



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for**

**FIVE MILE CREEK CULVERT REPLACEMENT
HIGHWAY 129
TOWNSHIP OF REANEY, ALGOMA DISTRICT, ONTARIO
ASSIGNMENT NO. 5013-E-0040
GWP 5222-05-00
SITE NO. 46-006/C
WP 5227-05-01**

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PML Ref.: 14TF038
Index No.: 060FIR and 061FDR
GEOCRES No.: 41O-14
September 27, 2016



PART A - PRELIMINARY FOUNDATION INVESTIGATION REPORT

for

FIVE MILE CREEK CULVERT REPLACEMENT

HIGHWAY 129

TOWNSHIP OF REANEY, ALGOMA DISTRICT, ONTARIO

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Part A – Preliminary Foundation Investigation Report
Five Mile Creek Culvert Replacement, Highway 129, Township of Reaney, Algoma District, Ontario
GWP 5222-05-00, WP 5227-05-01, Site No. 46-006/C, Index No.: 060FIR
PML Ref.: 14TF038, September 27, 2016, TOC 1 of 1

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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT

for
Five Mile Creek Culvert Replacement
Highway 129 (Site No. 46-006/C)
Township of Reaney, Algoma District, Ontario
GWP 5222-05-00, WP 5227-05-01

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 Five Mile Creek and Highway 129 (Sta. 10+000) in the Township of Reaney, Algoma District, Ontario.

The fieldwork was carried out on December 15, 2014, January 8, and January 9, 2015. The purpose of the investigation was to explore the subsurface conditions expected to influence of the preliminary design of the Five Mile 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 Five Mile Creek culvert replacement.

2. SITE DESCRIPTION

The proposed replacement culvert is located approximately 86.1 km north of the intersection of Highway 129 and Highway 556. The topography of the project area is generally flat, except for the highway embankments. Five Mile Creek flows from the east to Hill Lake located on the west side of Highway 129. Generally, the site surrounding the culvert is covered with bushes and grass. The area along the highway on both, north and south, sides is heavily wooded.

The existing culvert is a 3.90 m span and 24 m long corrugated steel elliptical pipe structure with a fill height of 1.2 m above the crown. This culvert was constructed in 1982 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.



3. FIELD INVESTIGATION PROCEDURES

The staff of PML visited the site on December 14, 2014 and January 7, 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 FM-1, FM-2, FM-3 and FM-4 to maximum depths ranging from 6.4 m to 8.1 m (El. 444.9 to El. 445.6). Boreholes FM-2 and FM-3 were located on the paved area of the road and these boreholes were advanced using hollow stem augers aided by a truck-mounted drill rig. Boreholes FM-1 and FM-4 were located at the outlet and inlet of the culvert, respectively. These two boreholes were advanced using a track-mounted drill rig. Location of boreholes is shown on the attached Drawing No. FM-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 (30)
- Grain size distribution analyses (11)

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 Figures FM-GS-1 to FM-GS-3. 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 project site lies in a deep valley located within Pre-Cambrian rock formations, which in this area consist mainly of granite and other intrusive rocks such as tonalite to granodiorite foliated to gneissic with minor supracrustal inclusions. During the last ice age, continental glaciers eroded much of the bedrock and laid down a shallow mantle of glacial debris that covered the area. Based on the Quaternary Geology map published by Ontario Ministry of Northern Development and Mines, the surface conditions in the vicinity of the project area consist of Glaciofluvial ice-contact deposits, which includes gravel, sand and, and minor till includes esker, kame, end moraine, ice-marginal delta and subaqueous fan deposit.

5.2 Subsurface Conditions

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 Drawing FM-1. The boundaries between



soil strata have been established at the borehole locations only. The boundaries of soil strata between and beyond the boreholes are assumed and it may vary from location to location.

In general, the subsoil conditions consist of 1.2 m to 4.9 m silty sand fill, followed by 1.5 m to 2.4 m sandy silt in two of the boreholes located on the pavement. In Borehole FM-4, the fill layer is underlain by 2.1 m thick peat deposit. Peat deposit was observed immediately below the 500 mm thick ice surface in FM-1, which is followed by 0.9 m sandy silt. The sandy silt layer is underlain by silty sand deposit, which extends to the maximum depth of investigation of 8.1 m and to elevation El. 444.9. For classification purposes, the soils encountered at this site can be divided into five distinct zones.

- a) Pavement Structure
- b) Silty Sand, Trace Clay (Granular Fill)
- c) Peat
- d) Sandy Silt, Trace Gravel, Trace Clay
- e) Silty Sand, Some Gravel, Trace Clay

5.2.1 Pavement Structure

Asphalt layer, approximately 200 mm to 230 mm thick, was encountered in two of the boreholes (FM-2 and FM-3) located on the paved area. Pavement structure consists of varying proportions of sand and gravel (base). The thickness of this granular base was observed to be about 100 mm to 570 mm and extends to a depth of 800 mm (El. 453.0 to El. 453.4). The moisture content of the granular base layer was about 5.7%.

5.2.2 Silty Sand, Trace Clay (Granular Fill)

This granular fill layer was encountered in three of the boreholes (FM-2, FM-3 and FM-4). This fill layer was encountered immediately below 200 mm topsoil in FM-4 and extends to a depth of 1.4 m below the surface. In Boreholes FM-2 and FM-3; it was encountered immediately below the pavement structure. This fill layer ranges in thickness from 3.5 m to 4.9 m, and extends to a maximum depth of 5.2 m (El. 448.5) below the existing grade. The SPT values in this fill layer varies from as low as 1 blows/300 mm to 12 blows/300 mm, indicating very loose to compact state of denseness.



The moisture content of this fill material varies from 10.9% to 29.8% with an average value of 21.3%. The results of the grain size distribution analyses performed on two representative samples from this fill layer are shown on Figure FM-GS-1. The test results reveal that the silty sand fill consists of 0% gravel, 70% to 80% sand, 19% to 29% silt and 1% clay.

5.2.3 Peat

This peat deposit immediately underlies the fill layer in Borehole FM-4. In FM-1, peat deposit was encountered immediately below 500 mm thick snow and ice. Peat deposit in FM-4 is laminated with thin seams of sand. The thickness of this peat deposit range from 1.6 m to 2.1 m, and extends to a maximum depth of 3.5 m (El. 448.6) below the existing surface at the inlet. The SPT values in this deposit range from 1 blows/300 mm to 6 blows/300 mm. Moisture content of this deposit varies widely from 25.5% to 51.9%.

