Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION INVESTIGATION AND DESIGN REPORT

for

GINN'S CREEK CULVERT REPLACEMENT
REHABILITATION OF HIGHWAY 8, GODERICH TO CLINTON
WP 189-89-03
TOWNSHIP OF GODERICH, ONTARIO

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PML Ref.: 05KF129A
Index No.: 088FIR and 089FDR
Geocres No.: 40P12-12
November 3, 2006

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FOUNDATION INVESTIGATION REPORT
for
Ginn's Creek Culvert Replacement
Rehabilitation of Highway 8, Goderich to Clinton
WP 189-89-03
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 replacement of the culvert located about 3.3 km west of the Hamlet of Holmesville (Ginn's Creek Culvert). This report was prepared for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario.

The Ginn's Creek Culvert is designated as C-84 (Site No. 12-390-C) and is located at the Ginn's Creek crossing of Highway 8. The culvert is located at approximate Station 20+630, Highway 8 chainage.

This report provides a summary of the factual information obtained during the field investigation conducted at the location of Ginn's Creek culvert.

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*, 3rd 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 April 20 to June 6, 2006 and comprised three boreholes drilled to depths of 6.5 to 11.3 m at the locations shown on Drawing 1.

The borehole layout was established by Peto MacCallum Ltd. (PML) in accordance with the requirements noted in the Request for Proposal. The locations of and ground surface elevations at the boreholes were also determined by PML. All elevations in this report are expressed in metres.

The boreholes were advanced with a track-mounted drill rig and a motorized tripod setup where 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.

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.

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 18 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, PC-2 and GS-1 to GS-3 respectively.

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 Drawing 1. 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 a surficial topsoil and/or embankment fill underlain by clayey silt till and silt till. Cobbles and boulders were encountered within the glacial till deposit during drilling. No natural gas emissions were noted from the boreholes during drilling. Groundwater measured in all the boreholes was at elevations 246.0 to 249.5. The strata encountered are summarised below.

4.2 Topsoil

Surficial topsoil was present in boreholes 1 and 3 put down near both ends of the culvert. The topsoil had a thickness of 600 to 700 mm. The moisture content of one of the topsoil samples was about 13%.

4.3 Fill

Variable fill soils making up the existing Highway 8 pavement and embankment were present surficially in borehole 2 advanced on the right shoulder of the highway. The shoulder pavement comprised of a 400 mm thick layer of sand and gravel trace silt.

The embankment of Highway 8 in borehole 2 was composed of firm clayey silt with pieces of asphalt at about 2 m depth and loose to compact sandy silt containing organics and topsoil and sand and gravel. The fill extended to 6.6 m depth and was penetrated at elevation 247.3.

A layer of fill was also buried beneath the topsoil in borehole 3 drilled at the west end of the culvert. Represented by sand and gravel, the fill was 3.4 m in thickness and penetrated at 4.1 m depth (elevation 246.0).

Standard penetration test N-values in the fill materials ranged widely from 4 to 75 blows for 300 mm penetration of the sampler indicating typically loose to compact conditions with dense to very dense state of compaction below about 2 m depth in borehole 3.

The results of one Atterberg limits test and grain size distribution analysis performed on the cohesive component of the fill are presented in Figures PC-1 and GS-1 respectively. The liquid limit of the clayey silt fill was 27 and the plastic limit 15, with a corresponding plasticity index of 12. The moisture content ranged from 8 to 25%.

4.4 Clayey Silt Till

Underlying the topsoil, fill and/or interbedded with silt till (described on Section 4.5) at depths of 0.6 to 6.6 m (elevations 245.3 to 250.0) in all the boreholes was a slightly cohesive clayey silt till. This deposit was stiff to hard in consistency, with the penetrometer test results indicating a shear strength of 90 to 175 kPa. Cobbles and boulders were encountered within the clayey silt till during drilling. In borehole 1, the clayey silt till extended to 5.7 m depth (elevation 244.9). The deposit was not penetrated in boreholes 2 and 3 at their termination depths of 6.5 and 11.3 m (elevations 243.6 and 242.6).

The results of Atterberg limits testing and grain size distribution analyses conducted on two samples of this material are presented in respective Figures PC-2 and GS-2. The liquid and plastic limits of the clayey silt till ranged from 18 to 20 and from 13 to 14 respectively, thus giving the plasticity index of 4 to 7. The moisture content of the deposit varied between 11 and 21%.

4.5 Silt Till

Cohesionless silt till was encountered below the fill or interbedded within the clayey silt till at depths of 4.1 to 8.7 m (elevations 244.9 to 246.0). This unit was compact to dense (N-values of 27 to greater than 50). Borehole 1 was terminated in the silt till at a depth of 6.7 m (elevation 243.9). In boreholes 2 and 3, the unit was 0.7 and 1.5 m thick and interbedded the clayey silt till from 4.1 to 4.8 m and from 8.7 to 10.2 m depths in boreholes 3 and 2 respectively (elevations 243.7 and 245.2 in borehole 2 and elevations 245.3 to 246.0 in borehole 3).

The results of grain size distribution analyses conducted on two samples of the silt till material are presented in Figure GS-3. The moisture contents of the silt till were 19 and 20%.

4.6 Groundwater

Water was observed in all the boreholes in the course of the field work. It was detected at depths of 0.8 to 5.3 m (elevations 247.9 to 249.3) in the process of augering. Upon completion of drilling, groundwater was measured at depths of 0.6 to 6.1 m (elevations 246.0 to 249.5). The water found in the culvert at its west end was about 0.4 m deep (elevation 249.3) on April 20, 2006.

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

5. CLOSURE

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 Geo-Environmental Drilling Inc.

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Contact, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

