FOUNDATION INVESTIGATION AND DESIGN REPORT
for
MEATBIRD CREEK CULVERT REPLACEMENT
HIGHWAY 17
1.2 KM WEST OF HIGHWAY 17/ REGIONAL ROAD 55 INTERCHANGE
AT SUDBURY WESTERLY
GREATER SUDBURY AREA, ONTARIO
SITE NO. 46-571
AGREEMENT NO. 5010-E-0027
G.W.P. 5146-09-00

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Index No.: 242FIR and 243FDR
Geocres No.: 41I-340
November 12, 2015

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for
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FOUNDATION INVESTIGATION REPORT

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION REPORT
for
Meatbird Creek Culvert Replacement
Highway 17
1.2 km West of Highway 17/Regional Road 55 Interchange
At Sudbury Westerly
Greater Sudbury Area, Ontario
GWP 5146-09-00

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1. INTRODUCTION

This report summarizes the results of the foundation investigations carried out for the proposed Meatbird Creek culvert replacement on Highway 17, located approximately 1.2 km west of Regional Road 55 in the Township of Waters, as part of the detail design for the Highway 17 resurfacing project, which extends from 0.8 km east of the Highway 17/Regional Road 55 interchange at Sudbury westerly 21.8 km. The investigation was carried out by Peto MacCallum Ltd. (PML) for AECOM Canada Ltd (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

The elevations in this report are expressed in meters, unless otherwise noted.

2. SOURCES OF INFORMATION

The following reports, including drawings, were available for the Meatbird Creek Culvert. Reference 1 is the original report for the site and Reference 2 is a subsequent foundation investigation report.

REFERENCE 1:

Soil Design Report, Highway 17, from 12.7 km east of Sec. Hwy 658 easterly (9.8 km), W.P. 62-74-01, District 17, Sudbury, by Material and Testing Office, Ministry of Transportation and Commutations, Northern Region – dated November,1975, GEOCRES 41I-104.

REFERENCE 2:

Foundation Investigation and Stability Analysis, Highway 17 Crossing of Meatbird Creek Valley, W.P. 62-74-01, District 17, Sudbury, by William Trow and Associates Ltd. – dated February 23, 1976, GEOCRES 41I-104.

Borehole Locations and Soil Strata, Drawing No. 1, Highway 17, Proposed Culvert Meatbird Creek, W.P. 62-74-01, District 17, Sudbury, by William Trow and Associates Ltd. – dated April, 1975, GEOCRES 41I-104.

Borehole Locations and Soil Strata, Drawing No. 1A, Highway 17, Proposed Culvert Meatbird Creek, W.P. 62-74-01, District 17, Sudbury, by William Trow and Associates Ltd. – dated October, 1975, GEOCRES 41I-104

In addition to the above GEOCRES reports, the following documents were also reviewed:

Ministry of Northern Development and Mines. 1991. Bedrock Geology of Ontario – Southern Sheet, Map 2544, Scale 1:1,000,000.

Chapman and Putnam. 1984. The Physiography of Southern Ontario, 3rd Edition.

Ontario Geological Survey. 1984. Physiography of Southern Ontario, Map 2715, Scale 1:600,000.

3. SITE DESCRIPTION AND GEOLOGY

The culvert is located within the Regional Municipality of Greater Sudbury within the Geographic Township of Waters. Photographs P1 and P2 (Appendix A) illustrate the site and surface conditions at the time of the investigation.

The Highway 17 corridor within the project limits is generally flanked by open water bodies, marsh areas and rock outcrops. The culvert is located south east of Lively, approximately 0.7 km east of Regional Road 24 and approximately 1.2 km west of Regional Road 55. There are no land use developments at the culvert location.

The project site is located within the Huronian Supergroup of the Canadian Shield. The typical rock types in the project area are argillite, siltstone and greywacke of the McKim Formation. The soil/bedrock interface is encountered at variable depths, but generally close to the surface.

The existing culvert was built within the rockfill embankment which carries the Highway 17 platform with side slopes in the order of 2.5H:1V. The rockfill embankment is up to 12 to 14 m high at the culvert location. At the east of the culvert, the east embankment was cut into a rock outcrop.

Meatbird Creek flows in the north to south direction and the creek water level was at a depth of 0.7 m (Elevation 244.0) at the time of the investigation.

4. INVESTIGATION PROCEDURES

Fourteen (14) boreholes were drilled at / near this site in 1975 and are presented in the Geocres report No.: 41I-104. Based on the previous Geocres report, old contract drawings and satellite photos of the area, this area has been modified since the previous boreholes were drilled with the construction of the existing 12 to 14 m high embankment.

The new fieldwork for the foundation investigation involved a total of 4 boreholes (numbered 101-1 to 101-4) that were carried out during the period from December 1 to 3, 2014. The boreholes were drilled to depths of 1.1 to 13.4 m at the approximate locations shown on Drawings MBC-1 and MBC-2. Boreholes were terminated by refusal on probable bedrock.

Two hand auger probes were also conducted at the north and south ends of the culvert near boreholes 101-1 and 101-4. The auger probes penetrated through silty/sandy soils and met refusal on probable boulder/bedrock at approximate depths of 0.6 to 1.8 m.

The boreholes were advanced using various methods including sonic drilling and manually operated continuous sampling equipment~~,~~ supplied and operated by specialist drilling contractors working under the full-time supervision of a PML field supervisor. Where site conditions dictated the use of a tripod system, a 70-pound hammer was used and a correction factor was applied to penetration test values obtained.

The following table, Table Section 4, summarizes the subsurface investigation program at the culvert location.

Table Section 4 - Details of Subsurface Investigation Program

Borehole No.	Location	Drilling Method	Depth (m)
101-1	Proposed Culvert Inlet	Tripod Continuous Sampling	4.1
AP-1		Hand Augering	1.8
101-2	Existing Highway 17 Embankment, WBL	Sonic and Casing	13.4
101-3	Existing Highway 17 Embankment, EBL	Sonic and Casing	11.7
101-4	Proposed Culvert Outlet	Tripod Continuous Sampling	1.1
AP-2		Hand Augering	0.6 (Boulder)

Representative samples of the soils were recovered at frequent depth intervals using a conventional split spoon sampler. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. The results of the field tests and observations are reported on the Record of Borehole sheets.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil and the sampler as the samples were retrieved. No groundwater observations could be made in the boreholes completed using the Sonic drilling, where casing was advanced by washboring techniques, as the drilling method continuously introduced outside water into the boreholes.

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

The coordinates including ground surface elevations at the borehole locations were established by exp Geomatics.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. The soil samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determination. The minimum soil recovery required to carry out laboratory tests including moisture content determinations was not obtainable from the SPT tests in the rockfill using tripod drilling. As a result, grain size distribution analyses (2) and moisture content determinations (3) were performed only on selected soil samples with sufficient soil available.

5. SUMMARIZED SUBSURFACE CONDITIONS

5.1 General

Refer to the attached Appendix A, Image 1 and 2 for general and detailed aerial view of the site and two photographs of the site. Refer to the attached Record of Borehole sheets for the details of the subsurface conditions including soil classifications, groundwater observations and inferred stratigraphy. The laboratory grain size distribution charts are presented in Figures GS-MB-1 and GS-MB-2. The test results are summarized on the attached Record of Borehole sheets.

Borehole locations and the stratigraphic profile and cross-sections prepared from the current borehole data are shown on Drawings MBC-1 and MBC-2. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The culvert is located under an approximately 12 to 14 m high rockfill embankment overlying natural ground. In summary, the subsurface stratigraphy revealed in boreholes located at the inlet and outlet of the culvert (boreholes 101-1 and 101-4 respectively) comprised a 100 mm thick topsoil layer underlain by an approximately 4.0 m thick non-cohesive deposit of silty/sandy soils at the inlet and an approximately 1.0 m thick cohesive clayey silt layer at the outlet. Probable bedrock was inferred at a depth of 1.1 m (Elevation 244.3) at the culvert outlet and at a depth of 4.1 m (Elevation 240.6) at the culvert inlet.

The subsurface stratigraphy revealed in the median boreholes 101-2 (Highway 17, westbound lane shoulder) and 101-3 (Highway 17, eastbound lane shoulder) generally consisted of pavement fill underlain by 9.5 and 11.0 m thick rockfill and 2.0 m thick non-cohesive sand layer. Boulders or rock fill up to 2 m in diameter were contacted at depths of 1.5 m and 3.9 m at approximately 1.5 to 2.0 m west of Borehole 101-3 and may exist throughout the rock fill. Probable bedrock was inferred by refusal at depths of 11.7 m (Elevation 241.9) in borehole 101-2 and 13.4 m (Elevation 241.0) in borehole 101-3.

The strata encountered are summarised below:

5.2 Topsoil

A 100 mm thick topsoil layer was encountered surficially in boreholes 101-1 and 101-4 (inlet and outlet of the culvert) and extended to Elevation 244.6 and 245.3, respectively.

5.3 Pavement Structure

Shoulder pavements of 100 mm asphaltic concrete with approximately 500 to 600 mm of sand and gravel courses were encountered surficially in boreholes 101-2 and 101-3 (WBL and EBL shoulders) that extended to depths of 0.7 and 0.6 m (Elevation 253.7 and 253.0), respectively.

5.4 Fill Material

A 1.4 m thick fill unit was encountered below the topsoil at a depth of 0.1 m (Elevation 244.6) in borehole 101-1 (culvert inlet) and extended to a depth of 1.5 m (Elevation 243.2). The fill was composed of silty sand mixed with organic materials. This unit was compact in relative density (SPT-'N' values of 14 and 16).

5.5 Rockfill

A 9.5 and 11.0 m thick rockfill unit was encountered below the pavement structure at depths of 0.7 and 0.6 m (Elevation 253.7 and 253.0) in boreholes 101-2 and 101-3 (WBL and EBL shoulders), respectively and was penetrated at depths of 10.2 and 11.6 m (Elevation 244.2 and 242.0).

Borehole 101-3 had to be relocated due to presence of a 1.0 to 2.0 m thick boulder that was contacted at depths of 1.5 m and 3.9 m at approximately 1.5 to 2.0 m west of borehole 101-3 (drilled on Highway 17, eastbound lane).

