Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT
WOODS CREEK CULVERT REPLACEMENT
GWP 154-91-00, SITE 35-212
REHABILITATION OF HIGHWAY 6 FROM NORTH LIMITS OF FERGUS, NORTHERLY 17.3 KM TO THE CONESTOGA RIVER BRIDGE IN ARTHUR
OWEN SOUND, ONTARIO

for

McCORMICK RANKIN CORPORATION

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Distribution:
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PML Ref.: 04KF136A
Geocres No.: 40P9-41
Index No.: 077FIR and 078FDR
February 16, 2006

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for
Woods Creek Culvert Replacement
GWP 154-91-00, Site 35-212
Rehabilitation of Highway 6 from North Limits
Of Fergus, Northerly 17.3 km to the
Conestoga River Bridge in Arthur
Owen Sound, Ontario

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1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed replacement of the Woods Creek culvert, (Site 35-212; Station 28+140) located on Highway 6 between Fergus and Arthur in the Township of Nichol, Ontario. The site is located approximately 5.0 km north of the Town of Fergus. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ontario Ministry of Transportation (MTO).

The existing culvert is a concrete rigid frame open footing culvert with a span of 6.1 m, height of 1.8 m and is 30.3 m long (ref.: RFP, Section 6.3.1, page 28). The invert of the existing culvert (measured during the field investigation) is near elevation 423.3. The height of fill above the culvert is about 1.5 m and the overall embankment height is approximately 2.5 m (measured during the field investigation).

Cracks exist on the 'roof' of the culvert at both the inlet and outlet of the structure. There is no evidence of cracks in the sidewalls of the culvert in the vicinity of the inlet/outlet of the culvert.

2. SITE DESCRIPTION

The existing culvert is located in a rural setting between the towns of Fergus and Arthur. Land use in the vicinity of the culvert is primarily agricultural with occasional treed and low lying areas. The topography adjacent to the site is typically flat and undulating with drainage provided by overland flow to the Woods Creek. Occasional residential properties are present along Highway 6. Refer to the photographs for typical conditions along the highway embankment.

3. PHYSIOGRAPHY AND GEOLOGY

The study area is located in the Dundalk Till Plain physiographic region. The native soils primarily consist of Georgian Bay Lobe Tavistock silt to clayey silt. In general, the thickness of the native soil overlying bedrock ranges from 20 to 30 m. The underlying bedrock consists of brown or tan Dolostone of the Guelph Formation.

4. INVESTIGATION PROCEDURES

The field work for the investigation was carried out on December 16, 2004 and January 10, 2005, and comprised six boreholes. Three boreholes were advanced to depths of 5.4 to 9.6 m to investigate foundation conditions along the culvert in accordance with the generic terms of reference for the project. Three additional boreholes were drilled through the embankment adjacent to the culvert to depths of 3.3 to 5.3 m to assess possible causes for the structural distress evident in the culvert. The borehole locations and stratigraphic profile prepared from the borehole data are presented on Drawing 1.

The locations of the boreholes were established in the field relative to the existing culvert by Peto MacCallum Ltd. (PML). The ground surface elevations were surveyed in the field by Callon Dietz Inc. surveyors.

The boreholes were advanced using continuous flight solid stem augers, powered by truck and track mounted CME 55 and 75 drillrigs, supplied and operated by specialist drilling contractors, working under the full time supervision of members of our engineering staff.

Representative samples of the overburden were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata.

Soils were identified visually in the field in accordance with the MTO Soil Classification procedures. The ground water conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and by measurement of the water level in the open boreholes. All the boreholes were backfilled with a bentonite/soil mixture in accordance with O.Reg 903 and the MTO guidelines for borehole abandonment procedures.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. Grain size distribution analyses, Specific Gravity and Atterberg Limits tests were carried out on selected samples. The results of the Grain Size Distribution analysis and the Atterberg Limits tests are presented on Figures 1 to 3, and on the Record of Borehole sheets.

5. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations, standard penetration test N values and ground water observations. The results of laboratory grain size distribution analyses, Atterberg Limits tests, and moisture content determinations are also shown on the borehole logs.

The subsurface stratigraphy revealed in the boreholes drilled through the highway embankment consisted of a sand fill overlying topsoil/topsoil fill. A surficial layer of topsoil and fill was penetrated in the borehole drilled at the east end of the culvert; topsoil was encountered surficially in the borehole drilled at the west end of the culvert. Topsoil was encountered in the boreholes drilled near the ends of the existing culvert. Silt till was encountered beneath the topsoil in all boreholes to the termination of drilling.

Ground water was measured in three boreholes at elevation 419.3 to 424.5 during drilling. Ground water was not detected in the other three boreholes.

5.1 Fill

Fill was encountered in Boreholes 102, 104, 105 and 106 drilled through the road embankment. It comprised cohesionless sand, with gravel and was compact to dense (Standard Penetration Test N (N) values of 12 to 33). The moisture content of the sand fill ranged from 5 to 10%. A 200 mm layer of topsoil fill was encountered below the sand fill in Borehole 102. The fill was penetrated at depths of 2.1 to 3.2 m (elevation 423.4 to 424.7).

The results of a grain size analysis performed on a sample of the embankment fill is presented on Figure 1.

Fill was also identified below a surficial topsoil layer in Borehole 101 drilled near the east end of the culvert adjacent to a gabion basket retaining wall. It was 1.8 m thick and consisted of loose sandy silt retained by the gabion baskets.

5.2 Topsoil

A 200 mm thick layer of topsoil was encountered surficially in Boreholes 101 and 103. Topsoil was also encountered beneath the embankment fill in Boreholes 102, 104, 105 and 106. The moisture content of the topsoil beneath the embankment fill ranged from 11 to 24%. The topsoil revealed in the boreholes drilled through the embankment was 0.4 to 0.9 m thick and penetrated at depths of 3.0 to 3.6 m (elevations 422.9 to 423.8).

5.3 Silt Till

Non to slightly cohesive silt till was contacted beneath the embankment fill and/or topsoil deposits in all six boreholes. The silt till was compact to very dense (N values of 11 to 50 blows for 50 mm penetration).

The moisture content of the silt till typically ranged from 6 to 11%.

The results of grain size distribution analyses performed on representative samples of the silt till are presented on Figure 2.

The silt till had liquid and plastic limits of 15 to 20 and 10 to 13 respectively, indicating the material is non to slightly plastic. The results of Atterberg Limits testing conducted on samples of the silt till are presented on Figure 3.

Drilling was terminated within the till in all boreholes at depths of 3.3 to 9.6 m (elevations 417.2 to 423.4).

5.4 Ground Water

Water was measured after the field work was completed in Boreholes 101, 102 and 105 at depths of 2.3 to 6.1 m (elevations 419.3 to 424.5). At the completion of drilling, Boreholes 101 and 102 caved at depths of 7.0 and 5.6 m (elevations 418.4 and 421.2) respectively. Ground water was not detected in the remaining three boreholes.

The water level in Woods Creek at the time of the field investigation was near elevation 423.5, about 200 mm above the invert level of the culvert.

Cognizant of the composition of the subgrade soil (silt with sand to sandy), the stabilized ground water level is expected to be near the water level in the creek. Observed ground water levels are subject to seasonal fluctuations and rainfall patterns.

6. CLOSURE

The field work was carried out under the supervision of Mr. Rob Mount, BEng and direction of Mr. Phil Cullen, P.Eng. The equipment was supplied by Elite Drilling and Aardvark Drilling Inc. The laboratory tests were conducted in the Kitchener office of PML.

