



**FOUNDATION INVESTIGATION AND DESIGN REPORT
for
POTTAWATOMI RIVER TRIBUTARY CULVERT (SITE NO. 8-483C)
HIGHWAY 6, STA. 10+445
SPRINGMOUNT
G.W.P. 43-00-00
DISTRICT OF LONDON, ONTARIO**

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PML Ref.: 11KF065A-C1
Index No. 114FIR and 115FDR
GEOCRES No. 41A - 225
November 21, 2012



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for
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FOUNDATION INVESTIGATION REPORT
for
Pottawatomi River Tributary Culvert (Site No. 8-483C)
Highway 6, Sta. 10+445
Springmount
G.W.P. 43-00-00
District of London, Ontario

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the Pottawatomi River Tributary culvert replacement (Site No. 8-483C) as a part of rehabilitation of Highway 6, from Springmount to Hepworth. The study was carried out by Peto MacCallum Ltd. (PML) for McCormick Rankin (MRC), a member of MMM Group Ltd., on behalf of the Ministry of Transportation of Ontario (MTO).

The purpose of this report was to summarize the subsurface stratigraphy and groundwater conditions encountered in the boreholes advanced during the foundation investigation at the new culvert site.

2. SITE DESCRIPTION AND GEOLOGY

The existing culvert is located under the Highway 6 northbound and southbound lanes, about 500 m north of the intersection of Highway 6 and Highway 21 in the Town of Springmount. The existing culvert conveys the flow from a tributary of the Pottawatomi River.

Land use in the vicinity of the site includes the existing Highway 6 transportation corridor and commercial sites. The terrain includes level areas vegetated with grass, brush and scattered trees. The topography of the site is generally flat. Site photographs of the culvert location are attached in Appendix A.

Physiographically the site is located in the region referred to as the Bruce Peninsula. The surficial and bedrock geology consists of a relatively thin till soil cover less than 5 m thick over dolomite bedrock.



3. INVESTIGATION PROCEDURES

The subsurface investigation was carried out on January 12, 2012. Two boreholes (CT1-1 and CT1-2) were drilled to depths of 4.1 and 4.6 m as shown on Drawing PRT-1, appended. In view of the wet conditions at the time of the field work, boreholes were not drilled beyond the highway platform. The data in the boreholes was consistent and considered to be representative for the purpose of this project.

The boreholes were advanced with a truck-mounted CME 45 drill rig using continuous flight hollow stem augers. The equipment was supplied and operated by a specialist drilling contractor, working under the full-time supervision of a PML field supervisor.

Soil samples were recovered from the boreholes at regular 0.75 and 1.5 m depth intervals using the standard penetration test method. Standard penetration tests were conducted to assess the strength characteristics of the substrata. Soils were identified in accordance with the MTO soil classification manual procedures. The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, where encountered, by measuring the groundwater level in the open boreholes.

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

The co-ordinates and ground surface elevations at the boreholes were provided by MMM Group Ltd. All elevations are reported in metres.



The recovered soil samples were returned to our laboratory in Toronto for detailed visual examination, laboratory testing and classification. The laboratory testing program included the following tests:

- Natural moisture content determinations (7)
- Grain size distribution analyses (4)
- Atterberg limits tests (4)

The grain size distribution charts are presented in Figures CT1-GS-1 to CT1-GS-3. The plasticity charts are presented in Figures CT1-PC-1 to CT1-PC-3. All of the test results are shown on the Record of Borehole sheets.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, standard penetration test results as well as groundwater observations. The results of grain size distributions, Atterberg limits tests and moisture content determinations are also shown on the Record of Borehole sheets.

The borehole locations and stratigraphic profile prepared from the borehole data are presented on the foundation Drawing PRT-1.

Boreholes CT1-1 and CT1-2 were drilled along the alignment of this culvert to depths of 4.1 and 4.6 m, elevation 231.8 and 231.2, respectively. The subsurface stratigraphy revealed in the boreholes generally comprised a road embankment fill underlain by native silty clay. Bedrock was inferred by auger refusal at depths of 4.1 and 4.6 m, elevation 231.8 and 231.2, respectively. Groundwater was observed on completion of drilling in both boreholes.



4.1.1 Fill

From the ground surface, a 2.1 m thick fill extending to elevation 233.8 and 233.7 was encountered in boreholes CT1-1 and CT1-2 respectively. The fill layer includes a surficial 0.6 m thick compact sand and gravel from the Highway 6 shoulder pavement underlain by soft silty clay. The sand and gravel fill locally below the shoulder pavement contained asphalt pieces while the silty clay fill locally contained rootlets, topsoil and organic inclusions. SPT N values in the fill ranged from 2 to 21.

The results of grain size distribution analysis for the silty clay fill contacted in borehole CT1-1 are included in Figure CT1-GS-1. A plasticity chart of the silty clay fill sample is presented in Figure CT1-PC-1. The Atterberg liquid and plastic limits were 41 and 22 respectively, with a plasticity index of 19. The moisture content of the tested sample was 15%.

The results of grain size distribution analysis for the silty clay with organics fill contacted in borehole CT1-2 are included in Figure CT1-GS-2. A plasticity chart of the silty clay with organics fill sample is presented in Figure CT1-PC-2. The Atterberg liquid and plastic limits were 57 and 31 respectively, with a plasticity index of 26. The moisture content of the sample was 45% (elevated due to the presence of organics in the fill).

4.1.2 Silty Clay

Below the fill in boreholes CT1-1 and CT1-2, a 2.0 and 2.5 m thick, firm silty clay stratum was contacted to the borehole termination depths of 4.1 m, elevation 231.8, and 4.6 m, elevation 231.2, respectively. SPT N values in this stratum were 7 and 8.

The results of grain size distribution analyses for the silty clay are included in Figure CT1-GS-3. A plasticity chart of the two silty clay samples is presented in Figure CT1-PC-3. The Atterberg liquid limits were 35 and 36, the plastic limit of both test samples was 20 and the plasticity indices were 15 and 16 respectively. The moisture content of the samples ranged from 21 to 23%.



4.1.3 Bedrock

Bedrock was inferred by auger refusal in boreholes CT1-1 and CT1-2 at depths of 4.1 and 4.6 m, elevation 231.8 and 231.2, respectively. From the local geology, it is inferred that the bedrock consists of dolomite.

4.1.4 Groundwater

During drilling, groundwater was only observed in borehole CT1-2 at 3.8 m depth, elevation 232.0. Groundwater was observed in boreholes CT1-1 and CT1-2 at respective depths of 4.1 and 3.4 m, elevation 231.8 and 232.4, on completion of drilling. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

5. MISCELLANEOUS

Mr. Alan Lo carried out the field investigation for this study under the supervision of Mrs. N .S. Balakumaran, P. Eng. Aardvaark Drilling Ltd. supplied the drill rig for the subsurface exploration. The laboratory testing of the selected samples was carried out in the PML laboratory in Toronto.