5.6 Sand

A 2.0 and 2.6 m thick deposit of non-cohesive sand was contacted below the fill units at depths of 1.5 and 10.2 m (Elevation 243.2 and 244.2) in boreholes 101-1 (culvert inlet) and 101-2 (WBL shoulder) and extended to the probable bedrock/boulder at depths of 4.1 to 12.2 m (Elevation 240.6 and 242.2). SPT-'N' values ranged from 9 to 41 within the sand deposit indicating a variable loose to dense relative density.

The results of grain size distribution analyses of a sand sample is included in Figure GS-MB-1. This deposit was moist to wet.

5.7 Clayey Silt

A 1.0 m thick cohesive clayey silt was encountered below the topsoil at a depth of 0.1 m (Elevation 245.3) in borehole 101-4 (culvert outlet) and extended to probable bedrock at a depth of 1.1 m (Elevation 244.3). The clayey silt was very soft to soft in consistency (SPT-'N' values of 1 and 5).

The results of grain size distribution analyses conducted on a clayey silt sample are included in Figure GS-MB-2. The minimum soil recovery required to carry out the Atterberg Limits test for this sample was not obtained. This moisture content of the clayey silt was about 27%.

5.8 Bedrock

Based on the previous GEOCRES reports (identified in Section 2 under References), satellite photos of the area, geological maps, visual inspections and previous rock samples, the typical rock types in the project area are argillite, siltstone and greywacke of the McKim Formation.

Although the bedrock interface was not verified by coring and the borehole refusal could be on bedrock or boulders, the inferred bedrock interface was encountered at variable depths along the culvert alignment. The probable bedrock/boulder surfaces were inferred by refusal at depths 4.1 and 1.1 (Elevation 240.6 and 244.3) in boreholes 101-1 and 101-4, near the inlet and outlet of the culvert, respectively. Below the highway rockfill, the probable bedrock was inferred by refusal at depths of 13.4 and 11.7 m (Elevation 241.0 and 241.9) in boreholes 101-2 and 101-3, respectively. A 1.2 m thick layer of cobbles and boulders was contacted in borehole 101-2.

5.9 Groundwater

The Meatbird Creek is about 4.5 to 5.0 m wide at the culvert location. The water level in the creek flows from north to south and was at Elevation 244.0 at the time of the current investigation. The water level in the creek governs the water level at the site.

In the process of augering and upon completion of drilling, water was at depths of 0.7 and 0.6 m (Elevation 244.0 and 244.8) in boreholes 101-1 and 101-4, respectively. The groundwater in borehole 101-4 was likely perched water above the local bedrock. No groundwater observations could be made in boreholes 101-2 and 101-3 as drilling water was continuously introduced into the boreholes as a result of the Rotosonic drilling.

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

6. CLOSURE

Mr. A. Lo and Mr. S. Aziz carried out the field investigations under the supervision of Mr. K. Daly, BEng, Project Supervisor, EIT and Mr. C. M. P. Nascimento, P. Eng., Project Manager. LandCore Drilling Ltd. and Underground Sonic Drilling Services Inc. supplied the drill equipment for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.

This report was prepared by Ms. Marzieh Kamranzadeh, MSc, Project Supervisor, EIT and reviewed by Mr. David Dundas, 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.

LICENSED PROFESSIONAL ENGINEER
D. H. DUNDAS
Nov. 12, 2015
PROVINCE OF ONTARIO

Marzieh Kamranzadeh, MSc, EIT
Project Supervisor, Geotechnical Services

David Dundas, P.Eng.
Senior Engineer, Geotechnical Services

REGISTERED PROFESSIONAL ENGINEER
C M P Nascimento
Nov. 12, 2015
PROVINCE OF ONTARIO

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

MK/DD/CN:mk-mi-jk

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/0.3 m)	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
r_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_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO

n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No 101-1

METRIC

G.W.P. 5146-09-00 LOCATION Coords: 5 143 321.2 N; 294 370.9 E ORIGINATED BY S.A.

DIST Sudbury **HWY** 17 BOREHOLE TYPE Tripod and Continuous Sampling COMPILED BY M.K.

DATUM Geodetic DATE December 02, 2014 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	NATURAL MOISTURE CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
244.7	Ground Surface							
244.6 / 0.1	Topsoil							
	Silty sand, organics, Compact, Grey/dark brown, Wet (FILL)	1	SS	14	▽* ▼*	244		
		2	SS	16			~20	
243.2 / 1.5	Sand, some silt, some gravel, trace clay, Compact to dense, Grey, Wet	3	SS	21		243		16 66 16 2
		4	SS	22				
		5	SS	32		242		
		6	SS	36				
		7	SS	41		241		
240.6 / 4.1	End of borehole. Refusal on probable boulder							

* 2014 12 02

▽ Water level observed during drilling

▼ Water level measured after drilling

Auger probe was carried out at the culvert end and 1.2 to 1.8m thick silty/sandy soils were encountered and auger probe met refusal on probable boulder at 1.8m depth.

Borehole was drilled using 70lb hammer and 'N' values were adjusted accordingly.

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+7, ×5: Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No 101-2

METRIC

G.W.P. 5146-09-00 LOCATION Coords: 5 143 290.1 N; 294 360.9 E ORIGINATED BY A.L.

DIST Sudbury **HWY** 17 BOREHOLE TYPE Sonic Drilling and Casings COMPILED BY M.K.

DATUM Geodetic DATE December 02, 2014 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%)
254.4	Ground Surface						
0.0	100mm asphalt over 60mm base-course over sand and gravel (PAVEMENT FILL)	1	GS				
253.7 / 0.7	Rockfill						
244.2 / 10.2	Sand trace to some gravel trace silt. Loose to compact, Brown, Wet	2	SS	9		o (approx. 2-3%)	
		3	SS	11			
242.2 / 12.2	cobbles and boulders	4	SS	17			
241.0 / 13.4	End of borehole. Refusal on probable bedrock						Borehole was drilled using 70lb hammer and 'N' values were adjusted accordingly.

* Borehole charged with drilling water

Elevation scale: 254, 253, 252, 251, 250, 249, 248, 247, 246, 245, 244, 243, 242, 241

DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100

PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

UNIT WEIGHT γ kN/m³; GR SA SI CL

+7 ,×5 : Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No 101-3

METRIC

G.W.P. 5146-09-00 LOCATION Coords: 5 143 232.1 N; 294 356.9 E ORIGINATED BY A.L.

DIST Sudbury **HWY** 17 BOREHOLE TYPE Sonic Drilling and Casings COMPILED BY M.K.

DATUM Geodetic DATE December 1 and 3, 2014 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE) 20 40 60 80 100	PLASTIC LIMIT W_P / NATURAL MOISTURE CONTENT W / LIQUID LIMIT W_L — WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.6	Ground Surface										
0.0	100mm asphalt over 50mm base-course over sand and gravel (PAVEMENT FILL)										
253.0 / 0.6	Rockfill						253				
							252				
251.2 / 2.4	probable boulder						251				
							250				
							249				
248.8 / 4.8	layers of silty/sand soils						248				
							247				
246.3 / 7.3	layers of dense gravelly soils						246				
							245				
							244				
							243				
							242				
241.9 / 11.7	End of borehole Refusal on probable bedrock Note: 1.0m to 2.0m thick boulder contacted at 1.5m and 3.9m depth in nearby boreholes, first and second trial)(at approximately 1.5 to 2.0m west of BH 101-3. * Borehole charged with drilling water										Borehole was drilled using 70lb hammer and 'N' values were adjusted accordingly.

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

20 / 15 —○— 5 / 10 (%) STRAIN AT FAILURE

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No 101-4

METRIC

G.W.P. 5146-09-00 LOCATION Coords: 5 143 214.1 N; 294 344.9 E ORIGINATED BY A.L.

DIST Sudbury **HWY** 17 BOREHOLE TYPE Tripod and Continuous Sampling COMPILED BY M.K.

DATUM Geodetic DATE December 01, 2014 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
245.4	Ground Surface													
245.3 / 0.1	Topsoil													
	Clayey silt some to with sand, trace gravel		1	SS	1	▽* ▼*	245							
244.3 / 1.1	Very soft Brown Moist to soft		2	SS	5				○		9	20	55	16
	End of borehole													
	Refusal on probable bedrock													

* 2014 12 010

▽ Water level observed during drilling

▼ Water level measured after drilling

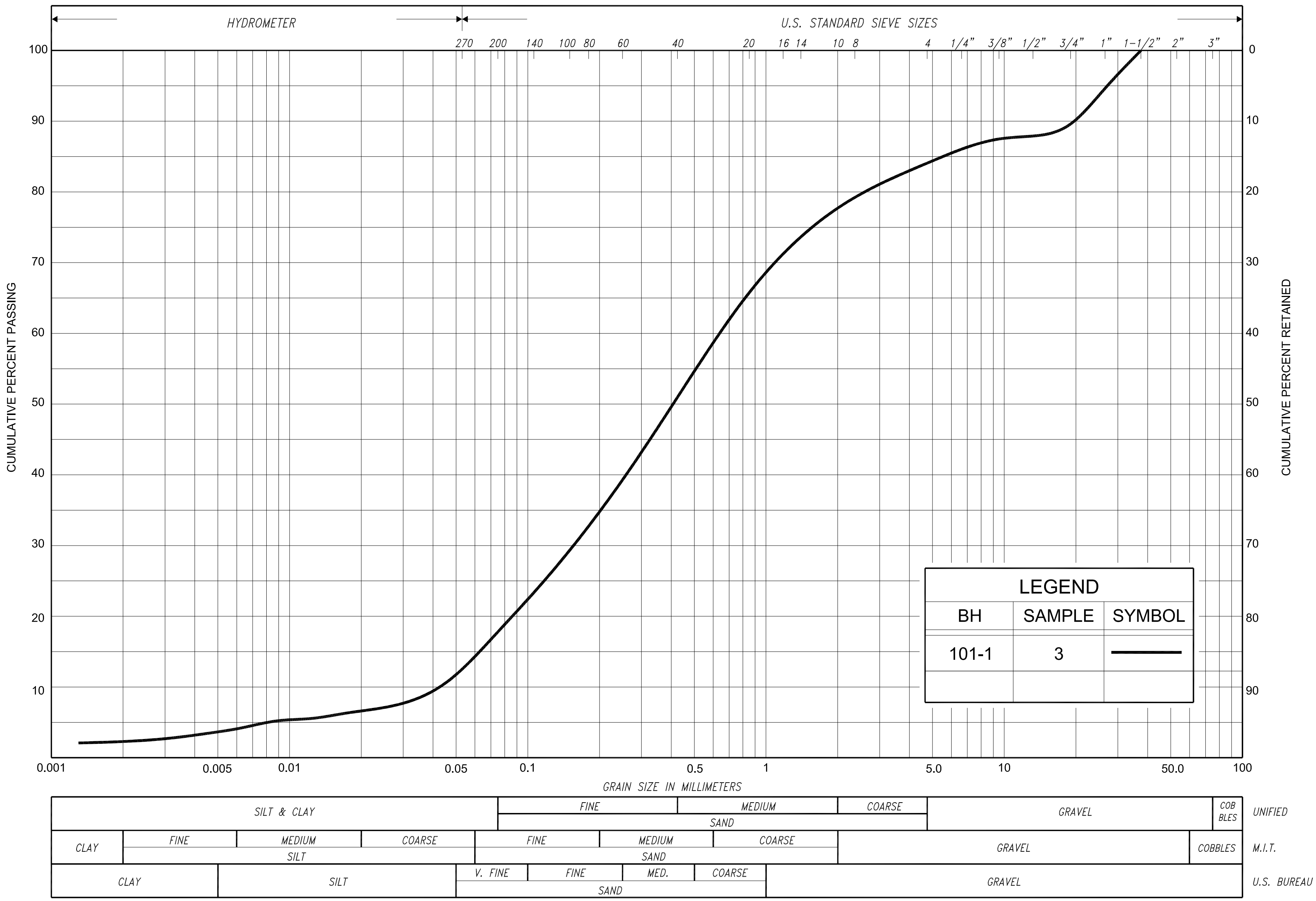
Auger probe was carried out at the culvert end and 0.6m thick silty/sand soild were encountered auger probe met refusal on probable bedrock at 0.6m depth.

