



**SUPPLEMENTAL FOUNDATION INVESTIGATION AND DESIGN  
REPORT**

**for**

**CULVERT REPLACEMENT STA. 20+510  
CULVERT REHABILITATION STA. 22+350  
REHABILITATION OF HIGHWAY 8, GODERICH TO CLINTON  
WP 189-89-00  
TOWNSHIP OF GODERICH, ONTARIO**

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PML Ref.: 05KF129A-1  
Index No.: 129FIR and 130FDR  
GEOCRES No.: 40P12-12  
March 24, 2009



**SUPPLEMENTAL FOUNDATION INVESTIGATION REPORT**

**for**

**CULVERT REPLACEMENT STA. 20+510**

**CULVERT REHABILITATION STA. 22+350**

**REHABILITATION OF HIGHWAY 8, GODERICH TO CLINTON**

**WP 189-89-00**

**TOWNSHIP OF GODERICH, ONTARIO**

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**SUPPLEMENTAL FOUNDATION INVESTIGATION REPORT**

for

Culverts at Sta. 20+510 and 22+350  
Rehabilitation of Highway 8, Goderich to Clinton  
WP 189-89-00  
Township of Goderich, Ontario

---

**1. INTRODUCTION**

Planned under this project is the rehabilitation of Highway 8 from the Goderich east limits easterly for 16.8 km to the Clinton west limits in the Township of Goderich, Ontario. Part of this project involves the replacement or rehabilitation of two existing culverts located about 2.7 and 4.5 km northwest of the Hamlet of Holmesville. This report is supplemental to the previous report issued for the replacement of the Ginn's Creek Culvert on November 3, 2006 Geocres No.: 40P12-12. This report was prepared for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario.

The replacement of the culvert located at Sta. 20+510 and the rehabilitation to the inlet end of the existing culvert at Sta. 22+350 is proposed. The culvert at Sta. 20+510 is part of the tributary system to Ginn's Creek and is located about 130 m north of the Ginn's Creek Culvert.

This report provides a summary of the factual information obtained during the field investigation conducted the culverts located at Sta. 20+510 and 22+350.

**2. SITE DESCRIPTION AND GEOLOGY**

Highway 8 within the project limits is primarily situated in a rural setting with rolling terrain containing streams and swampy streaks in depressions. Land use along the study corridor is mainly agricultural with some forested/swamp areas and local residential development.

The project area lies in the physiographic region known as the Huron Slope characterised by an undulating till plain, with the till often coming to the surface and resting on stratified clay. The principal surficial soil along the study corridor is clayey silt till interbedded with silt, sand and gravel (L.J.Chapman & D.F.Putnam, *The Physiography of Southern Ontario*, 3<sup>rd</sup> Edition, Ontario Research Foundation, 1984).



The bedrock underlying the site is at an approximate depth of 60 m and comprises limestone of the Dundee Formation, based on the Ontario Geological Survey map of Bedrock Resources, ARIM 177-2C for Huron County.

The frost penetration depth for design purposes as outlined in the Pavement Design and Rehabilitation Manual is 1.3 m.

### **3. INVESTIGATION PROCEDURES**

The field work for this study was carried out in the period of January 21 to 26, 2009 and comprised three boreholes advanced to depths of 3.4 to 6.7 m at the locations shown on Drawings C1 and C2.

The borehole locations and ground surface elevations at the boreholes were established by Peto MacCallum Ltd. (PML). All elevations in this report are expressed in metres.

The boreholes were advanced with a track-mounted drill rig and the use of manual hand equipment where site conditions and access restrictions required. The equipment was supplied and operated by a specialist drilling contractor working under the full-time supervision of a member of our engineering staff. PML advanced the boreholes where manual sampling equipment was required.

Representative samples of the soil 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. Penetrometer testing was also performed to further assess the shear strength of the cohesive soils encountered. The penetrometer test results typically give lower shear strengths than the actual values due to sample disturbance. The manual sampler was driven with a drop hammer delivering 150 J (110 ft-lb) of energy. The values shown on the borehole logs have been converted to equivalent standard penetration resistance N values which is based on a drop hammer delivering 475 J (350 ft-lb) of energy.



Soils were identified visually in the field in accordance with the MTO Soil Classification procedures. The groundwater 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, when appropriate, by measurement of the water level in the open boreholes. All the boreholes were backfilled with a bentonite/cement mixture in accordance with the MTO guidelines for borehole abandonment procedures.

The recovered samples were returned to our laboratory for detailed visual examination and classification. The laboratory testing program consisting of 17 moisture content determinations as well as 3 Atterberg limits tests and 5 grain size distribution analyses was carried out on selected samples. Atterberg limits were not determined on samples deemed to be non-plastic by visual and tactile examination. The results of laboratory Atterberg limits testing and grain size distribution analyses are presented in Figures PC-1, GS-1 and GS-2.

#### **4. SUMMARISED SUBSURFACE CONDITIONS**

##### **4.1 General**

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 and pocket penetrometer test data, groundwater observations and moisture content determinations. The results of laboratory Atterberg limits testing and grain size distribution analyses conducted on selected samples are also shown on the Record of Borehole sheets.

The borehole locations are shown on Drawings C1 and C2. The boundaries between soil strata have been established only at the borehole locations. Between boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised surficial topsoil and embankment fill underlain by sand and gravel (Sta. 20+510) or a clayey silt till deposit (Sta. 22+350). Cobbles and boulders were encountered within the sand and gravel and glacial till deposits. Groundwater was measured during the field work in the boreholes at



Sta. 20+503 and 20+518 at levels ranging from 0.9 to 1.4 m depths, elevations 251.3 to 251.9 and at 1.4 m depth, elevation 278.3 within the borehole at Sta. 22+355. The strata encountered in these boreholes are summarised below.

## **4.2 Culvert at Sta. 20+510**

### **4.2.1 Fill**

Surficial topsoil fill was present in boreholes 201 and 202 put down near the inlet and outlet ends of the culvert. The topsoil fill had a thickness of 300 to 500 mm and extended to elevations of 253.0 and 251.7, respectively.

Underlying the surficial topsoil fill at 0.3 and 0.5 m depth, a 1.3 and 0.6 m thick clayey silt fill deposit was contacted in boreholes 201 and 202, respectively. The fill extended to depths of 1.6 and 1.1 m and was penetrated at elevations 251.7 and 251.1 on the underlying sand and gravel deposit and the buried topsoil layer in boreholes 201 and 202, respectively. The moisture content of the samples of fill ranged from 16 to 23%.

Standard penetration test N-values in the fill materials ranged 6 to 12 indicating firm to stiff conditions.

### **4.2.2 Topsoil**

A 400 mm thick buried topsoil layer was contacted in borehole 202 at 1.1 m depth (elevation 251.1). The buried topsoil was penetrated at 1.5 m depth (elevation 250.7) on the underlying sand and gravel. The moisture content of the buried topsoil sample was 36%.