5.2.4 Sandy Silt, Trace Gravel, Trace Clay

The peat deposit in Boreholes FM-1 and FM-4, and the fill layer in boreholes FM-2 and FM-3 are immediately underlain by this sandy silt layer, which extends to a maximum depth of 6.7 m (El. 447.0). The SPT values in this deposit range from as low as 1 blows/300 mm to 28 blows/300 mm, indicating very loose to compact state of compaction.

The moisture content of samples tested from this layer varied from 11.4% to 25.9%. The results of the sieve analysis test performed on four representative samples from this deposit are provided on Figure FM-GS-2. The test results indicate that this deposit consists of 0% to 2% gravel, 9% to 49% sand, 50% to 88% silt and 1% to 3% clay.



5.2.5 Silty Sand, Some Gravel, Trace Clay

The sandy silt deposit is immediately underlain by this silty sand layer, which extends to the maximum depth of investigation of 8.1 m and to elevation El. 444.9. The SPT values in this deposit range from as low as 3 blows/300 mm to as high as 108 blows/300 mm, indicating very loose to very dense state of compaction.

The moisture content of samples tested from this layer varied from 7.8% to 25.8%. The results of the sieve analysis test performed on five representative samples from this deposit are provided on Figure FM-GS-3. The test results indicate that this deposit consists of 11% to 18% gravel, 50% to 62% sand, 12% to 32% silt and 6% to 8% clay.

5.2.6 Groundwater

The groundwater was observed during and upon completion of drilling. The groundwater levels were measured at a depth of 2.13 m to 5.94 m (El. 451.8 to El. 447.9), below the existing grade of the road. However, it was observed at a depth of 0.46 m and 1.4 m (El. 450.8 and El. 450.7) below the ground surface of the inlet and outlet.

The groundwater level may fluctuate due to the influence of precipitation and seasonal changes.



6. CLOSURE

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

This report was prepared by Ms. Asieh Khadem, M.Sc. Eng., EIT, Project Supervisor and reviewed by Mark Vasavithasan, M.Sc. 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.

Asieh Khadem

Asieh Khadem, M.Sc. Eng., EIT
Project Supervisor, Geotechnical Services



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

AK/MV/CN:mk



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

Part A – Preliminary Foundation Investigation Report

Five Mile Creek Culvert Replacement, Highway 129, Township of Reaney, Algoma District, Ontario

GWP 5222-05-00, WP 5227-05-01, Site No. 46-006/C, Index No.: 060FIR

PML Ref.: 14TF038, September 27, 2016



APPENDIX A

Site Photographs



Photograph P1: Looking south-east at the location of Borehole FM-1. (January 9, 2015)



Photograph P2: Looking east from Highway 129 northbound lane shoulder. Borehole FM-4 was drilled at southeast of the culvert outlet. (January 8, 2015)



Photograph P3: Looking southeast at the culvert outlet. (December 15, 2014)



Photograph P4: Looking south from Highway 129 southbound lane shoulder. (January 8, 2015)



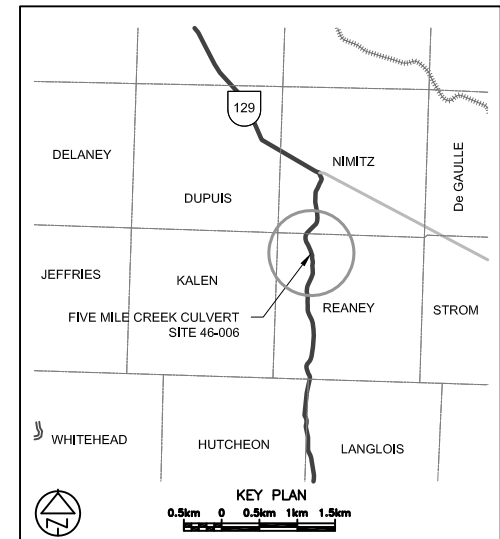
APPENDIX B

Borehole Locations Plan and Soil Strata at Five Mile Creek Culvert

Explanation of Terms Used in Report

Record of Borehole Sheets

Results of Grain Size Distribution Analyses – Figures FM-GS-1 to FM-GS-3



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
FM-1	451.3	5 270 083.0	364 136.2
FM-2	453.8	5 270 077.0	364 148.7
FM-3	453.7	5 270 087.3	364 152.5
FM-4	452.1	5 270 079.6	364 162.5