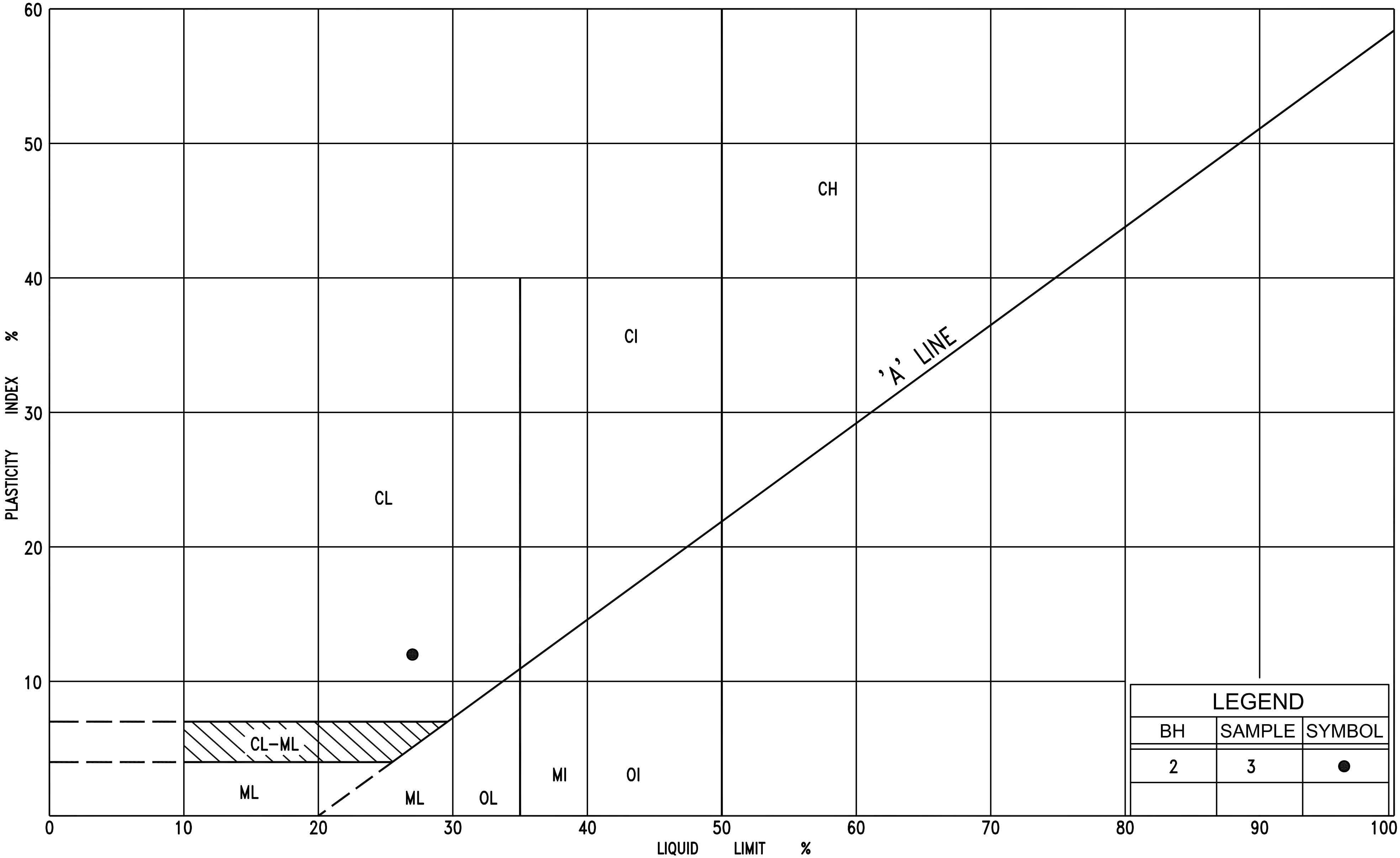
Carlos M. P. Nascimento, P.Eng.
Senior Project Engineer

REGISTERED PROFESSIONAL ENGINEER
C M P Nascimento
PROVINCE OF ONTARIO

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

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
PROVINCE OF ONTARIO

GD/CN:mi



LEGEND		
BH	SAMPLE	SYMBOL
2	3	●

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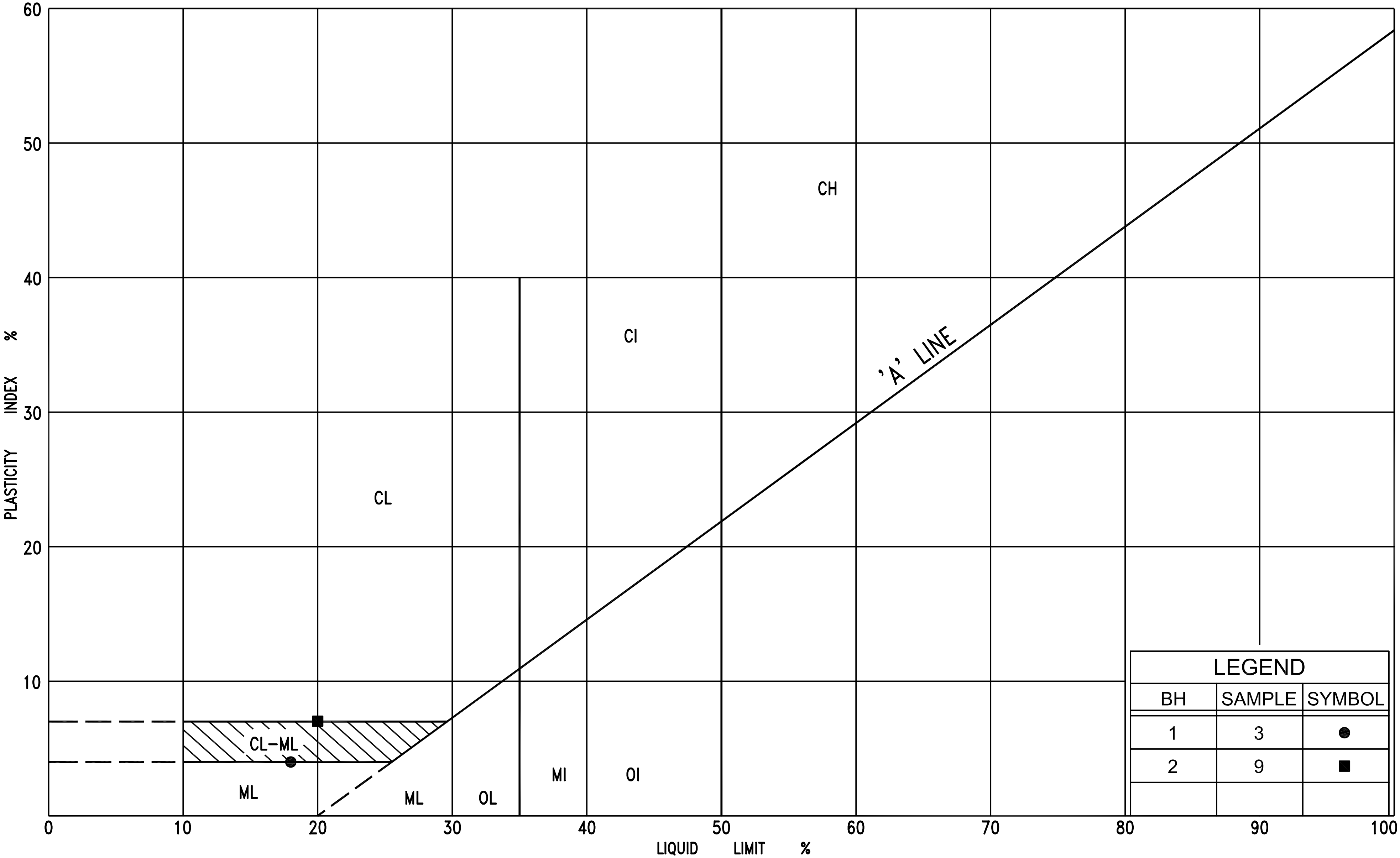
PLASTICITY CHART

CLAYEY SILT, some sand, trace gravel
(FILL)