The report was prepared by Mr. Phil Cullen, P.Eng., Project Engineer, and Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer. It was reviewed by Mr. Dennis Kerr, P.Eng., Chief Foundation Engineer. Mr. Brian Gray, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Sincerely

Peto MacCallum Ltd.

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Photograph 1: Woods Creek Culvert, East Side, Viewing South.

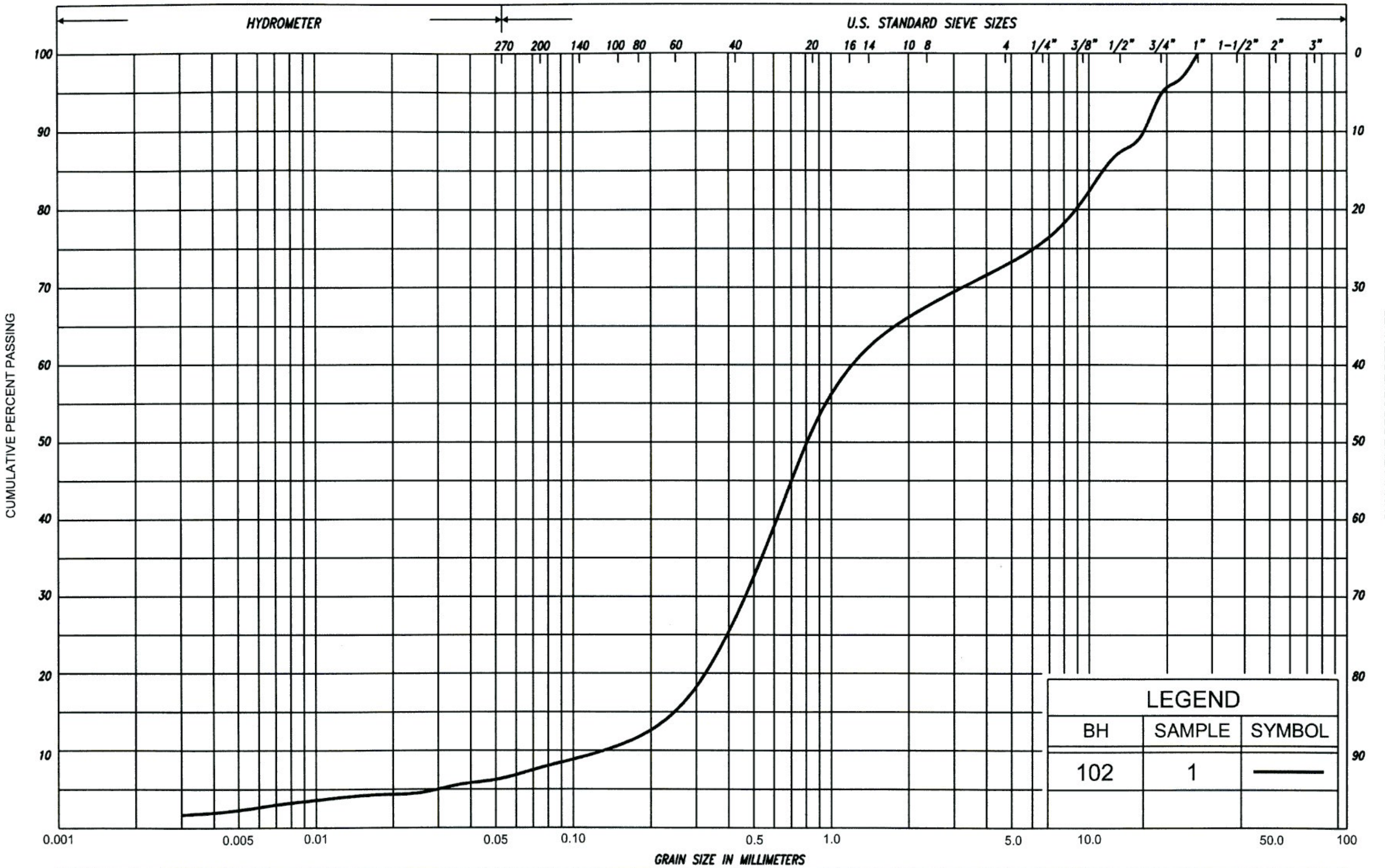
Photograph 2: Woods Creek Culvert, West Side, Viewing South.

Photograph 3: View north of culvert west side of road.

Photograph 4: View west of inlet to culvert. Note crack in concrete.

Photograph 5: View northeast of culvert outlet from south side of creek. Note crack in concrete.

Photograph 6: View southeast of culvert outlet from north side of creek. Note crack in concrete.