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

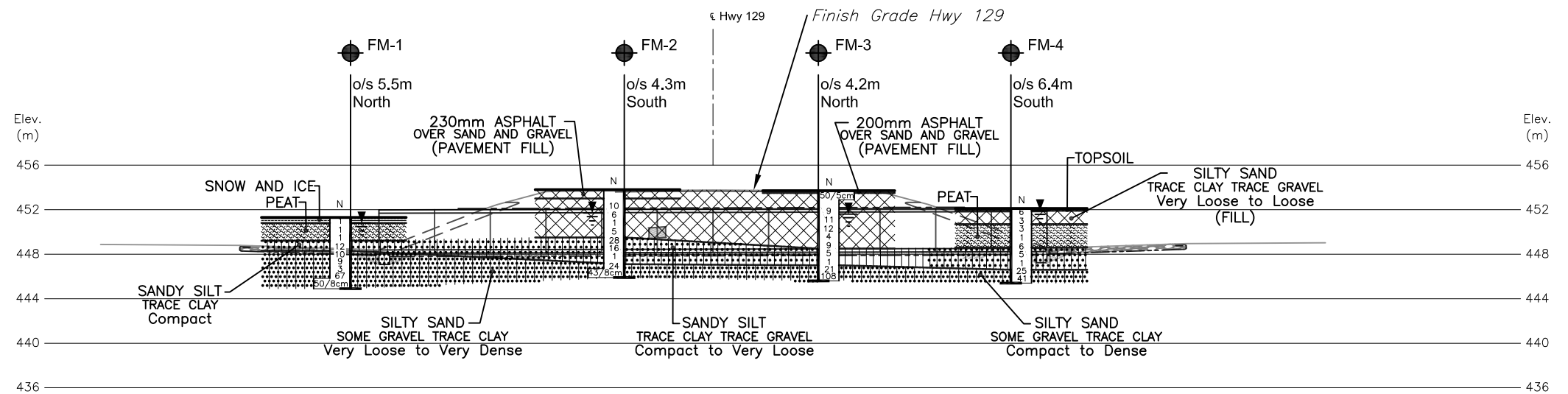
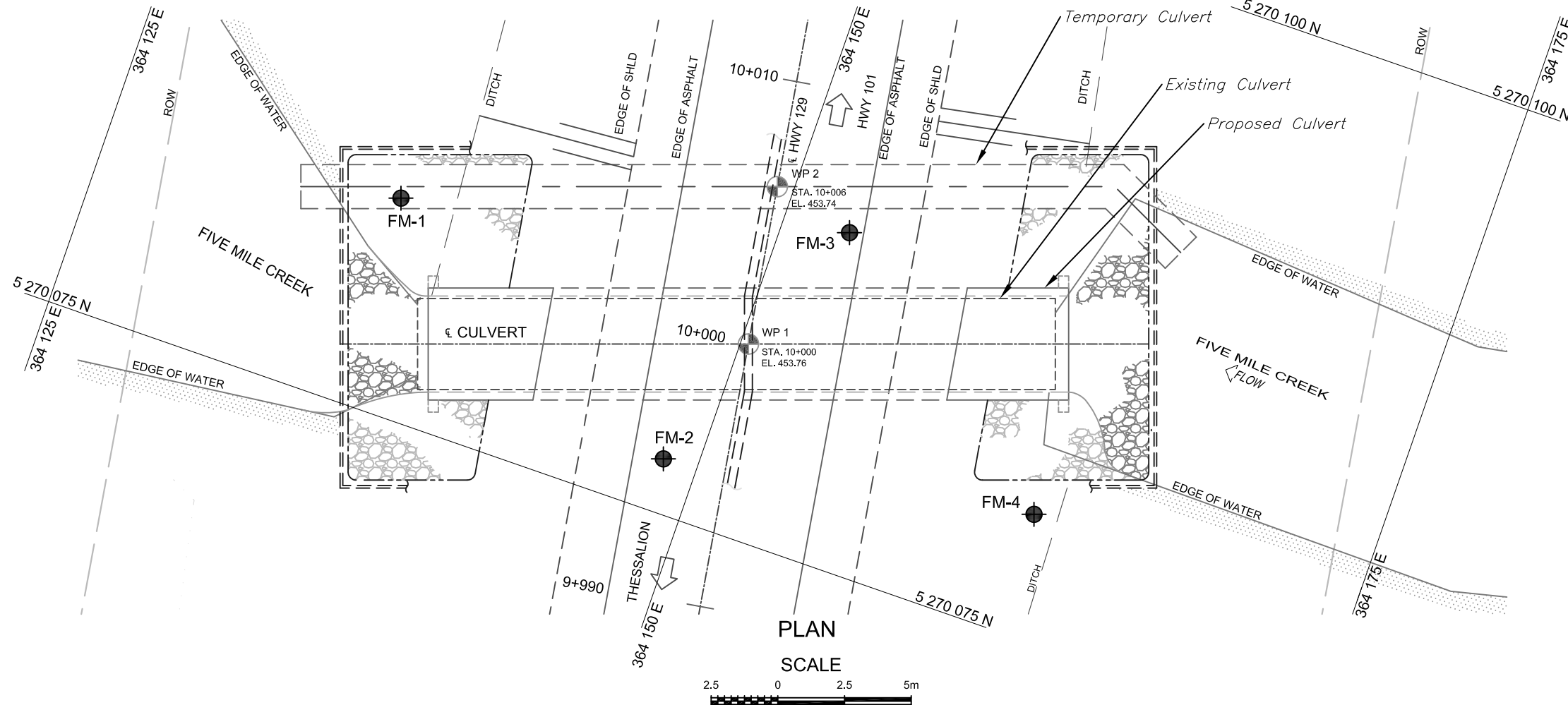
DATE	BY	DESCRIPTION

Geocres No. 400-14

HWY No	129	CHECKED	AK	DATE	SEPT. 27, 2016	DIST	ALGOMA
SUBM'D	NA	CHECKED	AK	DATE	SEPT. 27, 2016	SITE	46-006/C
DRAWN	NA	CHECKED	MV	APPROVED	CN	DWG	FM-1



REF AECOM Drawing: 60333079-P30.dwg dated June 2015



PROFILE ALONG CENTRELINE FIVE MILE CREEK CULVERT
SCALE
HORIZONTAL



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				i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

RECORD OF BOREHOLE No FM-1

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 083.0 N; 364 136.2 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE January 09, 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 kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								20 40 60 80 100									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
20 40 60 80 100					WATER CONTENT (%)												
451.3	Ground Surface																
0.0	Snow and ice		1	SS	1	▽*	451										
450.8	Peat, fine fibrous																
0.5	Dark Brown		2	SS	1		450										
			3	SS	1												
449.2	Sandy silt, trace clay						449										
2.1	Compact Grey Wet		4	SS	12							○				0 49 50 1	
448.3	Silty sand some gravel, trace clay						448					○					
3.0	Very loose Grey Moist to loose to wet		5	SS	10							○				16 40 40 4	
			6	SS	9		447					○					
			7	SS	3							○					
			8	SS	67		446					○				11 51 31 7	
444.9	Very dense																
6.4	End of borehole		9	SS	50/8cm		445					○					
	Sample 9: Sampler bouncing																
	* 2015 01 09																
	▽ Water level observed during drilling																
	▼ Water level measured after drilling																
	NOTE: Borehole caved in at 2.7m																

RECORD OF BOREHOLE No FM-2

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 077.0 N; 364 148.7 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE December 15, 2014 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 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												
453.8	Ground Surface						20	40	60	80	100									
0.0	230mm asphalt over sand and gravel		1	AS																
	(PAVEMENT FILL)																			
453.0	Silty sand, trace clay organics						453													
0.8	Loose to Grey Wet very loose		2	SS	10		452													
	(FILL)		3	SS	6		451									0 80 19 1				
			4	SS	1		450													
449.5			5	SS	5		449													
4.3	Sandy silt trace clay, trace gravel		6	SS	28		448									1 13 83 3				
	Compact to Grey Wet very loose		7	SS	16		447													
			8	SS	1		446									18 44 30 8				
447.1	Silty sand some gravel, trace clay		9	SS	24															
6.7	Compact to Grey Moist very dense		10	SS	43/8cm															
445.9	End of borehole																			
7.9	Sample 10: Sampler bouncing																			
							</													