FIG No.	PC-1
HWY	8
W.P. No.	189-89-03



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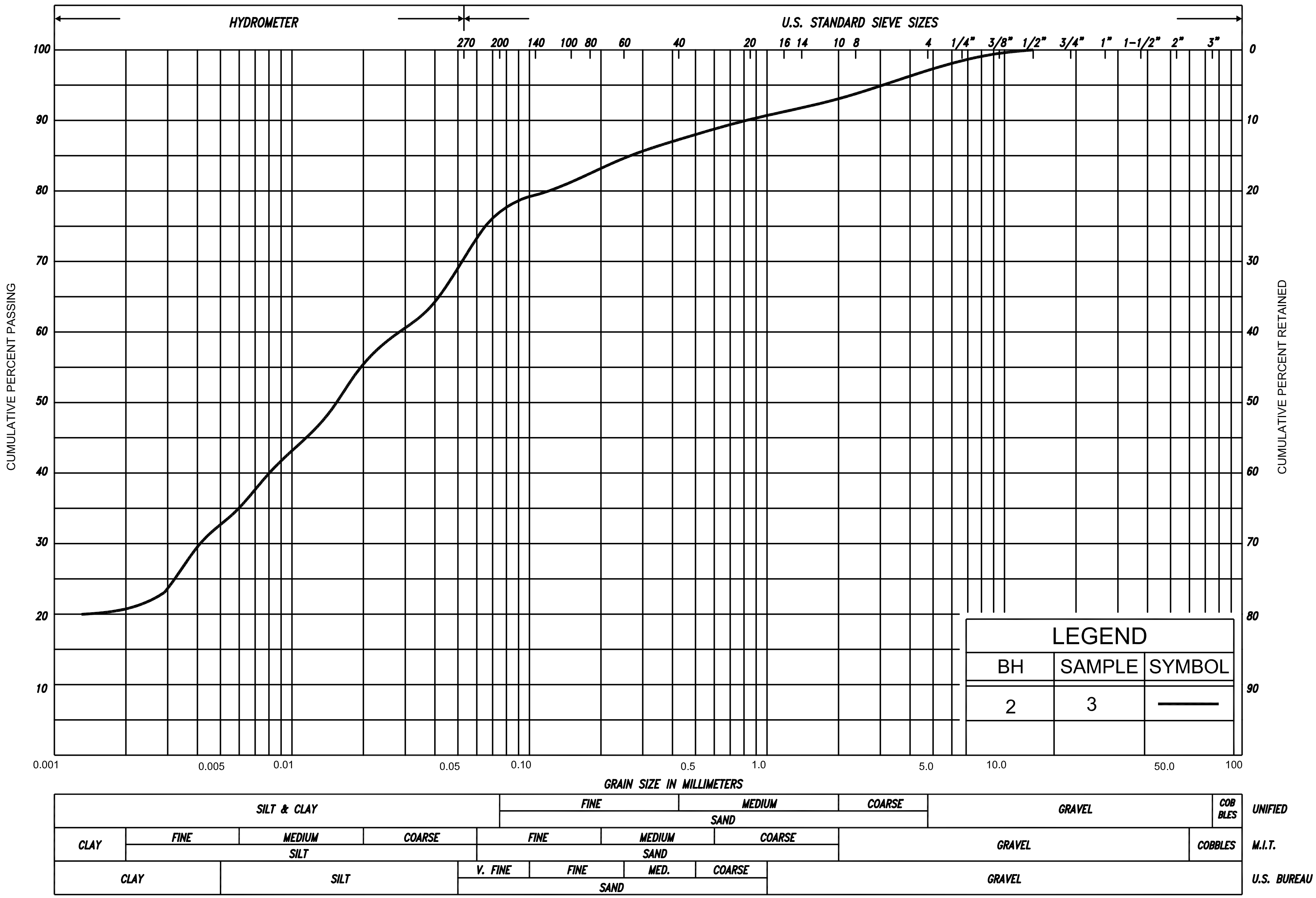
PLASTICITY CHART

CLAYEY SILT, some sand, trace gravel
(TILL)

FIG No.	PC-2
HWY	8
W.P. No.	189-89-03



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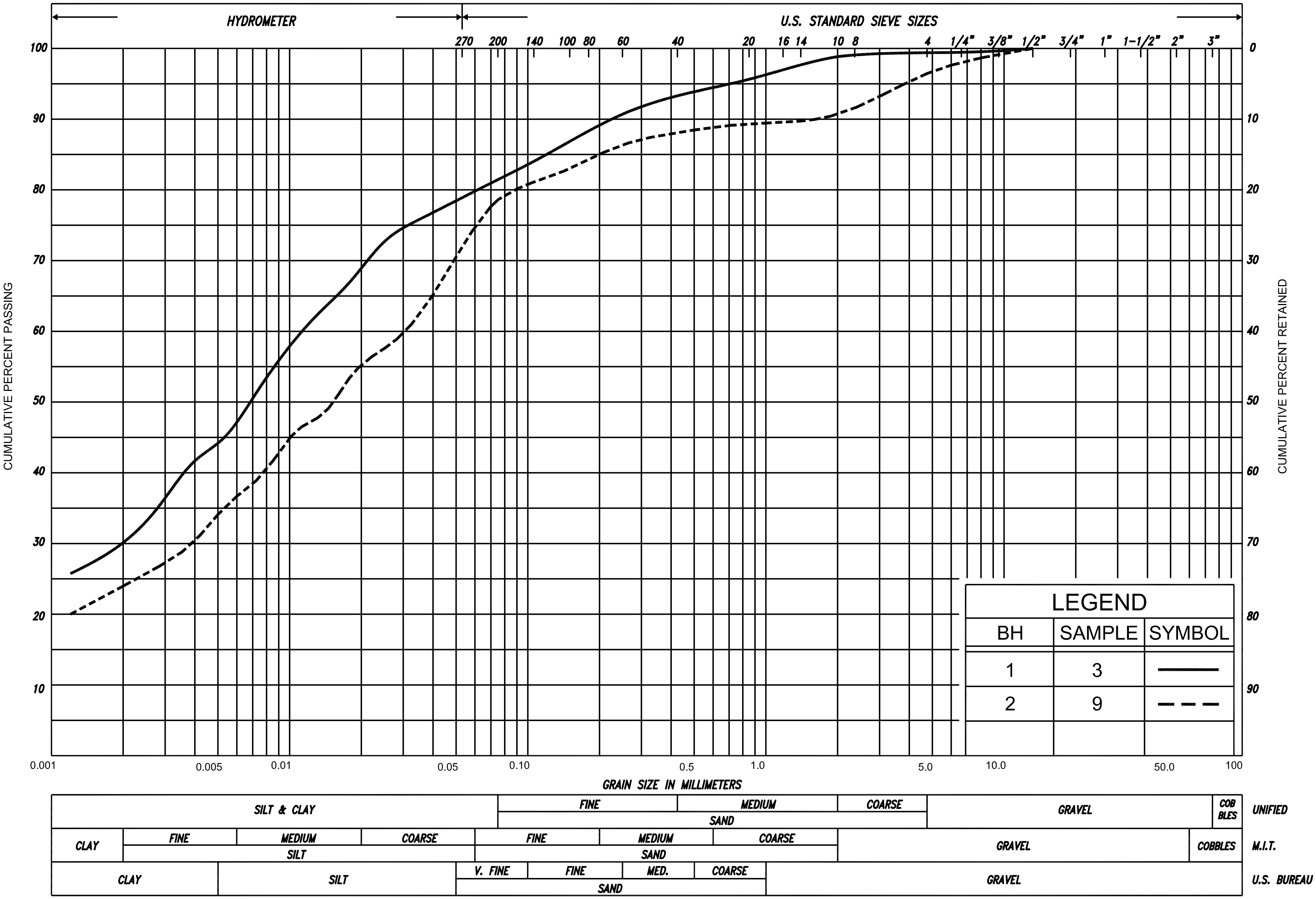
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some sand, trace gravel
(FILL)