6. CLOSURE

This Foundation Investigation Report was prepared by Mr. H. Gharegrat, P.Eng., and reviewed by Mr. G. Degil, PhD, P.Eng., Senior Foundation Engineer. Mr. C. M. P. Nascimento, P. Eng., Project Manager and MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



Harry Gharegrat, MS, P.Eng.
Project Engineer

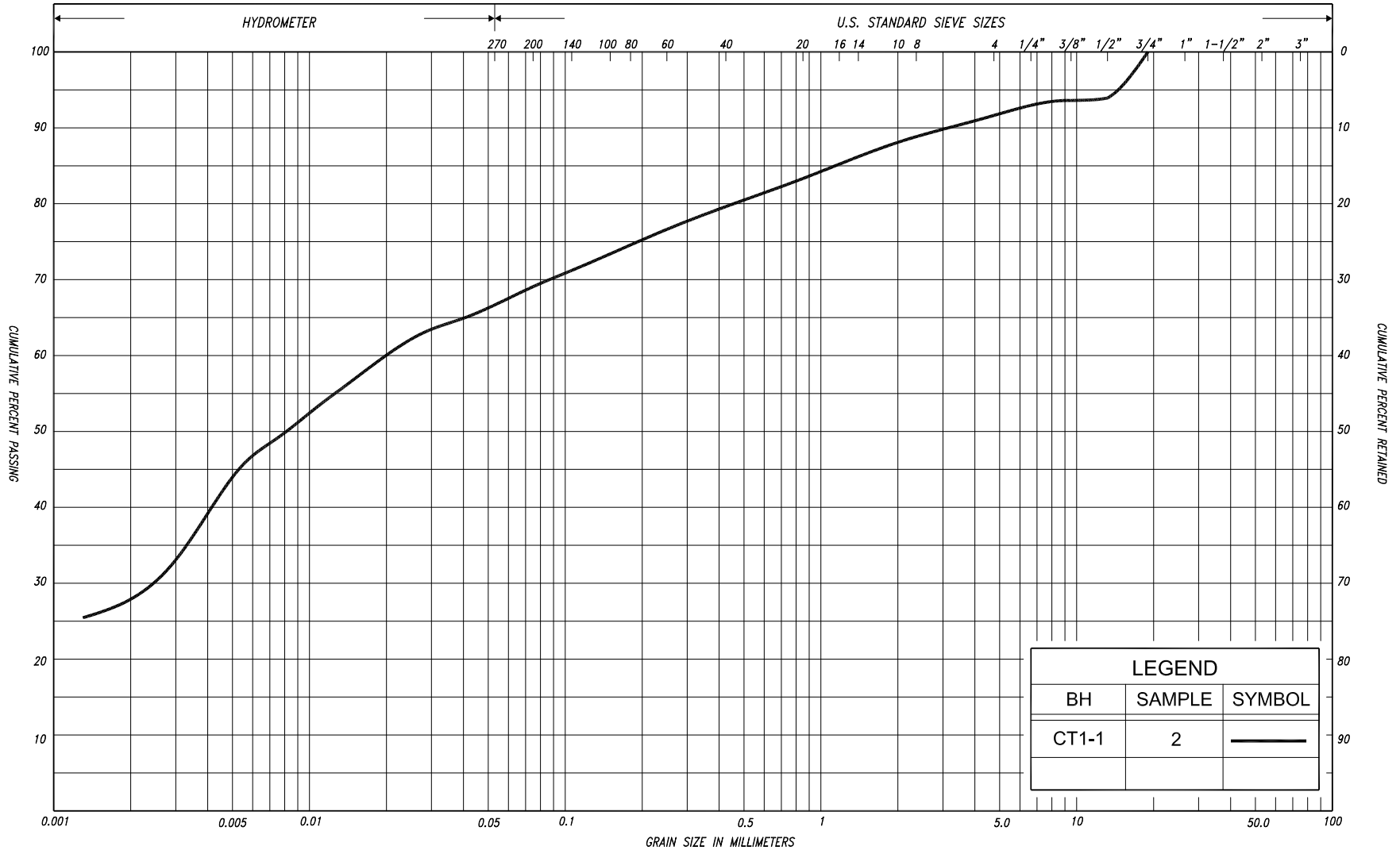


Grigory Degil, PhD, P.Eng.
