole locations and elevations, and provided the co-ordinates for locations in MTM NAD 83 northing and easting. PML used the survey data provided by AECOM for preparing this report. All elevations reported in this report are referred to Geodetic and expressed in meters.

The equipment used for drilling was owned and operated by Landcore Drilling of Chelmsford, Ontario. Landcore Drilling is a specialist drilling contractor was working under the full time supervision of a PML field supervisor. The investigation included advancing four (4) boreholes numbered PCN-1, PCN-2, PCN-3 and PCN-4 to maximum depths ranging from 2.1 m to 7.2 m (El. 438.9 to El. 436.7). Boreholes PCN-2 and PCN-3 were located on the paved area of the road and these boreholes were advanced using solid stem augers aided by a truck-mounted B-75 drill rig. Boreholes PCN-1 and PCN-4 were located at the inlet and outlet of the culvert, respectively. These two boreholes were advanced using CME-55 Bombardier and tripod drill rig employing 75 mm diameter casings and wash boring method. The drill rig was equipped with 63.5 kg (140 lb) automatic hammer calibrated to fall freely through 760 mm (30 in.). Boreholes PCN-2 and PCN-3 were sampled to a maximum depth of 5.2 m and below this depth, the boreholes were advanced by conducting Dynamic Cone Penetration Test (DCPT). Standard penetration and dynamic cone penetration (DCP) tests were conducted to assess the strength characteristics of the substrata. Location of boreholes is shown on the attached Drawing No. PCN -1.



Representative soil samples were recovered from the boreholes at 0.75 m intervals using a conventional 51 mm O.D split spoon sampler in accordance with the Standard Penetration Test (SPT) procedure. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata.

The groundwater conditions at the borehole locations were observed during the drilling by visual examination of the soil samples, sampler and drill rods as the samples were retrieved. In addition, water level measurements were taken in open boreholes. Upon completion of drilling, the boreholes were backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.

The recovered soil samples were returned to our laboratory for detailed visual examination, and index tests.

4. LABORATORY TEST PROCEDURES

Laboratory tests on representative SPT samples recovered during the fieldwork were carried out by the laboratory owned by PML, located in Toronto. The laboratory testing program included the following:

- Natural moisture content determinations (6)
- Grain size distribution analyses (3)

The laboratory tests to determine the index properties were performed in accordance with the MTO test procedures, which follow American Society for Testing Materials (ASTM) test procedures, with the exception of hydrometer test (LS-702). The results of the grain size distribution analyses tests are presented in Figure PCN-GS-1. All of the test results are summarized on the attached Record of Borehole sheets.



5. SITE GEOLOGY AND SUBSURFACE CONDITIONS

5.1 Site Geology

The Map 2543 (Bedrock Geology of Ontario, East-Central Sheet, 1991) published by the Ministry of Northern Development and Mines (MNDM), indicates that the bedrock formation of the project area is known to be in Neo to Mesoproterozoic Group, which consists mainly of gneissic tonalite suite intrusive rocks comprised of minor supracrustal inclusions. The Map 2555 (Quaternary Geology of Ontario, East-Central Sheet, 1991) published by MNDM indicates that the surface conditions of the project area consists of Glaciofluvial ice-contact and Glaciolacustrine Coarse-textured deposit such as gravel, sand, and minor till includes esker, kame, end moraine, ice-marginal delta and subaqueous fan deposit.

5.2 Subsurface Conditions

The existing culvert is located within approximately 2.0 to 3.0 m high embankment consisting of rock fill placed over the native soils. In summary, the subsurface stratigraphy consists of 600 mm pavement structure followed by 4.1 m to 4.6 m rock fill. The rock fill is underlain by sandy silt to silty sand, which extends to the maximum depth of investigation of 7.2 m (El. 436.7). The snow and 500 mm thick ice at the outlet is immediately followed by 1.6 m peat layer, which is underlain by silty sand to gravelly sand to the termination depth of borehole of 5.3 m (El. 436.7). Peat layer was not encountered in the borehole located at the inlet. For classification purposes, the soils encountered at this site can be divided into four distinct zones.

- a) Pavement Structure
- b) Peat
- c) Rock Fill
- d) Sandy Silt to Silty Sand, Trace Clay

The subsurface conditions encountered during the course of the investigation, together with the field and laboratory test results are shown on the attached Record of Borehole Sheets. The borehole locations and stratigraphic profile sections are shown on Drawings DGW PCN-1. The boundaries between soil strata have been established at the borehole locations only. The boundaries between and beyond the boreholes are assumed and may vary from location to location. Description of the soil strata encountered are summarised below.



5.3 Pavement Structure

Approximately, 180 mm asphaltic concrete over 420 mm of sand and gravel (base course) were encountered in Boreholes PCN-2 and PCN-3 located on the shoulders of Highway 129. The base course consists of varying proportions of sand and gravel and extends to about El. 443.4 m to El. 443.3 m.

5.4 Rock Fill

The pavement structure is immediately followed by rock fill in boreholes located on the highway shoulders. This rock fill layer ranges in thickness from 4.1 m to 4.6 m and extends to a depth ranging from 4.7 m to 5.2 m (El. 439.2 to El. 438.8) below the asphalt surface. The rock fill consists of gravelly size particles to cobbles ranging in particle size from 100 mm to 150 mm.