### **4.2.3 Sand and Gravel**

A deposit of sand and gravel was contacted at 1.6 and 1.5 m depth (elevations 251.7 and 250.7) overlain by fill in borehole 201 and the buried topsoil in borehole 202. The sand and gravel deposit extended to elevations 248.8 and 249.0 which was 3.4 and 4.3 m depth of exploration at



the borehole locations. The sand and gravel deposit comprised trace to some silt, with gravelly sand layers and cobbles and boulders encountered within the deposits in boreholes 201 and 202, respectively.

Standard penetration test N-values in the sand and gravel typically ranged from 10 to 31 indicating generally compact conditions. Hydrostatic disturbances within sand and gravel deposit resulted in lower standard penetration test N-values of 6 and 8 at around 2.5 m depth in both borehole locations.

The results of three grain size distribution analysis performed on the sand and gravel deposits and a gravelly sand layer are presented on Figure GS-1. The moisture content ranged from 13 to 26%.

#### 4.2.4 Groundwater

Water was observed in both boreholes during the course of the field work. It was detected at depths of 0.9 and 1.2 m (elevations 251.3 and 252.1) in the process of sampling at the inlet and outlet ends of the existing culvert. Upon completion of sampling, groundwater was measured at depths of 0.9 and 1.4 m (elevations 251.3 and 251.9). The observed groundwater levels are subject to seasonal fluctuations and precipitation patterns.

### 4.3 Culvert at Sta. 22+350

#### 4.3.1 Topsoil/Fill

Surficial topsoil was contacted in borehole 101 put down near the inlet end of the culvert. The topsoil was 600 mm thick and extended to elevation 279.1. The topsoil was underlain by 0.5 m of fill which was penetrated at 1.1 m depth, elevation 278.6 and contacted the underlying clayey silt till deposit. The fill comprised clayey silt with some sand and traces of gravel. Organics were also noted within the fill. The moisture content of the fill was 25%.





#### 4.3.2 Clayey Silt Till

Overlain by topsoil and fill at 1.1 m depth, elevation 278.6 in borehole 101 was a cohesive clayey silt till deposit. The till deposit was firm to very stiff in consistency, with penetrometer test results indicating a range in shear strength of 50 to 200 kPa. Standard penetration test N-values ranged from 5 to 26. Cobbles and boulders were encountered within the clayey silt till during drilling. The deposit was not penetrated at the termination depth of 6.7 m, elevation 273.0.

The results of Atterberg limits testing and grain size distribution analyses conducted on two samples of this material are presented in respective Figures PC-1 and GS-2. The liquid and plastic limits of the clayey silt till ranged from 20 to 24 and from 13 to 14 respectively, thus giving the plasticity index of 7 to 10. The moisture content of the deposit varied between 14 and 19%.

#### 4.3.3 Groundwater

Water was observed in the borehole in the course of the field work. It was detected at 0.9 m depth, elevation 278.8 in the process of augering. Upon completion of drilling, groundwater was measured at 1.4 m depth, elevation 278.3.

The observed groundwater levels are subject to seasonal fluctuations and precipitation patterns.



## 5. MISCELLANEOUS

The field work was carried out under the supervision of Mr. G. Idzik and direction of Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. The soil drilling equipment was supplied by Direct Environmental Drilling Inc. The laboratory work was carried out in the PML laboratory in Toronto.

This report was prepared by Mr. C.M.P. Nascimento, P.Eng., with the assistance of Mr. M.J. Narduzzi, BEng., and was independently reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly

Peto MacCallum Ltd.