Senior Foundation Engineer

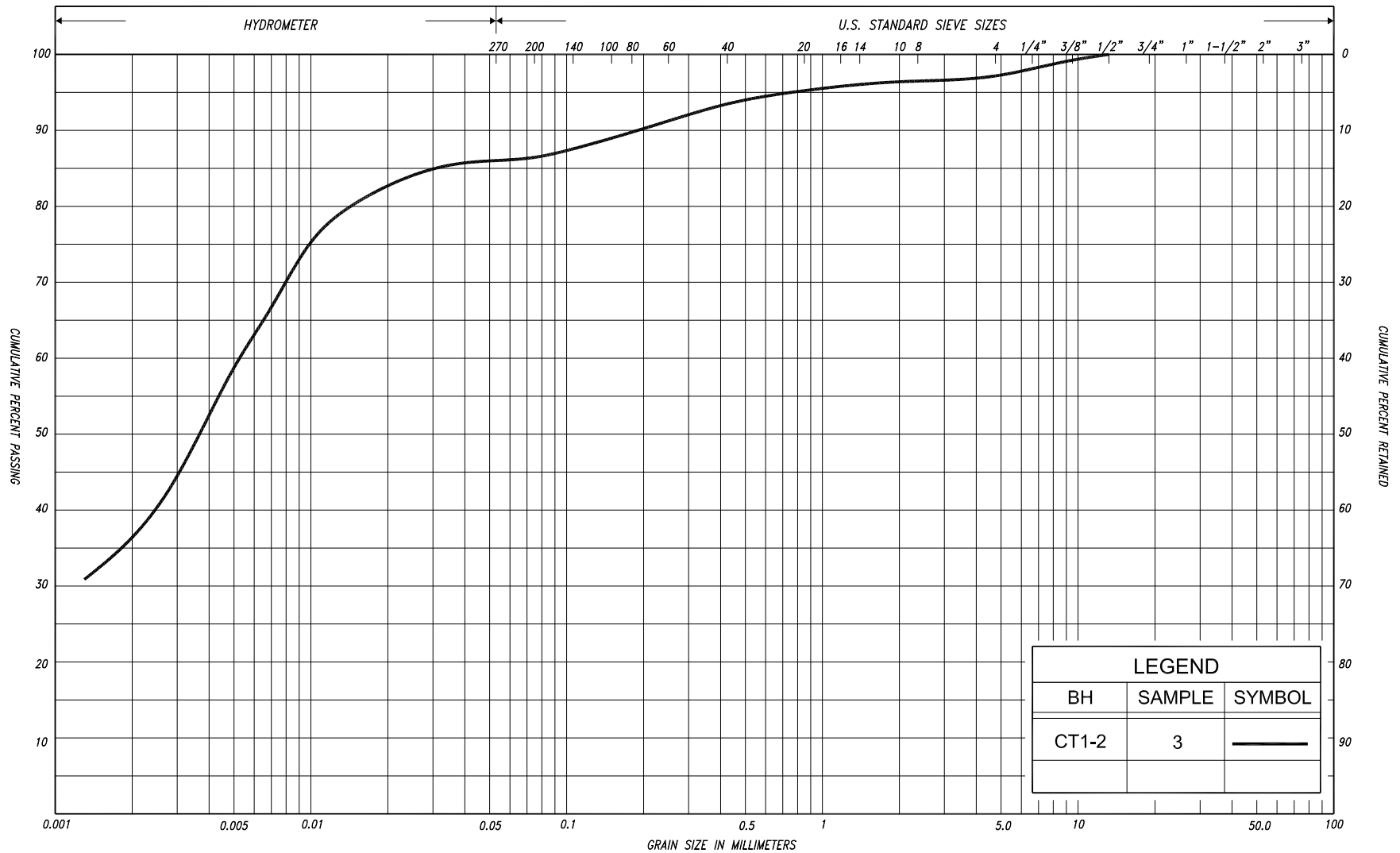


Carlos M.P. Nascimento, P.Eng
Project Manager and
MTO Designated Principal Contact

HG/GD/CN:hg-nk



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



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

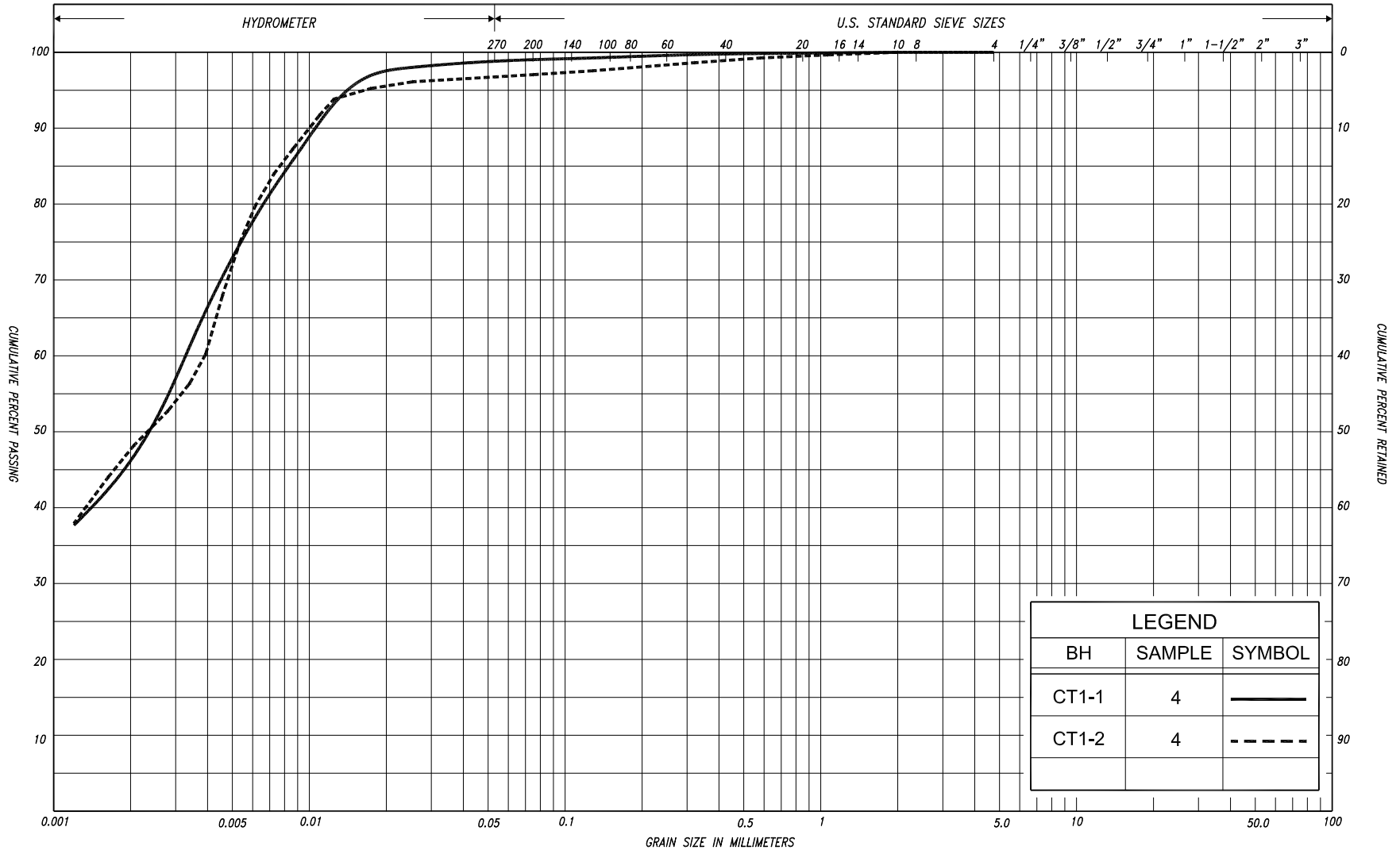


GRAIN SIZE DISTRIBUTION
SILTY CLAY, some sand, trace gravel, organics (CI/MH)
(FILL)