FIG No.	GS-1
HWY	8
W.P. No.	189-89-03



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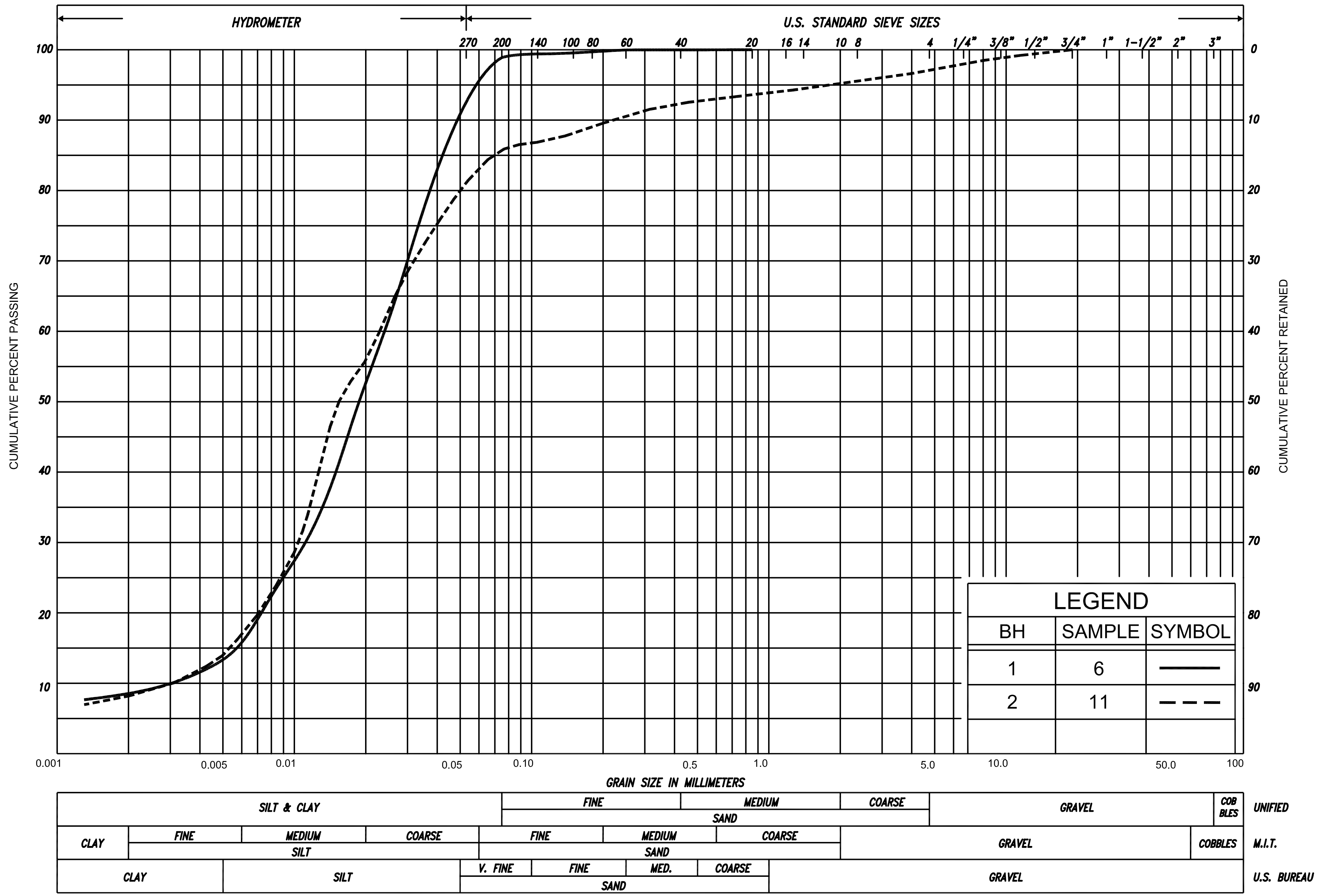
GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some sand, trace gravel
(TILL)

FIG No.	GS-2
HWY	8
W.P. No.	189-89-03



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GRAIN SIZE DISTRIBUTION

SILT, trace to some sand, trace clay, trace gravel (TILL)

FIG No.	GS - 3
HWY	8
W.P. No.	189-89-03

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
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

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

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

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 1

METRIC

G.W.P. 189-89-03 | LOCATION Co-ords: 4 837 105 N; 375 532 E | ORIGINATED BY G.I.

DIST Owen Sound HWY 8 | BOREHOLE TYPE Continuous Flight Solid Stem Augers | COMPILED BY G.D

DATUM Geodetic | DATE May 24, 2006 | CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	PENETROMETER (kPa, approx.)	WATER CONTENT	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
250.6	Ground Surface								
0.0	Topsoil	1	SS	10		250		○	
250.0 / 0.6	Clayey silt, some sand, trace gravel, organics in upper 1.0m. Stiff to very stiff, Brown, Moist	2	SS	8			■	○	
	cobbles and boulders	3	SS	16		249		⊢○⊣	1 17 52 30
	Grey				∇*	248			
		4	SS	14		247	■	○	
	silt seams, sand and gravel seams, Wet (TILL)	5	SS	16	▼*	246		○	
244.9 / 5.7	Silt, trace sand, trace clay. Compact, Grey, Wet (TILL)	6	SS	28		245 / 244		○ Non-plastic	0 1 91 8
243.9 / 6.7	End of borehole								

* 2006 05 24

∇ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

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

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

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

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Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 2

METRIC

G.W.P. 189-89-03 | LOCATION Co-ords: 4 837 102 N; 375 512 E | ORIGINATED BY G.I.