However, boulders ranging in diameter from 200 mm to as high as 1.0 m were encountered in borehole PCN-2 below about El. 440.5. Coring was carried out below El. 440.5 to advance the borehole through the rock fill. Presence of 1.0 m size boulder in the rock fill was confirmed by coring.

5.5 Peat

This peat deposit was encountered only in Borehole PCN-4 located near the outlet of the culvert. It was intercepted immediately below 500 mm thick ice and extends to about 2.1 m (EL. 439.9). The thickness of this deposit was about 1.6 m and the moisture content determined from a representative sample was about 70%.

5.6 Sandy Silt to Silty Sand, Trace Clay

This sandy silt to silty sand deposit underlie the rock fill embankment and peat deposit at the outlet. However, this sandy deposit encountered at the outlet (Borehole PCN-4) may be classified as sandy silt to silty sand. This sandy deposit extends to the maximum depth of investigation of 7.2 m (El. 436.7).



The “N”-values measured within this deposit ranged from 10 blows/300 mm to 35 blows/300 mm, indicating a compact to dense state of compaction.

The moisture content of this material varies from 9% to as high as 24%. The results of the grain size distribution analyses of three representative samples from this sandy layer are shown on Figure PCN-GS-1 appended to this report. The test results reveal that the sandy silt to silty sand consists of 0% to 35% gravel, 42% to 64% sand, 12% to 57% silt and 0% to 2% clay.

5.7 Groundwater

Poulin Creek flows from west to east and the water level of the creek was observed at about El. 441.5 at the time of the investigation. In view of the relatively pervious nature of the soils at this site, the groundwater level will be governed or influenced by the water level in the creek.

The groundwater was observed during and upon completion of drilling. The groundwater level was measured at a depth of 4.6 m (El. 439.3) below the existing grade of the road. However, it was observed at a depth of 500 mm (El. 441.5) below the ground surface of the inlet and outlet. It should be noted that the water level of the creek is subject to seasonal fluctuations and rainfall patterns.



6. 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, B.Sc., EIT. Project Supervisor and reviewed by Mark Vasavithasan, MSc.Eng., 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, B.Sc., EIT.
Project Supervisor, Geotechnical Services



Mark Vasavithasan, M.Sc. Eng., P.Eng.
Senior Engineer, Geotechnical Services



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



APPENDIX A

Site Photographs



Photograph P1: Looking south at the location of Borehole PCN-2. The borehole drilled at Highway 129 SB shoulder. (December 16, 2014)



Photograph P2: Looking east from toe of Highway 129 embankment. Borehole PCN-1 was drilled at south-west of the culvert inlet. (January 16, 2015)



Photograph P3: Looking west from toe of Highway 129 embankment. Borehole PCN-4 was drilled at northeast of the culvert outlet. (January 16, 2015)



Photograph P4: Looking north at the location of Borehole PCN-3. The borehole drilled at Highway 129 NB shoulder. (January 6, 2015)



APPENDIX B

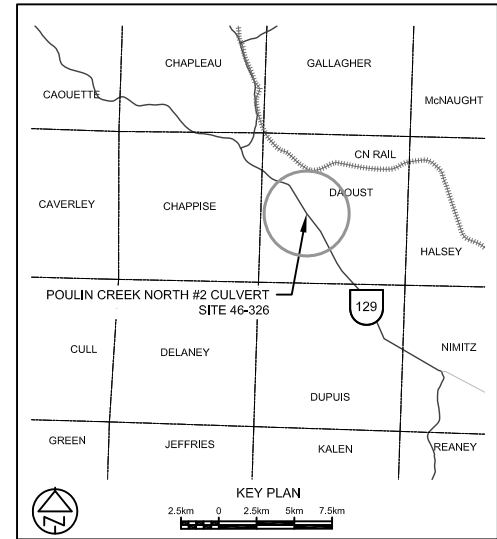
Drawings PCN-1

Borehole Locations Plan and Soil Strata at Poulin Creek Culvert

Explanation of Terms Used in Report

Record of Borehole Sheets – PCN1 to PCN4

Results of Grain Size Distribution Analysis – Figure PCN-GS-1



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 Jan. 2015		
*	Water level not established		
	Head		
	ARTESIAN WATER Encountered		
	PIEZOMETER		
BH No	ELEVATION	NORTHINGS	EASTINGS
PCN-1	441.9	5 284 874.9	356 186.6
PCN-2	443.9	5 284 879.3	356 176.9
PCN-3	444.0	5 284 865.9	356 176.0
PCN-4	442.0	5 284 887.7	326 189.2