FIG No. CT1-GS-2

HWY: 6

G.W.P. No. 43-00-00



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



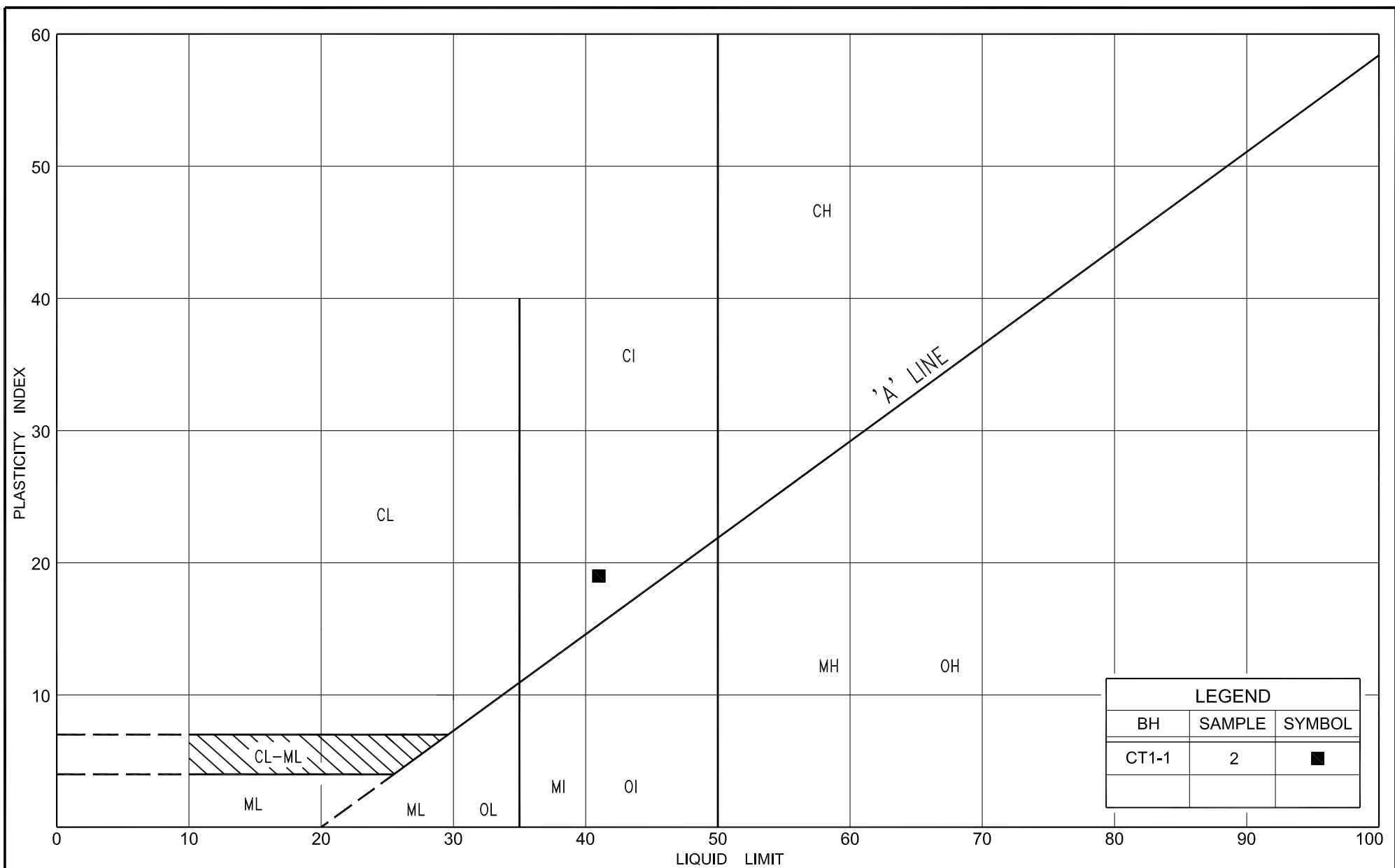
GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand (CL-CI)

FIG No. CT1-GS-3

HWY: 6

G.W.P. No. 43-00-00

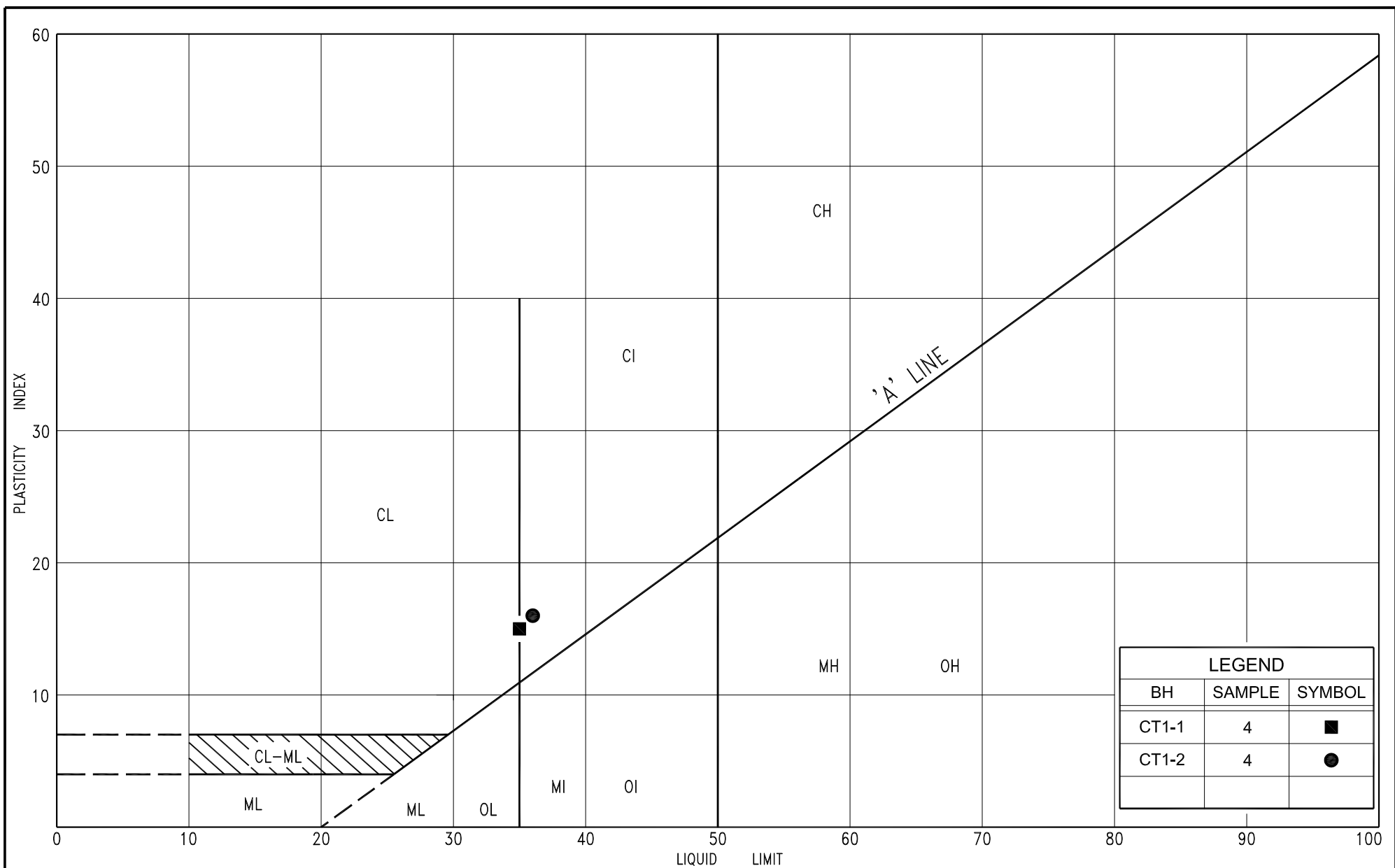


PLASTICITY CHART
SILTY CLAY, with sand, trace gravel (CI)
(FILL)

FIG No. CT1-PC-1

HWY: 6

G.W.P. No. 43-00-00



PLASTICITY CHART

SILTY CLAY, trace sand (CL-CI)

FIG No. CT1-PC-3

HWY: 6

G.W.P. No. 43-00-00

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.

