



**FOUNDATION INVESTIGATION AND DESIGN REPORT  
BRANT DRAIN CULVERT REPLACEMENT  
GWP 154-91-00, SITE 35-451  
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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PML Ref.: 04KF136B  
Geocres No.: 40P16-18  
Index No.: 079FIR and 080FDR  
February 16, 2006





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### Site Photographs

Figure 1 - Grain Size Distribution Chart - Sand, Some Gravel, Trace of Silt (Fill)

Figure 2 - Grain Size Distribution Chart - Silt Till, With Sand

Figure 3 - Plasticity Chart - Silt Till

Explanation of Terms Used in Report

Record of Boreholes 201 to 203

Drawing 1 - Borehole Locations and Soil Strata



## **FOUNDATION INVESTIGATION REPORT**

for  
Brant Drain Culvert Replacement  
GWP 154-91-00, Site 35-451  
Rehabilitation of Highway 6 from North Limits  
of Fergus, Northerly 17.3 km to the  
Conestoga River Bridge in Arthur  
Owen Sound, Ontario

---

### **1. INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for the proposed replacement of the Brant Drain culvert (Station 30+070, Site 35-451) located on Highway 6 between Fergus and Arthur in the Township of Nichol, Ontario. The site is located approximately 7.0 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 non rigid frame open footing culvert with a span of 4.2 m, height of 1.2 m and length of 27.3 m (ref.: RFP, Section 6.3.1, page 28). The invert of the existing culvert measured during this investigation was near elevation 446.5. The existing height of fill above the culvert is approximately 2.0 m (measured during field investigation).

### **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 Brant Drain. 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 January 11, 2005 and comprised three boreholes advanced to depths of 6.2 to 7.6 m below existing grade, about 5 to 7 m below the invert of 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 a track mounted CME 55 drillrig, supplied and operated by a specialist drilling contractor, working under the full time supervision of a member 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 classified 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 and Atterberg Limits tests were carried out on selected samples. The Grain Size Distribution and





Atterberg Limits test results are presented on Figures 1 to 3, and on the Record of Borehole sheets.

## **5. SUMMARIZED SUBSURFACE CONDITONS**

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 borehole drilled through the highway embankment consisted of a cohesionless sand with some gravel to a depth of 3.5 m underlain by a 0.5 m thick layer of topsoil. Topsoil, 0.5 and 0.8 m thick, was encountered surficially in the boreholes drilled near the ends of the existing culvert.

Compact to very dense silt till was encountered beneath the embankment fill and/or topsoil in all three boreholes at depths of 0.5 to 4.0 m. This deposit was not penetrated at the termination of drilling.

Ground water was measured at depths of 4.8 and 5.4 m (elevations 441.9 and 441.2) in Boreholes 203 and 201 respectively at the completion of drilling (boreholes located near the ends of the culvert). Water was observed in Borehole 202 at a depth of 3.0 m (elevation 446.1) during drilling.

### **5.1 Fill**

The fill material encountered in Borehole 202 drilled through the road embankment comprised cohesionless sand with some gravel. The fill was loose (based on Standard Penetration Test N values of 6 and 7). The moisture content of the fill was 8%. The fill was penetrated at a depth of 3.5 m (elevation 445.6).





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

## **5.2 Topsoil**

A deposit of topsoil was encountered surficially in boreholes 201 and 203 and beneath the embankment fill in Borehole 202. The moisture content of a sample of the topsoil beneath the embankment fill was 17%. The topsoil thickness ranged from 0.5 to 0.8 m and was penetrated in Borehole 202 at a depth of 4.0 m (elevation 445.1).

## **5.3 Silt Till**

Non to slightly cohesive silt till was contacted beneath the embankment fill and/or topsoil in all three boreholes. The silt till was compact to very dense (N values of 16 to 50 blows for 25 mm penetration). The moisture content of the silt till typically ranged from 5 to 17%. 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 18 to 20 and 12 to 13 respectively, indicating the material has low plasticity. 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 6.2 to 7.6 m (elevation 439.1 to 441.6).

## **5.4 Ground Water**

Water was observed after drilling in Boreholes 203 and 201 at depths of 4.8 and 5.4 m (elevations 441.9 and 441.2) respectively. Water was observed in Borehole 202 at a depth of 3.0 m (elevation 446.1) during drilling. At the completion of drilling, the borehole caved to a depth of 3.0 m (elevation 446.1). The water level in the watercourse at the time of the field investigation was near elevation 447.0.





Cognizant of the composition of the subgrade soil (silt with sand seams), 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 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.