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

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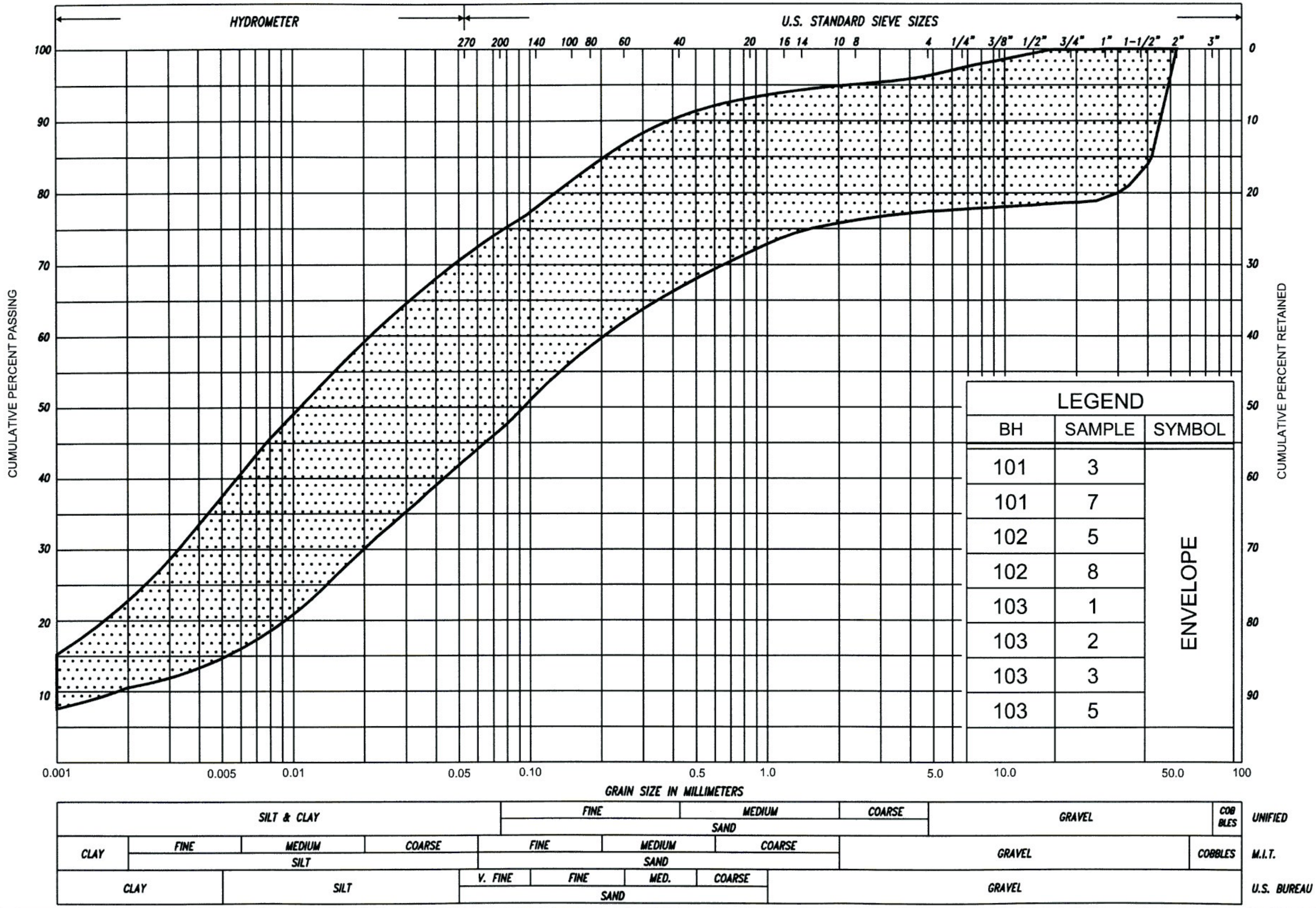
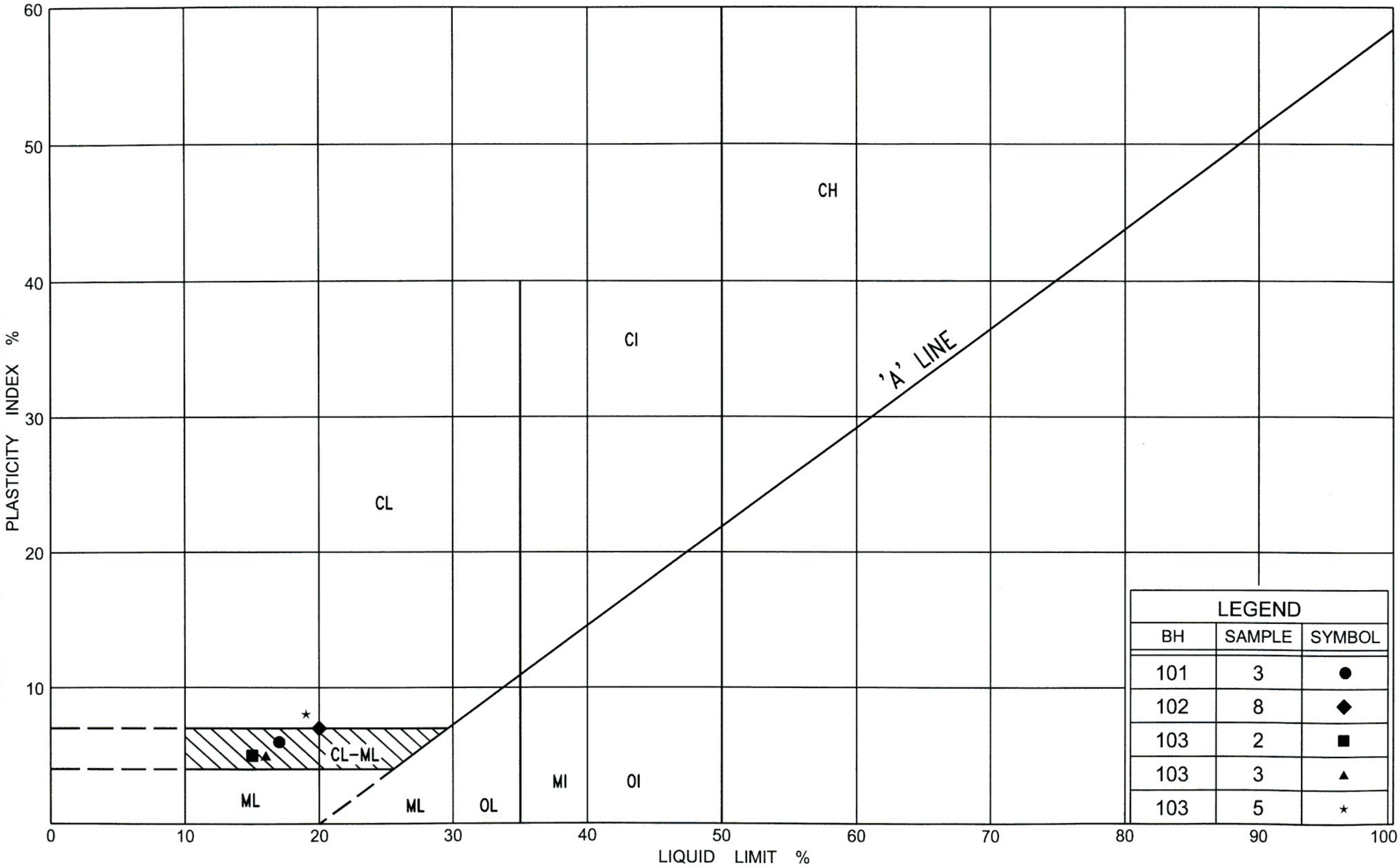
GRAIN SIZE DISTRIBUTION
SAND, with gravel
(FILL)

FIG No.	1
HWY	6 Culverts
W.P. No.	154-91-00



Ministry of Transportation Ontario	GRAIN SIZE DISTRIBUTION SILT TILL, with sand to sandy	FIG No. 2 HWY 6 Culverts W.P. No. 154-91-00



LEGEND		
BH	SAMPLE	SYMBOL
101	3	●
102	8	◆
103	2	■
103	3	▲
103	5	★

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PLASTICITY CHART
SILT TILL

FIG No.	3
HWY	6 Culverts
W.P. No.	154-91-00

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 $\overline{N}$.

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.

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
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	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				v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$	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			

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 101

METRIC

W.P. 154-91-00 | LOCATION Hwy 6, Woods Creek Station 28+142, 16.4 m Rt. | ORIGINATED BY RM

DIST 33 HWY 6 | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY PC

DATUM Geodetic | DATE January 10, 2005 | CHECKED BY DK

ELEV / DEPTH	SOIL PROFILE DESCRIPTION
425.4	
0.0	Topsoil
0.2	Sandy silt, trace gravel. Loose, Dark brown, Moist (FILL)
423.4 / 2.0	Silt, with sand to sandy, some to trace gravel, some clay. Dense to very dense, Brown, Moist (Till)
417.2 / 8.2	End of Borehole

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	4	
2	SS	35	
3	SS	76/ 28cm	15 28 43 14
4	SS	93/ 28cm	
5	SS	50/ 13cm	
6	SS	50/ 10cm	
7	SS	51	8 38 43 11

Column headings: GROUND WATER CONDITIONS; ELEVATION SCALE (425, 424, 423, 422, 421, 420, 419, 418); DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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.