COMPOSITION: SECONDARY SOIL COMPONENTS ARE DESCRIBED ON THE BASIS OF PERCENTAGE BY MASS OF THE WHOLE SAMPLE AS FOLLOWS:

PERCENT BY MASS	0 - 10	10 - 20	20 - 30	30 - 40	> 40
	TRACE	SOME	WITH	ADJECTIVE (SILTY)	AND (AND SILT)

CONSISTENCY: COHESIVE SOILS ARE DESCRIBED ON THE BASIS OF THEIR UNDRAINED SHEAR STRENGTH (c_u) AS FOLLOWS:

c_u (kPa)	0 - 12	12 - 25	25 - 50	50 - 100	100 - 200	> 200
	VERY SOFT	SOFT	FIRM	STIFF	VERY STIFF	HARD

DENSENESS: COHESIONLESS SOILS ARE DESCRIBED ON THE BASIS OF DENSENESS AS INDICATED BY SPT N VALUES AS FOLLOWS:

N (BLOWS/0.3m)	0 - 5	5 - 10	10 - 30	30 - 50	> 50
	VERY LOOSE	LOOSE	COMPACT	DENSE	VERY DENSE

ROCKS ARE DESCRIBED BY THEIR COMPOSITION AND STRUCTURAL FEATURES AND / OR STRENGTH.

RECOVERY: SUM OF ALL RECOVERED ROCK CORE PIECES FROM A CORING RUN EXPRESSED AS A PERCENT OF THE TOTAL LENGTH OF THE CORING RUN.

MODIFIED RECOVERY: SUM OF THOSE INTACT CORE PIECES, 100mm+ IN LENGTH EXPRESSED AS A PERCENT OF THE LENGTH OF THE CORING RUN. THE ROCK QUALITY DESIGNATION (RQD), FOR MODIFIED RECOVERY, IS:

RQD (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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
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_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	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 CT1-1

1 of 1

METRIC

G.W.P. 43-00-00 **LOCATION** Co-ords: 4 936 978.6 N ; 424 607.9 E **ORIGINATED BY** A.L.
DIST London **HWY** 6 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** H.G.
DATUM Geodetic **DATE** January 12, 2012 **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 (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100					w _p	w	w _L					
								SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
235.9 0.0	Ground Surface																			
235.3 0.6	Sand and gravel		1	SS	21															
	Compact Brown (FILL)																			
	Silty clay with sand, trace gravel rootlets, topsoil inclusions		2	SS	3															
	Soft Dark moist grey (FILL)		3	SS	2															
233.8 2.1	Silty clay, trace sand																			
	Firm Greyish Moist brown		4	SS	8															
	Grey		5	SS	7															
231.8 4.1	End of borehole																			
	Refusal on probable bedrock																			
	* 2012 01 12																			
	Water level measured after drilling																			

RECORD OF BOREHOLE No CT1-2

1 of 1

METRIC

G.W.P. 43-00-00	LOCATION	Co-ords: 4 936 967.4 N ; 424 619.5 E	ORIGINATED BY A.L.
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DIST	London	HWY	6	BOREHOLE TYPE	Continuous Flight Hollow Stem Augers	COMPILED BY	H.G.
-------------	--------	------------	---	----------------------	--------------------------------------	--------------------	------

DATUM Geodetic **DATE** January 12, 2012 **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV. DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED	● QUICK TRIAXIAL	+ FIELD VANE	× LAB VANE									
235.8 0.0	Ground Surface						20	40	60	80	100									
235.2 0.6	Sand and gravel		1	SS	12															
	Compact Brown (FILL)																			
	Sand and gravel mixed with asphalt pieces		2	SS	15															
	Silty clay, some sand trace gravel, rootlets topsoil, organic inclusions		3	SS	2															
233.7 2.1	Soft Dark moist grey (FILL)																			
	Silty clay, trace sand		4	SS	8															
	Firm Grey Moist		5	SS	7															
231.2 4.6	End of borehole		6	SS	50/0cm															
	Refusal on probable bedrock																			

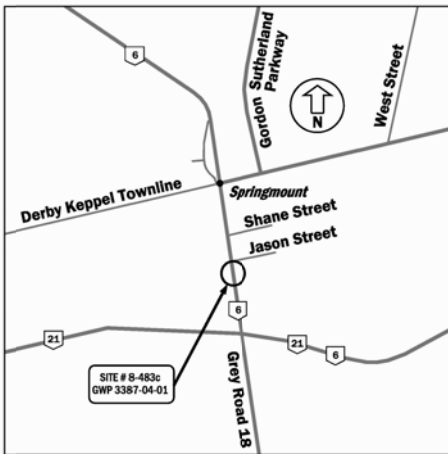
CONT No
GWP No 43-00-00



POTTAWATOMI RIVER TRIBUTARY CULVERT
HIGHWAY 6 Sta. 10+445
SPRINGMOUNT
BOREHOLE LOCATIONS AND SOIL STRATA

SHEET

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



KEY PLAN

NOT TO SCALE

LEGEND

- Borehole
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL at time of investigation Jan. 2012
- * Water level not established
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
CT1-1	235.9	4 936 978.6	424 607.9
CT1-2	235.8	4 936 967.4	424 619.5