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

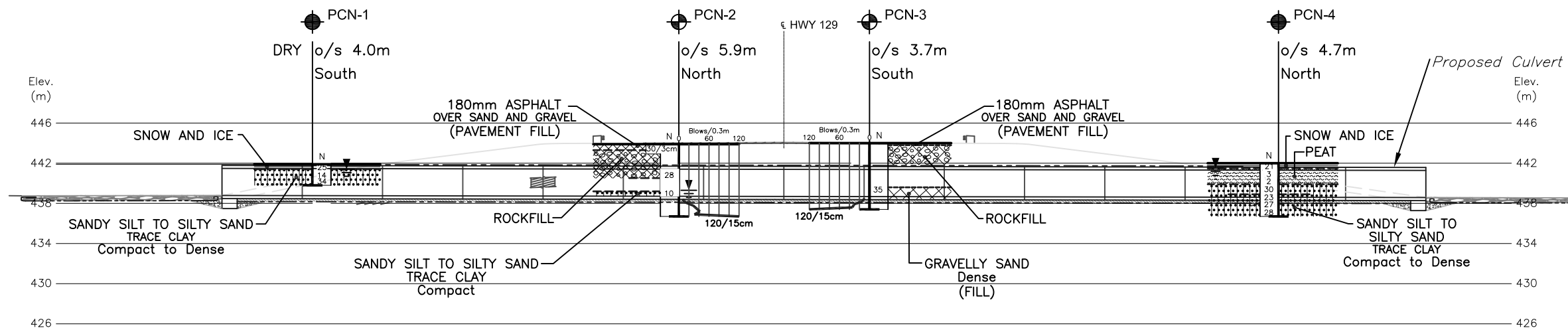
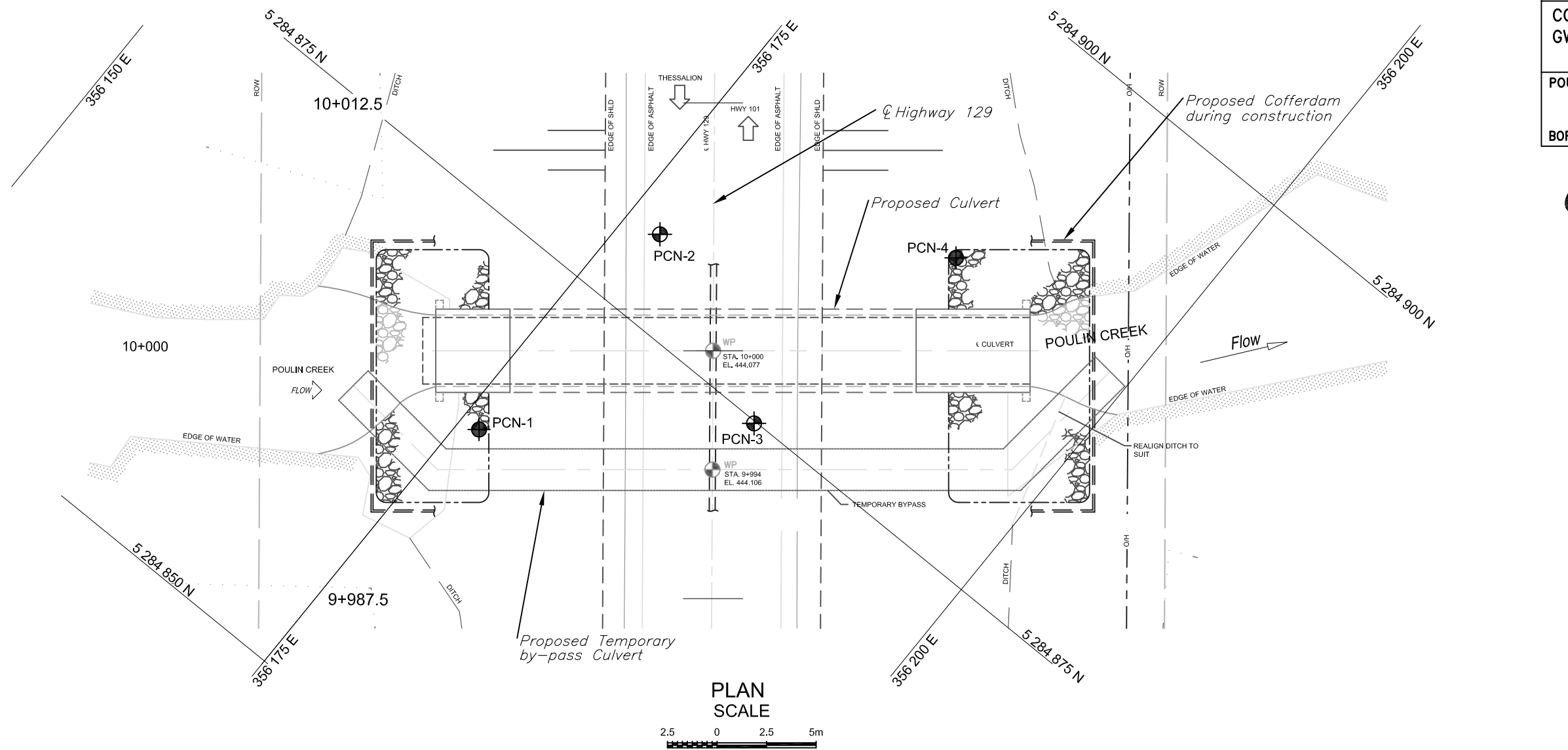
DATE	BY	DESCRIPTION

Geocres No. 410-18

HWY No 129	CHECKED M.Kh	DATE SEPT. 21, 2016	DIST SAULT ST. MARIE
SUBM'D NA	CHECKED M.V	APPROVED CN	SITE 46-326
DRAWN NA	CHECKED M.V	APPROVED CN	DWG PCN-1



REF AECOM Drawing: 6033379-P40.dwg dated June 2015



NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
- 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.