▼ Water level measured after drilling

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

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 102

METRIC

W.P. 154-91-00 — LOCATION Hwy 6, Woods Creek Station 28+149, 6.5 m Rt. — ORIGINATED BY RA

DIST 33 HWY 6 — BOREHOLE TYPE Continuous Flight Solid Stem Augers — COMPILED BY PC

DATUM Geodetic — DATE December 16, 2004 — CHECKED BY DK

ELEV / DEPTH	DESCRIPTION
426.8 / 0.0	Sand with gravel, trace silt, trace clay. Compact Brown Moist (FILL)
424.7	Topsoil Fill
2.1	Topsoil
423.8 / 3.0	Silt, sandy to with sand, some to with clay, with to trace gravel. Compact to dense Brown Moist (TILL)
421.8 / 5.0	Very dense
417.2 / 9.6	End of Borehole

SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	23	27 65 6 2
2	SS	12	
3	SS	4	
4	SS	29	
5	SS	16	21 32 37 10
6	SS	35	
7	SS	60	
8	SS	48	4 21 52 23
9	SS	50/ 13cm	
10	SS	90/ 28cm	

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES (NUMBER, TYPE, "N" VALUES); GROUND WATER CONDITIONS; ELEVATION SCALE (426, 425, 424, 423, 422, 421, 420, 419, 418); DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (O 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

▼* Water level measured after drilling

ON_MOT VER 3A 04KF1136 GINT LOGS.GPJ ON_MOT.GDT 9/30/2005 9:06:03 AM

+7, ×5 : Numbers refer to Sensitivity — 20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

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Foundation Design

RECORD OF BOREHOLE No 103

METRIC

W.P. 154-91-00 | LOCATION Hwy 6, Woods Creek Station 28+134, 16.0 m Lt. | ORIGINATED BY RM

DIST 33 HWY 6 | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY PC

DATUM Geodetic | DATE January 10, 2005 | CHECKED BY DK

ELEV / DEPTH	DESCRIPTION (SOIL PROFILE)	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT / LIMITS	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
424.0					*			
0.0	Topsoil							
0.2	Silt, sandy to with sand, some clay, trace to with gravel. Compact to very dense, Brown, Moist (TILL)							
		1	SS	20		423	o	12 34 37 17
		2	SS	11		422	⊢o⊣	5 30 51 14
		3	SS	50/ 14cm			o ⊢⊣	17 30 39 14
						421		
		4	SS	50/ 12cm			o	
						420		
		5	SS	50/ 5cm		419	⊢o⊣	23 22 40 15
418.6 / 5.4	End of Borehole							

* Borehole dry on completion of drilling

Chart headings: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O 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

RECORD OF BOREHOLE No 104

METRIC

W.P. 154-91-00 LOCATION Hwy 6, Woods Creek Station 28+139, 6.0 m Lt. ORIGINATED BY RM

DIST 33 HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY PC

DATUM Geodetic DATE January 10, 2005 CHECKED BY DK

SOIL PROFILE		SAMPLES							
ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
426.6					*					
0.0	Sand, with gravel, trace silt, trace clay Compact Brown Moist (Fill)					426				
		1	SS	13		425				
		2	SS	12		424				
423.4 / 3.2	Topsoil	3	SS	21						
423.0 / 3.6	Silt, with sand, some clay, trace of gravel Compact Brown Moist (Till)					423				
		4	SS	17		422				
421.3 / 5.3	End of Borehole									

* Borehole dry on completion of drilling

Chart legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: O 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

$+^7, \times^5$: Numbers refer to Sensitivity   20 / 15 — 5 / 10 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 105

METRIC

W.P. 154-91-00 | LOCATION Hwy 6, Woods Creek Station 28+127, 6.0 m Lt. | ORIGINATED BY RM

DIST 33 HWY 6 | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY PC

DATUM Geodetic | DATE January 10, 2005 | CHECKED BY DK

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) (approx.)
426.5 / 0.0	Sand, with gravel, trace silt, trace clay; Dense to Compact, Brown, Moist (FILL)					426	
		1	SS	33		425	~5
		2	SS	16		424	~5
423.5 / 3.0	Topsoil	3	SS	25		423	~10
422.9 / 3.6	Silt, with sand, some clay, trace gravel; Dense, Brown, Moist (TILL)	4	SS	30	▼*		~5
422.1 / 4.4	End of Borehole						

Column headings: SOIL PROFILE; SAMPLES; 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

▼ Water level measured after drilling

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

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 106

METRIC

W.P.	154-91-00	LOCATION	Hwy 6, Woods Creek Station 28+138, 6.5 m Rt.	ORIGINATED BY	RM
DIST	33 HWY 6	BOREHOLE TYPE	Continuous Flight Solid Stem Augers	COMPILED BY	PC
DATUM	Geodetic	DATE	January 10, 2005	CHECKED BY	DK

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
426.7 / 0.0	Sand, with gravel, trace silt, trace clay. Compact Brown Moist (FILL)				*	426			
		1	SS	20		425			
		2	SS	19					
424.0 / 2.7	Topsoil					424			
423.5 / 3.2		3	SS	18					
423.4 / 3.3	Silt, with sand, some clay, trace gravel. Compact Brown Moist (TILL)								
	End of Borehole								

* Borehole dry on completion of drilling

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20, 40, 60, 80, 100); SHEAR STRENGTH kPa: O 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

ON_MOT VER 3A 04KF1136 GINT LOGS.GPJ ON_MOT.GDT 9/30/2005 9:09:18 AM

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

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

METRIC

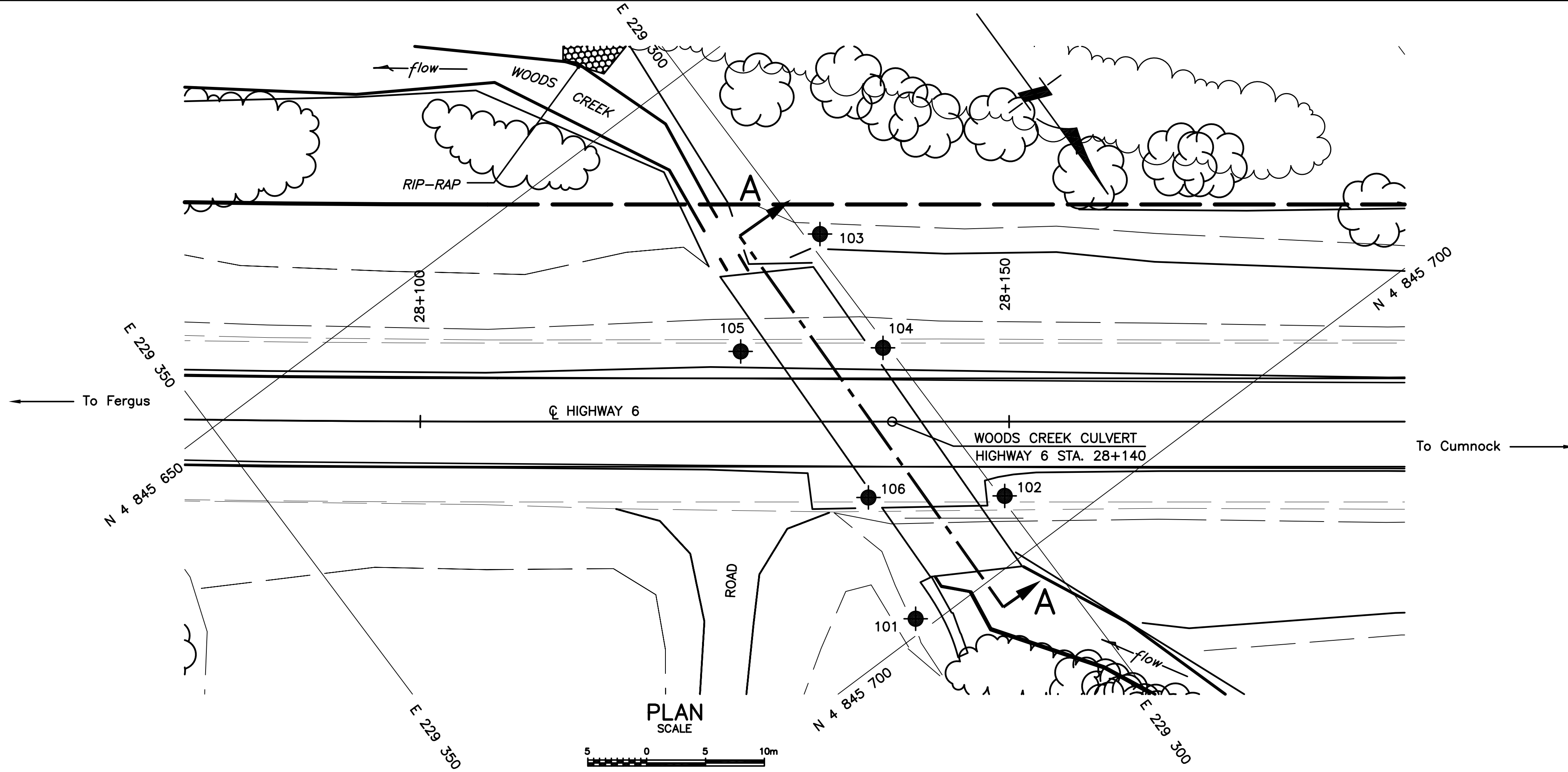
DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS IN KILOMETRES + METRES