Grigory O. Degil, PhD, P.Eng.  
Senior Foundation Engineer



Dennis W. Kerr, MEng, P.Eng.  
Chief Foundation Engineer



Brian R. Gray, MEng, P.Eng.  
MTO Designated Contact



PC:sd/lad-mi





Photograph 1: Brant Drain Culvert, East Side, Viewing North.



Photograph 2: Brant Drain Culvert, West Side, Viewing North.





Photograph 3: View south of existing culvert, east side of road.



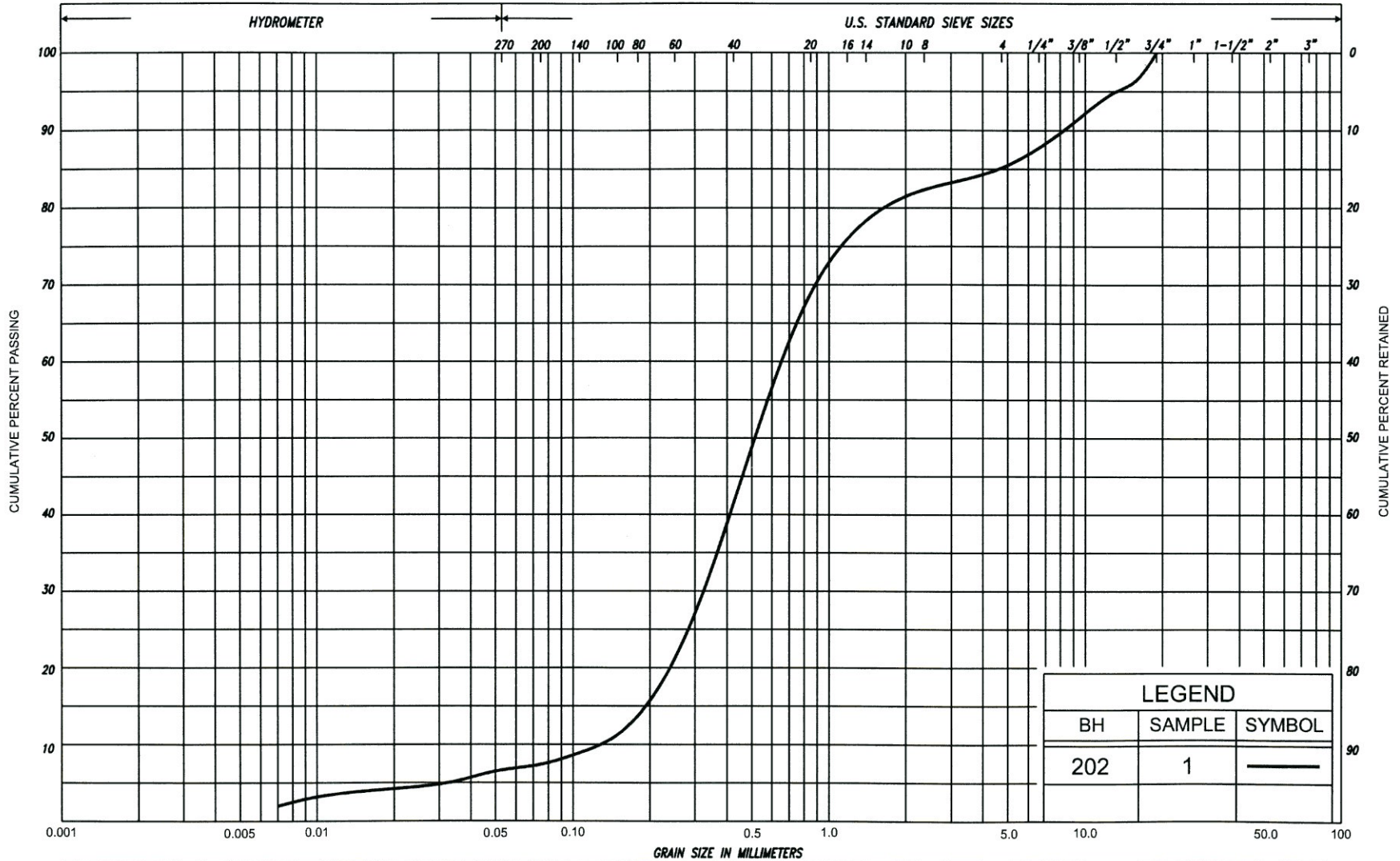
Photograph 4: View north of outlet on west side of road (opposite fence posts).