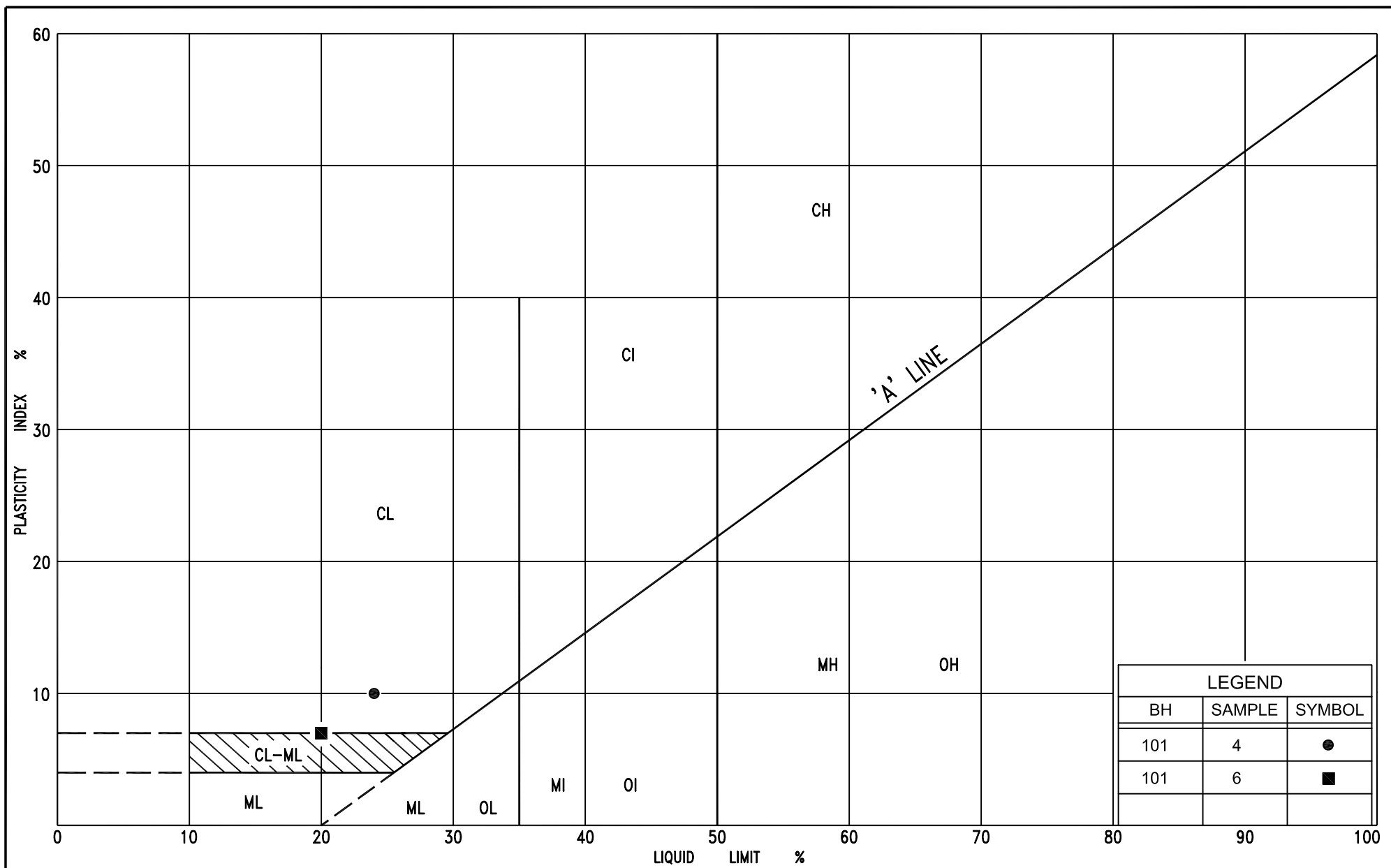


Carlos M. P. Nascimento, P.Eng.  
Senior Project Engineer



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

MN/CN:mi-nk



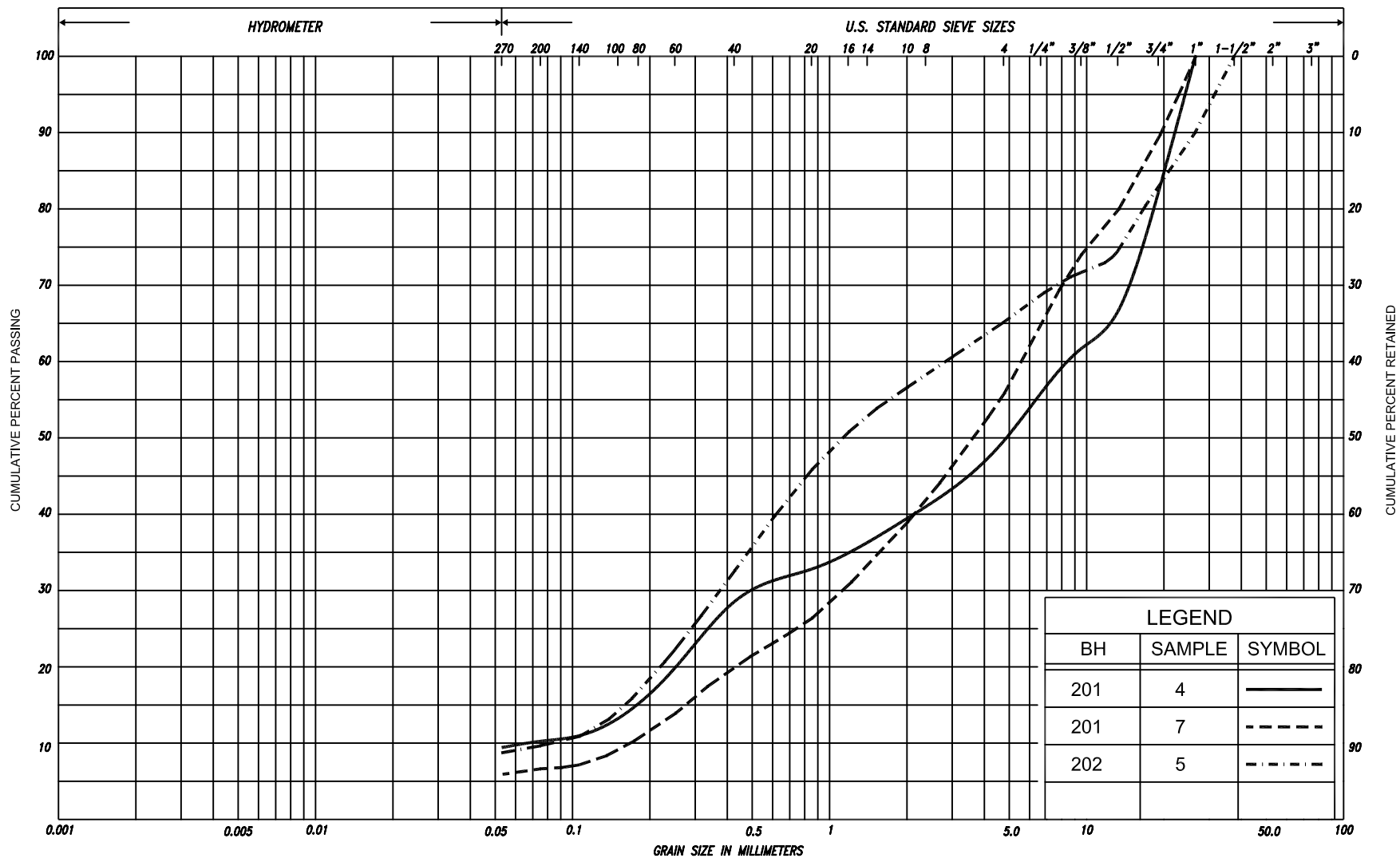
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PLASTICITY CHART  
CLAYEY SILT, with to some sand, trace gravel  
(TILL)

FIG No. PC-1

HWY: 8

W.P. No. 189-89-00



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



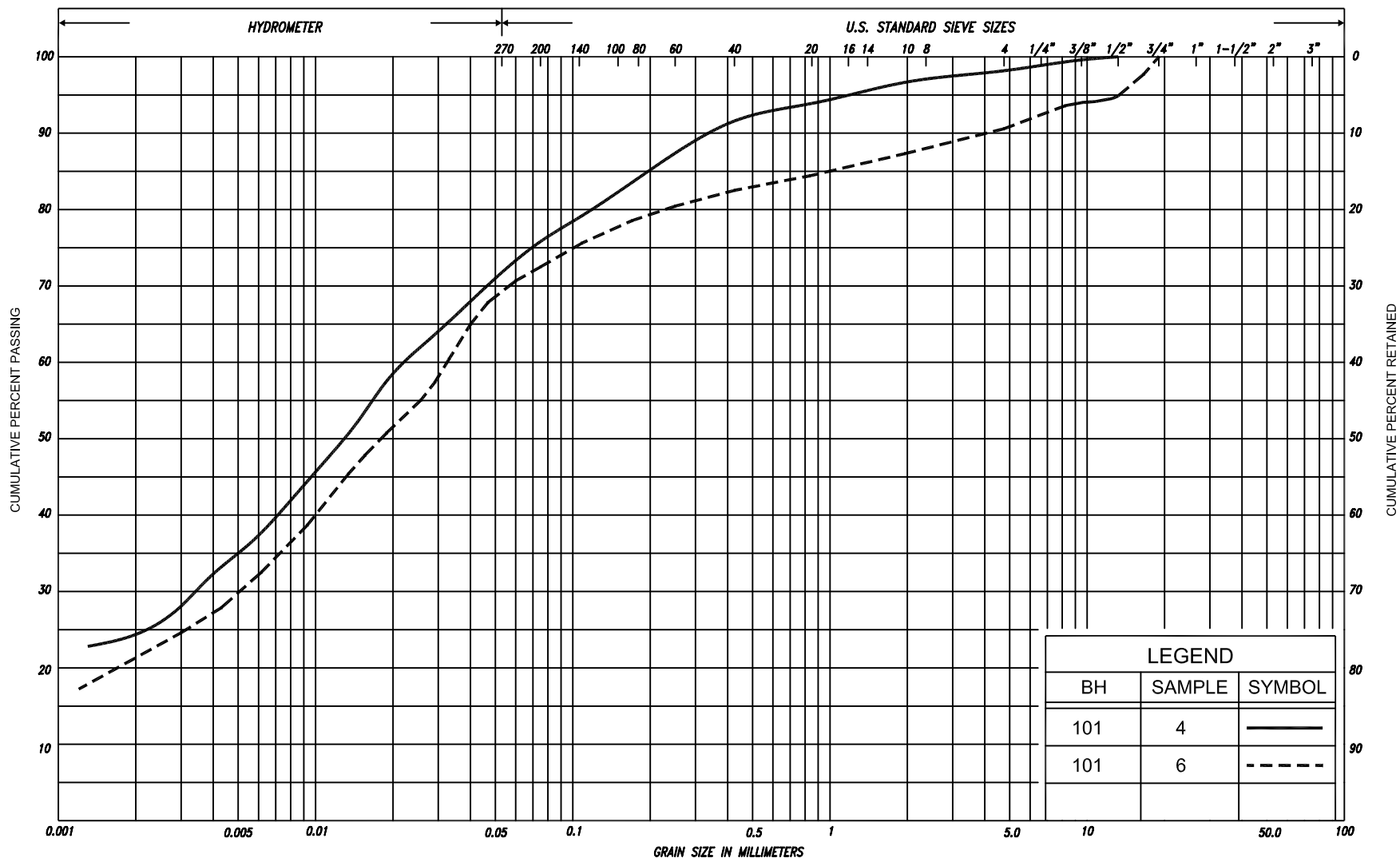
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# GRAIN SIZE DISTRIBUTION SAND AND GRAVEL / GRAVELLY SAND trace to some silt