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'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_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 PCN-1

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Poulin Creek Coords: 5 285 664.0 N; 326 267.0 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Tripod and Casing COMPILED BY M.Kh.
DATUM Geodetic DATE January 16, 2015 CHECKED BY M.V.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE										○		
								● QUICK TRIAXIAL × LAB VANE												
441.0	Ground Surface						20	40	60	80	100	20	40	60	kn/m³	GR SA SI CL				
0.0	Snow and ice		1	SS	25	▽* ▼*														
440.5	Sandy silt to silty sand trace clay																			
0.5	Compact Grey Wet to dense		2	SS	14												0 42 57			
			3	SS	34															
438.9	End of borehole						440													
2.1	Casing refusal on probable boulders						439													
	<div>* 2015 01 16</div> <div>▽ Water level observed during drilling</div> <div>▼ Water level measured on completion</div>																			

RECORD OF BOREHOLE No PCN-2

1 of 1

METRIC

G.W.P. 5222-05-00

LOCATION

Poulin Creek

Coords: 5 284 879.3 N: 356 176.9 E

ORIGINATED BY F.P.

DIST Alqoma HWY 129

BOREHOLE

TYPE C.F.S.S.A. + Casing and Dynamic Cone Penetration Test

COMPILED BY M.Kh.

DATUM Geodetic

DATE _____

December 16, 2014

CHECKED BY M.V.

[illegible]

RECORD OF BOREHOLE No PCN-3

1 of 1

METRIC

G.W.P. 5222-05-00

LOCATION

Poulin Creek

Coords: 5 284 874.9 N: 356 186.6 E

ORIGINATED BY F.P.

DIST Alqoma HWY 129

BOREHOLE

TYPE C.F.S.S.A. + Casing + Dynamic Cone Penetration Test

COMPILED BY M.Kh.

DATUM Geodetic

DATE _____

January 06, 2015

CHECKED BY M.V.

[illegible]