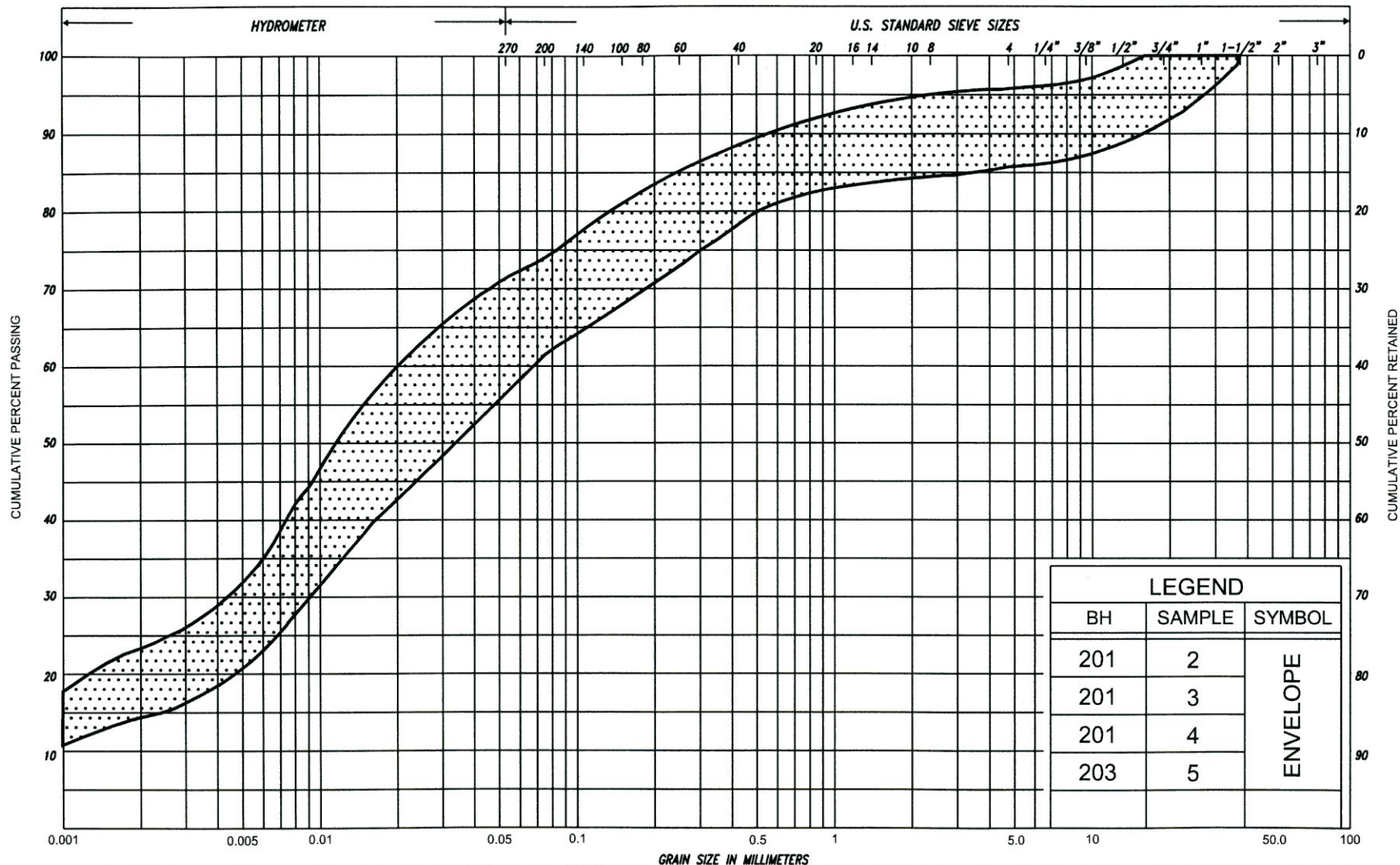
Photograph 5: View east along creek channel from culvert inlet on east side of road.