Borehole was drilled using 70lb hammer and 'N' values were adjusted accordingly.

Legend: SHEAR STRENGTH kPa — ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE. PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L. REMARKS & GRAIN SIZE DISTRIBUTION (%).

+7, ×5 : Numbers refer to Sensitivity

20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

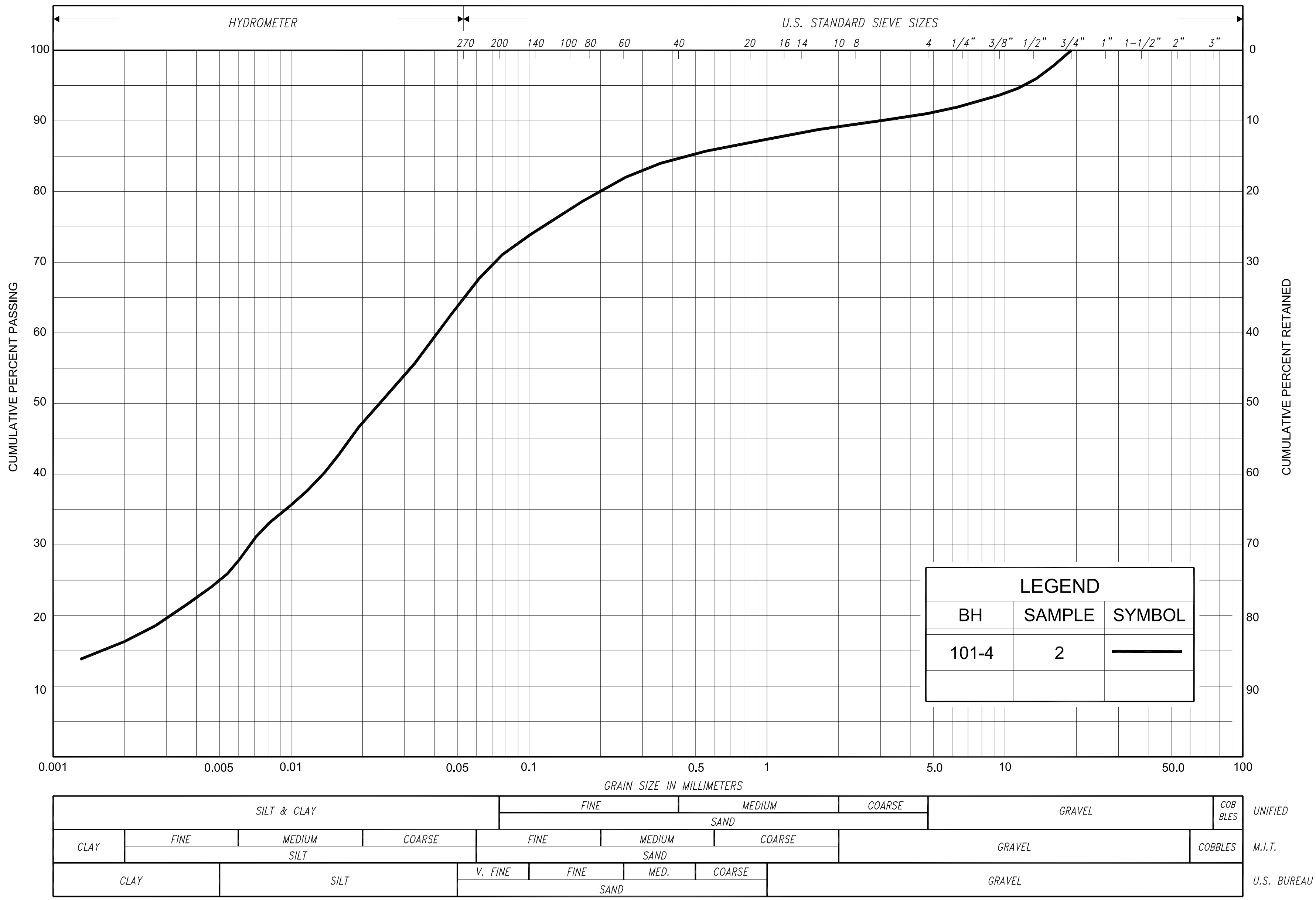


Ontario

GRAIN SIZE DISTRIBUTION

SAND, some silt, some gravel, trace clay

FIG No.	GS-MB-1
HWY:	17
G.W.P. No.	5146-09-00



Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some to with sand, trace gravel

FIG No.	GS-MB-2
HWY:	17
G.W.P. No.	5146-09-00

CONT No GWP No 5146–09–00 WP No	
MEATBIRD CREEK CULVERT HIGHWAY 17 BOREHOLE LOCATIONS & SOIL STRATA	SHEET

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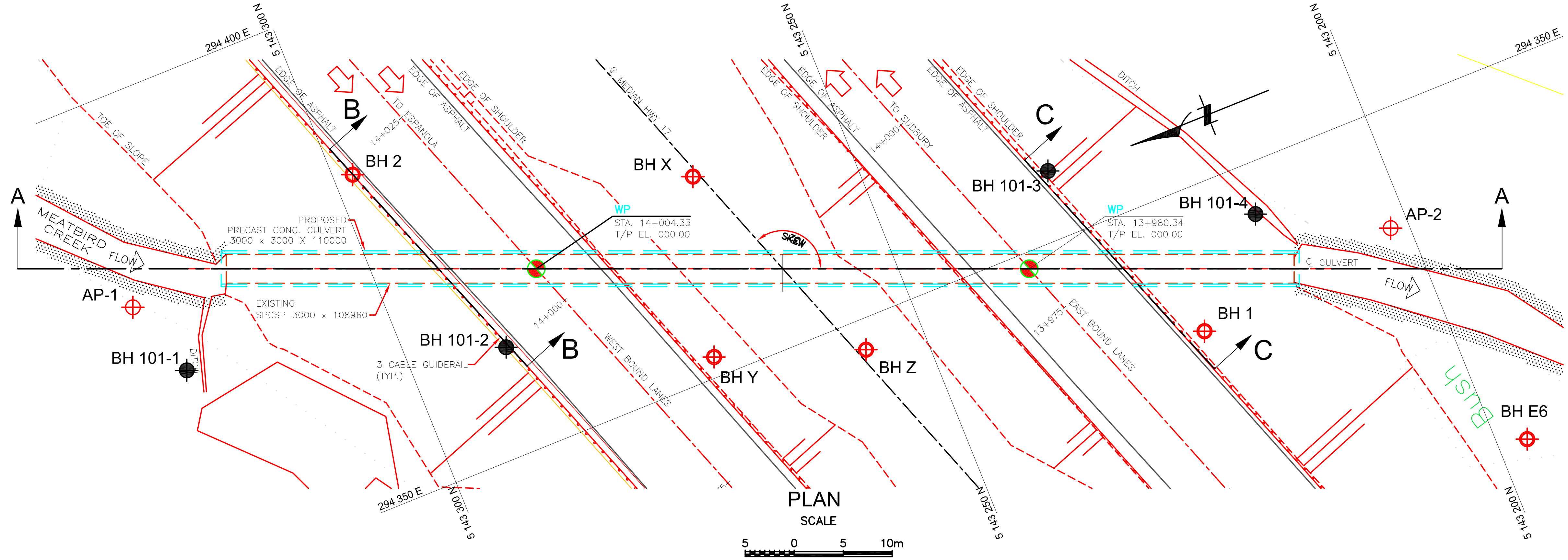
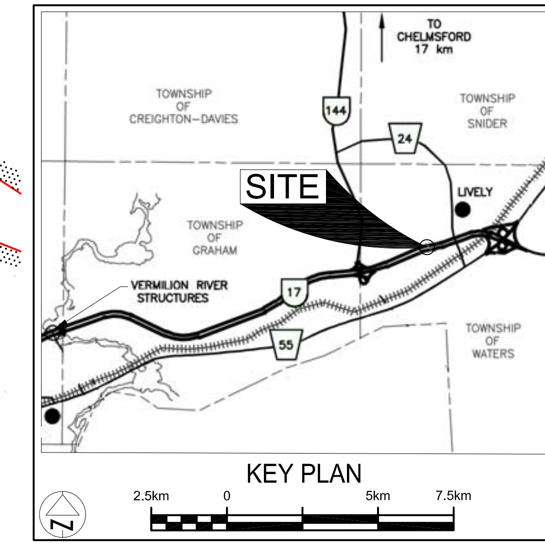
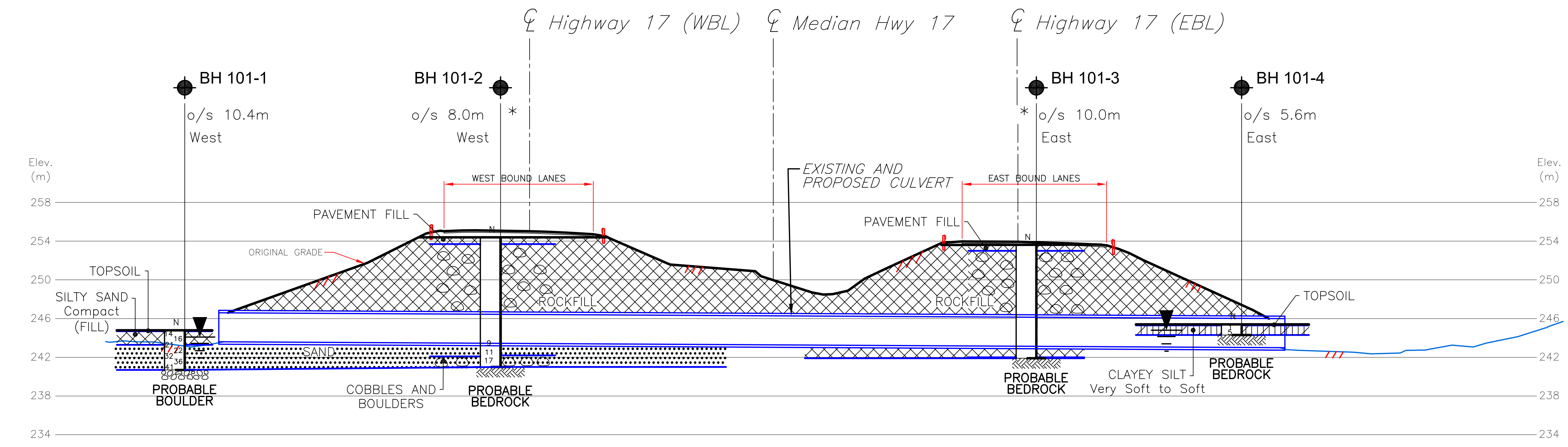