CONT No
GWP No 154-91-00

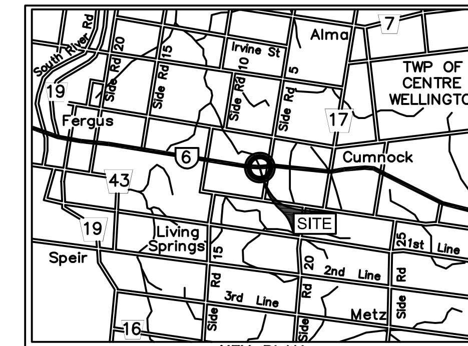
HIGHWAY 6
WOODS CREEK CULVERT
STA.28+140
BOREHOLE LOCATIONS & SOIL STRATA

SHEET

Peto MacCallum Ltd.
CONSULTING ENGINEERS



PLAN
SCALE

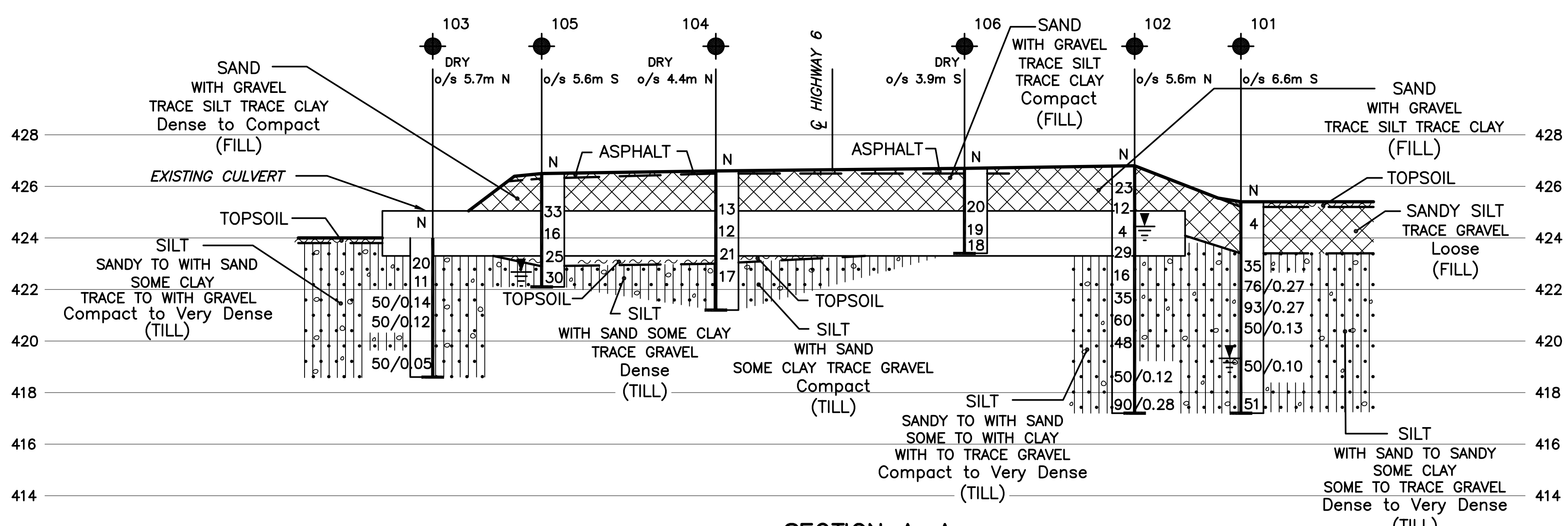


KEY PLAN
SCALE

LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N — Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE — Blows/0.3m (60° Cone, 475 J / blow)
- W L at time of investigation Dec 2004 to Jan 2005
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
101	425.4	4 845 699	229 312
102	426.8	4 845 695	229 300
103	424.0	4 845 668	229 299
104	426.6	4 845 679	229 300
105	426.5	4 845 672	229 310
106	426.7	4 845 688	229 309



SECTION A-A
SCALE

NOTE:

SECTIONS ARE PROVIDED SOLELY FOR ILLUSTRATIVE PURPOSES. REFER TO RECORD OF BOREHOLES FOR DETAILED DESCRIPTION OF SUBSURFACE CONDITIONS, IN-SITU TEST DATA AND LABORATORY TEST RESULTS.

– NOTE –

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

REGISTERED PROFESSIONAL ENGINEER D. W. KERR PROVINCE OF ONTARIO

REGISTERED PROFESSIONAL ENGINEER B. R. GRAY PROVINCE OF ONTARIO

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 40P9-41

HWY No 6			DIST 33
SUBM'D PC	CHECKED PC	DATE FEB. 16, 2006	SITE 35-212
DRAWN MM/NA	CHECKED DWK	APPROVED BRG	DWG 1

REF No E-BASEPLAN(NICHOL).dwg; DESIGN ETRS(NICHOL).dwg; January 2005

FOUNDATION DESIGN REPORT
WOODS CREEK CULVERT REPLACEMENT
GWP 154-91-00, SITE 35-212
REHABILITATION OF HIGHWAY 6 FROM NORTH LIMITS OF FERGUS, NORTHERLY 17.3 KM TO THE CONESTOGA RIVER BRIDGE IN ARTHUR
OWEN SOUND, ONTARIO

for

McCORMICK RANKIN CORPORATION

PETO MacCALLUM LTD.
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5 cc: McCormick Rankin Corporation for distribution to MTO, Project Manager plus one digital copy
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1 cc: PML Hamilton
1 cc: PML Kitchener
1 cc: PML Toronto

PML Ref.: 04KF136A
Geocres No.: 40P9-41
Index No.: 078FDR
February 16, 2006

TABLE OF CONTENTS

FOUNDATION DESIGN REPORT
for
Woods Creek Culvert Replacement
GWP 154-91-00, Site 35-212
Rehabilitation of Highway 6 from North Limits
of Fergus, Northerly 17.3 km to the
Conestoga River Bridge in Arthur
Owen Sound, Ontario

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1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding replacement of the Woods Creek culvert, (Site 35-212; Station 28+140) located on Highway 6 between Fergus and Arthur in the Township of Nichol, Ontario. The site is located approximately 5.0 km north of the Town of Fergus, along the Highway 6 corridor. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ontario Ministry of Transportation (MTO).

The existing culvert is a concrete rigid frame open footing culvert with a span of 6.1 m, height of 1.8 m and is 30.3 m long (ref.: RFP, Section 6.3.1, page 28). The invert of the existing culvert (measured during the field investigation) is near elevation 423.3. The height of fill above the culvert is about 1.5 m and the overall embankment height is approximately 2.5 m (measured during the field investigation).

Cracks exist in the 'roof' of the culvert at both the inlet and outlet of the structure. There was no evidence of cracks in the sidewalls of the culvert in the vicinity of the inlet/outlet of the culvert.