FIG No. GS-1

HWY: 8

W.P. No. 189-89-00



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

# GRAIN SIZE DISTRIBUTION CLAYEY SILT, with to some sand, trace gravel (TILL)

FIG No. GS-2

HWY: 8

W.P. No. 189-89-00

## 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  $\bar{N}$ .

**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 (R Q D), FOR MODIFIED RECOVERY, IS:

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

**JOINTING AND BEDDING:**

SPACING	50mm	50 - 300mm	0.3m - 1m	1m - 3m	> 3m
JOINTING	VERY CLOSE	CLOSE	MOD. CLOSE	WIDE	VERY WIDE
BEDDING	VERY THIN	THIN	MEDIUM	THICK	VERY THICK

## ABBREVIATIONS AND SYMBOLS

### FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$u$	1	PORE PRESSURE RATIO
$\sigma$	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

$m_v$	$kPa^{-1}$	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	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
$\sigma'_{vo}$	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
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

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

**RECORD OF BOREHOLE No 101**

1 of 1

**METRIC**

G.W.P. 189-89-00 LOCATION Coords: 4 835 497.2 N; 376 154.7 E  
Hwy 8, Sta. 22+355, o/s 15.0m Lt. CL Med. ORIGINATED BY G.I.  
DIST 54 HWY 8 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY M.N.  
DATUM Geodetic DATE January 21, 2009 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)				
								○ UNCONFINED   + FIELD VANE		● QUICK TRIAXIAL   × LAB VANE					w <sub>p</sub> w   w <sub>L</sub>				
279.7 0.0	Ground Surface						20	40	60	80	100	20	40	60	GR	SA	SI	CL	
	Topsoil		1	SS	3								○						
279.1 0.6	Clayey silt with sand, trace gravel organics																		
278.6 1.1	Firm      Dark      Wet brown (FILL)		2	SS	6								○						
	Clayey silt with sand, trace gravel cobbles		3	SS	5								○						
	Firm to      Brown      Moist very stiff		4	SS	16						150		○			2	22	52	24
	thin fine sandy silt seams																		
			5	SS	21						175		○						
	some sand																		
	(TILL)		6	SS	18								○			9	18	52	21
			7	SS	22								○						
			8	SS	26														
273.0 6.7	End of borehole										200								