LEGEND

- Borehole
- Borehole and Cone
- Geocres Borehole (41I–104)
- Auger Probe (AP)
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation (Dec. 2014)
- * Borehole Charged With Drilling Water

BH No	ELEVATION	NORTHINGS	EASTINGS
101–1	244.7	5 143 321.2	294 370.9
101–2	254.4	5 143 290.1	294 360.9
101–3	253.6	5 143 232.1	294 356.9
101–4	245.4	5 143 214.1	294 344.9



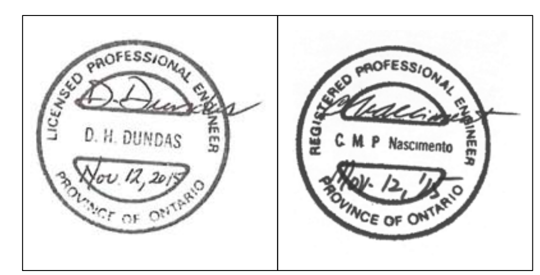
PROFILE A - A ALONG MEATBIRD CREEK CULVERT

SCALE

5 0 5 10m

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
3. REFER TO DRAWING MBC–2 FOR SECTIONS B–B AND C–C.
4. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



REF AECOM Drawing: 6027641–P1 Geotech.dwg, undated

– NOTE –

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

REVISIONS DATE	BY	DESCRIPTION

Geocres No. 41I-340

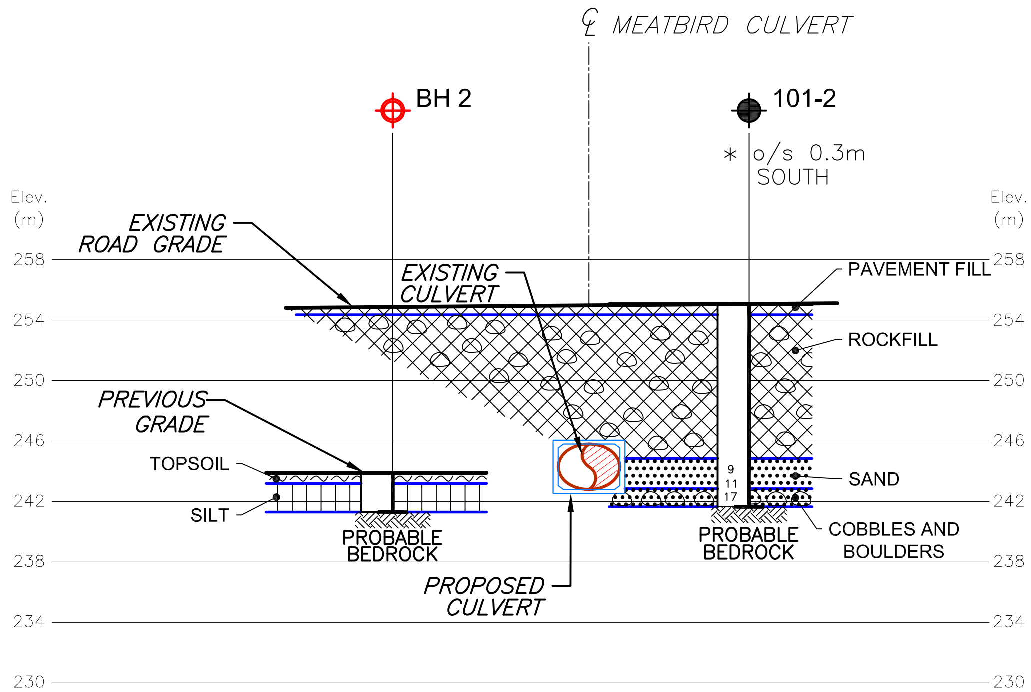
HWY No 17			DIST SUDBURY
SUBM'D NA	CHECKED MK	DATE NOV. 12, 2015	SITE 30–334/1
DRAWN NL	CHECKED DD	APPROVED CN	DWG MBC–1

CONT No
GWP No 5146-09-00
WP No

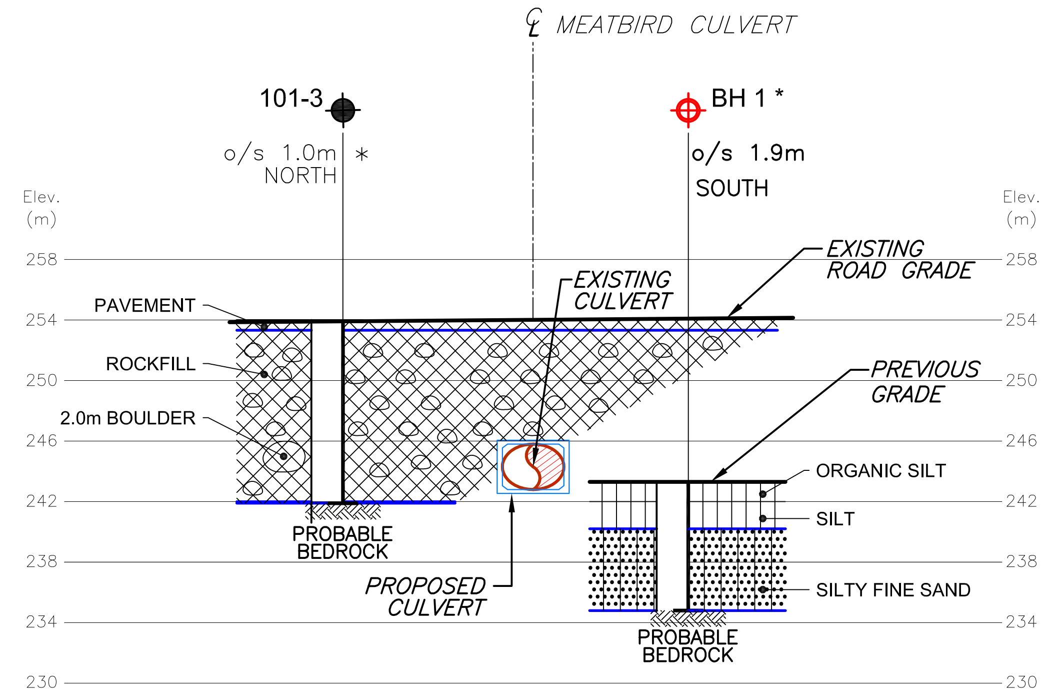
MEATBIRD CREEK CULVERT
HIGHWAY 17
SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



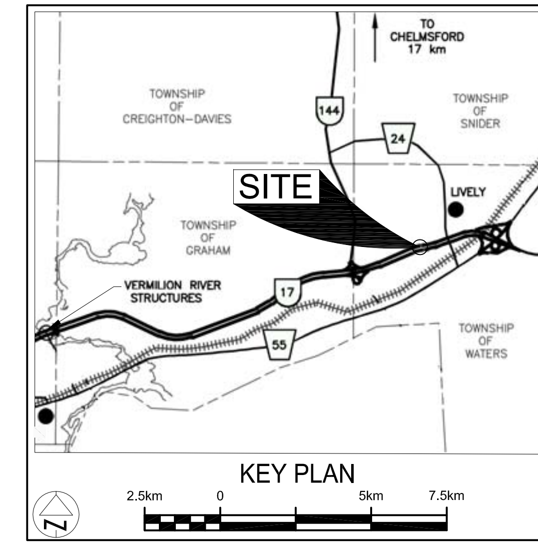
SECTION B - B



SECTION C - C

SCALE

5 0 5 10m



KEY PLAN

LEGEND

- Borehole
- Borehole and Cone
- Geocres Borehole (41I-104)
- Auger Probe (AP)
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation (Dec. 2014)
- ∗ Borehole Charged With Drilling Water

BH No	ELEVATION	NORTHINGS	EASTINGS
REFER TO DWGS MBC-1, FOR DETAILS			

NOTE: TYPE OF CULVERT SHOWN FOR ILLUSTRATION ONLY

NOTES:

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

REF AECOM Drawing: 6027641-P1 Geotech.dwg, undated

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

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No. 41I-340

HWY No 17			DIST SUDBURY
SUBM'D NA	CHECKED MK	DATE NOV. 12, 2015	SITE 30-334/1
DRAWN NL	CHECKED DD	APPROVED CN	DWG MBC-2

APPENDIX A

Image 1– General Aerial View of the Site

Image 2 – Detailed Aerial View of the Site

Site Photographs

IMAGE 1 – GENERAL AERIAL VIEW OF THE SITE

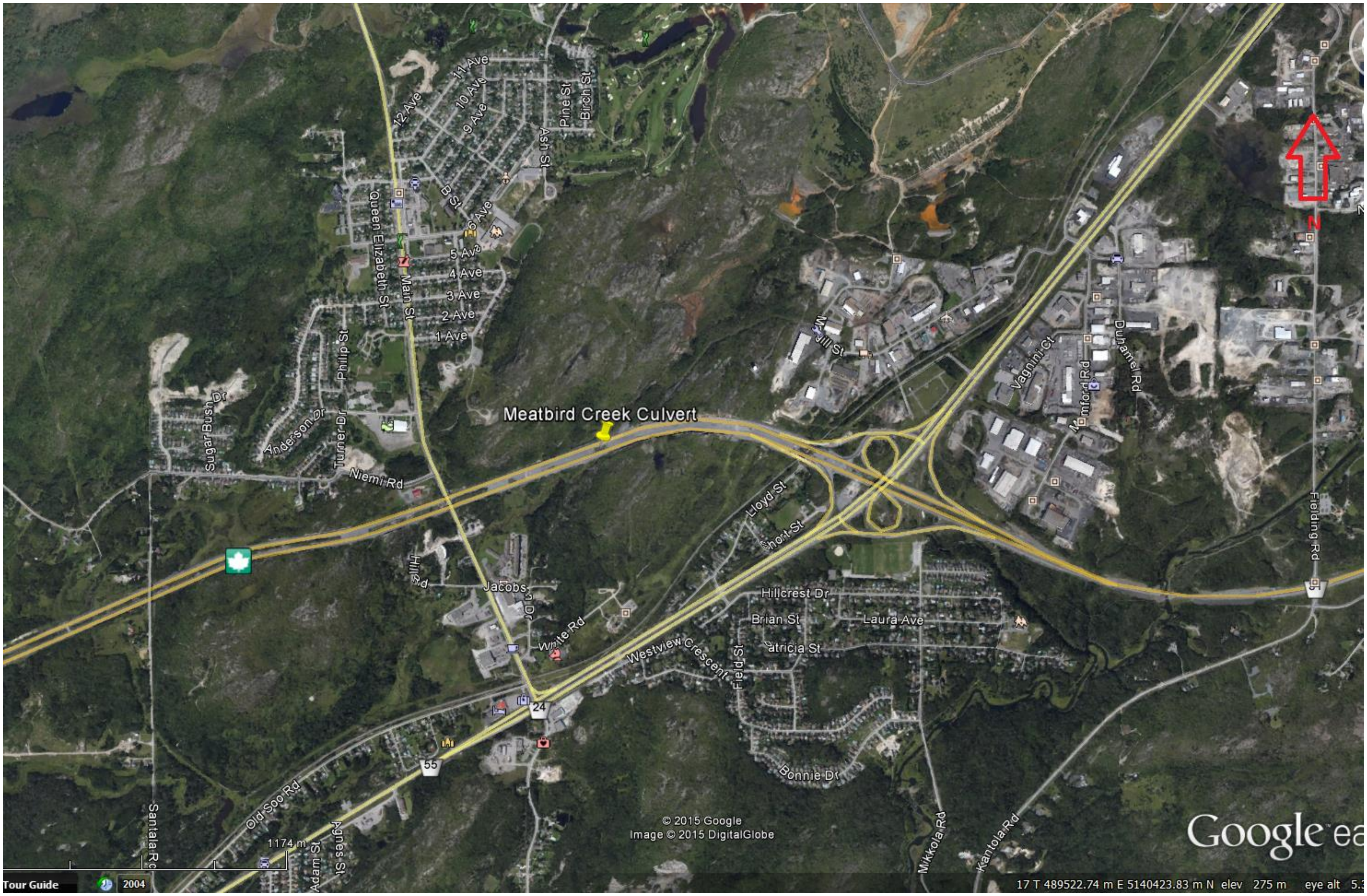


IMAGE 2 – DETAILED AERIAL VIEW OF THE SITE

Photograph P1: Looking north from the existing Highway 17 WBL at the location of the Borehole 101-1 (culvert inlet). Surficial boulders are visible. (December 2, 2014)

Photograph P2: Looking west from the existing Highway 17 median. Borehole 101-2 advanced using the Sonic drilling techniques at this location. (December 1, 2014)

FOUNDATION DESIGN REPORT
for
MEATBIRD CREEK CULVERT REPLACEMENT
HIGHWAY 17
1.2 KM WEST OF HIGHWAY 17/ REGIONAL ROAD 55 INTERCHANGE
AT SUDBURY WESTERLY
GREATER SUDBURY AREA, ONTARIO
SITE NO. 46-571
AGREEMENT NO. 5010-E-0027
G.W.P. 5146-09-00

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PML Ref.: 12KF053
Index No.: 243FDR
Geocres No.: 41I-340
November 12, 2015

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FOUNDATION DESIGN REPORT

FOUNDATION DESIGN REPORT
for
Meatbird Creek Culvert Replacement
Highway 17
1.2 KM West of Highway 17/Regional Road 55 Interchange
At Sudbury Westerly,
Greater Sudbury Area, Ontario
GWP 5146-09-00

1. PROJECT DESCRIPTION

This report pertains to design and construction of the proposed culvert replacement described below.

A new 110.0 m long concrete box culvert is proposed to replace the existing CSP culvert under the westbound and eastbound lanes of Highway 17. The invert levels of the proposed 3.0 m high and 3.0 m wide culvert are specified to be at approximate Elevation 243.3 at the inlet and 242.7 at the outlet of the culvert.

The following table, Table Section 1, indicates the approximate location of the proposed culvert, type and size of the existing and proposed culvert on Highway 17:

Table Section 1 - Details of Existing and Proposed Culverts

Approximate Location of Culvert	Type and Size of the Existing Culvert	Type and Size of the Replacement Culvert
Highway 17, Sta. 14+000, Township of Waters, Meatbird Creek	CSP, 3050 mm Diameter, 110 m long	Precast Concrete Box 3.0m×3.0m×110.0m

2. DISCUSSION OF FOUNDATION ALTERNATIVES

The existing culvert was built within a rockfill embankment that carries the Highway 17 platform. Based on the previous GEOCRES reports (identified in Section 2 of the Foundation Investigation Report under References), old contract drawings and satellite photos of the area, this area has been modified since the previous boreholes were drilled for construction of Highway 17 and Meatbird Creek culvert in 1975, with the construction of the 12 to 14 m high rockfill embankment.

The construction operations such as excavation, shoring, groundwater control, backfill and bedding should be considered in the selection of the replacement culvert type and the replacement culvert installation technique.

2.1 Culvert Type

Based on discussions with AECOM, the following culvert type alternatives are proposed for consideration for the culvert replacement:

1. Precast Concrete Box Culvert
2. Cast-In-Place Concrete Box Culvert
3. Cast-In-Place Concrete Open-Footing Culvert
4. Propriety Steel Arch Culvert (CIP Concrete Footings on H-Piles or Micro-Piles)

The following table, Table Section 2.1, compares the advantages, disadvantages, risks / consequences and relative costs of each alternative from the foundation perspective:

Table Section 2.1 - Comparison of Alternative Options

Culvert Type (Alternatives)	Advantages	Disadvantages	Risks/Consequences	Relative Costs
1. Precast Concrete Box Culvert	- less time required for construction - less complex dewatering or potentially construction in the wet - more tolerant to settlement than CIP options	- temporary drainage is required while the new culvert is installed along the existing alignment but partial dewatering with installation in the wet is possible - transportation of culvert segments - limited size of off-the-shelf culvert segments - precast concrete provides lower sliding resistance than CIP concrete	- differential settlement requiring gaskets between box segments needs to be considered	- less cost due to shorter construction time, but cost of transportation of materials has to be considered - cost of temporary drainage installation, if needed, has to be considered
2. CIP Concrete Box Culvert	- less transportation for materials than precast option - more flexibility in sizing - CIP concrete provides higher sliding resistance than precast concrete	- longer culvert construction schedule than precast concrete box culvert construction - more rigorous dewatering required than precast concrete box culvert - less tolerant to settlement	- differential settlement could cause cracking of concrete in the culvert	- more costly construction than precast concrete box culvert due to longer construction time - higher cost for dewatering than for concrete precast box culverts due to requirements for construction in the dry

Table Section 2.1 - Comparison of Alternative Options

Culvert Type (Alternatives)	Advantages	Disadvantages	Risks/Consequences	Relative Costs
3. CIP Concrete Open-Footing Culvert	- CIP concrete provides higher sliding resistance than precast concrete	- longer culvert construction schedule than precast concrete box culvert construction - more complex dewatering required than precast concrete box culvert complicated by the uneven bedrock and sandy overburden - less tolerant to settlement than CIP concrete box culvert	- increased importance of positive dewatering at strip footings increases risk of dewatering claims	- higher cost for dewatering than for concrete precast box culverts due to requirements for construction in the dry - cost of temporary drainage installation, if needed, has to be considered
4. Propriety Steel Arch Culvert (CIP Concrete Footings on H-Piles or mirco-Piles)	- temporary drainage not a concern if culvert overarches existing culvert left in place during construction	- more time required for construction of foundations (pile driving perhaps keyed into bedrock or spread footings anchored into bedrock) - complex design to determine optimum lateral resistance for 3-sided structure type - more complex dewatering required than precast concrete box culvert complicated by the uneven bedrock and sandy overburden - less tolerant to settlement than box culverts	- increased importance of positive dewatering at strip footings increases risk of dewatering claims - difficult conditions for developing resistance to horizontal loads required at base of proprietary arch culverts could require socketing of piles or anchoring of footings and could lead to construction claims	- high costs for foundations due to longer construction time and more complex design, dewatering and footing requirements with potential very high costs if piling or anchor installation equipment required

In general, the critical foundations engineering issues for this project are the existence of rockfill embankment at the culvert location, potential difficulties in dewatering (fissures in rock and sandy zones in overburden) and a potentially uneven bedrock surface.