The current design calls for the replacement culvert to be located on the same alignment as the existing culvert.

The field investigation revealed that the highway embankment consists of sand fill constructed on topsoil. A surficial topsoil layer was also revealed in boreholes drilled near the ends of the culvert. The topsoil is underlain by compact to very dense silt till.

Ground water was measured at depths of 2.3 to 6.1 m (elevations 419.3 to 424.5) in three boreholes at the completion of drilling and not detected in the remaining three boreholes. The water level in Woods Creek at the time of the field investigation was near elevation 423.5.

The information revealed in the boreholes indicate the native inorganic silt till that exists below the culvert is a competent bearing material. The boreholes drilled through the roadway embankment adjacent to the culvert identified the presence of some 0.4 to 0.9 m of topsoil above the silt till. The surface elevation of the silt till revealed in these boreholes along with other pertinent details are noted in the following table:

BOREHOLE	FILL THICKNESS (m)	TOPSOIL THICKNESS (m)	SURFACE ELEVATION OF SILT TILL
102	2.1	0.9	423.8
104	3.2	0.4	423.0
105	3.0	0.6	422.9
106	2.7	0.5	423.5

As noted previously, the existing culvert is a rigid frame open footing structure with the invert at elevation 423.3. Considering the normal depth of embedment to account for erosion and frost penetration, it is likely that the footings are founded in the silt till. Since the silt till is a competent bearing material, the cracks observed in the culvert are not attributed to foundation conditions.

A list of the MTO documents referred to in subsequent sections of the report is provided in Table 1.

2. FOUNDATIONS

It is expected that the invert of the proposed culvert will be near the same elevation as the existing culvert (elevation 423.3). Therefore, the design subgrade level of the foundation for a new open footing culvert is interpreted to be near or below elevation 421.6 and 422.8 if a box culvert is constructed.

The subgrade material revealed in the boreholes below elevation 422.8 consisted of compact to dense silt till whereas below elevation 421.6 it comprised dense to very dense silt till.

The embankment height at this location is about 2.5 m.

It is considered that the compact to dense silt till is capable of supporting the stress imposed by the embankment and culvert foundation.

Culvert foundations constructed on the silt till at the elevations noted previously should be designed using the following geotechnical resistance at ultimate and serviceability limit states (ULS and SLS):

	Open Footing Culvert (1) Founded at Elevation 421.6	Box Culvert (2) Founded at Elevation 422.8
Factored Geotechnical Resistance at ULS	1100 kPa	700 kPa
Geotechnical Resistance at SLS	500 kPa	250 kPa

(1) 500 mm wide (2) 6 m wide base

The recommended resistance at SLS allows for 25 mm of total settlement; differential settlement along the length of the culvert is expected to be less than 75% of this value.

The topsoil and any soft/wet soils revealed below the subgrade during construction should be excavated prior to construction of the culvert foundation and the subgrade raised to the design level with engineered fill. Additional comments in this regard are provided in the paragraph that deals with site review during construction (SP 902S01).

Fill placed under the culvert to accommodate any variation in the level of the native surface and/or removal of any topsoil/alluvium deposits extending below the design founding level should comprise granular material (Granular B Type I and Granular B Type II) compacted to at least 95% of the target density in conformance with Ontario Provincial Standard Specification (OPSS) 501 and Special Provision (SP) 105S10. The granular fill zone should extend beyond the culvert base a minimum 0.5 m and down to the subgrade at 1 horizontal to 1 vertical (1H:1V) and be established by a site specific survey.

Preparation of the subgrade for construction of the culvert should be performed and monitored in accordance with SP 902S01 to verify the competency of the founding surface. This should include site review by geotechnical personnel during preparation of the subgrade as well as during placement and compaction of the fill material. It is noted that the silt till ranged in relative density from just compact in Borehole 103 to dense in Borehole 101. Particular attention is needed to ensure removal of any poor quality material below the design founding level of the culvert to minimize the potential for post construction settlement.

Subgrade preparation, cover backfill and frost treatment for the culvert should be carried out in accordance with Ontario Provisional Standard Drawing (OPSD) 803.010 and OPSS 422. The bedding material for a precast box culvert, if utilised, should comprise a minimum 150 mm thick layer of Granular A.

Use of driven piles or caissons to support the culvert was considered. Cognizant of the composition and engineering properties of the soil on site, use of spread footings to support the foundation loads is considered to be the preferred means of supporting the culvert from a foundation engineering perspective.

The seismic coefficient for the conditions at this site is 1.0 (Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00, March 2001). The zonal acceleration ratio is 0.05.

The culvert site is located in Seismic Performance Zone 1. The liquefaction potential of the clayey soils was evaluated by consideration of the grain size distribution (% of particles <0.005 mm), liquid limit values and the ratio of the water content to the liquid limit. Based on the research by Marcuson et al (1990), we believe liquefaction of the fine grained soils (more than 35% of the soil particles passing the No. 200 sieve) is unlikely (clause 4.6.2 of the CHBDC).

3. CULVERT BACKFILL AND RETAINING WALLS

Backfill adjacent to the culvert should be placed in general accordance with OPSD 803.010, 3121.150 and OPSS 422. Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. This should include placement of the fill simultaneously on each side of the culvert and restricted operation of heavy equipment within 0.5 times the height of the culvert (each side) to minimize the potential for movement and / or damage of the culvert due to the lateral earth pressure induced by compaction. Refer to SP 105S10 for additional information.

The culvert and retaining walls, if required to retain the embankment beyond the end of the culverts, must be designed to support the stress imposed by the overlying fill as well as to resist the unbalanced lateral earth pressure and compaction pressure imposed by the backfill adjacent to the culvert walls.

The lateral earth and water pressure, p (kPa), should be computed using the equivalent fluid pressures presented in Section 6.9 of the CHBDC, or employing the following equation assuming a triangular pressure distribution:

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

where p = lateral pressure (kPa)

K = lateral pressure coefficient

γ = unit weight of free draining granular material above the design water level (kN/m^3)

γ' = unit weight of submerged free draining granular material below the design water level (kN/m^3)

γ_w = unit weight of water
= 9.8 kN/m^3

h_1 = depth below final grade (m), above design water level

h_2 = depth below design water level (m)

q = any surcharge load (kN/m^2)

C_p = compaction pressure (refer to clause 6.9.3 of CHBDC)

C_s = earth pressure induced by seismic events (refer to clause 4.6.4 of CHBDC)

where Φ = angle of internal friction of retained soil (35° for Granular B Type II)

δ = angle of friction between the soil and the wall (23.5° for Granular B Type II)

The following parameters are recommended for design:

PARAMETER	GRANULAR A	GRANULAR B TYPE II	EXCAVATED GRANULAR MATERIAL
Angle of Internal Friction (degrees)	35	35	30
Unit Weight (kN/m^3)	22.8	22.8	20.0
Coefficient of Active Earth Pressure (K_a)	0.27	0.27	0.33
Coefficient of Earth Pressure At Rest (K_o)	0.43	0.43	0.50
Coefficient of Passive Earth Pressure (K_p)	3.69	3.69	3.00

The design should consider both the maximum and minimum water levels in the stream as well as the stabilized ground water level. The stabilized ground water level employed for design should be the same as the water level in the creek since the subgrade soil comprises silt with sand to sandy silt. The maximum and minimum stream water levels will be dictated by flood flow conditions and should be defined by the project hydraulic engineer.