**RECORD OF BOREHOLE No 201**

1 of 1

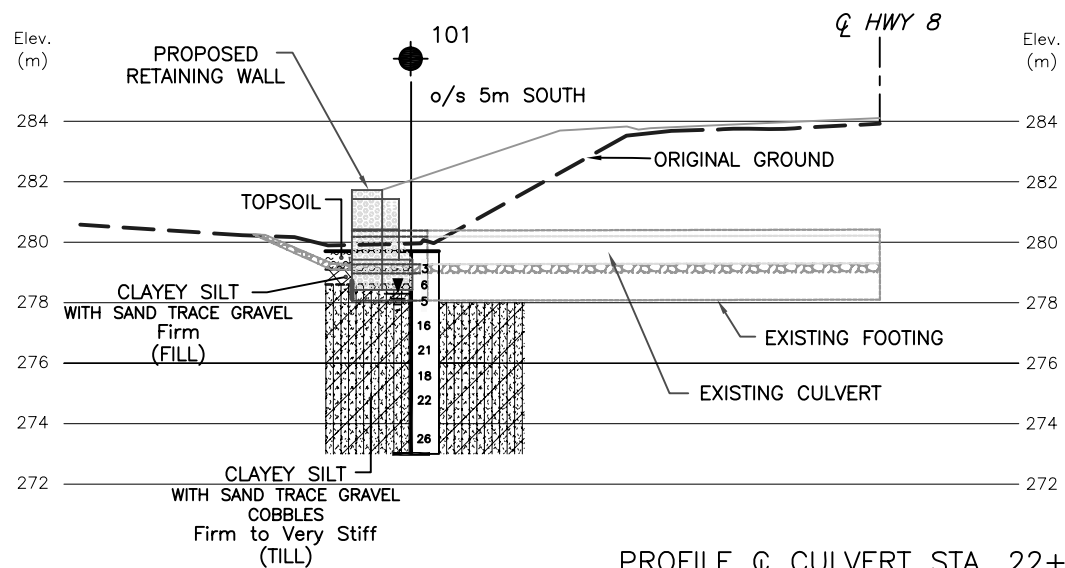
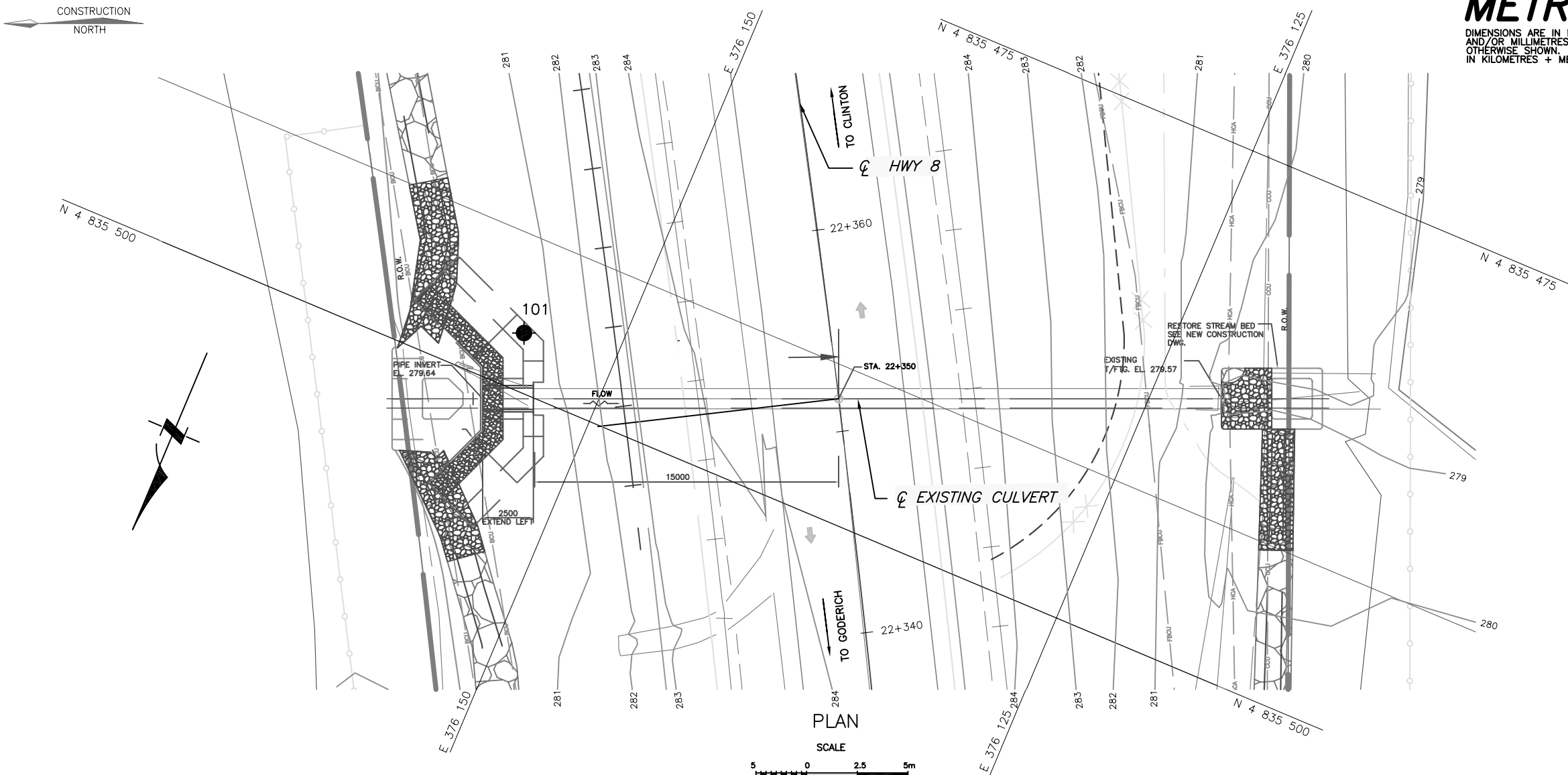
**METRIC**

G.W.P. 189-89-00 LOCATION Coords: 4 837 212.5 N; 375 453.4 E  
Hwy 8, Sta. 20+503, o/s 20.5 Rt. CL Med. ORIGINATED BY G.I.  
DIST 54 HWY 8 BOREHOLE TYPE Manual Sampling COMPILED BY M.N.  
DATUM Geodetic DATE January 26, 2009 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT  γ  kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					WATER CONTENT (%)					
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    x LAB VANE										
253.3	Ground Surface							20	40	60	80	100						
0.0	Topsoil		1	SS	8		253											
253.0	Clayey silt trace sand, trace gravel		2	SS	12		252											
0.3	Firm to Brown Moist stiff (FILL)		3	SS	10		251											
251.7	Sand and gravel trace to some silt		4	SS	19		250											
1.6	Compact Brown Wet		5	SS	8													
			6	SS	8													
			7	SS	17													
249.0	End of borehole						249											
4.3	"N" Values obtained with manual sampling equipment have been converted to standard penetration resistance "N" values  *    2009   01   26  ▽    Water level observed during drilling  ▼    Water level measured after drilling																	



RECORD OF BOREHOLE No 202										1 of 1		METRIC				
G.W.P. 189-89-00			LOCATION			Coords: 4 837 217.3 N; 375 498.6 E Hwy 8, Sta. 20+518, o/s 23.5 Lt. CL Med.			ORIGINATED BY G.I.							
DIST 54 HWY 8			BOREHOLE TYPE Manual Sampling						COMPILED BY M.N.							
DATUM Geodetic			DATE			January 26, 2009			CHECKED BY C.N.							
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 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE								
252.2 0.0	Ground Surface															
251.7 0.5	Topsoil		1	SS	6		252									
251.1 1.1	Clayey silt some sand, trace gravel		2	SS	10		251									
250.7 1.5	Firm Dark brown (FILL) Moist		3	SS	10		251									
	Topsoil															
	Sand and gravel, some silt															
	Compact to dense Brown Wet		4	SS	6		250									
	gravelly sand layers		5	SS	31											
	Cobbles and boulders		6	SS	31		249									35 55 (10)
248.8 3.4	End of borehole															
<p>Sample 6: Sampler bouncing</p> <p>"N" Values obtained with manual sampling equipment have been converted to standard penetration resistance "N" values</p> <p>* 2009 01 26</p> <p> Water level observed during drilling</p> <p> Water level measured after drilling</p>																



NOTES:

1. THE GABION WALLS WERE CHANGED TO CAST-IN-PLACE CONCRETE RETAINING WALLS AFTER THE FIELD INVESTIGATION.
2. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

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

CONT No 2008-3006

WP No 189-89-00

CULVERT STA. 22+350

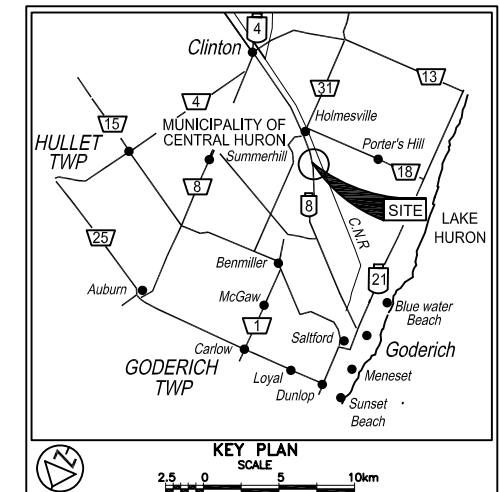
HIGHWAY 8 GODERICH TO CLINTON

BOREHOLE LOCATION AND SOIL STRATA



SHEET

**PMI Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



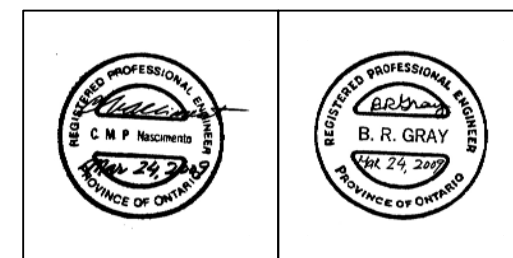
LEGEND

- 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 2009
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES	
		NORTHINGS	EASTINGS
101	279.7	4 835 497.2	376 154.7