Based on the foundation investigations at the existing culvert location, replacement of the culverts by the open excavation method is considered feasible from a geotechnical viewpoint. Trenchless methods such as the jack and bore or pipe ramming are considered to be technically challenging and may not be feasible.

Depending on the construction staging and traffic interruption constraints, the hydraulic capacity and size of the existing and proposed culvert, the following alternatives for replacing the culvert are prioritized from a foundations engineering viewpoint:

Option 1 (Precast Concrete Box Culvert)
Option 2 (CIP Concrete Box Culvert)
Option 3 (CIP Concrete Open-Footing Culvert)
Option 4 (Propriety Steel Arch Culvert – CIP Concrete Footings on H-Piles or Micro-Piles)

From a foundations engineering perspective, alternatives with less onerous dewatering requirements and less requirements for anchoring footings would be preferable.

2.2 Culvert Installation Technique

One option of construction staging for the culvert replacement could consist of temporary closing of Highway 17 in one direction and detouring traffic and hence, minimizing requirements for shoring to maintain traffic.

Due to presence of large boulders (from approximately 0.6 m to over 1 m and in some cases up to 2 m in diameter) within the rockfill, it is considered that the trenchless technologies would be technically challenging and may not be feasible at this site. However, MTO may be considering this site as a trial project for tunneling technologies through rock fill.

The following table (Table Section 2.2) summarizes the advantages and disadvantages of a suite of tunnelling methods. For the selection of the preferred alternative, consideration should be given to difficulties inherent in driving liners through rock fill and coring through discontinuous materials that may require prior grouting for success. If a tunnelling method is considered, it is recommended that the design should be coordinated with an expert tunnelling contractor.

Table Section 2.2 – Advantages and Disadvantages of Tunneling Methods

Tunnelling Method	Advantages	Disadvantages
Steerable Jack and Bore	▪ Contractor availability ▪ Grade control is available for alignment corrections ▪ Typically least costly method ▪ Can accommodate variable soils (sand layers in the onsite till) without major tooling adjustments ▪ Small staging areas compared to HDD ▪ Minimal tunnel diameter of 610 mm	▪ Ground water control may be required for the bore and staging pits ▪ Elevated potential for ground subsidence if adequate ground water control is not achieved ▪ Recommended for drier seasons ▪ Minor residual space may be present surrounding liner exterior, which could require grouting ▪ Once operation is started it should continue without major stoppage until completion to mitigate potential for sloughing of face and void formation ▪ Typically not suitable for rock fill
Pipe Ramming	▪ Low risk for loss of ground from over augering / collapses at the bore ▪ Low sensitivity to ground water seepage compared to jack and bore ▪ Can accommodate variable soils (sand layers, cobbles in the onsite till) without major tooling adjustments ▪ Small staging areas compared to HDD ▪ Has been reported to be effective through rock fill with some reservations	▪ High ramming resistance required for liner penetration in stiff clayey silt till overburden ▪ Thicker steel needed to sustain ramming stresses ▪ Poor grade control compared to jack-and-bore, micro-tunneling and HDD ▪ Ground water control may be required for staging works ▪ More costly than jack and bore ▪ May require encroachment into right of way to maintain grade control ▪ Grades cannot be corrected once installation has started

Tunnelling Method	Advantages	Disadvantages
Horizontal Directional Drilling	▪ Does not require deep staging pits ▪ Minimal ground water control required during drilling ▪ No wet season restrictions	▪ Largest tunnel diameter, 900 mm envisaged ▪ Requires long slot trench and layout area, likely extending beyond the right of way ▪ Potential for inadvertent drilling fluid returns ▪ Larger HDD equipment may be required given 900 mm tunnel diameter and the nature of the till deposits ▪ Requires drilling fluid to maintain the bore which could allow subsidence ▪ May have poor grade control in gravelly soils and very loose or soft material. ▪ Potential for oval tunnel cross section ▪ Typically not suitable for rock fill due to high potential to lose drilling fluid
Micro-tunnelling	▪ May not require ground water lowering for the tunnel ▪ Machine can be designed to be able to counter- balance earth and water pressures in a controlled manner, thereby reducing the risk of ground losses during tunnelling ▪ Good grade control ▪ Can be effective in rock fill if pregrouting binds the rock fill	▪ Limited contractor availability ▪ Cost effectiveness depends on availability of existing adequate tunnel boring machines ▪ Ground water control is required for staging pits ▪ Typically more costly than jack-and-bore, or pipe-ramming
Conventional Mining	▪ Capable of advancing tunnels in through soil embankments, mixed embankments and rock fill embankments	▪ High cost ▪ Larger tunnel diameter required

3. FOUNDATION RECOMMENDATIONS

3.1 Site and Project Summary

At the Meatbird Creek site, Highway 17 is a 4 lane highway with centre median with a road level width of 12 m. The highway crosses the valley of Meatbird Creek in a rockfill embankment zone that is about 20 m long and with a variable height that ranges up to12 to 14 m.

Meatbird Creek is about 4.5 to 5.0 m wide at the culvert location. The water level in the creek flows from north to south and was relatively shallow at the time of the investigation (Elevation 224.0).

The existing 110 m long and 3050 mm diameter corrugated steel pipe (CSP) culvert was built within the local rockfill embankment, which carries the Highway 17 platform.

In general, the subsurface conditions at the site consist of 12 to 14 m of rock fill that is underlain, depending on the depth to bedrock, by relatively thin deposits of various cohesive and non-cohesive soils or on the bedrock.

In general within the specific plan limits of the proposed culvert and its backfill and bedding, the culvert bedding is inferred to be underlain by relatively thin deposits of various cohesive and non-cohesive soils or directly by bedrock.

It is understood that based on the evaluation of alternatives, the existing Meatbird Creek Culvert will be replaced by a precast segmental box culvert founded as spread footings.

The following table, Table Section 3.1, summarizes the approximate invert elevations (at the inlet and outlet of the culvert) of the proposed culvert that were obtained from the profile drawings provided by AECOM dated June 8, 2015.

Table Section 3.1 - Proposed Culvert Invert Elevations

Location	Proposed Inlet Invert Elevation	Proposed Outlet Invert Elevation
Meatbird Creek Culvert	243.3	242.7

The new precast concrete box culvert is proposed to be installed at the existing culvert location and a temporary diversion culvert will be installed within the general excavation to the west of the new permanent culvert alignment. Excavation to the anticipated founding levels of the permanent culvert is expected to extend through the rockfill and native cohesive and non-cohesive deposits. Depending on the depth to bedrock along the temporary culvert diversion, bedrock excavation may be required where water flow relies on gravity drainage and not pumping.

The foundation design recommendations and construction considerations for the new permanent culvert are detailed in the following sections.

3.2 Excavation

The soil and rock fill under the plan limits of the culvert should be sub-excavated to bedrock or to non-cohesive soil. The minimum depth of excavation should allow for the levelling and base course requirements. Where non-cohesive soil is encountered at the founding level, the depth of subexcavation should extend a minimum of 1 m below the culvert base level. Where bedrock is at the founding level, the depth of subexcavation should extend to a minimum depth of 0.5 m to permit installation of the levelling and bedding layers for the culvert.

For design and construction documentation purposes, the depth to bedrock is illustrated in the relevant boreholes and stratigraphical sections presented in the Foundation Investigation part of this report. The depth to bedrock was determined only at borehole locations and will be variable along and across culvert alignment. Although the bedrock surface is expected to be variable along and across culvert alignment, the bedrock surface has been interpolated between the top of bedrock levels at the borehole locations.

Excavation can be carried out in-the-wet or in-the-dry.

Excavation of the soils should be feasible using conventional excavation equipment. All excavations should be undertaken in accordance with OPSS 902.

If bedrock is encountered above the planned excavation elevation, excavation of bedrock will be required to attain the subexcavation geometry and to provide base padding to improve the consistency of settlement performance with adjoining sections over more compressible ground. Although the selection of equipment and construction procedures should be the responsibility of the Contractor, rock excavation techniques such as blasting per OPSS 120 and possibly jack-hammering should be suitable. Near vertical sidewalls may be utilised for excavations in bedrock.

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

According to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria, native loose to compact noncohesive soils are classified as Type 3 soils necessitating temporary cut slopes to be inclined at 1H:1V. The very soft to soft cohesive soils and very loose noncohesive soils are classified as Type 4 soils necessitating temporary cut slopes to be inclined at 3H:1V or flatter.

Temporary roadway protection will be required where excavation slopes are steeper than 1H:1V from the base of the existing highway embankment to the base of excavation. Temporary roadway protection should be designed in accordance with OPSS 539 providing a minimum performance level 2.

3.3 Subgrade Preparation

Preparation of the subgrade for construction of the culvert should be carried out in accordance with OPSS 902 and SP 902S01.

For the culvert replacement, it is recommended to provide a minimum 500 mm of combined granular bedding and levelling course with a minimum 300 mm thick granular bedding below the culvert. The granular bedding material should be composed of Granular A or Granular B Type II and compacted in conformance with OPSS 501 (Method A). Alternatively, 19 mm diameter clear stone can be utilized for granular bedding and levelling course. Clear stone should be placed in accordance with OPSS.PROV 1004.

Subsurface soil conditions will vary between the boreholes drilled at the site and the thicknesses of overburden and the depths to bedrock should be expected to vary between the borehole locations. The soil and bedrock encountered should be excavated to the required bedding level prior to placement of levelling course and granular bedding for culverts. Levelling course and granular bedding can be placed below water level if the material is sufficiently self-compacting or by overbuilding above the water level by 1 m and then compacting and trimming to the bedding level.

Rock fill should be placed in accordance with OPSS.PROV 206 and SP 206S03. This is particularly important above the water level within the zone of influence of the culvert, defined by an imaginary line inclined downwards at 2H:1V from a point located at the invert level 1 m beyond the edge of the culvert.

For culverts placed on and within rock fill, the granular bedding material and rock fill material should be separated by a geosynthetic filter fabric to prevent loss of the granular materials into the voids of the rock fill. The rock fill surface should be chinked in accordance with the requirements of OPSS.PROV 206 and SP 206S03, prior to placing the geotextile. The filter fabric should conform to OPSS 1860 and comprise a Class II non-woven geotextile with a filtration opening size (FOS) of 105 to 210 µm. The filter fabric should be placed beneath the bedding and extend up each side and to the top of the bedding and/or granular cover material.