RECORD OF BOREHOLE No FM-3

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 087.3 N; 364 152.5 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE December 15, 2014 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 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												
453.7	Ground Surface						20	40	60	80	100									
0.0 453.4 0.3	200mm asphalt over sand and gravel (PAVEMENT FILL)		1	AS	50/5cm															
	Silty sand, trace clay organics																			
	Loose to Brown Wet very dense to grey		2	AS	9															
			3	SS	11															
			4	SS	12															
			5	SS	4															
	(FILL)		6	SS	9															
448.5																				
5.2	Sandy silt trace clay, trace gravel		7	SS	5															
	Loose to Grey Moist very loose																			
			8	SS	1															
447.0																				
6.7	Silty sand some gravel, trace clay																			
	Compact to Grey Moist very dense		9	SS	21															
			10	SS	108															
445.6																				
8.1	End of borehole																			
	Sample 1: Sampler bouncing																			

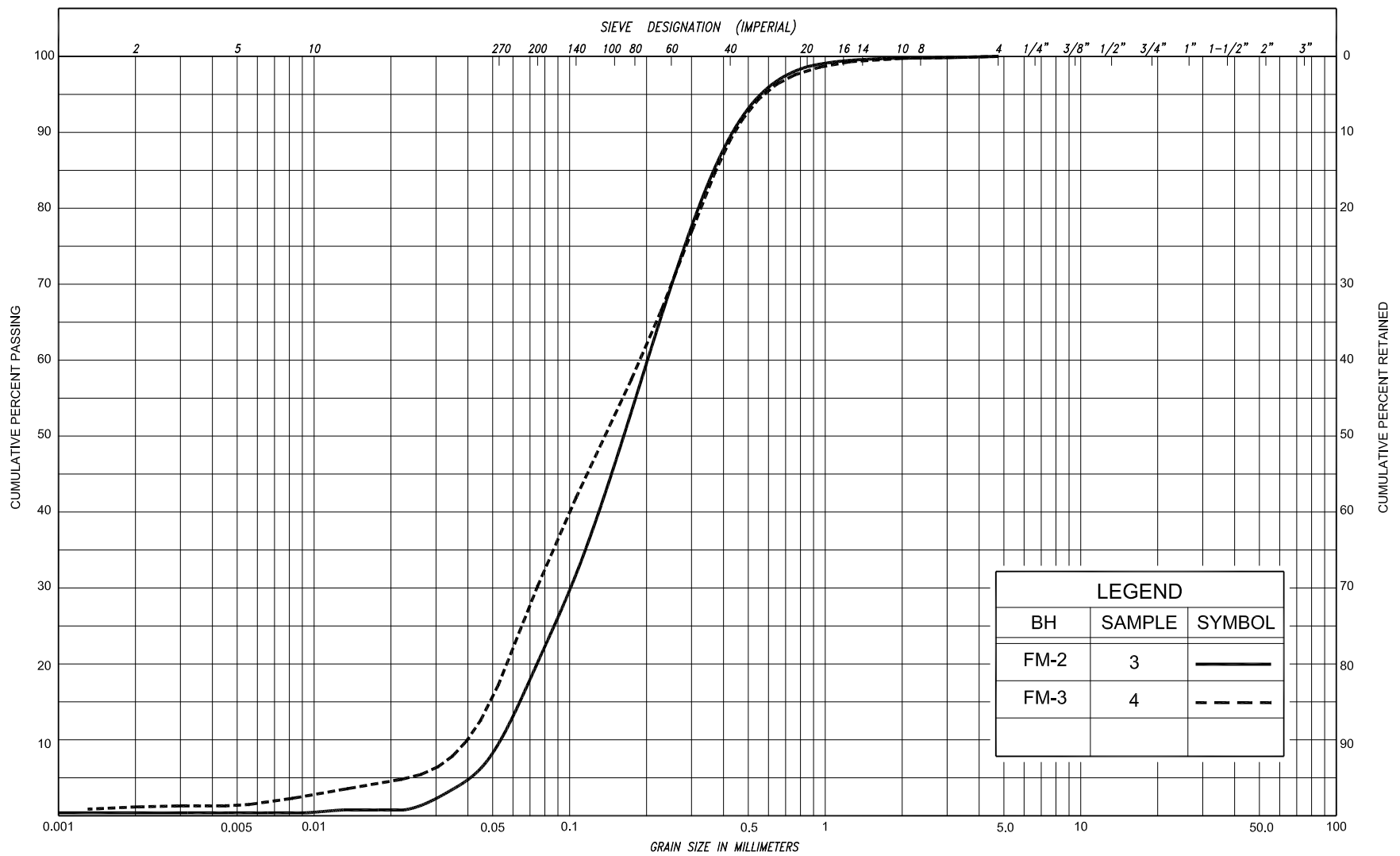
RECORD OF BOREHOLE No FM-4

1 of 1

METRIC

G.W.P. 5222-05-00 LOCATION Five Mile Creek Coords: 5 270 079.6 N; 364 162.5 E ORIGINATED BY F.P.
DIST Algoma HWY 129 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY A.K.
DATUM Geodetic DATE January 08, 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 kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
452.1	Ground Surface					▽*	452													
451.9	Topsoil																			
0.2	Silty sand, trace gravel organics		1	SS	6															
	Very loose Brown Wet to loose (FILL)		2	SS	3															
450.7																				
1.4	Peat, amorphous sand seams		3	SS	3															
	Dark brown		4	SS	1															
448.6			5	SS	6															
3.5	Sandy silt trace clay, trace gravel																			
	Loose to Grey Moist very loose to wet		6	SS	5															
			7	SS	1															
446.6			8	SS	25															
5.5	Silty sand some gravel, trace clay																			
	Compact Grey Moist to dense		9	SS	41															
445.4																				
6.7	End of borehole																			