NOTE

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



REF No.: TSH DRAWINGS S6357-300-098GA.dwg; h6357xa1.dwg;  
h6357xb1.dwg; h6357xn1.dwg; h6357xu1.dwg;  
h6357xy1.dwg Modified on January 03, 2008;  
h6357xb2.dwg Modified on July 17, 2008

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 40P12-12

HWY No	8	DIST	Owen Sound
SUBM'D	MN	CHECKED	MN
DRAWN	NA	CHECKED	CN
DATE	MAR. 24, 2009	APPROVED	BRG
SITE	--	DWG	C2





**SUPPLEMENTAL FOUNDATION DESIGN REPORT**

**for**

**CULVERT REPLACEMENT STA. 20+510**

**CULVERT REHABILITATION STA. 22+350**

**REHABILITATION OF HIGHWAY 8, GODERICH TO CLINTON**

**WP 189-89-00**

**TOWNSHIP OF GODERICH, ONTARIO**

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March 24, 2009



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TABLE 1 – List of Standard Specifications Referenced in Report

**SUPPLEMENTAL FOUNDATION DESIGN REPORT**

for  
Culverts at Sta. 20+510 and 22+350  
Rehabilitation of Highway 8, Goderich to Clinton  
WP 189-89-00  
Township of Goderich, Ontario

---

**1. INTRODUCTION**

This report provides foundation engineering comments and recommendations for the proposed construction of concrete headwalls/wing walls and/or gabion basket walls at the replacement and rehabilitation of two culvert locations. The works are proposed in connection with the rehabilitation of a 16.8 km long section of Highway 8 between the Goderich east limits and the Clinton west limits. This report was prepared for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

This report is supplemental to the previous report issued for the replacement of the Ginn's Creek Culvert on November 3, 2006 GEOCREC No.: 40P12-12.

The replacement of the culvert located at Sta. 20+510 and the rehabilitation to the inlet end of the existing culvert at Sta. 22+350 is proposed.

In summary, the subsurface stratigraphy revealed in the boreholes drilled at the culvert sites generally comprised surficial topsoil and the highway embankment fill underlain by compact sand and gravel (Sta. 20+510) or a firm to very stiff clayey silt till deposit (Sta. 22+350). Cobbles and boulders were encountered within the sand and gravel and glacial till deposits. Groundwater was measured in the boreholes at Sta. 20+510 at levels ranging from 0.9 to 1.4 m depths, elevations 251.3 to 251.9 and at 1.4 m depth, elevation 278.3 in the borehole drilled at the Sta. 22+350 site.

The culvert at Sta. 20+510 is part of the tributary system to Ginn's Creek and is located about 130 m north of the Ginn's Creek Culvert on Highway 8. The existing open footing concrete culvert structure at Sta. 20+510 is proposed to be replaced with a 38 m long precast concrete box culvert, with 2.4 by 1.5 m opening size and cast-in-place concrete headwalls at the inlet and outlet. The



Cast-in-place concrete wing walls are to be constructed at both ends of this culvert, extending 3 to 7 m from the sides of the box culvert and from 3 to 5 m in height.

The existing open footing concrete culvert structure at Sta. 22+350 is proposed to be rehabilitated at the inlet end. A concrete pipe extension of 2.5 m along with the construction of gabion basket walls around the extension is proposed. The gabion basket walls are to extend about 5 to 6 m from the centerline of the culvert along the toe of the embankment and range from 3 to 4 m in height.

This report pertains to the design and construction of the proposed cast-in-place headwalls/wing walls and/or gabion basket walls and associated backfill zones. Recommendations for the replacement or rehabilitation of the culverts were not within the scope of this supplemental study.

## **2. FOUNDATIONS**

### **2.1 General**

The feasibility and potential construction concerns for the design and construction of the retaining walls are discussed in the following sections of this report. It is noted that no responsibility or liability is assumed by the consultants or MTO for alerting the contractor and to “red-flag” all critical issues. The requirement to deliver acceptable construction quality remains the responsibility of the contractor.

A list of the standard specifications referenced in this report is compiled in Table 1. All elevations in this report are expressed in metres.

The foundation frost depth for structures foundations at this site is 1.3 m, according to OPSD-3090.101.



#### 2.1.1 Culvert Replacement – Sta. 20+510

The invert levels of the proposed 2.4 m wide replacement box culvert at Sta. 20+510 are specified at elevation 251.4 at the inlet and elevation 250.8 at the outlet. The cast-in-place headwalls should be founded at or below the 1.3 m foundation frost protection level, elevations 249.5 and 250.1 at the outlet and inlet, respectively. Proposed finished grading in front of the cast-in-place wing walls will provide the required frost protection to found the walls at elevations 249.9 and 250.5 at the outlet and inlet, respectively.

The subgrade material revealed in the boreholes below the founding levels comprises compact sand and gravel. The groundwater level at the time of the field investigation was at elevations 251.3 to 251.9 at the outlet and inlet, some 1.8 m above the founding levels.

The construction of cast-in-place headwalls/wing walls at the proposed box culvert inlet and outlet is considered feasible. The groundwater should, however be controlled or temporarily diverted to provide adequate subgrade for construction of the foundations. Alternatively a dewatering system, such as well points may be utilized.

#### 2.1.2 Culvert Rehabilitation – Sta. 22+350

The inlet invert level of the proposed 2.5 m long concrete pipe extension at Sta. 22+350 is specified at elevation 279.6. The proposed founding level for the gabion basket wall is at approximate elevation 278.5.

The subgrade material revealed in the borehole at the proposed founding level comprises firm clayey silt till, becoming very stiff below elevation 277.5. The groundwater level at the time of the field investigation was at elevation 278.3, some 0.2 m below the proposed founding level.

The construction of gabion basket walls at the proposed extended culvert inlet is considered feasible and the groundwater should be adequately controlled with sump pumping.





## 2.2 Bearing Resistance

### 2.2.1 Culvert Replacement - Sta. 20+510

The headwall/wing wall foundations constructed on the compact sand and gravel should be designed using the following geotechnical resistances at ultimate and serviceability limit states (ULS and SLS) for the minimum 1.0 m wide footing:

WALL LOCATION	REF. FOUND ELEV. (m)	REFERENCE BOREHOLE	FOUNDING CONDITIONS	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL RESISTANCE AT SLS (kPa)
INLET	250.1 – 250.5	202	Compact Sand and Gravel	250	150
OUTLET	249.5- 249.9	201	Compact Sand and Gravel	250	150

The resistance at SLS allows for 25 mm settlement of the founding medium. A foundation embedment depth of 1.2 m and groundwater at about the level of the culvert invert were assumed for computation of the geotechnical resistance.