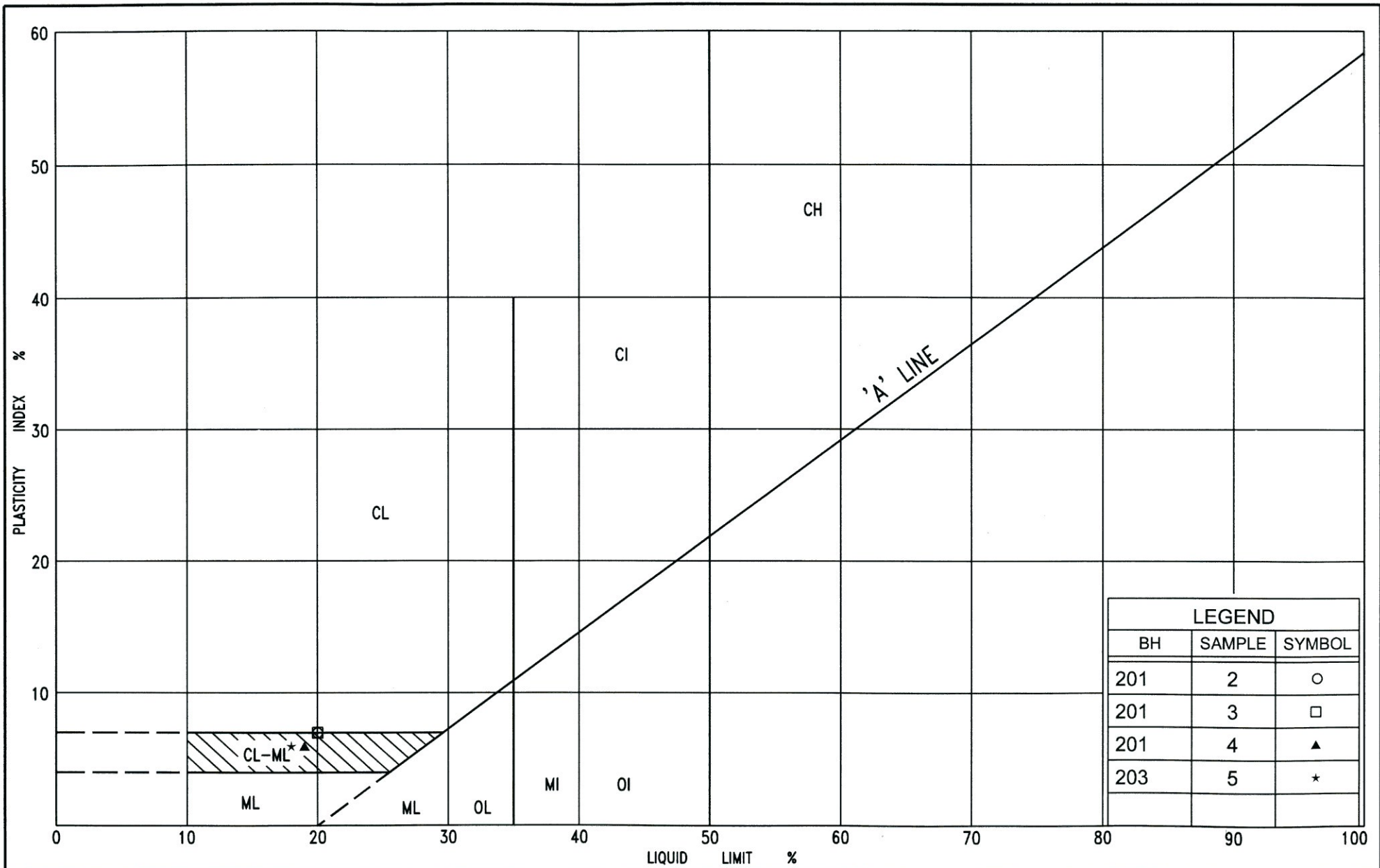
SILT & CLAY				FINE		MEDIUM		COARSE		GRAVEL				COBBLES	UNIFIED			
				SAND														
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.	
	SILT																	
CLAY		SILT				V. FINE	FINE		MED.	COARSE		GRAVEL						U.S. BUREAU
						SAND												





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







## 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.

**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 (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE
FV	FIELD VANE		

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
$E$	kPa	MODULUS OF LINEAR DEFORMATION
$G$	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
$H$	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
$U$	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_f$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$n$	1, %	POROSITY	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE
$\gamma_s$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$w$	1, %	WATER CONTENT	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$S_r$	%	DEGREE OF SATURATION	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\gamma_w$	kn/m <sup>3</sup>	UNIT WEIGHT OF WATER	$w_L$	%	LIQUID LIMIT	$D$	mm	GRAIN DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_p$	%	PLASTIC LIMIT	$D_n$	mm	n PERCENT - DIAMETER
$\gamma$	kn/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_s$	%	SHRINKAGE LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\gamma_d$	kn/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	$q$	m <sup>3</sup> /s	RATE OF DISCHARGE
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	$v$	m/s	DISCHARGE VELOCITY
$\gamma_{sat}$	kn/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL				$i$	1	HYDRAULIC GRADIENT
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL			DTPL	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\gamma'$	kn/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL			APL	$j$	kn/m <sup>3</sup>	SEEPAGE FORCE
$e$	1, %	VOID RATIO			WTPL			