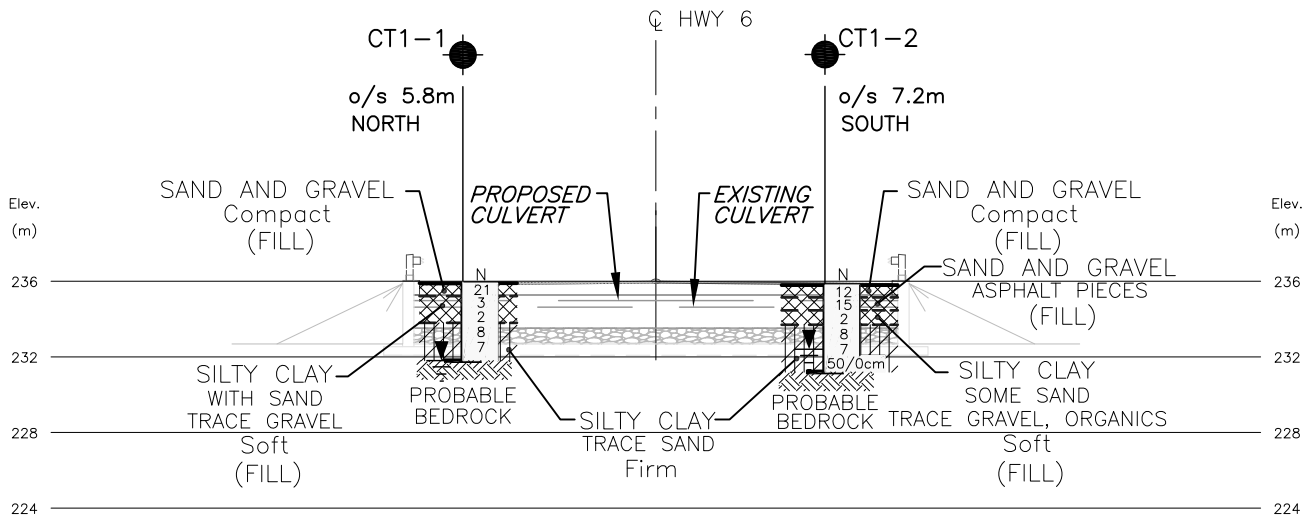
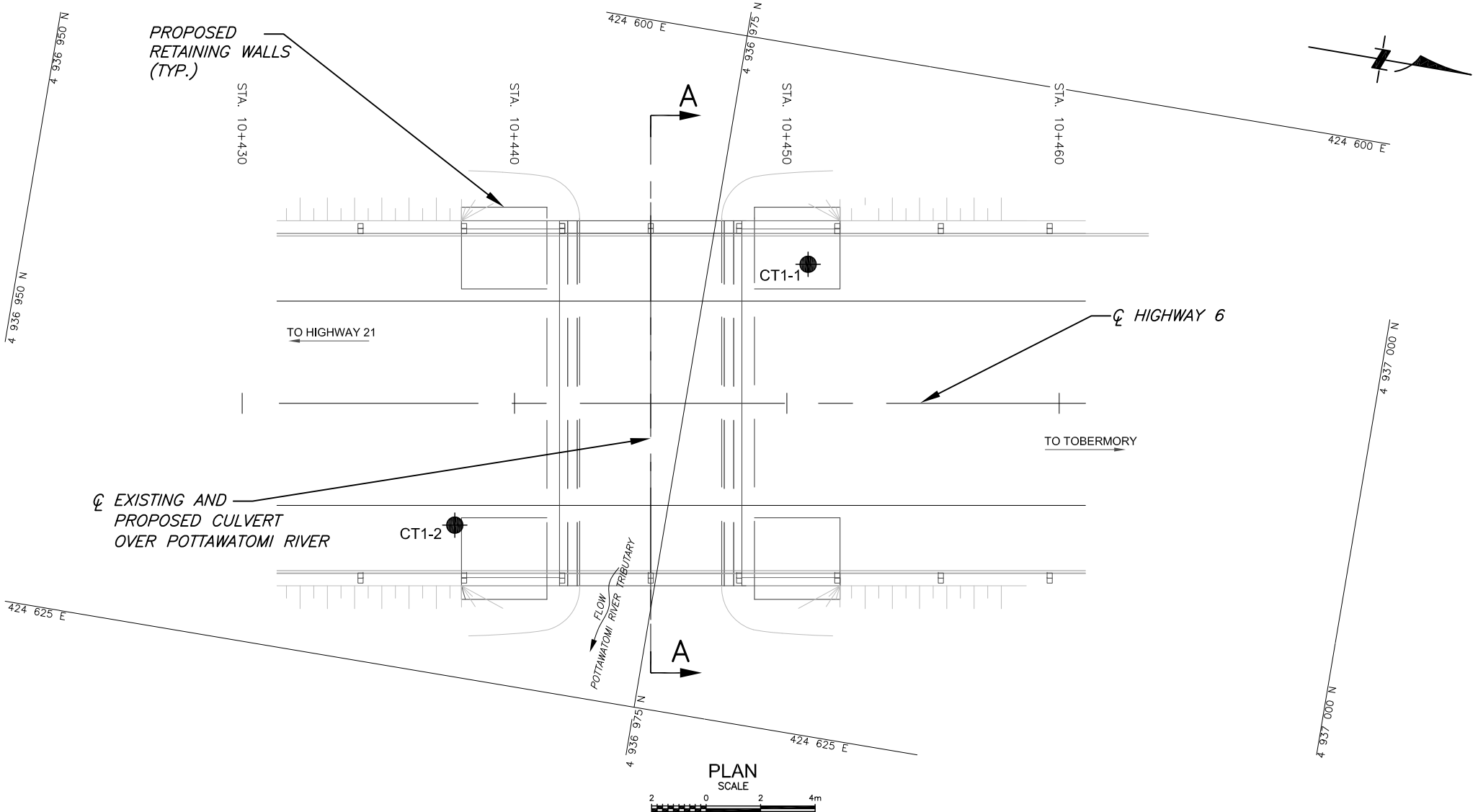
NOTE

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

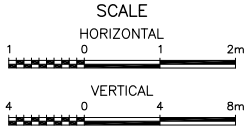
REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 41A-225

HWY No	6	CHECKED HG	DATE NOV. 16, 2012	DIST	London
SUBM'D	NA	CHECKED GD	APPROVED CN	SITE	8-483C
DRAWN	NA	CHECKED GD	APPROVED CN	DWG	PRT-1



PROFILE A-A ALONG CULVERT C



NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF THE REPORT AND RECORD OF BOREHOLE LOGS.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



REF MRC Drawing: 3811011-340-001GA_D.dwg dated May 2012



APPENDIX A

Site Photographs



Photograph 1: Facing north from east side of Highway 6 towards culvert.
(January 2012)



Photograph 3: Looking south from west side of Highway 6 towards the culvert.
(January 2012)



**FOUNDATION DESIGN REPORT
for
POTTAWATOMI RIVER TRIBUTARY CULVERT (SITE NO. 8-483C)
HIGHWAY 6, STA. 10+445
SPRINGMOUNT
G.W.P. 43-00-00
DISTRICT OF LONDON, ONTARIO**

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PML Ref.: 11KF065A-C1
Index No. 115FDR
GEOCRES No. 41A - 225
November 21, 2012



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Table 1 – List of Standard Specifications Referenced in Report

FOUNDATION DESIGN REPORT

for

Pottawatomi River Tributary Culvert (Site No. 8-483C)
Highway 6, Sta. 10+445
Springmount
G.W.P. 43-00-00
District of London, Ontario

1. INTRODUCTION

The installation of a 6.1 x 2.4 m precast concrete open footing culvert at Station 10+445 on Highway 6 (Pottawatomi River Tributary culvert) is planned as part of the rehabilitation of Highway 6, from Springmount to Hepworth. This report was prepared for McCormick Rankin (MRC), a member of MMM Group Ltd., on behalf of the Ministry of Transportation of Ontario (MTO).