### 2.2.2 Culvert Rehabilitation - Sta. 22+350

The gabion basket wall foundations constructed on the firm to very stiff clayey silt should be designed using the following geotechnical resistances at ultimate and serviceability limit states (ULS and SLS) for the minimum 1.0 m wide gabion basket footing width:

WALL LOCATION	REF. FOUND ELEV. (m)	REFERENCE BOREHOLE	FOUNDING CONDITIONS	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL RESISTANCE AT SLS (kPa)
INLET	278.5	101	Firm Clayey Silt Till	150	100
INLET	277.5	101	Very Stiff Clayey Silt Till	250	150



The resistance at SLS allows for 25 mm settlement of the founding medium. A foundation embedment depth of 1.2 m and groundwater at about the level of the culvert invert were assumed for computation of the geotechnical resistance.

The design of the walls should be checked for sliding resistance using the geotechnical parameters provided in Section 2.3.

### 2.2.3 Subgrade Preparation

Preparation of the subgrade for construction of the headwalls/wing walls and/or gabion basket walls should be performed and monitored in accordance with OPSS 902 and SP 902S01. This should include site review by qualified geotechnical personnel during preparation of the subgrade as well as during placement and compaction of granular fill or, if required, mass concrete fill. The removal of existing footings/structures at proposed headwall/wing wall and/or gabion basket wall locations should be carefully inspected and approved by qualified geotechnical personnel.

The topsoil/fill and other deleterious soils and soil disturbed by the removal of the existing footings should be excavated. The geometry of the subgrade preparation, cover backfill and frost taper treatment for the headwalls/wing walls should be carried out in accordance with OPSD-3121.150.

Structural fill placed under the gabion walls or retaining walls to accommodate any variation in the level of the native surface and/or replace any deleterious soils extending below the design founding level should comprise Granular A material placed in maximum 200 mm thick lifts, compacted to at least 98% of the ASTM D698 (standard proctor) maximum dry density with conformance to OPSS 501. The limit of the granular fill zone should extend laterally outward a minimum 0.3 m beyond the wall base and down to the subgrade at 45° to the horizontal and be established by a site specific survey.

If engineered fill is required the anticipated thickness is less than 1.0 m, the wall foundation on the engineered fill at the two sites should be designed using the previously recommended geotechnical resistances for native soils in this section of the report.



It is noted that the depth of excavation for the new foundations will be within the existing embankment fill. Where the excavation extends into the existing embankment, road protection will require bracing to support the cut slopes. Refer to Section 7 of this report for further comments.

It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the excavation at the culvert at Sta. 22+350. Dam and pump techniques or temporary creek flow diversion will be required for the culvert at Sta. 20+510 in view of the relatively pervious local soils. Further comments in this regard are provided in subsequent Section 7 of this report.

### 2.3 Sliding Resistance

The following parameters should be used for sliding resistance of wall foundations.

#### Culvert Replacement - Sta. 20+510

PARAMETER	GRANULAR A OR GRANULAR B, TYPE II	COMPACT SAND AND GRAVEL Elevation 249.5 – 250.5
Friction Angle, degrees	35	32
Cohesion, kPa	0	0
Unit Weight, kN/m <sup>3</sup>	22.8	20.5

#### Culvert Rehabilitation - Sta. 22+350

PARAMETER	GRANULAR A OR GRANULAR B, TYPE II	FIRM CLAYEY SILT TILL Elevation 277.5	VERY STIFF CLAYEY SILT TILL Elevation 278.5
Friction Angle, degrees	35	0	0
Cohesion, kPa	0	50	100
Unit Weight, kN/m <sup>3</sup>	22.8	19.0	20.0



### **3. WALL BACKFILL**

The backfill behind the cast-in-place concrete walls should consist of suitable free draining granular materials such as Granular A or Granular B Type I or Type II and the backfill geometry should be according to OPSD-3121.150. The backfill should be placed and compacted to at least 95% of the standard Proctor maximum dry density. A geotextile/fabric should be placed between the free draining wall backfill and existing embankment fill (or any new non-free draining backfill) to mitigate the migration of fines.

Backfilling adjacent to retaining structures should be carried out in conformance with OPSS 501 and SP105S10. 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. Refer to SP 105S10 for additional information in this regard

All backfilling and compaction operations should be supervised on a full-time basis by geotechnical personnel to examine and approve backfill materials, evaluate placement operations and verify that the specified degree of compaction is achieved uniformly throughout the fill.

The grading of the earth fill slopes should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required.

The upper 600 mm of backfill against the wall should consist of relatively impermeable material to mitigate infiltration.

### **4. LATERAL EARTH PRESSURE**

The cast-in-place concrete walls and gabion basket walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure,  $p$  (kPa) may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation.



$p = K(\gamma h + q) + C_p + C_s$   
where  $K$  = coefficient of lateral earth pressure (dimensionless)  
 $\gamma$  = unit weight of free-draining granular material,  $\text{kN/m}^3$   
 $h$  = depth below final grade, m  
 $q$  = surcharge load, kPa, if present  
 $C_p$  = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)  
 $C_s$  = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)  
where  $\phi$  = angle of internal friction of retained soil  
 $\delta$  = angle of friction between the soil and wall

Free-draining granular material should be used as backfill behind the cast-in-place concrete walls. The following parameters are recommended for design:

PARAMETERS	Granular A, Granular B Type II	Granular B Type I	Excavated Sand and Gravel
Internal Friction Angle, $\phi$ (degrees)	35	32	30
Unit weight, $\gamma$ ( $\text{kN/m}^3$ )	22.8	21.0	20.5
Coefficient of Active Earth Pressure, $K_a$	0.27	0.31	0.33
Coefficient of Earth Pressure At Rest, $K_o$	0.43	0.47	0.5
Coefficient of Passive Earth Pressure, $K_p$	3.69	3.25	3.00

The coefficient of earth pressure at-rest should be used for design of rigid and unyielding walls, the active earth pressure coefficient for unrestrained structures. The earth pressure coefficients should be reviewed if the slope of the backfill exceeds  $10^\circ$  to the horizontal. Alternatively, the material above the top of the wall could be treated as a surcharge load ( $q$  in the preceding equation).

The magnitude of the passive resistance is dependent on the actual lateral movement of the structure toward the retained soil. We refer to Figure C6.16 of the CHBDC for this computation. The subsoil/backfill should be considered as medium dense sand for the project.

The horizontal force at the base of the walls will be resisted along the interface between the granular base or cast-in-place footing and the founding soil, subject to site specific design details.



An unfactored friction factor of 0.7 is considered to be appropriate for the interface between the granular base and founding soil or cast-in-place footing and founding soil, respectively.