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												

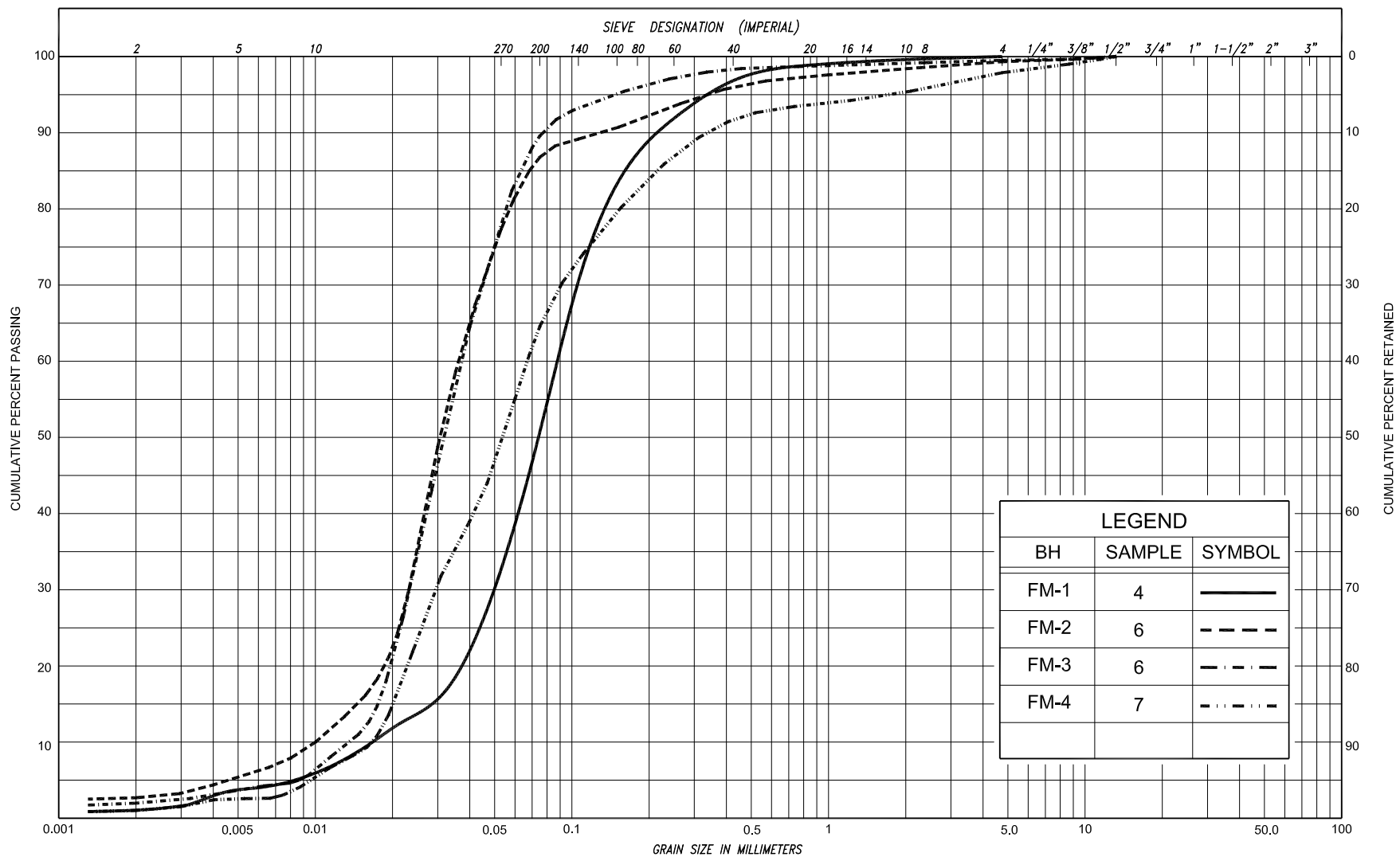
GRAIN SIZE DISTRIBUTION SILTY SAND, trace clay (FILL)

FIG No. FM-GS-1

HWY: 129

G.W.P. No. 5222-05-00





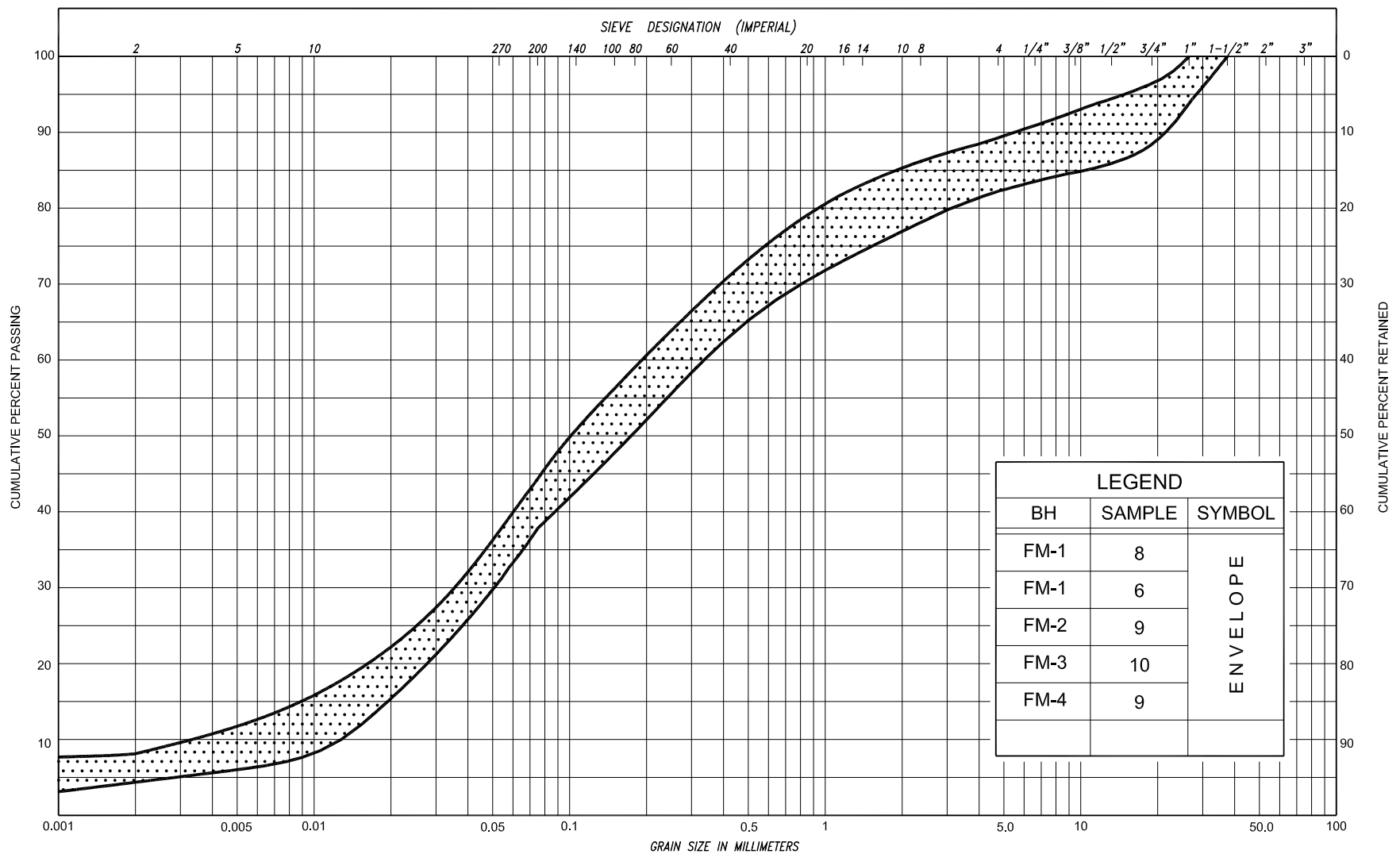
SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED	
CLAY	FINE		MEDIUM		COARSE	SAND									
	SILT					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	M.I.T.
CLAY		SILT			V. FINE	FINE	MED.	COARSE	GRAVEL						U.S. BUREAU
				SAND											