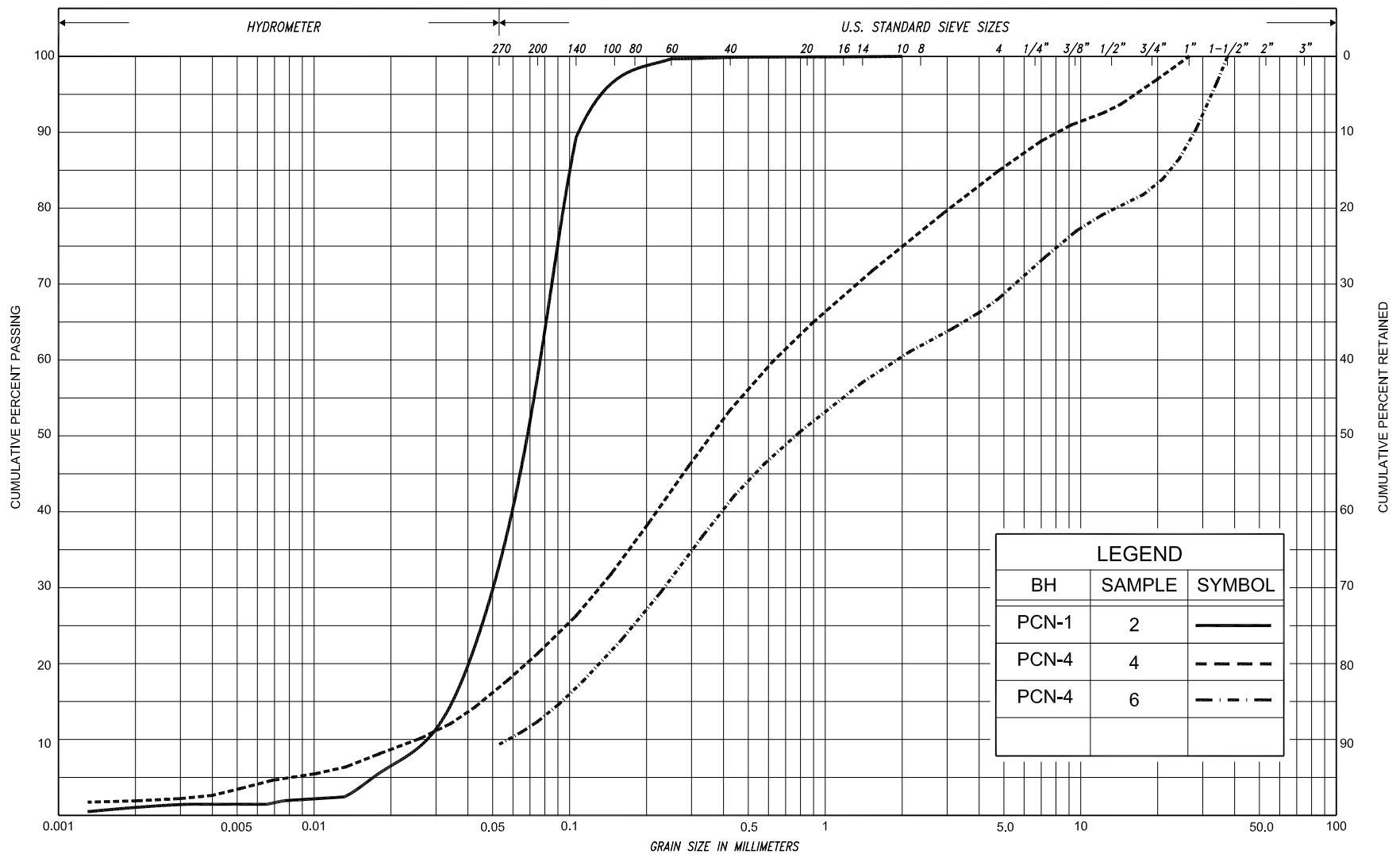
RECORD OF BOREHOLE No PCN-4

1 of 1

METRIC

G.W.P.	5222-05-00	LOCATION	<div> <div>Foulin Creek</div> <div>Coords: 5 285 684.0 N; 326 280.0 E</div> </div>	ORIGINATED BY	F.P.
DIST	Algoma	HWY	129	BOREHOLE TYPE	Tripod and Casing
DATUM	Geodetic	DATE	January 16, 2015	CHECKED BY	M.V.
COMPILED BY	M.Kh.				

[illegible]



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



GRAIN SIZE DISTRIBUTION

SANDY SILT TO SILTY SAND, trace to some gravel, trace clay

FIG No.	PCN-GS-1
HWY:	129
G.W.P. No.	5222-05-00



PART B - PRELIMINARY FOUNDATION DESIGN REPORT

for

POULIN CREEK CULVERT REPLACEMENT

HIGHWAY 129

TOWNSHIP OF DAOUST, ALGOMA DISTRICT, ONTARIO

ASSIGNMENT NO. 5013-E-0040

G.W.P. 5222-05-00

SITE NO. 46-326/C

WP NO. 5230-05-01

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PART B
PRELIMINARY FOUNDATION DESIGN REPORT

for
Replacement of Poulin Creek Culvert
Highway 129 (Site No.46-326/C)
Township of Daoust, Ontario
GWP 5222-05-00, WP No. 5230-05-01

7. INTRODUCTION

This foundation investigation and design report with the interpretation and recommendations are intended for the use of AECOM Canada Ltd on behalf of the ministry of transportation, and shall not be used or relied upon for any other purposes or by any other parties including the construction or design-build contractor. The design-build contractor must make their own interpretation based on the factual data in Part A of the report. Where comments are made on construction, they are provided only to highlight those aspects, which could affect the design of the project. Contractors must make their own interpretation of the factual information provided in Part A of the report, as it may affect equipment selection, proposed construction methods and scheduling.

8. PROJECT DESCRIPTION

8.1 General

This report provides preliminary foundation design recommendations based on interpretation of the geotechnical data presented in the factual report (Part A) to assist the design team in the selection of a suitable type of foundation for the culvert replacement at the crossing of Highway 129 and Poulin Creek in the Township of Daoust, District of Algoma. Based on the General Arrangement drawings (GA) provided by AECOM, it is proposed to replace the existing corrugated steel pipe structure with a 3.6 m wide and 2.7 m high precast concrete box culvert.

The discussions and recommendations presented in this report are based on the GA received by PML and the factual data obtained during the preliminary geotechnical investigation carried out by PML. The designers must review the geotechnical data presented to determine the adequacy of the information for the detail design of the proposed structure. Additional geotechnical investigation must be carried out if the geotechnical data presented is inadequate.



8.2 Existing Culvert

The General Arrangement drawing for the Poulin Creek culvert is appended in Appendix C. The proposed culvert to be replaced is located at the crossing of Poulin Creek and Highway 129 (Sta. 10+000). The existing culvert is a 3.5 m span and 21 m long corrugated steel pipe structure with a fill height of 2.1 m above the crown. Based on the GA drawing provided by AECOM, the invert of the existing culvert at the centerline of Highway 129 (Sta. 10+000) is located at approximate elevation of El. 438.58 and the embankment above the creek bed is approximately 5.5 m high. There is no riprap on either side of the creek, i.e., inlet or outlet, to protect against scour or erosion. The width of the creek in the vicinity of the culvert inlet is about 10.0 m, which indicates that the banks of the creek has been eroded over the years.

This culvert was constructed in 1975 and the road accommodates two lanes of vehicular traffic. The RFP reveals that the condition of the existing culvert is poor with minor deterioration of several elements of the culvert including noticeable sagging in middle section. Further, significant deterioration of the culvert barrel, barrier posts and the structural steel coating require replacement of the existing culvert.

8.3 Proposed Culvert

The RFP specifies that the viability of the following three options required to be evaluated for replacing the existing culvert along the same vertical and horizontal alignments:

- Replacement with a precast concrete box culvert,
- Replacement with a cast-in-place concrete box culvert, and
- Replacement with a three-sided open footing concrete culvert.

Further, it requires that the new culvert shall be designed to be as short as possible with concrete headwalls and wing walls to contain the roadway embankment.