## METRIC

20  
15 — 5 (%) STRAIN AT FAILURE  
10



RECORD OF BOREHOLE No 202

1 of 1

METRIC

W.P. 154-91-00 LOCATION Hwy 6, Brant Drain Station 30+072, 5.5 m Rt. ORIGINATED BY RM  
DIST 33 HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY PC  
DATUM Geodetic DATE January 11, 2005 CHECKED BY DK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
449.1																	
0.0	Sand, some gravel, trace of silt Loose Brown Moist (FILL)		1	SS	6		449										
							448										
							447										15 78 7 0
							446										
445.6			2	SS	7		445										
3.5	Topsoil						444										
445.1			3	SS	33		443										
4.0	Silt, with sand, some clay, trace gravel Dense to very dense Brown Moist (TILL)		4	SS	41		442										
			5	SS	91/ 14cm												
			6	AS													
441.6																	
7.5	End of Borehole																
	▽ Water level observed during drilling																



RECORD OF BOREHOLE No 203

1 of 1

METRIC

W.P. 154-91-00 LOCATION Hwy 6, Brant Drain Station 30+075, 15.5 m Lt. ORIGINATED BY RM  
DIST 33 HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY PC  
DATUM Geodetic DATE January 11, 2005 CHECKED BY DK

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
446.7								20	40	60	80	100					
0.0	Topsoil																
446.2																	
0.5	Silt, with sand, some clay, trace gravel with occ. thin sandy layers  Compact Brown Moist to very dense  (TILL)																
			1	SS	19												
			2	SS	60												
			3	SS	50/ 14cm												
			4	SS	50/ 14cm												
			5	SS	50/ 12cm												
			6	AS	50/ 3cm												
			7	AS	50/ 8cm												
439.1	End of Borehole																
7.6	▼ Water level measured after drilling																



**METRIC**

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

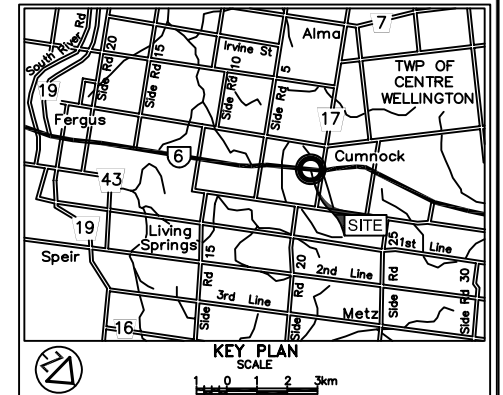
CONT No  
GWP No 154-91-00

HIGHWAY 6  
BRANT DRAIN CULVERT  
STA. 30+070  
BOREHOLE LOCATIONS & SOIL STRATA



SHEET

**PML Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



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 Jan 2005
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
201	446.6	4 846 959	227 867
202	449.1	4 846 957	227 854
203	446.7	4 846 944	227 837