DIST Owen Sound HWY 8 | BOREHOLE TYPE Continuous Flight Hollow Stem Augers | COMPILED BY G.D

DATUM Geodetic | DATE April 20, 2006 | CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa (Penetrometer)	WATER CONTENT (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
253.9	Ground Surface								
0.0	Sand and gravel, trace silt. Brown (PAVEMENT FILL)								
	Clayey silt, some to with sand, trace to some gravel, with sand and gravel pockets, pieces of asphalt at 2.0m depth. Firm, Dark brown/brown, Moist (FILL)	1	SS	14		253			
		2	SS	5		252			
		3	SS	4					3 20 56 21
		4	SS	7		251	■		
		5	SS	5		250	■		
	Sandy silt, organics, topsoil. Loose to compact, Dark brown	6	SS	9		249			
		7	SS	17	▽*				
	Sand and gravel, trace silt, thin layer of clayey silt. Brown, Wet	8	SS	22	▼*	248			
247.3 / 6.6	Clayey silt, some sand, trace gravel, cobbles and boulders. Very stiff, Brown, Moist; silt seams, Grey (TILL)	9	SS	18		247	175		3 18 55 24
		10	SS	17		246			
245.2 / 8.7	Silt, some sand, trace gravel, trace clay, sand and gravel seams. Compact, Grey, Wet (TILL)	11	SS	27		245		Non-plastic	3 11 78 8
243.7 / 10.2	Clayey silt, some sand, trace gravel, silt seams. Very stiff, Grey, Moist (TILL)	12	SS	22		244, 243			
242.6 / 11.3	End of borehole								

* 2006 04 20

▽ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa: O UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE (20 40 60 80 100); PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³

+[7], ×[5]: Numbers refer to Sensitivity

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

Ministry of Transportation Ontario

Peto MacCallum Ltd.
Foundation Design

RECORD OF BOREHOLE No 3

METRIC

G.W.P. 189-89-03 LOCATION Co-ords: 4 837 083 N; 375 509 E ORIGINATED BY G.I.

DIST Owen Sound HWY 8 BOREHOLE TYPE Motorized Tripod COMPILED BY G.D

DATUM Geodetic DATE June 05 & 06, 2006 CHECKED BY C.N.

ELEV DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE
250.1	Ground Surface					
0.0	Topsoil	1	SS	7		250
249.4 / 0.7	Sand and gravel trace silt Compact Brown Wet (FILL)	2	SS	20	▼* ▽*	249
		3	SS	21		248
	with silt Very dense	4	SS	69		247
		5	SS	75		
246.0 / 4.1	Silt, some sand trace gravel, trace clay Very dense Grey Moist (TILL)					246
245.3 / 4.8	Clayey silt with sand, trace gravel cobbles and boulders Hard Grey Moist (TILL)	6	SS	57		245
						244
243.6		7	SS	50/5cm		
6.5	End of borehole					

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100
SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE: 20 40 60 80 100
PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L; WATER CONTENT (%) 20 40 60
UNIT WEIGHT γ kN/m³
REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