However, the GA provided by AECOM indicate that the proposed replacement structure will be a 30.0 m long precast concrete box culvert with an opening size of 3.6 m in span, 2.7 m in rise and a wall thicknesses of 300 mm. The proposed replacement culvert is 9.0 m longer than the existing and does not include headwalls or wing walls on the GA provided to PML. The proposed invert of the box culvert slopes from about El. 438.78 at the inlet to an elevation of El. 438.58 at the outlet. The founding level of the subgrade at the inlet and outlet is proposed to be at El. 438.18 and El.



437.98, respectively. It is proposed to construct the replacement culvert along the same vertical and horizontal alignment and grade of the road at the culvert location will be maintained at the existing elevation of El. 444.077, which will result in a fill height, including the pavement structure, of about 2.0 m above the box culvert.

There is no local detour available to divert the traffic and the construction of the replacement culvert will be carried out in two stages by allowing the traffic to use one side of the highway. A properly designed temporary roadway protection along the centerline of the road will be required.

8.4 Structure Foundation

In summary, the subsurface stratigraphy at the proposed culvert generally consists of 600 mm pavement structure over 4.1 m to 4.6 m rockfill, followed by sandy silt to silty sand deposits to the maximum depth of investigation of 7.2 m (El. 436.7). Peat deposit was observed immediately below the surface in Borehole PCN-4 located near the outlet. The peat in PCN-4 is followed by sandy silt to silty sand deposit, which extends to the borehole termination depth of 5.3 m (EL. 436.7).

The groundwater level was observed between El. 441.5 and at El. 439.3 during the fieldwork. However, the GA drawing indicate an approximate creek water level of El. 440.1 in September 2014, which is about 1.4 m lower than the highest groundwater level observed during the investigation.

Considering the subsoil conditions, the recommendations for the replacement culvert are provided below in the order of preference. A comparison of the technical advantages and disadvantages for the replacement culvert are presented in Appendix D. The discussions with AECOM suggests that option of using pipe culvert (CSP) need not be considered. Therefore, this report does not include any discussion on the option of using CSP culvert.

8.4.1 Option 1: Precast Concrete Box Culvert

Based on the GA drawing, it is assumed that the precast concrete box culvert will be placed at about elevation El. 438.1±. The subsoil conditions below the proposed founding level is capable of supporting the proposed box culvert. The option of a precast box culvert will require at least 75 mm of levelling course meeting the requirement of OPSS 422.07.08 and bedding material as



specified in OPSS 422.05.13. The bedding for the replacement culvert should be placed in accordance with Section 422.07.07 of OPSS 422.

As required by Clauses 1.9.5.6 and 1.9.11.6.5 of Canadian Highway Bridge Design Code (CHBDC, 2014), cut-off walls at both ends of the culvert shall be provided. Cut-off walls shall be in accordance with OPSD 812.010 or made of precast concrete with similar dimensions to prevent washout of granular bedding with provision to protect the loose sandy subgrade material below invert.

For the design of the 4.2 m wide precast box culvert, a geotechnical resistance of 300 kPa at ULS and 200 kPa at SLS shall be utilized. The design should meet the requirements of Clauses 1.9.5.6 and 1.9.11.6.5 of CHBDC, 2014.

There will be no grade raise of the road to impose additional load on the culvert to cause settlement. Considering the thickness (1.1 m to 1.6 m) of sandy material and the probable very dense or hard layer below the invert, the settlement induced by the bearing resistance at SLS recommended will be about 10 mm to 15 mm and most of the settlement will occur upon completion of construction. Continuing settlements to cause any differential settlements will be negligible.

8.4.2 Option 2: Cast-in-Place Reinforced Concrete Box Culvert

The existing sandy material below the proposed founding surface will require a cut-off wall to prevent washout or undermining of subgrade. In addition, the weight of the cast-in-place concrete culvert will induce substantial settlement compared to precast concrete box culvert and construction under about 3.0 m of ground water will impose greater difficulties.

If this option is considered, the dewatering scheme shall be used to provide working platform for form work and placing concrete. In this case, the footing of box culvert may be placed at about elevation of El. 438.0 if the material below this level is replaced with concrete. The cast in place concrete box culvert may be designed using a geotechnical resistance of 450 kPa if the structure is founded on a concrete working slab placed on very dense non-cohesive soils. The dewatering to construct the cast in place culvert in dry conditions will be a major concern. In view of the construction difficulties, this option is not preferred.



8.4.3 Option 3: Three-Sided Open Concrete Culvert on Strip Footing

The compact sandy material encountered below the proposed founding level of the replacement culvert is susceptible for scour. Section C1.9.11.1 of the Canadian Highway Bridge Design Code commentary (CHBDC, 2014) suggests avoiding placing open footing on material that is susceptible to scour. Same as in Option 2, all the loose sandy material above the very dense or hard layer within the width of the strip footing need to be removed and replaced with lean concrete to the founding level of the footing. It is recommended that the footings for the open culvert be placed at or below El. 438.0 on a concrete working slab placed on very dense non-cohesive material, and designed using a geotechnical resistance of 450 kPa at U.L.S.

Geotechnical resistance or the bearing pressure at SLS will not govern because of the material at the founding elevation as recommended and the load expected from the proposed culvert. The load required to produce detrimental settlement of the structure will be at 300 kPa. The dewatering to construct the cast in place culvert in dry will be a major concern. For these reasons, this option is not preferred.