Where very loose to loose non-cohesive soils and very soft to soft clayey soils are anticipated at subgrade level and above the prevailing groundwater level, the subgrade should be covered with a layer of biaxial geogrid and backfilled with the select bedding material. It is recommended to provide 300 mm of granular bedding above the geogrid. The bedding material should be composed of Granular A or clear stone, installed and compacted as detailed above. Below the prevailing ground water level, geogrid is not applicable and granular fill or rock fill may be end dumped directly into the excavation.

If rock fill is placed as a backfill material below the culvert over loose non-cohesive soils, settlements of the culverts may exceed the 25 mm compression of the founding medium normally allowed for by SLS resistance values due to the rock fill sinking into underlying softer materials and fine soils infiltrating the rock fill voids. A minimum of 300 mm thick granular bedding material should be placed over the rock fill immediately beneath the culvert. The granular backfill should be shaped to conform to the shape of the invert of the culvert to reduce the structural distress that may result from the differential settlement as well as to minimise “low areas” in the culvert when settlement is complete.

Where the culvert invert is on bedrock, bedrock excavation may be required to provide the minimum bedding depth. The minimum combined depth of levelling course and culvert bedding should be 0.5 m. Mass concrete could be used to level bedrock, if required. Mass concrete could also be placed to provide a level founding surface for the wing wall or head wall footings, if required. Alternatively, the rock surface could be “stepped” to follow variations in the bedrock surface elevation thereby creating a level subgrade by a combination of rock excavation and placement of mass concrete.

3.4 Bearing Resistance

The bearing resistances in the following table, Table Section 3.4, are recommended at the culvert/ground interface for various subgrade material categories that could apply at this site depending on the depth to bedrock and design choices.

Table Section 3.4 - Bearing Resistances

Foundation Type	Subgrade Material Category	Factored Geotechnical Resistance at ULS (kPa)	Geotechnical Reaction at SLS (kPa)	Risk-Managed Geotechnical Reaction at SLS (kPa)
Box Culvert	Bedrock	>10000	>10000 (with no settlement)	Not required
	Rockfill Pad on Bedrock	750	350 (with <50mm settlement)	200 (with 25mm settlement)
	Rockfill Pad on Soil	375	250 (with <100mm settlement)	N/A (with 25mm settlement)
	Granular A or Granular B Type II Bedding on bedrock *	750	350 (with <25mm settlement)	Not required

** Note: Bearing resistance values apply where compaction in conformance to MTO standards is confirmed. This condition may not be feasible to achieve below the prevailing groundwater level.*

Watertight flexible joints to accommodate the indicated settlement for the identified subgrade material category in the above table should be provided between culvert segments. Where portions of culverts are founded on different subgrade materials, the flexible joints at the interface between segments founded on different subgrade materials should accommodate the differential settlement.

3.5 Lateral Resistance

The lateral earth and water pressure, p (kPa), will only be applicable for retaining structures such as head walls and wing walls and should be computed using 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 K = lateral earth pressure coefficient
γ = unit weight of free draining granular material above the design water level (kN/m^3)
γ' = unit weight of backfill submerged below the design water level (kN/m^3)
h_1 = depth below final grade (m), above the design water level
h_2 = depth below the design water level (m)
q = any surcharge load (kN/m^2)
γ_w = unit weight of water equal to 9.8 kN/m3
C_p = compaction pressure (refer to clause 6.9.3 of CHBDC)
C_s = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)

where Ø = angle of internal friction of retained soil (35º for Granular A)
δ = angle of friction between soil and wall (23.5º for Granular A)

The following parameters are recommended for design:

Table Section 3.5 - Lateral Earth Pressure Parameters

Parameter	Granular A, Granular B Type II	Rockfill
Angle of Internal Friction, degrees	35	42
Unit Weight, kN/m^3	22.8	18.0
Active Earth Pressure Coefficient (K_a)	0.27	0.20
At-Rest Earth Pressure Coefficient (K_o)	0.43	0.33
Passive Earth Pressure Coefficient (K_p)	3.69	5.04

The design should consider both the maximum water level and the stabilised groundwater level condition. Seasonal perched water should be anticipated above the clayey silt deposits at the culvert location.

The coefficient of earth pressure at rest should be employed to design rigid and unyielding walls and the active earth pressure coefficient for unrestrained structures. Concrete culverts are considered to be constrained.

3.6 Slope Stability and Settlement

The geometry of the existing embankment will be reinstated after the installation of the permanent culvert.

No slope stability issues are anticipated as demonstrated by the acceptable performance of the existing embankment.

No significant settlement issues are anticipated, provided that the materials in the reinstated embankments are constructed and compacted in conformance with requirements. No settlement is anticipated in the ground below the embankment fill as demonstrated by the acceptable performance of the existing embankment.

3.7 Seismic Site Coefficient

From a foundations engineering perspective, seismic conditions are not a consideration at this site. The site is not in a critical seismic load zone and the works are resistant to the effects for dynamic loading being a box culvert founded on or near bedrock within a rock fill embankment.

3.8 Frost Protection

A foundation frost penetration depth in the area is 2.0 m according to OPSD 3090.100. The granular aggregate materials should conform to OPSS.PROV 1010. Frost protection is not required for box culverts where the structural frame can withstand frost pressures.

3.9 Culvert Backfill

Backfill adjacent to the box culvert should be placed in accordance with OPSS 501, OPSD 803.010, OPSS 422 and SP 422S01.

Backfill should be brought up simultaneously on each side of the box culvert. The operation of heavy equipment within a horizontal distance defined as 0.5 times the height of the culvert should be restricted to minimise the potential for movement and/or damage of the culvert due to the lateral earth pressure induced by compaction.

The box culverts must be designed to resist the unbalanced lateral earth pressure and compaction pressure exerted by the backfill adjacent to the box culvert walls.

3.10 Embankment Fill

Embankment fill should be comprised of suitable earth fill, granular fill or rockfill. At this site, rockfill is preferred as it will not require erosion protection on the slopes and since rockfill is readily available in the area.

All embankment fill, above the prevailing groundwater, should be placed and compacted in accordance with OPSS.PROV 206. This is of particular importance within the zone of influence of the culvert, as defined by an imaginary line at a 2H:1V gradient inclined upwards from the invert level of the culvert and extending to the highway grade.

A transition zone is required to provide smooth settlement transitions along the highway alignment and to mitigate excessive differential settlements between the new installed embankment over the culvert zone and the adjacent highway embankments. In order to provide these transitions zones, excavation should extend to specified depths under the plan limits of culvert and then with geometry of 2H:1V or flatter backslopes from the base of sub-excavations to the ground surface.

The placement below the prevailing groundwater will be in-the-wet, and as such materials should be end-dumped without compaction and up to a minimum of 1 m above the groundwater level. The material should be then compacted in accordance with OPSS 501.

The rockfill embankment side slopes should be inclined no steeper than 1.25H:1V. If earth slope flattening is indicated, a vegetation cover over slope flattening material or other measures to control surface runoff and minimise erosion of the embankment slopes should be implemented.

Design should be in accordance with the MTO memorandum in Appendix F entitled “Post Construction Rockfill Settlement and Guidelines for Estimating Rockfill Quantity” dated September 14, 2010, which provides direction for design and construction including magnitude of post-construction settlement of rockfill and bulking factor assumptions.

3.11 Erosion Control

The protective measures noted in the OPSD 800 series to deal with erosion (inlet/outlet treatment, headwalls, cut-off walls etc.) are considered to be appropriate. The backfill should comprise OPSS Granular A or Granular B Type II.

Inlet and outlet protection in accordance with OPSS 511, OPSS.PROV 1004 and OPSD 810.010 is recommended to prevent erosion adjacent to the culvert as well as scour.

It is recommended that horizontal inlet cut-offs and rock protection and outlet erosion protection should be considered instead of vertical cut-offs and structural head walls in order to minimize excavation into bedrock and construction below the groundwater level. In this case, the following recommendations apply:

> The length and width of horizontal cut-off aprons shall be a minimum of 2.0 m or twice the diameter of the culvert, whichever is less.
>
> The rock protection shall conform to OPSS 511 with a minimum dimension of 0.3 m and a minimum thickness of 0.5 m and extend to a minimum of 0.3 m above the culvert obvert level.

Clay seals at the inlet shall be in conformance with OPSS 1205 and extend over the area defined under rock protection.

Drainage and/or filter blankets at the outlet shall extend over the area defined under rock protection and may consist of a natural filter consisting of a minimum thickness of 0.3 m of Granular A or non-woven Class II geotextile with an FOS of 75-150 μm according to OPSS 1860. The filter shall be placed below the rock protection to minimize the potential for erosion of fine particles from below the treatment.

Where earth slopes are inclined at 2.5H:1V or steeper, the permanent earth slopes should be protected with erosion control blankets. Where embankments are composed of earth, they should be covered with topsoil or suitable excess earth material from swamps or muskeg areas and seeded in accordance with OPSS 802 and 804 as soon after grading as possible to prevent erosion. Alternatively rock fill embankments without erosion protection or a minimum 0.5 m thickness of rock protection over earth fill may be applied for this purpose.

Refer to OPSS 511 - Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting, for design and installation requirements for these types of erosion control treatments.

Refer to OPSS.PROV 804 - Construction Specification for Seed and Cover, for design and installation requirement for Matrix Bonded Fabric (BMF) for erosion control.

4. CONSTRUCTION CONSIDERATIONS

4.1 Groundwater Control

Groundwater is near the surface along the culvert alignment.

For construction in-the-dry, it would be necessary to implement measures to control the surface water flow and the groundwater. Conventional procedures such as dam and pump and/or diversion of the stream may be sufficient to control surface water flow. It is noted that the groundwater levels are subject to seasonal fluctuations and precipitation patterns. The contract documents should include an NSSP stating that the groundwater level should be lowered to a minimum 0.5 m below the proposed founding levels for construction in-the-dry, except in bedrock where it should be lowered to the surface of the bedrock. Refer to Appendix B for the draft NSSP - Surface Water Control and Dewatering. Dewatering along the culvert alignment would be challenging due to the nature of the ground and may require an enclosed cofferdam for construction in-the-dry.