GRAIN SIZE DISTRIBUTION

SANDY SILT, trace clay, trace gravel

FIG No.	FM-GS-2
HWY:	129
G.W.P. No.	5222-05-00



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



GRAIN SIZE DISTRIBUTION

SILTY SAND, some gravel, trace clay

FIG No.	FM-GS-3
HWY:	129
G.W.P. No.	5222-05-00



PART B – PRELIMINARY FOUNDATION DESIGN REPORT

for

FIVE MILE CREEK CULVERT REPLACEMENT

HIGHWAY 129

TOWNSHIP OF REANEY, ALGOMA DISTRICT, ONTARIO

ASSIGNMENT NO. 5013-E-0040

GWP 5222-05-00

SITE NO. 46-006/C

WP 5227-05-01

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Index No.: 061FDR
GEOCRES No.: 41O-14
September 27, 2016



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Appendix D – Comparison of Alternate Culvert Options

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PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for
Five Mile Creek Culvert Replacement
Highway 129 (Site No. 46-006/C)
Township of Reaney, Algoma District, Ontario
GWP 5222-05-00, WP 5227-05-01

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 Five Mile Creek in the Township of Reaney, District of Algoma. Based on the General Arrangement drawings (GA) provided by AECOM, it is proposed to replace the existing culvert with a 3.6 m wide and 3.3 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 proposed culvert to be replaced is located at the crossing of Five Mile Creek and Highway 129 (Sta. 10+000). The existing culvert is a 3.90 m span and 24 m long corrugated steel ellipse pipe structure with a fill height of 1.2 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. 448.66 and the embankment above the creek bed is approximately 5.4 m high. There is no riprap on either side of the creek, i.e., inlet or outlet, to protect against scour or erosion. A review of the Google Earth Map indicates that the toe of the embankment on both sides of the culvert is eroded and undermined.

This culvert was constructed in 1982 and the road accommodates two lanes of vehicular traffic. The RFP reveals that the condition of the existing culvert is poor with deficiency in several elements and more significantly, deterioration of the coating protecting the culvert. Further, culvert barrel is experiencing sagging in the mid section and requires replacement.

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 24.0 m long precast concrete box culvert with an opening size of 3.6 m in span, 3.3 m in rise and a wall thicknesses of 300 mm. The proposed replacement culvert does not include headwalls or wing walls on the GA provided to PML. The proposed invert of the box culvert slopes from about El. 448.53 at the inlet to an elevation of 448.38 at the outlet. The founding level of the subgrade at the inlet and outlet is proposed to be at El. 447.93 and El. 447.78, 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. 453.76, which will result in fill height including the pavement structure of 1.8 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 consist of 1.2 m to 4.9 m silty sand fill, followed by 1.5 m to 2.4 m sandy silt in two of the boreholes located on the pavement. In Borehole FM-4, the fill layer is underlain by 2.1 m thick peat deposit. Peat deposit was observed immediately below the 500 mm thick ice surface in FM-1, which is followed by 0.9 m sandy silt. The sandy silt layer is followed by silty sand deposit, which extends to the maximum depth of investigation of 8.1 m and to elevation El. 444.9. The groundwater level was observed between El. 451.8 to El. 447.9 during the fieldwork. However, the GA drawing indicate an approximate creek water level of El. 449.8 in September 2014, which is about 2 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 a corrugated steel plate arch or pipe culvert need not be considered. Therefore, this report does not include any discussion on the option of using an arch or pipe culvert.

8.4.1 Option 1: Precast Concrete Box Culvert

Based on the GA drawing, it is assumed that the precast box culvert will be placed at about El. 447.8±. The subsoil conditions below the proposed founding level is capable of supporting 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 (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 silty subgrade material below invert. For the design of the 4.2 m wide precast box culvert, a geotechnical resistance of 200 kPa at ULS and 130 kPa at SLS shall



be utilized. The design of cut-off wall shall meet the requirements of clauses 1.9.5.6 and 1.9.11.6.5 of CHBDC 2014, to protect against scour or undermining.

The grade of the road is expected to be maintained at the existing elevation. Therefore, there will be no additional load on the culvert to induce settlement. Considering the thickness (0.7 m to 1.2 m) of silty material below the invert level, the total 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 silt to silty sand material below the proposed founding surface will require a cut-off wall to prevent scour or washout. In addition, the weight of the cast-in-place concrete culvert will induce substantial settlement compared to precast concrete box culvert and construction under 4.0 m of ground water will impose greater difficulties for construction in dry conditions.

If this option is considered, the dewatering scheme shall be used to provide working platform for form work and placing of concrete. In this case, the footing of box culvert may be placed at about El. 447.8 if the loose to compact sandy material below this level to about El. 446.0 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 dense sandy soil. The dewatering to construct the cast in place culvert in dry condition will be costly and impose greater difficulties. In view of the construction difficulties, this option is not preferred.