The coefficient of earth pressure at rest should be employed to design rigid and unyielding walls; the coefficient of active earth pressure is suitable for retaining walls constructed beyond the end of the culvert. The horizontal force imposed on the walls of a box culvert, if employed, will be resisted by the base slab. For an open footing culvert, lateral resistance will be provided by the passive pressure developed by the soil and the frictional resistance along the base of the footing. The passive pressure should be computed using the equation provided above. The frictional resistance developed between the underside of the footing bearing on the compact silt till should be computed using an unfactored friction factor of 0.5.

A weeping tile system and/or weep holes should be installed to minimize the build up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or non woven Class II geotextile (with an FOS of 75-150 µm according to OPSS 1860) placed to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost free outlet.

4. EXCAVATION AND GROUND WATER CONTROL

The ground water level at the time of the field investigation was 1 to 3 m above the anticipated depth of excavation. Cognizant of the type of soil to be excavated (silt with sand to sandy silt), conventional sump pumping techniques may not be sufficient to control ground water seepage into the excavation and installation/operation wells or well points for a period of about two weeks prior to excavation may be required to provide a stable base and sidewalls of the excavation.

The contract should call for ground water control to be the responsibility of the contractor and the contractor to retain a specialist dewatering contractor to assess the preferred means of ground water control and a performance specification to maintain and control the ground water at least 0.6 m below the excavation base in order to provide a stable excavation.

It will be necessary to implement measures to control water flow in the stream. Conventional procedures such as draining and/or diversion of the stream should be sufficient. Observed ground water levels are subject to seasonal fluctuations and rainfall patterns.

Excavation to the anticipated founding level is expected to extend some 4 to 5 m below grade within the embankment through the pavement structure, sand fill and into the native silt till. The in situ materials above the ground water level are classified as Type 3 soils according to Occupational Health and Safety Act criteria. Temporary cut slopes inclined at 1H:1V from the base of the excavation should be employed. Below the ground water table, the materials are classified as Type 4 soils necessitating 3H:1V slopes.

It is recommended that the work be carried out during the dry summer months to minimize the amount of ground water inflow to be handled and the volume of surface water, if any, to be diverted from the construction area.

All construction work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

Shoring will be required to support the walls of the excavation and adjacent traffic lanes during installation of the replacement culvert if traffic is maintained on Highway 6.

The magnitude and distribution of the lateral earth pressures acting on a braced excavation wall is dependent upon the support system used, the number of supports, the allowable movements and the construction sequence. The recommended design earth pressure distribution for singly and multi braced walls, for the conditions that exist at the site, are presented in Figures 1 and 2 respectively. Recommendations concerning design and construction of the braced excavation support systems are provided in the figures.

A soldier pile and lagging system may be considered. Provided the spacing between soldier piles is at least five pile diameters, the unfactored lateral passive resistance developed on the face of the soldier pile below the base of the excavation may be taken as the passive earth pressure developed over a width equivalent to three times the pile diameter and depth of six times the pile diameter.

The following geotechnical parameters should be employed to design the wall:

Angle of Internal Friction, degrees	30
Unit Weight, kN/m^3	20.0
Coefficient of Active Earth Pressure K_a	0.33
Coefficient of Earth Pressure at Rest K_o	0.50
Coefficient of Passive Earth Pressure, K_p	3.00

Additional lateral resistance could be provided by installing tiebacks anchored in the sand fill or silt till. The unfactored pull out resistance (R) of anchors grouted in cohesionless material can be estimated using the following equation:

$$R = \sigma'_z A_s L_s K_f$$

where R = pullout resistance (kN)

σ'_z = effective vertical stress at midpoint of load carrying length (kN/m^2)
= γh_1 if total anchorage length is above the design ground water level
= $\gamma h_1' + \gamma' h_2$ if design ground water level is above the anchor

γ = bulk unit weight of soil above design ground water level
= 20 (kN/m^3)

γ' = buoyant unit weight of soil below design ground water level
= 10.2 (kN/m^3)

h_1 = depth below ground surface to midpoint of anchor (m)

h_1' = depth below ground surface to design ground water level (m)

h_2 = depth below design ground water level to midpoint of anchor (m)

A_s = circumference of fixed length of anchor (m)

L_s = effective embedment length of the anchor (m)

K_f = anchorage coefficient
= 0.8 for sand fill and dense silt till

A geotechnical resistance factor of 0.4 should be applied to the computed anchor capacity to determine the ULS resistance.

The ground surface adjacent to the excavation is expected to experience some inward movement and vertical settlement. The magnitude of movements adjacent to a braced cut can be limited by selection of an appropriate lateral earth pressure coefficient (see Figures 1 and 2) provided good quality workmanship and construction practice is employed. The anticipated magnitude of movements is as follows:

	MOVEMENT (% OF EXCAVATION DEPTH)
Lateral Movement	
Braced Excavation	0.2
Anchored Wall	0.1
Vertical Movement	0.05

Construction procedures should be specifically suited to limit any consequent settlement of the pavement subgrade behind the excavation face.

Foundations of heavily loaded/settlement sensitive structures and/or utilities located within close proximity to the excavation, may require underpinning to preserve the integrity of these structures. Further comments and general recommendations in this regard are provided in Figure 3.

5. EMBANKMENT FILL

It is anticipated that the embankment height at the culvert location will not exceed 3 m.

The anticipated subgrade for the embankments typically comprises compact silt. Topsoil was encountered in the boreholes drilled beyond the toe of the existing embankment as well as below the fill in the borehole advanced on the road shoulder. The topsoil and other excessively loose, soft, organic or otherwise deleterious materials within the limits of the embankment fill should be subexcavated prior to placement of the fill.

The embankment side slopes should be inclined no steeper than 2H:1V. A vegetation cover or other measures should be established to control surface runoff and minimise erosion of the embankment slopes.

It is considered that the subgrade soil is capable of supporting the embankment. Settlement of the embankment material is expected to be in the order of 25 mm. The settlement is expected to occur as the fill is placed and be essentially complete within a few months following placement of the fill.

6. EROSION CONTROL

The protective measures noted in the OPSD 800 series (in particular OPSD 810.010 and 810.020 for box culverts) to deal with erosion (inlet/outlet treatment, headwalls, cut-off walls) are considered to be appropriate. The backfill should comprise OPSS Granular A or Granular B Type II. The cut-off walls should extend at least 600 mm into the silt till to prevent flow below the culvert that could erode the bedding material and extend laterally to protect the granular material. The requirements of the CHBDC Clause 1.10.5.6 and 1.10.11.6.5 should be applied.

Inlet and outlet protection in accordance with OPSS 511 and 1004 is recommended to prevent erosion adjacent to the culvert as well as scour that could undermine the culvert and/or embankment foundation. The actual design requirements (length and width of the aprons at the inlet/outlet of the culvert as well as the rock size, apron thickness and height of erosion protection on the embankment slope) will be dictated by stream hydraulics, stream configuration as well as the water level in the creek and should be established by a hydraulic engineer. A non woven, Class I or II geotextile with an FOS as specified in the contract document, according to OPSS 1860, should be placed below the rip-rap to minimize the potential for erosion of fine particles from below the treatment.

All newly constructed embankment slopes and retained soils should be topsoiled and seeded (as per OPSS 570 and 572) as soon after grading as possible to prevent erosion. Where slopes are inclined at 2.5H:1V or steeper, the permanent slopes should be protected with erosion control blankets. Also, sod (as per OPSS 571) shall be placed where it currently exists for aesthetic reasons. Additional appropriate erosion control measures for the project should be assessed by MRC using the following erodibility K factor:

SOIL TYPE	K FACTOR
Sand, some gravel	0.15

7. CLOSURE

The report was prepared by Mr. Phil Cullen, P.Eng., Project Engineer, and Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer. It was reviewed by Mr. Dennis Kerr, P.Eng., Chief Foundation Engineer. Mr. Brian Gray, P.Eng., MTO Designated Contact, carried out an independent review of the report.