This report provides foundation engineering comments and recommendations for design and construction of the new Pottawatomi River Tributary culvert at Station 10+445 and a temporary roadway protection which will be required for the staged construction.

The culvert will replace the existing culvert at Sta. 10+445 on Highway 6, in the Township of Springmount. The proposed new culvert will be 6.1 x 2.4 m precast concrete open footing culvert with a total length of about 13.4 m. The retaining walls at the four corners will be constructed using temporary roadway protection while maintaining two lanes of traffic.

In summary, the subsurface stratigraphy revealed in the boreholes generally comprised a road embankment fill underlain by native silty clay. Bedrock was inferred by auger refusal at 4.1 and 4.6 m, elevation 231.8 and 231.2, respectively. Groundwater was observed in the boreholes at depths of 4.1 and 3.4 m, elevation 231.8 and 232.4, on completion of drilling.

The foundation frost penetration depth at the site is 1.4 m according to OPSD 3090.101.

It is understood that a replacement precast concrete open footing culvert is proposed at the same culvert location. Therefore, recommendations for a cast-in-place concrete culvert and a discussion of the advantages and disadvantages of the two culvert options are not included in this



report. Although the GA drawing indicates that the culvert will be an open footing culvert, MRC has requested recommendations for both open footing and closed box options.

Since the replacement culvert is planned at the same location as the existing culvert, the foundation subgrade has been preloaded with the weight of the previous culvert which will mitigate settlements. When removing the existing culvert, care should be taken not to disturb the founding subgrade to minimize the preparation needed for placing the new culvert. It is recommended that the entire existing culvert together with the footing be removed to provide an uniform subgrade for support of the new box culverts.

It is noted that no responsibility or liability is assumed by the consultants and MTO for alerting the contractors, 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. POTTAWATOMI RIVER TRIBUTARY CULVERT (NEW) – STA. 10+445

2.1 Foundations

2.1.1 Open Footing Culvert

The invert level of the new open footing culvert is specified to be near elevation 233.5 (top of substrate material) at the inlet and the culvert will have a 0.2% grade down to the outlet. The culvert will be countersunk with approximately 400 mm depth of substrate inside. The invert of the culvert at the inlet is specified at elevation 233.1 while the bottom of the retaining wall footings will be at approximate elevation 232.1. The proposed road grade elevation at the new culvert will be about 236.0, indicating that there will be a minimum of 600 mm soil over the top of the culvert with about 0.8 m soil cover at the centreline of the road.



The subgrade soils revealed in the boreholes at the culvert substrate invert level (elevation 233.5) and at the four retaining walls subgrade level (elevation 232.1) comprised 0.3 to 0.9 m thick firm silty clay over assumed bedrock at both ends of the culvert.

Groundwater was observed at 4.1 and 3.4 m depth, elevation 231.8 and 232.4 in boreholes CT1-1 and CT1-2 on completion of drilling.

Based on the encountered subsurface condition, the culvert footings can be adequately supported on the firm silty clay or on bedrock.

The recommended factored geotechnical bearing resistance at ultimate limit states (ULS) and geotechnical reaction at serviceability limit states (SLS) for the proposed concrete spread footings constructed on the native firm silty clay are as follows:

CULVERT SECTION	SUBGRADE SOIL TYPE	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL REACTION AT SLS (kPa)
Entire Length	Firm silty clay	150	100

The geotechnical reaction at SLS normally allows for 25 mm compression of the founding medium (firm silty clay). The provided geotechnical bearing resistances on clayey soils are independent of footing width for this project. The minimum footing width will be governed by structural requirements.

If a higher bearing resistance is required, the excavation may be extended to bear on assumed bedrock, however excavations of 0.3 to 0.9 m below the currently proposed founding elevation will be required to achieve bearing on bedrock. The recommended factored geotechnical bearing resistance at ultimate limit state (ULS) for footings bearing on the bedrock is 1,000 kPa. A lower bearing resistance than would normally be available for bedrock is provided since the bedrock depth and quality were not confirmed by coring.



The geotechnical reaction at SLS is not applicable for footings founded on bedrock, since the bedrock is considered to be non-yielding. For structural computation purpose the geotechnical reaction at SLS may be taken as 1,000 kPa.

The estimated total and differential settlement of culvert foundations bearing on bedrock will be negligible.

2.1.2 Precast Concrete Box Culvert

If a closed precast box culvert is selected, the assumed invert level will be at elevation 232.6. At this elevation firm silty clay was contacted in both boreholes. A closed box culvert supported on the firm silty clay can be designed for the bearing resistance provided in Section 2.1.1.

2.2 General Comments

2.2.1 Subgrade Preparation

Preparation of the subgrade for construction of the culvert should be performed and monitored in accordance with OPSS 902. A site review should be conducted by qualified geotechnical personnel during preparation of the subgrade and compaction of the granular fill.

Deleterious or disturbed soils revealed during the preparation at or below the footing subgrade should be excavated and replaced with compacted Granular A or Granular B Type II. Granular B Type II should be preferred for construction under wet conditions.

A minimum 300 mm thick granular bedding (OPSS Granular A) is recommended below the closed box culvert.

The backfill and bedding material should be compacted to 100% of the ASTM D-698 (standard Proctor) maximum dry density in conformance to OPSS 501 (Method A).



The geometry of the subgrade preparation, cover backfill (if applicable) and frost taper treatment for the precast culvert should be carried out in accordance with MTOD 803.021, OPSS 422 and MTO SP 422S01.

2.2.2 Modulus of Subgrade Reaction

The estimated value of the modulus of subgrade reaction for a closed box culvert constructed on the undisturbed subgrade native firm silty clay at elevation 232.6 is as follows:

SOIL TYPE	MODULUS OF SUBGRADE REACTION, MN/m ³
Native firm silty clay	10

2.2.3 Sliding Resistance

The following parameters should be used to compute sliding resistance of precast box culvert if applicable and cast-in-place footings, including headwalls and wing walls. The friction angles have been reduced by a factor of 0.67 for precast box culvert foundations to account for the smooth concrete base.

SOIL TYPE	FOUNDATION FRICTION ANGLE, DEGREES		COHESION, kPa	UNIT WEIGHT, kN/m ³
	CAST-IN-PLACE	PRECAST		
Granular A or Granular B Type II	35	23	0	22.8
Firm Silty Clay	12	8	50	20.0
Bedrock (Rough surface)	45	30	0	23.5

The structural designer should use a factor of 0.8 for the friction angle and cohesion values when performing the sliding resistance check.