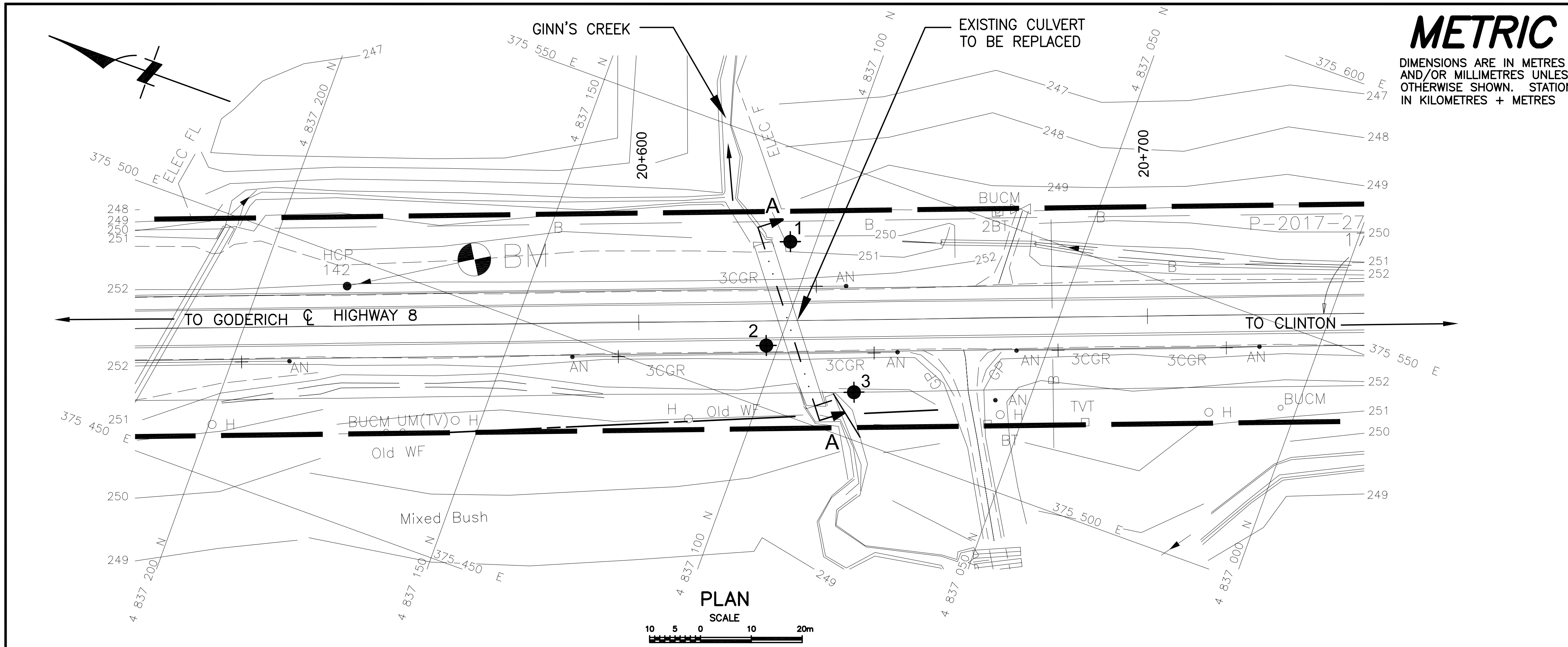
* 2006 06 05/06

▽ Water level observed during drilling

▼ Water level measured after drilling

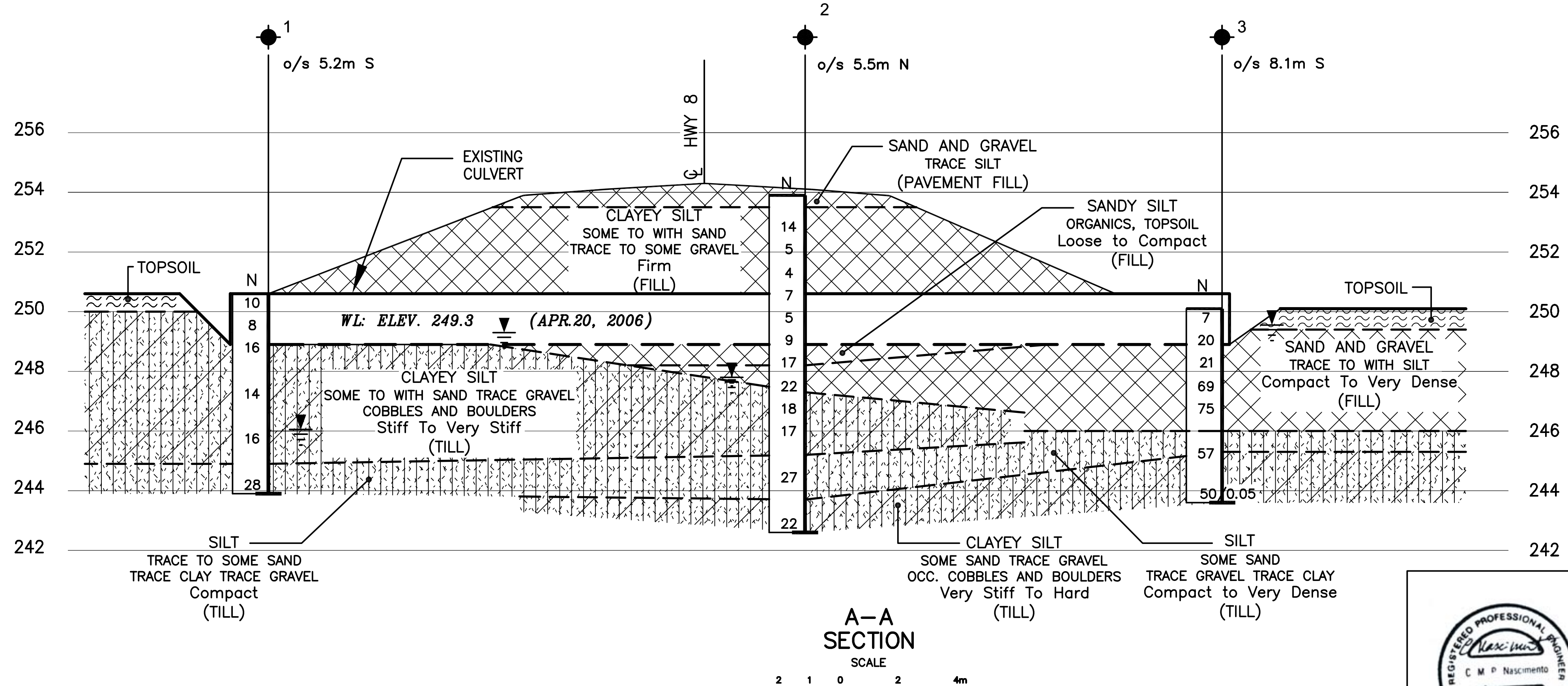
+7, ×5 : Numbers refer to Sensitivity

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



METRIC

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



CONT No

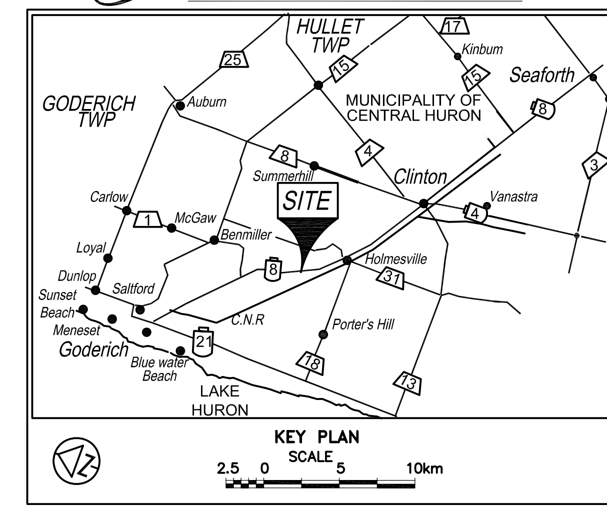
WP No 189-89-03

SHEET

GINN'S CREEK CULVERT
REPLACEMENT AT STA. 20+630

BOREHOLE LOCATIONS AND SOIL STRATA

Peto MacCallum Ltd.
CONSULTING ENGINEERS



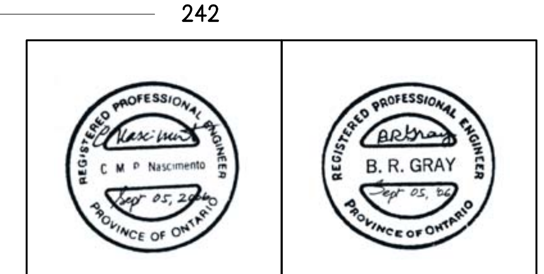
KEY PLAN
SCALE
2.5 0 5 10km

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 the time of investigation April–May 2006
- Head ARTESIAN WATER Encountered

BH No	ELEVATION	CO-ORDINATES NORTH	EAST
1	250.6	4 837 105	375 532
2	253.9	4 837 102	375 512
3	250.1	4 837 083	375 509

– 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: 40P11–12

HWY No. 8			DIST Owen Sound
SUBM'D GD	CHECKED GD	DATE SEPT. 05, 2006	SITE 12–390–C
DRAWN GI/NA	CHECKED CN	APPROVED BRG	DWG 1

REF No: 008god–8a.dwg; April 29, 2005

FOUNDATION DESIGN REPORT

for

GINN'S CREEK CULVERT REPLACEMENT
REHABILITATION OF HIGHWAY 8, GODERICH TO CLINTON
WP 189-89-03
TOWNSHIP OF GODERICH, ONTARIO

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5 cc: McCormick Rankin Corporation for distribution to MTO, Project Manager + one digital copy
1 cc: McCormick Rankin Corporation for distribution to MTO, Pavements and Foundations Section + one digital copy
2 cc: McCormick Rankin Corporation + one digital copy
1 cc: PML Kitchener
1 cc: PML Toronto

PML Ref.: 05KF129A
Index No.: 089FDR
Geocres No.: 40P12-12
November 3, 2006

TABLE OF CONTENTS

FOUNDATION DESIGN REPORT
for
Ginn's Creek Culvert Replacement
Rehabilitation of Highway 8, Goderich to Clinton
WP 189-89-03
Township of Goderich, Ontario

1. INTRODUCTION

This report provides foundation engineering comments and recommendations for the proposed replacement of the Ginn's Creek Culvert during 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).

The Ginn's Creek Culvert is designated as C-84 (Site No. 12-390-C) and is located at the Ginn's Creek crossing of Highway 8. The culvert is located at approximate Station 20+630, Highway 8 chainage. The Ginn's Creek Culvert is an approximately 33.5 m long concrete, rigid frame open footing structure, with 3.6 by 1.5 m opening size.

This report pertains to design and construction of the proposed culvert replacement and associated backfill zones.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised surficial topsoil and/or the Highway 8 shoulder pavement and embankment fill underlain by stiff to hard clayey silt till and compact to dense silt till. Compact to very dense sand and gravel fill extended below the culvert invert level in borehole 3, drilled on the west side of the Highway 8. Cobbles and boulders were encountered within the glacial till deposits during drilling of the boreholes. Groundwater was measured in all the boreholes at levels ranging from 0.6 to 6.1 m depths, elevations 246.0 to 249.5.