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

The loose sandy silt 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. It is recommended that the footings for the open culvert be placed at or below El. 446.0 on dense sandy material, and designed using a geotechnical resistance of 450 kPa at ULS. Alternatively, the sandy material below the proposed invert level to El. 446.0 is replaced with concrete and the footing may be placed at El.447.8.



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 much larger than the recommended value for factored geotechnical resistance at ULS.

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 447.8 is the preferred option for the replacement of the existing culvert.

Options 2 and 3 are technically feasible. However, considering the construction difficulties and cost of dewatering 5.8 m high groundwater, 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)
 h_1 = depth below final grade (m), above design water level
 h_2 = 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 \emptyset = angle of internal friction of retained soil (35° for Granular A or 30° for Granular 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 10 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 5.4 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. 453.76. 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 silt deposit. For OHSA classification purposes, the fill materials and loose sandy silt 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 very dense material at shallow depths 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. 451.8 to El. 447.9 and the excavation to the founding level will have to be carried out under 4.0 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 coffer dam. If any environmental restrictions are imposed on placing clay puddle in the lake, 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

This Foundation Investigation and Design Report was prepared by Ms. Asieh Khadem, M.Sc. Eng., 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.

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Project Manager and
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Part B – Preliminary Foundation Design Report

Five Mile Creek Culvert Replacement, Highway 129, Township of Reaney, Algoma District, Ontario

GWP 5222-05-00, WP 5227-05-01, Site No. 46-006/C, Index No.: 061FDR

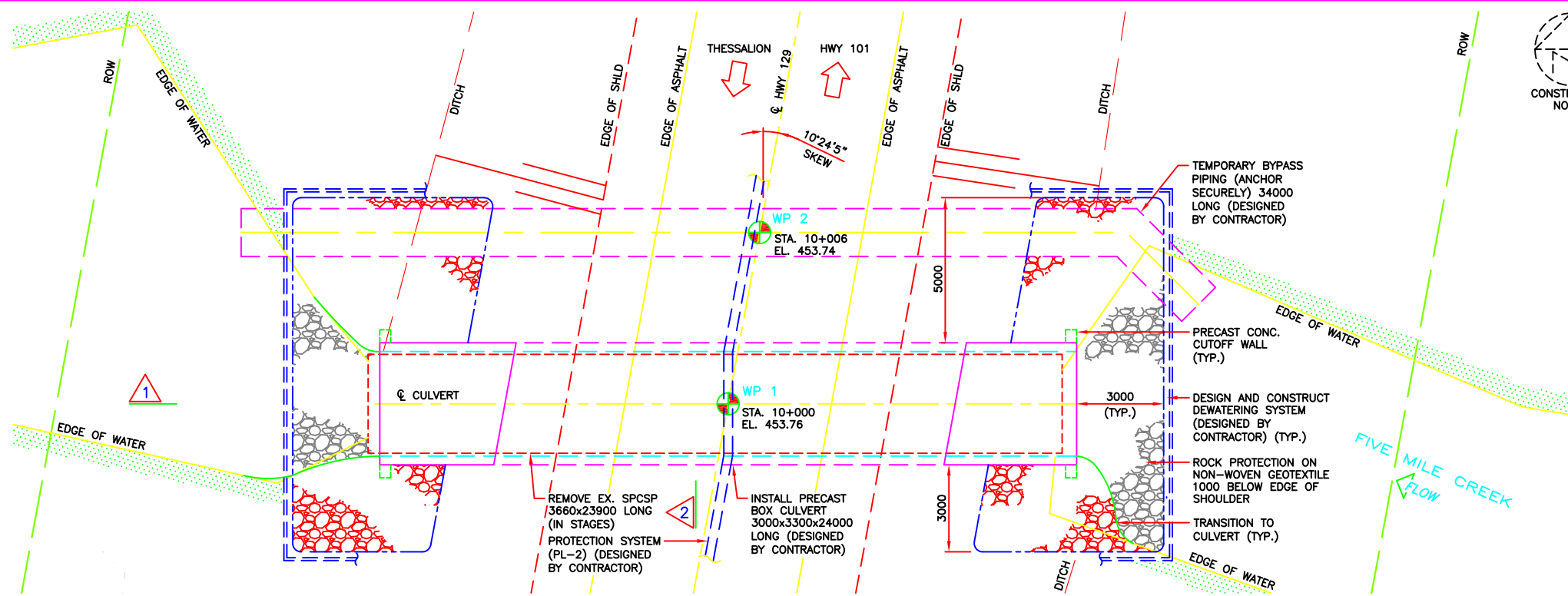
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APPENDIX C

Five Mile Creek Culvert General Arrangement

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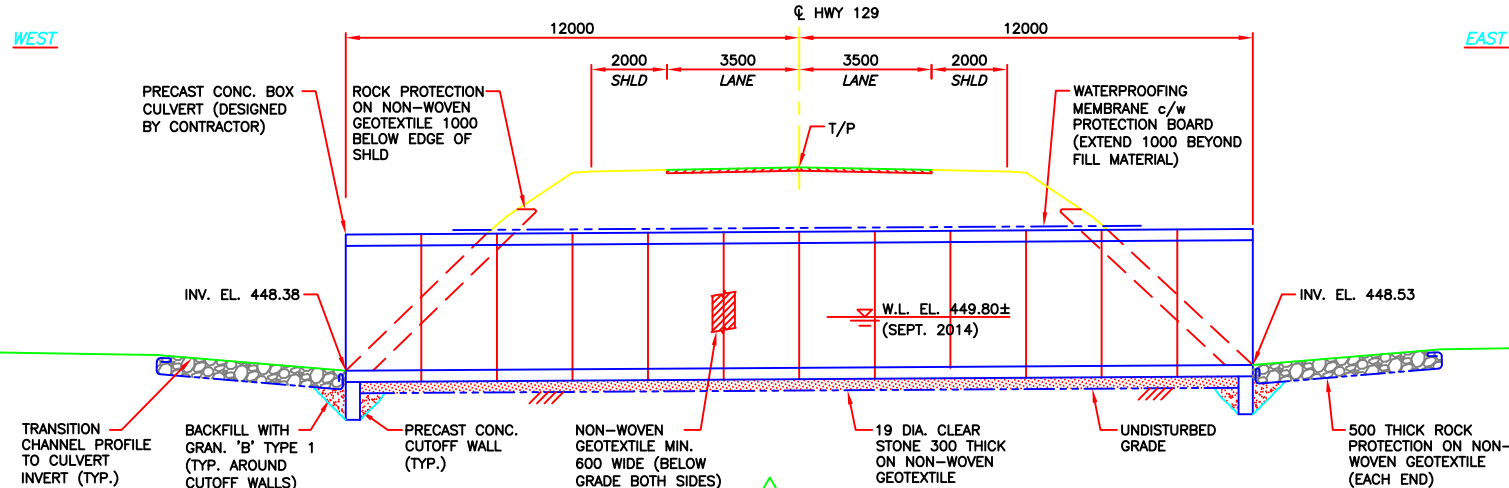