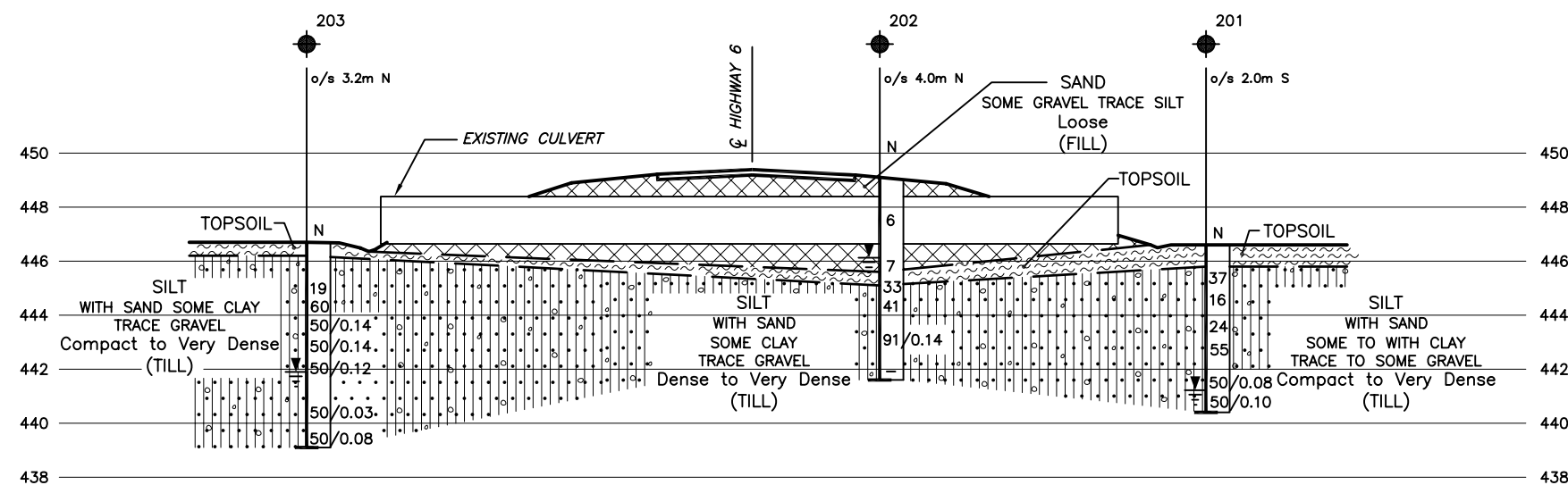
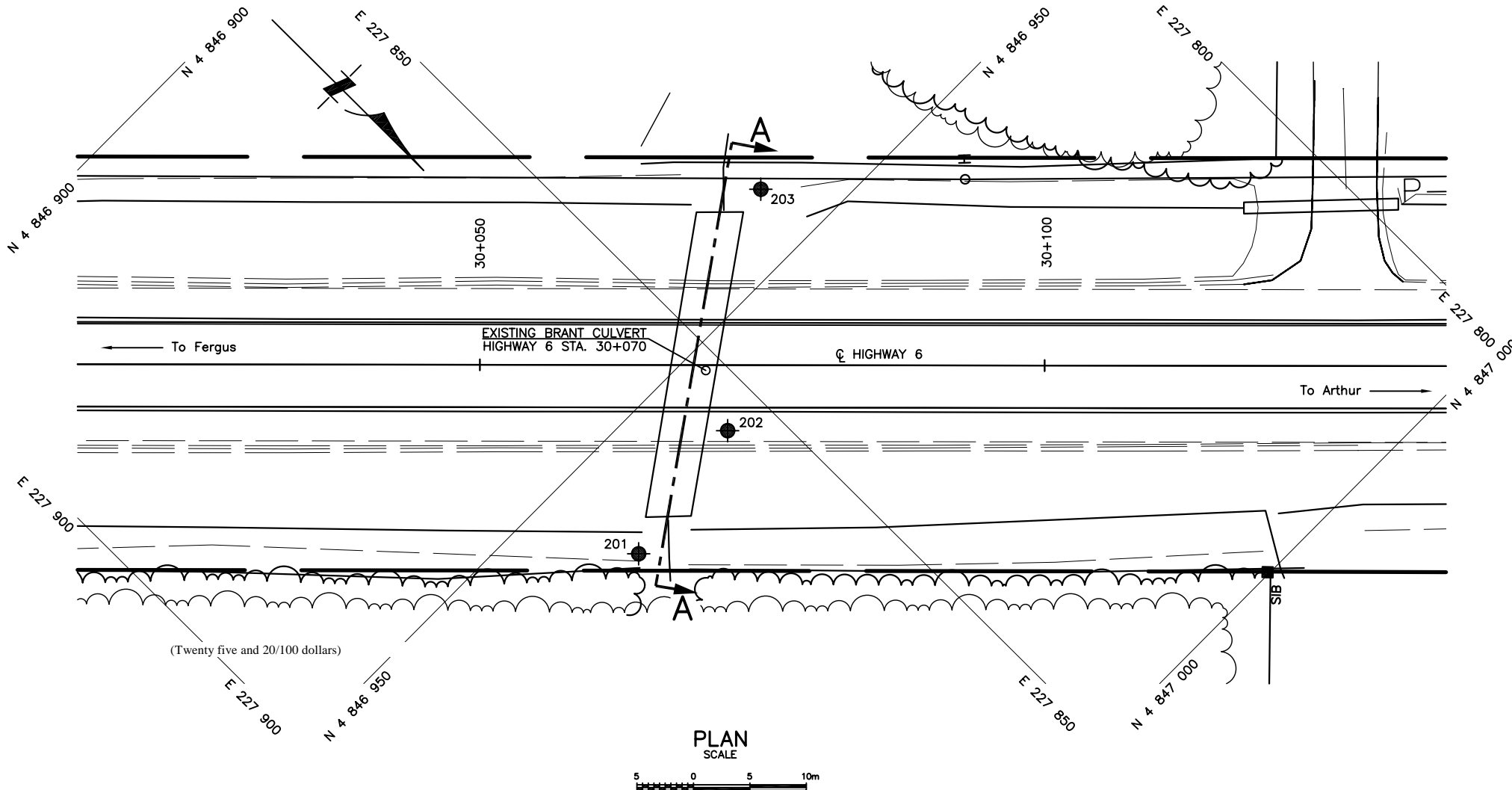
NOTE

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. 40P16-18

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



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.