The site conditions indicate that the proposed replacement culvert may be designed as an open footing structure. Alternatively, a precast or cast-in place concrete box culvert may be used. A

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discussion of alternatives is presented in this report. Care should be taken with subgrade preparation and erosion protection as outlined in this report.

It is understood that construction and traffic staging at this site will be accomplished by limiting traffic to one lane for the duration of the replacement culvert construction. Temporary road protection will be required along the centreline of the highway as a minimum. In view of the presence of cohesionless sand and gravel fill at the subgrade level it is anticipated that the road protection will need to comprise of sheetpiling. Where dewatering is required it should be designed to prevent affecting existing water wells.

We also understand that the alignment of the replacement culvert will be changed slightly from the existing alignment. Consequently the new footing (in the open footing option) will not be located over the existing footing founding subgrade.

It is noted that no responsibility or liability is assumed by the consultants 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.

2. FOUNDATIONS

2.1 General

The invert of the existing open footing culvert is deduced to be near elevation 248.9. The existing subgrade founding level of the spread footings is inferred to be at elevation 248.0 based on the footing depth of about 0.9 m indicated on Drawing 1 of MTO Contract No. 2004-3404 ETR 171-8/21-0 for Concrete Culvert Rehabilitation dated December 2003.

The subgrade material revealed in the boreholes just below the founding level comprises stiff to very stiff clayey silt till under the eastern half of the culvert length and compact to very dense sand

and gravel fill under the centre and western half of the culvert length. It is inferred that the compact to very dense sand and gravel fill encountered below the culvert invert was placed as controlled fill at the time of the construction of the culvert. The high groundwater level at the time of the field investigation was at elevations 246.0 to 249.5, some 2.9 m below to 1.5 m above the inferred subgrade level of the existing footings.

Based on ground surface elevations at the borehole locations and the road grade (elevation 254.3), the embankment fill at the location of the culvert is assessed to be about 4 m high.

The replacement culvert may consist of an open footing culvert founded at or below the 1.3 m frost protection level, elevation 247.6.

A precast or cast-in concrete box culvert is also considered a feasible replacement. The base of the new precast culvert may be founded at about elevation 248.1 allowing for 150 mm thick granular bedding to be placed over the level of the existing footings (elevation 248.0). The founding level may be established at elevation 248.0 for the cast-in-place box culvert alternative since a granular bedding is not required under these culverts. The design invert level of the crossing will be established on fill placed inside the culvert. Alternatively, the box culvert base may be placed about 0.4 m below the design invert level at about elevation 248.5 using engineered fill, mass concrete or unshrinkable fill to backfill the excavation carried out to remove the existing footings and unsuitable fill potentially present between and beyond the footings. A discussion of the advantages and disadvantages of the three foundation alternatives outlined above is presented in Section 7 of this report.

Preparation of the subgrade for construction of the culvert 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 existing compact to very dense sand and gravel fill should be carefully inspected and approved where reused under the new culvert foundation.

The topsoil and other deleterious soils revealed at and below the subgrade and soil disturbed by the removal of the existing footings should be excavated. Any grade differences found under the foundations of the open footing culvert should be made up with mass concrete fill.

The geometry of the subgrade preparation, cover backfill and frost taper treatment for the culvert should be carried out in accordance with OPSD 803.010 and OPSS 422. A frost penetration depth of 1.3 m should be employed for the design.

2.2 Open Footing Culvert Replacement

For the open footing culvert design, the new footings should be founded at elevation 247.6 about 0.4 m lower than the inferred elevation of the existing footings, to provide 1.3 m of frost protection. Construction of the foundations for the culvert replacement on spread footings bearing on the stiff to very stiff clayey silt till and/or compact to very dense sand and gravel fill found in the boreholes within the zone of influence of the new foundations is considered to be feasible.

The culvert foundations constructed on the stiff to very stiff clayey silt till or compact to very dense sand and gravel fill should be designed using the following geotechnical resistances at ultimate and serviceability limit states (ULS and SLS) for the minimum 0.5 m wide footing:

Factored Geotechnical Resistance at ULS	=	300 kPa
Geotechnical Resistance at SLS	=	200 kPa

The resistance at SLS allows for 25 mm settlement of the founding medium. Total and differential settlements along the culvert are expected to be negligible in view of the relatively low net bearing pressure exerted by the culvert foundations. Therefore, provision for camber is not considered necessary for the replacement culvert.

2.3 Box Culvert Replacement

As indicated previously, the base of the replacement precast concrete box culvert may be placed at about elevation 248.1 that is about 150 mm above the level of the existing footing subgrade. A replacement cast-in-place box culvert may be founded at about elevation 248.0 since a bedding layer is not considered required for this culvert design. Alternately, the base could be established at about elevation 248.5 that is about 0.4 m below the inferred invert level for the replacement culvert. These levels allow for the thicknesses of the culvert base slab and granular bedding.

The box culvert founded at the deeper level about elevation 248.1 will be placed on the stiff to very stiff clayey silt till or on existing compact to very dense sand and gravel fill. The following geotechnical resistances at ultimate and serviceability limit states (ULS and SLS) should be used for design.

Factored Geotechnical Resistance at ULS	=	300 kPa
Geotechnical Resistance at SLS	=	200 kPa

For the box culvert replacement option at the shallower level (base slab at elevation 248.5) the existing footings and the organic or loose materials fill present between and outside of the footings should be removed to allow placement of engineered fill, unshrinkable fill or mass concrete to make up the grade below the culvert bedding.

Engineered fill placed under the culvert 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 compacted to at least 95% of the target density with conformance to OPSS 501 and SP 105S10. The limit of the granular fill zone should extend laterally outward a minimum 0.3 m beyond the culvert base and down to the subgrade at 45° to the horizontal and be established by a site specific survey.

The geometry of the subgrade preparation, cover backfill and frost taper treatment for the culvert should be carried out in accordance with OPSD 803.010 and OPSS 422. The bedding material for a precast box culvert should comprise a minimum 150 mm thick layer of Granular A.

Since the estimated thickness of engineered fill is less than 1.0 m, the box culvert foundation on the engineered fill at this site should be designed using the previously recommended geotechnical resistances for native soils in this section of the report.

Settlement considerations for the box culvert replacement are the same as those provided previously in the report for the open footing type culvert.

It is noted that the depth of excavation for the new foundations will be about 3 m beyond the toe of the existing embankment and up to 8 m 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 5 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. Further comments in this regard are provided in subsequent Section 5 of this report.

2.4 Sliding Resistance

The following parameters should be used for sliding resistance of cast-in-place culvert foundations. The friction angle and cohesion for precast concrete culverts should be reduced by a factor of 0.67.