PLAN
1 : 200

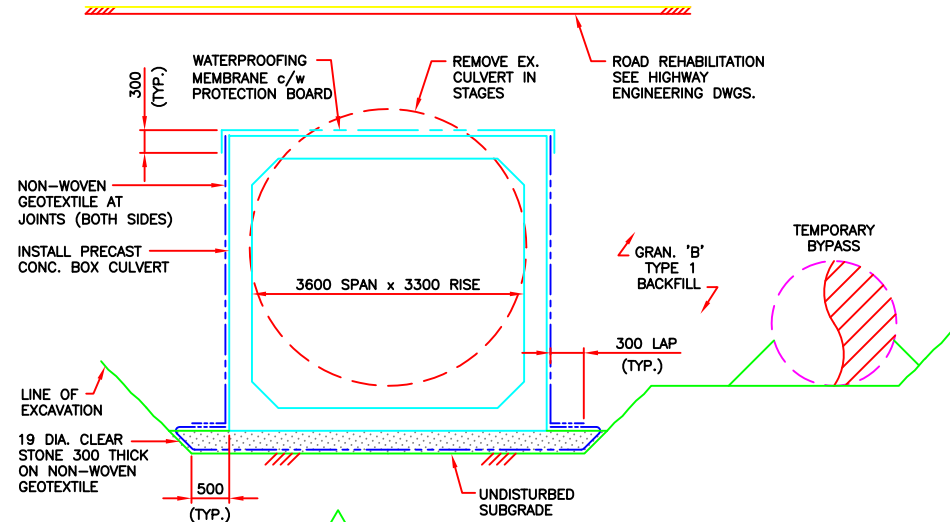
WEST

EAST

EL. 454.00
EL. 453.00
EL. 452.00
EL. 451.00
EL. 450.00
EL. 449.00
EL. 448.00



SECTION
1 : 200



SECTION
2 : 100



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

HWY 129
CONT No
WP No 5227-05-01



FIVE MILE CREEK CULVERT
STA. 10+000, REANEY TWP.
GENERAL ARRANGEMENT

SHEET

AECOM

GENERAL NOTES :

1. CLASS OF CONCRETE : PRECAST 40 MPa
2. CLEAR COVER TO REINFORCING STEEL : PRECAST 50 ± 10
3. REINFORCING STEEL :
 1. REINFORCING STEEL SHALL BE GRADE 400W UNLESS OTHERWISE SPECIFIED.
 2. UNLESS SHOWN OTHERWISE, TENSION LAP SPLICES FOR REINFORCING STEEL SHALL BE CLASS B.
 3. BAR HOOKS SHALL HAVE STANDARD HOOK DIMENSIONS USING MINIMUM BEND DIAMETERS, WHILE STIRRUPS AND TIES SHALL HAVE MINIMUM HOOK DIMENSIONS. ALL HOOKS SHALL BE IN ACCORDANCE WITH THE STRUCTURAL STANDARD DRAWING SS12-1, UNLESS INDICATED OTHERWISE.
4. GEOTEXTILE :
 1. NON-WOVEN, CLASS II, FOS 50 TO 100µm.

CONSTRUCTION NOTES :

1. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS OF THE EXISTING WORK AND ALL DETAILS ON SITE AND REPORT ANY DISCREPANCIES TO THE CONTRACT ADMINISTRATOR BEFORE PROCEEDING WITH THE WORK.
2. THE CONTRACTOR SHALL CARRY OUT SITE SURVEYS TO DETERMINE THE EXISTING ELEVATIONS OF ASPHALT PRIOR TO REMOVALS.
3. BACKFILL SHALL BE PLACED SIMULTANEOUSLY BEHIND BOTH CULVERT WALLS, KEEPING THE HEIGHT OF BACKFILL APPROXIMATELY THE SAME, AT NO TIME SHALL THE DIFFERENCE IN BACKFILL HEIGHTS BE GREATER THEN 200mm.
4. ALL SITE ACCESS TO COMPLETE THE WORK IS THE RESPONSIBILITY OF THE CONTRACTOR.

APPLICABLE STANDARD DRAWINGS :

OPSD 3941.200 FIGURES IN CONCRETE, SITE NUMBER, AND DATE, LAYOUT

LIST OF ABBREVIATIONS :

CL	CENTRELINE	SBGR	STEEL BEAM GUIDE RAIL
CONC.	CONCRETE		
c/w	COMPLETE WITH	SPCSP	STRUCTURAL PLATE CORRUGATED STEEL PIPE
DIA.	DIAMETER		
DWG.	DRAWING		
EL.	ELEVATION (METRES)	STA	STATION
EX.	EXISTING	TYP.	TYPICAL
MIN.	MINIMUM	T/P	TOP OF PAVEMENT
ROW	RIGHT OF WAY	W.L.	WATER LEVEL
SHLD	SHOULDER		

DRAWING NOT TO BE SCALED
50 mm ON ORIGINAL DRAWING

REVISIONS	DATE	BY	DESCRIPTION
DESIGN	J.P.	CHK G.M.	CODE CHBDC 2006 LOAD CL.-625-ONT
DRAWN	T.G.	CHK J.P.	SITE 46-006/C STRUCT SCHEME DWG P1

Part B – Preliminary Foundation Design Report

Five Mile Creek Culvert Replacement, Highway 129, Township of Reaney, Algoma District, Ontario

GWP 5222-05-00, WP 5227-05-01, Site No. 46-006/C, Index No.: 061FDR

PML Ref.: 14TF038, September 27, 2016



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 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.