If Options 2 and 3 are considered, additional investigation will be required to establish the type of material presence below about El. 436.7.

8.4.4 Recommended Option for Culvert Replacement

From a geotechnical perspective and based on the subsurface conditions, precast concrete box culvert placed at about El. 438.3 is the preferred option for the replacement of the existing culvert.

Option 2 and 3 are technically feasible. However, considering the construction difficulties and cost of replacement of sandy material, these options are not recommended.

8.4.5 Lateral Earth Pressure

Earth pressure for the concrete structure should be computed as per the Clause 6.12.2 (b) of Canadian Highway Bridge Design Code (CHBDC, 2014). Sufficient movement of the structure wall may not be permitted for all three options and “at rest” conditions may be assumed for the calculation of earth pressure. The earth pressure calculation should include maximum water level expected in the creek. The lateral earth and water pressure, p (kPa), may be computed using the equivalent fluid



pressures presented in Section 6.12 of the CHBDC, 2014 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 ($\gamma - \gamma_w$) material below design water level (kN/m³)

γ_w = unit weight of water (9.8 kN/m³)

h₁ = depth below final grade (m), above design water level

h₂ = depth below design water level (m)

q = Surcharge load (kPa)

C_p = compaction pressure (refer to Clause 6.12.3 of CHBDC, 2014)

C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.5 of CHBDC, 2014)

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 (24° for Granular A or B Type II)

The seismic site coefficient for the conditions at this site is provided in Section 9.8 of this report. Granular 'A' or 'B' should be utilized as backfill material and should be carried out in accordance with the requirements specified in the OPSS 902. The following parameters are recommended for the granular backfill:

Table 8.4.5: Recommended Geotechnical Parameters

Geotechnical Parameter	Granular 'A'	Granular B Type II
Angle of Internal Friction, degrees	35°	30°
Unit Weight, kN/m ³	22.5± 0.3	21.5 ± 0.3
Coefficient of Active Earth Pressure (K _a)	0.27	0.33
Coefficient of Earth Pressure at Rest (K _o)	0.43	0.5
Coefficient of Passive Earth Pressure (K _p)	3.69	3

Backfill shall be placed simultaneously behind both sides of the culvert, maintaining the height of backfill approximately the same. At no time should the difference in backfill elevation from one side to the other be greater than 500 mm.

8.5 Approach Embankment

The height of the existing approach fill is approximately 5.3 m above the creek bed. PML understands that there will be no increase in the profile grade of the road and it will be maintained



at El. 440.08. No major instability problems are anticipated for the embankment constructed with 2H: 1V side slope. Considering the high water level, the fill should consist of well compacted granular material, preferably Granular B Type II. Any spongy or soft area observed within the base of the embankment should be removed before placing the fill. Rip-rap should be provided on both, the upstream and downstream sides of the creek to protect the toe of the embankments and to prevent erosion of creek bed in the proximity of the culvert. Rip-rap shall be in accordance with OPSD 810.010 and provided to a minimum height of 1.0 m above the high flood level expected in the creek.

9. FOUNDATION FROST DEPTH

In accordance with OPSD 3090.101, a minimum of 2.3 m earth cover is required to protect against the frost penetration in the area where the site is located.

Frost tapers within the granular backfill should be constructed in accordance with OPSD 3101.150. The frost penetration depth, f , is measured from the top of the grade to the bottom of the footing.

10. SEISMIC CONSIDERATIONS

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

10.1 Cover and Backfill

Backfill materials shall meet the requirements of Group I, or Group II specified in OPSS 422.05.14, Table 1 and placed according to the procedures described in Section 422.07.11. It shall be placed in layers not exceeding 200 mm in thickness before compaction and compacted in accordance with OPSS 501. Backfill on each side of the box culvert shall be completed simultaneously and at no time, the levels on each side of the culvert exceeds more than 400 mm. Restrictions on compaction near the culvert shall be as specified in OPSS 902.07.06.02.

Cover material shall meet the requirements of OPSS 422.05.14 and placed in accordance with OPSS 422.07.12.



11. CONSTRUCTION CONSIDERATIONS

11.1 Excavation

Staged construction with a roadway protection system will be required to remove the existing culvert and to install the new culvert while maintaining traffic on Highway 129. 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 (OHSA) and MTO Regulations for Construction Projects. The protection system for excavations should be in accordance with OPSS 539, Construction Specification for Temporary Protection Systems, and OPSS 902, Construction Specifications for Excavating and Backfilling – Structures. Excavated material shall not be stockpiled on top of the excavation.

Based on the record of boreholes, the excavations for the construction of replacement culvert will be advanced through existing granular fill material underlain by native sandy deposit. For OHSA classification purposes, the fill materials and loose sandy deposit should be classified as Type 3 soils. For excavations through multiple soil types, the side slope geometry is governed by the soil with the highest number designation.