Sincerely

Peto MacCallum Ltd.

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PC:sd/lad-mi

TABLE 1

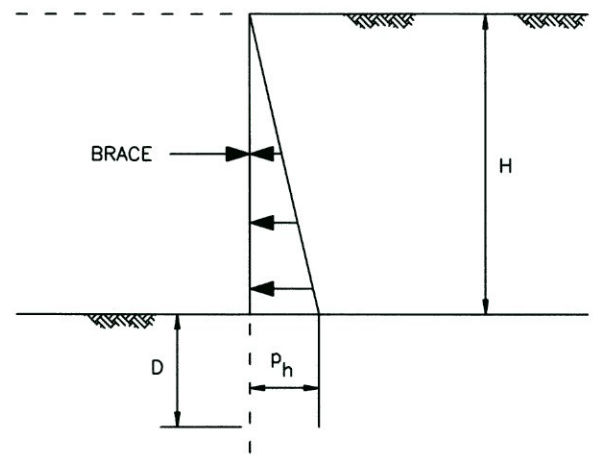
LIST OF MTO DOCUMENTS USED IN REPORT

NO.	TITLE	DATE
OPSD 803.010	Backfill and Cover for Concrete Culverts	November 1999
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets	November 2001
OPSD 810.020	Rip-Rap Treatment for Ditch Inlets	November 2001
OPSD 3121.150	Minimum Granular Backfill Requirements - Retaining Walls	November 2005
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut	April 2004
OPSS 501	Construction Specification for Compacting	November 2005
OPSS 511	Construction Specification for Rip-Rap Rock Protection and Granular Sheeting	November 2004
OPSS 570	Construction Specification for Topsoil	August 1990
OPSS 571	Construction Specification for Sodding	November 2001
OPSS 572	Construction Specification for Seed and Cover	November 2003
OPSS 1004	Material Specification for Aggregates – Miscellaneous	November 2005
OPSS 1860	Material Specification for Geotextiles	November 2004
SP 105S10	Soils Compaction – Quality Assurance and Quality Control	November 2004
SP 902S01	Earth and Rock Excavation for Structure	September 2003

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established. If groundwater table is well above base of excavation and/or artesian conditions exist, local lowering of the groundwater level will be necessary to prevent bottom heave/piping of the base of the excavation.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and apppropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, the earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM



p_h = design lateral earth pressure
= $K\gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

Refer to text of report for details

Peto MacCallum Ltd. CONSULTING ENGINEERS	LATERAL EARTH PRESSURE DISTRIBUTION SINGLY–BRACED CUTS IN COHESIONLESS SOILS	DATE MAY 2005	JOB NUMBER 04KF136A	FIGURE NO. 1

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.

2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established. If groundwater table is well above base of excavation and/or artesian conditions exist, local lowering of the groundwater level will be necessary to prevent bottom heave/piping of the base of the excavation.

3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).

4. Structural components of bracing system should be confirmed adequate for each level of excavation.

5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.

6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.

7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.

8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and apppropriate monitoring (surface and insitu) carried out during construction.

9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, earth pressure diagram must be reviewed, and remedial works may be required.

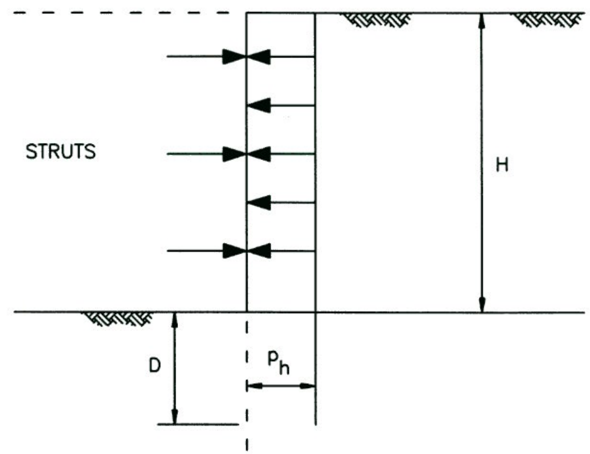
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.

11. Bracing system should be regularly examined for signs of distress.

12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.

13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM



p_h = design lateral earth pressure
= $0.65\ K\gamma H$

K = lateral earth pressure coefficient

γ = unit weight of soil

H = depth of excavation

D = depth of embedment of soldier piles (if used).

RECOMMENDED DESIGN PARAMETERS

Refer to text of report for details

Peto MacCallum Ltd. CONSULTING ENGINEERS	LATERAL EARTH PRESSURE DISTRIBUTION MULTI–BRACED CUTS IN COHESIONLESS SOILS	DATE	JOB NUMBER	FIGURE NO.
		MAY 2005	04KF136A	2

NOTES

1. The need to underpin existing footings/utilities is dependent upon soil type, proximity of the existing facility to the face of the excavation, loads imposed on the foundation and permissible movements.

   ZONE A:
   Foundations of relatively heavy and/or settlement sensitive structures/ utilities located in Zone A generally require underpinning.

   ZONE B:
   Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

   ZONE C:
   Utilities and foundations located within Zone C do not normally require underpinning.

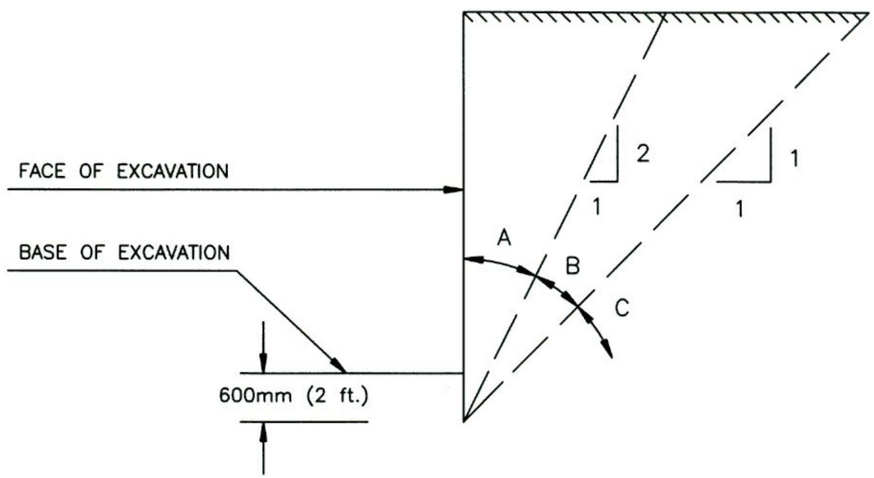
   Underpinning of foundations located in Zones A and B should extend at least into Zone C.

2. As an alternative to underpinning, it may be possible to control movement of existing utilities and foundations by supporting the face of the excavation with bracing/tiebacks or a rigid (caisson) wall. Horizontal and vertical earth pressures imposed on the excavation wall by non-underpinned foundations must be considered in the design of the support system.

3. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction to monitor any movement which may occur.

4. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.

5. This sheet is to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.



PML Peto MacCallum Ltd. CONSULTING ENGINEERS	STANDARD DRAWING GENERAL RECOMMENDATIONS REGARDING UNDERPINNING OF FOUNDATIONS/UTILITIES LOCATED CLOSE TO EXCAVATION	DATE MAY 2005	JOB NUMBER 04KF136A	FIGURE NO. 3