However, construction in-the-wet is feasible by excavating without dewatering, overbuilding the levelling course/bedding and compacting, then trimming to the required top of bedding elevation. Construction in-the-wet should be considered in order to avoid the challenges and costs associated with construction in-the-dry.

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

4.2 Planned Staging for Temporary Stream Diversion

Highway 17 consists of two embankments with a central median. One embankment carries the eastbound lane traffic and one embankment carries the westbound lane traffic. It is assumed that the construction staging will consist of diverting both EB and WB traffic onto the WB lanes to permit construction of the culvert under the EB lanes, followed by diversion of both EB and WB traffic back onto the completed EB lanes to permit construction of the culvert under the WB lanes and completion of the WB lanes embankment.

According to the preliminary design provided by AECOM, a temporary bypass pipe (130 m long) has been considered to divert the creek flow during the construction.

4.3 ‘Red Flag’ Issues

The “red-flag” issues outlined and the recommended methods of overcoming these issues noted in the following sections of this report are intended to alert and aid the designer and where appropriate to alert the Contractor through subsequent contract specification. It is noted that no responsibility or liability is assumed by the MTO or its design consultants for alerting the contractor to all “red-flag” issues. The requirement to deliver acceptable construction quality remains the responsibility of the Contractor.

The red-flag issues for this project include potentially complex dewatering challenges and the designation of ground and rock for contract purposes for subexcavation below ground surface.

All construction work should be carried out in accordance with the Occupational Health and Safety Act and with local/MTO regulations.

Refer to Appendix E, Standard Specifications Relevant to Report for a list of relevant OPSS’s and for draft NSSP’s that should be included in the contract documents.

4.4 Contract Specifications

A list of standard specifications and draft NSSP’s relevant to this report are compiled in Appendix E.

A critical contract issue will be designation and payment for excavation due to uncertainties about the depth to bedrock between borehole locations. In order to mitigate this risk, consideration should be given to implementing a process in the contract to account for this, such as the following:

- The measure for payment should be defined as cost per cubic metre of excavated material for both soil and bedrock/rockfill.

- The excavation geometry for payment purposes should be defined as to the specified depth beneath the plan limits of the culvert and from that depth at a back-slope of 1.5H:1V to the ground surface.
- The Contractor should submit prices per cubic metre for excavation of soil and for excavation of bedrock/rockfill. Payment should be for actual volumes removed.
- In order to avoid unbalanced bids, the Contractor should be advised that for bid evaluation purposes a presumed total excavation quantity of, for example, 5,000 cubic metres (or a more precise value for the planned volume of excavation) and a blended bid price of 60% weighting for rock fill, 20% weighting for soil and 20% for bedrock could be assumed.

5. CLOSURE

This Foundation Design Report was prepared by Ms. M. Kamranzadeh, MSc, EIT., and reviewed by Mr. D. Dundas, P.Eng., Senior Engineer, Geotechnical Services. Mr. C. M. P. Nascimento, P. Eng., Project Manager and MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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APPENDIX A

List of Standard Specifications Relevant to Report Report

Non-Standard Special Provisions (NSPP's)

LIST OF STANDARD SPECIFICATIONS RELEVANT TO REPORT

DOCUMENT	TITLE
OPSS 120	General Specification for Use of Explosives
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 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Excavation and Backfilling of Structures
OPSS 1205	Material Specification for Clay Seal
OPSS 1860	Material Specification for Geotextiles
OPSS.PROV 206	Construction Specification for Grading
OPSS.PROV 804	Construction Specification for Seed and Cover
OPSS.PROV 1004	Material Specification for Aggregates – Miscellaneous
OPSS.PROV 1010	Material Specification for Aggregates, Base, Subbase, Select Subgrade and Backfill Material
SP 206S03	Construction Specification for Grading
SP 422S01	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers
SP 902S01	Excavation and Backfilling of Structures
OPSD 803.010	Backfill and Cover for Concrete Culverts
OPSD 810.010	General Rip-Rap Layout Sewer and Culvert Outlets
OPSD 3090.100	Foundation Frost Depth for Northern Ontario

NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP – Variation in Depth to Bedrock between Boreholes (Addition to OPSS 902)

The Contractor is advised that the depth to bedrock between boreholes may vary along and across the culvert alignment.

NSSP - Variable Mixed Fill and Rock Fill at Embankments (Addition to OPSS 902 and OPSS539)

The Contractor shall be advised that the existing highway embankments and the ground in the vicinity of the embankments contain variable components of mixed fill and rock fill and that the Contractor shall use methods and equipment that are appropriate for the work.

NSSP – Surface Water Control and Dewatering (Addition to OPSS 902)

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 excavations for work in-the-dry in overburden and to the bedrock surface for work in-the-dry in bedrock. Although the Contractor shall be responsible for designing and implementing measures for surface water control and dewatering, the Contractor is advised that damming of Meatbird Creek and diversion of the flow through pumping through temporary conduits to accommodate construction staging will probably be required at this site.

NSSP – Installation of Shoring (Addition to OPSS 539)

The Contractor shall be advised that cobbles, boulders and rockfill may be encountered during the excavation and that the Contractor shall use appropriate methods for shoring installation.

APPENDIX B

MTO Guideline for Rockfill Settlement and Rockfill Quantity Estimate

Dated September 14, 2010

MTO Guideline for Rock Fill Settlement and Rock Fill Quantity Estimates

September 14, 2010

SUBJECT: **ROCKFILL SETTLEMENT AND ROCK FILL QUANTITY ESTIMATES**

PURPOSE: To provide direction for estimating settlements and quantity of rock fill used for the construction of new embankments. The criteria are to provide guidance for estimating settlement within rock fill (within the embankment proper exclusive of the settlement of the native subsoil) of new embankments; and outlining the information that should be provided for use in the estimation of the quantity of rock fill that may be required for construction. The criteria apply to strong, granitic-type rock fills (placed above and below original ground surface) that are up to 15 m in total thickness. The criteria should be reviewed and the designs modified for thicker/higher rock fill embankments and/or for weaker types of rock fill on a project specific basis.

BACKGROUND: If rock fill is used for the construction of embankments, there will be settlement due to compression of the rock fill. In highway embankments, settlement of rock fill during and after construction occurs as a result of re-arrangement of rock particles under load and as a result of crushing of rock particles at point contacts.

The magnitude of settlement of the rock fill depends on the following factors:

- type of rock/strength of particles;
- size and shape of rock particles;
- gradation of rock fill;
- total height/thickness of rock fill (stress level); and,
- method of construction and sequence of placement (including, lift thickness, compactive effort, and state of packing).

The magnitude of the short-term settlement (i.e. within about 1 year following completion of construction to full height) and long-term settlement (i.e. after 1 year, over the life of the embankment) of rock fill depends on amongst other variables the method of placement (compacted versus dumped) as discussed below.

Compacted Rock Fill

Where possible, rock fill should be placed in a controlled manner (i.e. not end dumped) in accordance with Special Provision 206S03. Blading, dozing and 'chinking' the rock to form a dense, compact mass will be required to minimize voids and bridging and should be used to construct rock fill embankments above the existing groundwater table. Rock size shall be controlled in accordance with SP206S03.

Dumped Rock Fill

If rock fill embankments are constructed by end dumping rock fill (for cases where Special Provision 206S03 cannot be applied) or when backfilling sub-excavated areas below the groundwater table by end dumping rock fill with little or no control on the lift thickness and compactive effort, the settlement of rock fill placed in this uncontrolled manner will be greater than that of compacted rock fill.

POLICY:

Section 1: Performance - Recommendation for Design

For rock fill embankments, both the short-term and long-term settlement of the fill should be considered in the design. Further, both the compacted and un-compacted portions of rock fill in the embankment should be considered when estimating the magnitude of settlement. In all cases, the total height of the rock fill embankment will be measured from the base of the rock fill.

1.1 Short-Term Rock Fill Settlement

For rock fill embankments constructed over a non-compressible subgrade, the percentages in Table 1.1 should be used for estimating the short-term settlement of the embankment.

Table 1.1: Short-Term Rock Fill Settlement

Height of Rock Fill, H (m)	Short-Term Settlement (m)	
	Compacted Rock Fill	Dumped Rock Fill
Up to 5	0.5%·H	1.0%·H
>5 to 10	0.75%·H	1.5%·H
>10 to 15	1.0%·H	2.0%·H

Short-term is defined as 1 year after the rock fill embankment is constructed to full height. Approximately 90% of the short-term settlement may be expected to be complete within 6 months following construction to full height (including surcharge, if applicable).

1.2 Long-Term Rock Fill Settlement

For rock fill embankments constructed over a non-compressible subgrade, the percentages in Table 1.2 should be used for estimating the long-term settlement of the embankment.

Table 1.2: Long-Term Rock Fill Settlement

Height of Rock Fill, H (m)	Long-Term Settlement (m)	
	Compacted Rock Fill	Dumped Rock Fill
Up to 15	0.1%·H	0.2%·H

Long-term is defined as being after 1 year following construction to full height, over the life of the embankment.

1.3 Rock Fill Embankments over a Compressible Subgrade

For rock fill embankments constructed over a compressible subgrade, the estimated settlement of the embankment must include the compression of the rock fill (short-term and long-term, as described in Section 1.1 and 1.2) plus the settlement of the compressible foundation soils.

Section 2: Guidelines for Estimating Rock Fill Quantities for Construction

Each fill material has its own unique quantity requirements that are dependent upon the type of material used. For the appropriate embankment fill item, the designer determines the quantity of material for backfill and embankment construction by considering the following:

- neat lines of the embankment;
- embedment of fill material into the founding stratum;
- settlement during construction of the underlying founding stratum;
- settlement during construction of the un-compacted fill material;
- settlement during construction of the compacted fill material; and,
- construction loss of material below the water line.

For each swamp crossing and high fill area, the Foundation Investigation and Design Report should include the following estimates:

- estimated max. embedment of fill into the founding stratum (m);
- estimated max. settlement of the founding stratum during construction (m); and
- estimated max. settlement within the fill itself (both compacted and un-compacted) (m)

The estimates of maximum embedment and foundation soil settlement during construction are to be considered by the designer when estimating the quantity of fill required for construction. To account for the settlement of rock fill during construction, the rock fill quantity should be estimated using the standard bulking factor(s) currently recommended by MTO.