11.2 Staged Construction

The construction of culvert replacement is expected to be carried out in two stages. The subsoil conditions encountered at this site and probable bedrock at shallow depth will impose greater difficulty to design and construct a shoring system to maintain traffic on Highway 129. A shoring system consisting of sheet pile wall may not be feasible and the soldier piles with timber lagging may have to be used. The soldier piles will have to be lowered in pre-augured boreholes to adequate depth to provide lateral resistance and to minimise the lateral movement. Alternatively, shoring system consisting of soldier piles may be supported by rock anchors. This type of shoring system will be very costly for the type of proposed structure.

Temporary roadway protection shall be designed to meet a Performance Level of 2 and constructed in accordance with OPSS 539 (Temporary Protection Systems). Additional foundation investigation may be required to determine the type and method of installation of temporary roadway protection system.



12. GROUNDWATER CONTROL

The groundwater level was encountered between El. 441.5 and El. 439.3 and the excavation to the founding level will have to be carried out under 3.2 m high water level. The groundwater level should be lowered to a minimum of 0.5 m below the proposed founding levels to allow for construction in the dry and to place bedding materials.

The creek may have to be temporarily diverted and a cofferdam may be required due to the relatively pervious nature of the subsoil. Cofferdam consisting of sand bags and clay puddle may be constructed by damming the upstream and downstream of the culvert. Dewatering may be carried out from the sumps located along the periphery of the cofferdam. If any environmental restrictions are imposed on placing clay puddle in the creek, the culvert replacement may have to be constructed under the prevailing water level. If the construction is carried out under water, the backfill material shall consist of Granular B Type II containing particle sizes not finer than 75 μm .

The contractor shall be responsible for the selection, performance and detailed design of the dewatering system including the cofferdam. 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 E.

Groundwater levels are subject to seasonal fluctuations and precipitation patterns.

13. SCOPE OF ADDITIONAL INVESTIGATION AND DESIGN SERVICES

The recommendations in this report are preliminary. Detailed foundation engineering services will be required during the Detail Design phase of the project.

The extent of further investigations at this site should include a minimum of 2 boreholes on the Highway 129 for recommendations on roadway protection. The boreholes should extend to sufficient depth to provide information for shoring and dewatering. It is recommended the boreholes at least 3.0 m in to bedrock if it encountered.



14. CLOSURE

The Preliminary Foundation Design portion of this report was prepared by Mr. Mansoor Khorsand, B.Sc., EIT. Project Supervisor and reviewed by Mr. Mark Vasavithasan, MSc.Eng., 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 blue ink signature of Mansoor Khorsand is written over a circular professional engineer stamp. The stamp contains the text 'REGISTERED PROFESSIONAL ENGINEER', 'C M P Nascimento', and 'Sep 21, 2016'.

Mansoor Khorsand, B.Sc., EIT.
Project Supervisor, Geotechnical Services



Mark Vasavithasan, M.Sc.Eng., P.Eng.
Senior Engineer, Geotechnical Services



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



APPENDIX C

Poulin Culvert General Arrangement



APPENDIX D

Comparison of Alternate Culvert Options



Comparison of Alternate Culvert Options

Option 1: Precast Concrete Box Culvert	Option 2: Cast In-Place Concrete Box Culvert	Option 3: Three Sided Open Culvert
Advantages: <ol style="list-style-type: none"> 1. High degree of quality and uniformity, design flexibility, superior strength and durability 2. Reduced weather dependency during installation 3. Reduced impact on traffic interruption 4. Ease of construction and installation in wet conditions is possible 	Advantages: <ol style="list-style-type: none"> 1. Reduces uneven settlement 2. Reduces water leakage and deterioration of culvert 3. Ability to withstand differential settlements 4. Longer life span of the structure 5. Degradation of subgrade can be avoided 6. Replacing of overburden with concrete can be done under water using tremie 	Advantages: <ol style="list-style-type: none"> 1. Generally allows for natural streambed to remain intact 2. Less accumulation of sediments in the upstream of channel 3. Permits the removal of overburden within the width of footing 4. Lower cost than other options for removal of overburden and to replace with lean concrete 5. High geotechnical resistance available to support the culvert on strip footings 6. Ease of installation for precast open culvert 7. Natural stream bed may be maintained
Disadvantages: <ol style="list-style-type: none"> 1. Natural stream bed will not remain intact 2. Cause sediment accumulation in the upstream of the channel 3. Higher cost for removal of overburden below groundwater level 4. Possibility for degradation of subgrade 	Disadvantages: <ol style="list-style-type: none"> 1. Natural stream bed will not remain intact 2. Cause sediments accumulation in the upstream of the channel 3. Higher cost for removal of overburden and replace with lean concrete 4. Weather dependent during construction 5. Major dewatering scheme is required to construct the floor slab under 3.5 m high water 	Disadvantages: <ol style="list-style-type: none"> 1. Increased impact on commuting and longer interruption of traffic 2. Weather dependent during construction
Cost of Construction: Total Cost \$ 13,000/m	Cost of Construction: Total Cost \$ 15,000/m	Cost of Construction: Total Cost \$ 14,000/m
Recommended	Technically Feasible but Not Recommended	Technically Feasible but Not Recommended



APPENDIX E

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 3101.150	Walls Abutment, Backfill Minimum Granular Requirement
OPSD 3121.150	Walls, Retaining, Backfill, Minimum Granular Requirement
OPSD 812.010	Cut off Wall for Structural Plate Pipe Arch and Circular CSP



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.