The design should consider both the maximum water level in the stream and the stabilised groundwater level conditions. The groundwater level measured during the field investigation was variable at the culvert locations, being 0.2 m below the inferred founding subgrade at the Sta. 22+350 culvert and 1.8 m above the proposed founding level at the Sta. 20+510 culvert. The highest stream water level will be dictated by flood flow conditions and should be defined by the project hydraulic engineer.

## **5. CONCRETE HEADWALLS/WING WALLS**

For headwall design at the culvert replacement site, the wall founding levels should match those of the respective culverts where the walls are designed integral with the culvert structure. For walls designed separately from the culvert structure, the founding levels should be established 1.3 m below the culvert invert level for adequate frost protection, elevations 249.5 and 250.1 at the outlet and inlet, respectively. Where the ground surface behind the walls is sloped, we refer to Section 6.9 of the CHBDC and respective commentary for structural computations using the medium dense soil type.

The design of the walls should be checked for sliding resistance using the geotechnical parameters provided in Section 2.3.

A subdrain system (SP 405F03) should be installed to minimize the build-up of hydrostatic pressure behind cast-in-place concrete walls.

## **6. GABION WALLS**

The construction of gabion basket walls is proposed for the culvert rehabilitation site. The granular pad founding levels should be established 1.3 m below finished grade for adequate frost protection. Where the ground surface behind the walls is sloped, we refer to Section 6.9 of the



CHBDC and respective commentary for structural computations using the medium dense soil type.

The design of the walls should be checked for sliding resistance using the geotechnical parameters provided in Section 2.3.

The gabion walls should be constructed in accordance with OPSS 512 for gradation, bedding and backfill, assembly and placement requirements. A geotextile/fabric should be placed along the interior of the gabion wall between the gabion basket and backfill to mitigate the migration of fines into the voids of the gabion stone. Additionally, geotextile/fabric should be wrapped around the lower courses of gabion wall that will be constructed below the stream invert level and/or exterior finished grade to also minimize the migration of fines into the voids of the gabion stone. A non-woven Class II geotextile with an FOS of 75-150  $\mu\text{m}$ , according to OPSS 1860, should be used.

## **7. EXCAVATION AND GROUNDWATER CONTROL**

Excavation to the anticipated founding level of the walls is expected to extend through the topsoil and/or fill into the compact sand and gravel (Sta. 20+510) or into a firm to very stiff clayey silt till deposit (Sta. 22+350). Provision for excavation of cobbles and boulders at the site should be made. Subject to adequate groundwater control, excavation of the soils should be feasible using conventional equipment. According to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria, the in-situ materials mentioned above are typically classified as Type 3 soils necessitating an inclination of temporary cut slopes at 1H:1V (horizontal to vertical). The need to excavate flatter sideslopes below the groundwater table or if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction should be considered. The full depth of the existing fill soils which may extend to the founding level of the existing footings should be supported.

It is anticipated that a suitable roadway protection scheme following SP 105S19 will be required to support the existing embankment and the walls of the excavation and adjacent traffic lanes during construction of the culvert at Sta. 20+510. Several protection scheme alternatives such as sheet



piling, sheeting supported by rakers or bracing, cantilever or anchored soldier piles and lagging may be considered. It is noted however that soldier pile and lagging schemes are not considered adequate where the excavation will be carried out through the sand and gravel material under the water table. The schemes should be designed for performance level 2 provided that groundwater control is in place. The contractor is responsible for the selection, preparation and performance of a detailed design for the road protection scheme.

For the culvert replacement site at Sta. 20+510 the groundwater level observed in the boreholes at the time of the field investigation was at elevations 251.3 to 251.9, up to 1.8 m above the anticipated level of excavation. At the inlet and outlet of this culvert, the excavation will be carried out through the existing topsoil/fill and into 1.2 to 1.6 m of the underlying the sand and gravel deposit. Given the depth within the permeable deposit the water flow should be temporarily diverted away from the excavation, using dam and pump techniques or a dewatering system, such as well points should be implemented.

For the culvert rehabilitation site at Sta. 22+350 the groundwater level observed in the borehole was found at elevation 278.3, some 0.2 m below to 0.8 m above the anticipated level of excavation. At the inlet of the culvert rehabilitation site at Sta. 22+350 the excavation will be carried out within the clayey silt till. Cognisant of the permeability characteristics of the clayey silt till, it is anticipated that conventional sump pumping will be adequate to control seepage of groundwater into the excavation.

The dewatering (if required) should follow the current OPSS 517. Where dewatering is required it should be designed to prevent affecting existing water wells. The contract documents should clearly state that the selection, design and implementation of dewatering of the excavations is the contractor's responsibility.

It is recommended that the work be carried out during the dry summer months to minimise the amount of groundwater 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 and with local/MTO regulations.

## **8. EROSION CONTROL**

The protective measures noted in the OPSS 577 series 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 materials.

Inlet and outlet protection in accordance with OPSS 511 and OPSS 1004 is recommended to prevent erosion adjacent to the culverts 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, the water level in the stream and should be established by a hydraulic engineer. A non-woven Class II geotextile with an FOS of 75-150  $\mu\text{m}$ , according to OPSS 1860, should be placed below the rip-rap to minimise the potential for erosion of fine particles from below the treatment.

All newly constructed embankment slopes and retained soils behind the headwalls and wing walls (if provided) should be covered with topsoil 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 with a view to aesthetics. Additional appropriate erosion control measures for the project should be assessed using the following erodibility K factor:

<b><u>SOIL TYPE</u></b>	<b><u>K FACTOR</u></b>
Clayey Silt Till	0.5
Sand and Gravel	0.1



## 9. CLOSURE

This report was prepared by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer, with the assistance of Mr. M.J. Narduzzi, BEng, and was independently reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact.

Yours very truly

Peto MacCallum Ltd.



Carlos M. P. Nascimento, P.Eng.  
Senior Project Engineer



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



**TABLE 1**  
**LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT**

<b>DOCUMENT</b>	<b>TITLE</b>
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting
OPSS 512	Construction Specification for Gabion Installations
OPSS 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation
OPSS 570	Construction Specification for Topsoil
OPSS 571	Construction Specification for Sodding
OPSS 572	Construction Specification for Seed and Cover
OPSS 577	Construction Specification for Temporary Erosion and Sediment Control Measures
OPSS 902	Excavation and Backfilling of Structures
OPSS 1004	Material Specification for Aggregates – Miscellaneous
OPSS 1860	Material Specification for Geotextiles
SP 105S10	Construction Specification for Compaction
SP 105S19	Construction Specification for Protection Systems
SP 405F03	Construction Specification for Pipe Subdrains
SP 902S01	Excavation and Backfilling of Structures
OPSD-3090.101	Foundation Frost Depths for Southern Ontario
OPSD-3121.150	Minimum Granular Backfill Requirements – Retaining Walls