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





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BRANT DRAIN CULVERT REPLACEMENT  
GWP 154-91-00, SITE 35-451  
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OF FERGUS, NORTHERLY 17.3 KM TO THE  
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**for**

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Table 1 - List of MTO Documents Used in Report

Figure 1 - Lateral Earth Pressure Distribution  
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Figure 2 - Lateral Earth Pressure Distribution  
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Figure 3 - General Recommendations Regarding Underpinning of Foundation/Utilities Located  
Close to Excavation



**FOUNDATION DESIGN REPORT**  
for  
Brant Drain Culvert Replacement  
GWP 154-91-00, Site 35-451  
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 existing Brant Drain culvert, (Site 35-451; Station 30+070) located on Highway 6 between the Towns of Fergus and Arthur in the Township of Nichol, Ontario. The site is located approximately 7.0 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 non rigid frame open footing culvert with a span of 4.2 m, height of 1.2 m and length of 27.3 m (ref.: RFP, Section 6.3.1, page 28).

Measurements conducted during the field investigation indicate the invert level of the existing culvert is near elevation 446.5, and the existing height of fill above the culvert is approximately 2.0 m. The total embankment height is 3.0 to 3.5 m.

The current design calls for the replacement culvert to be located on the same alignment as the existing culvert with the invert at/near the invert of the existing culvert.

The field investigation revealed that the highway embankment consists of sand fill and it was constructed on topsoil. Topsoil was encountered surficially in the boreholes drilled near the ends of the proposed culvert. The topsoil is underlain by compact to very dense silt till.





Ground water was measured at depths of 4.8 and 5.4 m (elevations 441.9 and 441.2) in the boreholes drilled near the ends of the proposed culvert and 3.0 m (elevation 446.1) in the borehole drilled through the embankment.

A list of MTO documents referred to in subsequent sections of this report are given 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 446.5). The design subgrade level of the foundation for a new open footing culvert is interpreted to be near or below elevation 444.8 and 446.0 if a box culvert is constructed

The subgrade material near elevation 446.0 revealed in the boreholes drilled at the ends of the culvert consisted of compact to dense silt till. Sand fill underlain by topsoil was identified to elevation 445.1 in the borehole drilled through the roadway embankment. The subgrade material below elevation 444.8 comprised dense to very dense silt till.

The embankment height at this location is about 3.0 to 3.5 m.

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





Open footing (0.5 m wide) and box (4 m wide) culvert foundations constructed at the elevations noted previously should be designed using the following geotechnical resistance at ultimate and serviceability limit states (ULS and SLS):

Factored Geotechnical Resistance at ULS	600 kPa
Geotechnical Resistance at SLS	225 kPa

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. A lower bound value for the recommended geotechnical resistance takes into account the variation of the subgrade material (compact silt till at each end, dense silt till in the middle).

The topsoil revealed in the boreholes during the field investigation noted previously as well as any soft/wet soils revealed below the subgrade during construction should be excavated prior to construction of the culvert foundation and replaced with engineered fill. Additional comments in this regard are provided in the paragraph that deals with site review during construction (SP 902S01).

The engineered fill should comprise granular material (Granular B Type I or 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 topsoil and/or fill extended to elevation 445.1 in Borehole 202 and the relative density of the underlying silt ranged from compact to very dense. 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 culvert, must be designed to support the stress imposed by the overlying fill as well as to resist the





unbalanced lateral earth pressure and compaction pressure imposed by the backfill adjacent to the culvert walls. The lateral earth and water pressure acting on the culvert and retaining walls,  $p$  (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

$\gamma$  = unit weight of free draining granular material above the design water level (kN/m<sup>3</sup>)

$\gamma'$  = unit weight of submerged free draining granular material below the design water level (kN/m<sup>3</sup>)

$\gamma_w$  = unit weight of water  
= 9.8 kN/m<sup>3</sup>

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

$h_2$  = depth below design water level (m)

$q$  = any surcharge load (kN/m<sup>2</sup>)

$C_p$  = 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  $\Phi$  = angle of internal friction of retained soil (35° for Granular B Type II)

$\delta$  = 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 <sup>3</sup> )	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 watercourse. Since the subgrade soil consists of silt with





sand seams, 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 ends 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  $\mu\text{m}$  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 near elevation 442 in holes drilled at the ends of the culvert and elevation 446 in the borehole drilled through the embankment. It is considered, therefore, that conventional sump pumping techniques should in general be suitable to control ground water seepage that enters the excavation. It must be noted, however, that the water level is subject to seasonal variations and weather conditions, particularly following heavy rainfalls when flood conditions may occur during construction of the culvert.