PARAMETER	GRANULAR A OR GRANULAR B, TYPE II	COMPACT TO VERY DENSE SAND AND GRAVEL FILL	VERY STIFF CLAYEY SILT TILL
Friction Angle, degrees	35	30	0
Cohesion, kPa	0	0	150
Unit Weight, kN/m^3	22.8	20.5	20.0

3. CULVERT BACKFILL

Backfill adjacent to the culverts should be placed in accordance with OPSD 803.010, OPSD 3121.150 and OPSS 422. The compaction of earth backfill against the culvert should follow the OPSS 501 and SP 105S10.

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

The replacement 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 exerted by the backfill adjacent to the culvert walls.

The lateral earth and water pressure, p (kPa), should be computed using the equivalent fluid pressures presented in Section 6.9 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00, March 2001, or employing the following equation assuming a triangular pressure distribution.

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

where p = lateral earth pressure (kPa)

K = lateral earth pressure coefficient

γ = unit weight of backfill material above design water level (kN/m^3)

γ' = unit weight of submerged backfill material below design water level (kN/m^3)
= $\gamma - \gamma_w$

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

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

h_2 = depth below design water level (m)

q = any surcharge load (kPa)

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

The following parameters are recommended for design:

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

The design should consider both the maximum water level in the stream and the stabilised groundwater level conditions. The groundwater level measured during the field investigation was variable at the culvert location, from 2.9 m below to 1.5 m above the inferred founding subgrade level of the existing culvert. The highest stream water level 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.

4. HEADWALLS AND WING WALLS

If headwalls and wing walls are utilised, the previous recommendations and geotechnical parameters for culvert foundations and backfill should be utilised for design of the foundations. 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. Where the ground surface behind the walls is sloped, we refer to Section 6.9.1 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 2.4 for cast-in-place concrete foundations.

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

5. EXCAVATION AND GROUNDWATER CONTROL

Excavation to the anticipated founding level of the culvert replacement is expected to extend through the topsoil and/or fill into the native deposit of clayey silt till and compact to very dense sand and gravel fill. 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 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 walls of the excavation and adjacent traffic lanes during construction. 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 fill material under the water table (boreholes 2 and 3). The schemes should be designed for performance level 2 provided that groundwater control is in place. Otherwise, a performance level 1a system is recommended to prevent movement of the existing embankment. The contractor is responsible for the selection, preparation and performance of a detailed design for the road protection scheme.

The groundwater level observed in the boreholes at the time of the field investigation was up to 2 m above the anticipated level of excavation. Cognisant of the permeability characteristics of the clayey silt till, it is anticipated that conventional sump pumping will be sufficient to control seepage of groundwater into the excavation within the eastern part of the culvert replacement. Where the excavation will be carried out within the existing sand and gravel fill encountered at the center and western part of the culvert (boreholes 2 and 3) a more positive dewatering system, such as well points should be implemented. The dewatering 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 will be necessary to implement measures to control water flow in the stream. Conventional procedures such as dam and pump and/or temporary diversion of the stream should be sufficient. Observed stream and groundwater levels are subject to seasonal fluctuations and precipitation patterns.

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.

6. EROSION CONTROL

The protective measures noted in the OPSD 800 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. Where required for the box culvert options, the cut-off walls should extend to a depth at least equal to the fluctuation of the water level at the culvert location to prevent flow below the culvert that could erode the granular base/bedding material as well as extend laterally to protect the granular backfill material. The cut-off walls should also protect the existing sand and gravel fill and new engineered fill which would be required to install a replacement box culvert. The requirements of CHBDC clauses 1.10.5.6 and 1.10.11.6.5 should be applied.

Inlet and outlet protection in accordance with OPSS 511 and OPSS 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, 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 µm, 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:

SOIL TYPE	K FACTOR
Clayey Silt Till	0.5
Sand and Gravel Fill	0.1

7. DISCUSSION OF FOUNDATION ALTERNATIVES

7.1 Advantages and Disadvantages of Foundation Alternatives

The following table summarizes the advantages and disadvantages and inferred risks/consequences of each of the foundation alternatives for the replacement culvert at the Ginn's Creek crossing.

ADVANTAGES AND DISADVANTAGES - GINN'S CREEK CULVERT

OPEN FOOTING		BOX CULVERT WITH BASE AT ELEV. 248.1		BOX CULVERT WITH BASE AT ELEV. 248.5	
ADVANTAGES	DISADVANTAGES	ADVANTAGES	DISADVANTAGES	ADVANTAGES	DISADVANTAGES
Design matches existing configuration. Cut-off walls not required between footings. Design incorporates full frost protection.	Erosion control of soil between the footings is required. Relatively high user cost due to longer construction schedule than precast culvert construction.	Precast concrete culvert alternative is feasible. Concrete base provides erosion protection. Excavation for precast culvert installation is narrower than for box culvert at Elev. 248.5. Precast culvert option expedites construction, minimizing user costs.	Requires fill placement inside culvert to make up grade to design invert level. Only partial frost protection is incorporated - frost tapers required.	Precast concrete culvert alternative is feasible. Concrete base provides erosion protection at invert level. Precast culvert option expedites construction, minimizing user costs.	Subgrade level needs to be adjusted using engineered fill. Cut-off walls should extend below engineered fill level (minimum). Requires wider excavation and adequate site conditions (dewatering) for engineered fill construction. Only partial frost protection is incorporated - frost tapers required.

Notes: Culvert base and invert elevations are inferred for the purpose of this report.

The precast concrete box culvert option founded at about elevation 248.1 is considered to be less costly at this site, since the construction will be expedited without the forming and the setting time required for cast-in-place concrete construction. In addition, the engineered fill below the granular base of the culvert will not be required. However, it is expected that the construction of cut-off walls will offset some of the cost advantages of box culvert construction.

7.2 Preferred Foundation Option Considerations

From the foundation perspective, either of the three foundation schemes is feasible, however the box culverts provide a more effective erosion protection. The box culvert at the lower level (about elevation 248.1) is considered the preferred option in view of the estimated lower cost of construction and user cost.

It is noted that the selected foundation alternative also depends on other considerations, such as potential fish habitat protection, which are being evaluated separately by MRC.

8. CLOSURE

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Senior Project Engineer. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Contact, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

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

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

GD/CN:mi

TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Culverts 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 517	Construction Specification for Dewatering of Pipeline, Utility, and Associated Structure Excavation	November 2005
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 902	Excavation and Backfilling of Structures	November 2002
OPSS 1004	Material Specification for Aggregates – Miscellaneous	November 2005
OPSS 1860	Material Specification for Geotextiles	November 2004
SP 105S10	Construction Specification for Compaction	November 2004
SP 105S19	Construction Specification for Protection Systems	March 2005
SP 902S01	Excavation and Backfilling of Structures	September 2003
OPSD 803.010	Backfill and Cover for Concrete Culverts	November 1999
OPSD 3121.150	Minimum Granular Backfill Requirements – Retaining Walls	November 2005