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. 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 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 <sup>3</sup>	20.0
Coefficient of Active Earth Pressure K <sub>a</sub>	0.33
Coefficient of Earth Pressure at Rest K <sub>o</sub>	0.50
Coefficient of Passive Earth Pressure, K <sub>p</sub>	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)

$$\begin{aligned} \sigma'_z &= \text{effective vertical stress at midpoint of load carrying length (kN/m}^2\text{)} \\ &= \gamma h_1 \text{ if total anchorage length is above the design ground water level} \\ &= \gamma h_1' + \gamma' h_2 \text{ if design ground water level is above the anchor} \end{aligned}$$

$$\begin{aligned} \gamma &= \text{bulk unit weight of soil above design ground water level} \\ &= 20 \text{ (kN/m}^3\text{)} \end{aligned}$$

$$\begin{aligned} \gamma' &= \text{buoyant unit weight of soil below design ground water level} \\ &= 10.2 \text{ (kN/m}^3\text{)} \end{aligned}$$

$$h_1 = \text{depth below ground surface to midpoint of anchor (m)}$$

$$h_1' = \text{depth below ground surface to design ground water level (m)}$$

$$h_2 = \text{depth below design ground water level to midpoint of anchor (m)}$$

$$A_s = \text{circumference of fixed length of anchor (m)}$$

$$L_s = \text{effective embedment length of the anchor (m)}$$

$$\begin{aligned} K_f &= \text{anchorage coefficient} \\ &= 0.8 \text{ for sand and dense silt fill} \end{aligned}$$





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:

	<b><u>MOVEMENT (% OF EXCAVATION DEPTH)</u></b>
<b>Lateral Movement</b>	
Braced Excavation	0.2
Anchored Wall	0.1
<b>Vertical Movement</b>	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 4 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:

<u>SOIL TYPE</u>	<u>K FACTOR</u>
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

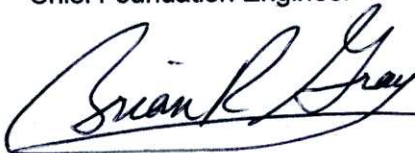
Peto MacCallum Ltd.



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PC:sd/lad-mi





**TABLE 1**  
**LIST OF MTO DOCUMENTS USED IN REPORT**

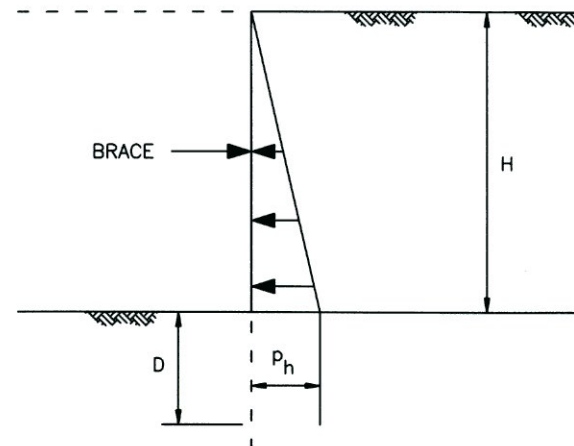
<b>NO.</b>	<b>TITLE</b>	<b>DATE</b>
OPSD 803.010	Backfill and Cover for Concrete Culverts	November 1999
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets	November 2001
OPSD 810.020	Rip-Rap Treatment for Ditch Inlets	November 2001
OPSD 3121.150	Minimum Granular Backfill Requirements - Retaining 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 appropriate 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



$$p_h = \text{design lateral earth pressure} \\ = K\gamma H$$

$K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of soil

$H$  = depth of excavation

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

## RECOMMENDED DESIGN PARAMETERS

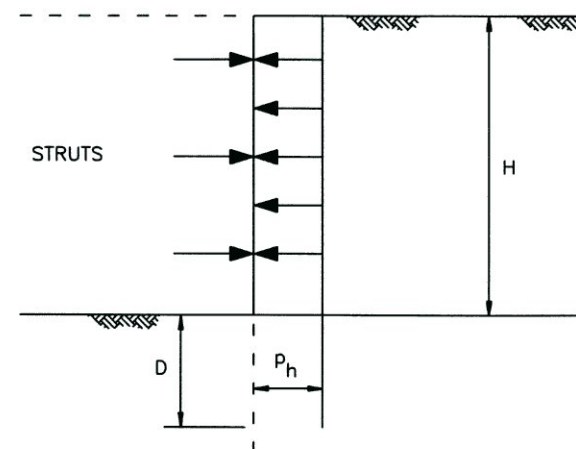
Refer to text of report for details



## 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.
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## EARTH PRESSURE DIAGRAM



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

$K$  = lateral earth pressure coefficient

$\gamma$  = 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



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.