2.2.4 Seismic Site Coefficient

The seismic site coefficient for the conditions at the subject site is 1.0 – Type I soil profile as per clause 4.4.6 of the CHBDC.



3. CULVERT AND RETAINING WALL BACKFILL

Backfill adjacent to the culverts should be placed in accordance with OPSS 422 and MTO SP 422S01 and backfill adjacent to headwalls and wing walls should be placed in accordance with OPSD 3121.150. Requirement for frost taper is provided in the Pavement Design Report.

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. Refer to MTO OPSS 501 for additional comments.

The new 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) or employing the following equation assuming a triangular pressure distribution:

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

where p = lateral earth pressure (kPa)

K = lateral earth pressure coefficient

γ = unit weight of backfill material above design water level (kN/m³)

γ' = unit weight of submerged backfill material below design water level (kN/m³)
= $\gamma - \gamma_w$

γ_w = unit weight of water
= 9.8 kN/m³

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)

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

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

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



The seismic site coefficient for the conditions at this site was provided in Section 2.2.4.

The following parameters are recommended for estimating the earth pressure for granular backfill:

PARAMETER	GRANULAR A OR GRANULAR B (Type II)
Angle of Internal Friction, degrees	35
Unit Weight, kN/m ³	22.8
Coefficient of Active Earth Pressure (K_a)	0.27
Coefficient of Earth Pressure At Rest (K_o)	0.43
Coefficient of Passive Earth Pressure (K_p)	3.69

The design should consider both the maximum water level in the stream and the stabilised groundwater level condition. The maximum stream water level will be dictated by flood flow conditions and should be defined by the project hydrological engineer.

The coefficient of earth pressure at rest should be employed to design rigid and unyielding walls.

4. HEADWALLS AND WING WALLS/RETAINING WALLS

For headwalls and wing walls or retaining walls design, the previous recommendations and geotechnical parameters for culvert foundations and backfill should be utilized for the design of the foundations. The wall founding levels should match those of the culvert where the walls are designed integral with the culvert structure. For walls designed separately from the culvert structure, the founding levels should be established with 1.4 m of earth cover for adequate frost protection.

The design of the walls should be checked for sliding resistance using the geotechnical parameters provided previously in Section 2 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 walls. 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.

5. CONSTRUCTION CONSIDERATIONS

5.1 Staged Construction

MRC is proposing a five day road closure remove the existing culvert and to install the new open footing culvert. Following this, the cast in place retaining walls would be constructed using roadway protection of all four corners while maintaining two lanes of traffic on Highway 6.

5.2 Roadway Protection

It is anticipated that a suitable roadway protection scheme following OPSS 539 will be necessary to support the walls of the excavation during construction of the retaining wall at all four corners.

A roadway protection system designed for performance level 2 system according to OPSS 539 is recommended to prevent excessive lateral and/or vertical movement of the existing embankment during construction of each new retaining wall. The contractor is responsible for the selection, performance and detailed design of the roadway protection scheme(s). The contractor should monitor the movement of the roadway protection system. To meet the performance Level 2, the maximum lateral displacement is limited to 25 mm with a maximum allowable angular distortion of 1:200.

In case excessive movement is experienced in the roadway protection system, a monitoring system should be implemented to check the horizontal and vertical displacements of the adjacent roadway surface during construction. A maximum of 12 mm of settlement should be allowed on the travelled highway section to be used as required.

Alternative roadway protection schemes such as sheet piling or anchored soldier piles and lagging were considered. Typically, sheet piling can be used to reduce loss of native soils below the



water table. Soldier piles and lagging are generally considered suitable for applications above groundwater table in cohesionless soils.

The following table presents an overview assessment of the advantages and disadvantages, including relative costs and risk/consequences of the roadway protection system alternatives from the foundation perspectives at the subject site.

ALTERNATIVES	ADVANTAGES	DISADVANTAGES
Sheet piles	<ul style="list-style-type: none"> • Sheet piles will be interlocked therefore loss of native soils will be negligible • Suitable for high water table • Suitable to drive for varying bedrock profile, if required • Low risk of soil loss 	<ul style="list-style-type: none"> • Higher cost than for soldier piles • May require soil anchors/rakers for lateral support • Larger construction equipment is required than for soldier piles
Soldier piles and lagging	<ul style="list-style-type: none"> • Lower cost than for sheet piles • Smaller construction equipment is required than for sheet piles 	<ul style="list-style-type: none"> • Excessive settlement may occur due to loss of cohesionless soils/ fill materials with high water table • Unsuitable with high water table • High risk of soil loss

Based on the above table and considering the low groundwater found at the site, an anchored or braced soldier pile and lagging system is considered feasible at the site. The presence of debris in the fill and cobbles and boulders in the native soils must be considered during installation of soldier piles.



5.3 Excavation

Excavation to the anticipated founding level of the new culvert is expected to extend through the fill and native silty clay. Subject to adequate groundwater control, excavation of the soils should be feasible using conventional equipment. All excavations should be conducted in accordance with OPSS 902.

According to OSHA criteria, soft silty clay fill materials are classified as Type 4 soils. The compact sand and gravel fill and the firm silty clay are considered as Type 3 soils. Since open cut procedures are governed by soils with the highest soil type number, temporary cut slopes over the full depth of excavation inclined at 3 horizontal to 1 vertical should be provided assuming adequate groundwater control measures are in place. Where space restrictions exist, a shoring system should be used.

5.4 Groundwater Control

Groundwater was observed at 4.1 and 3.4 m depth, elevation 231.8 and 232.4 in boreholes CT1-1 and CT1-2 on completion of drilling. These levels are at or immediately above the founding subgrade levels, elevation 232.1.

It is anticipated that, during the dry summer months, conventional procedures such as dam and pump will be sufficient to dewater the foundation excavation, including the removal of the existing culvert. These measures will likely be inadequate during the relatively wetter remainder of the year and additional groundwater control from sumps may be required. The contractor should be made responsible for the design and operation of the dewatering system.



6. EROSION CONTROL

The protective measures noted in the OPSD 800 series to deal with erosion (inlet/outlet treatment, headwalls, cut-off walls etc.) are considered to be appropriate. The backfill should comprise OPSS Granular A or Granular B Type II. The cut-off walls should extend laterally to protect the granular backfill material and 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. The requirements of CHBDC clauses 1.9.5.6 and 1.9.11.6.5 should be applied.

Inlet and outlet protection in accordance with OPSS 511 and 1004 and OPSD 810.010 is recommended to prevent erosion adjacent to the culvert as well as scour that could undermine the culvert foundation. The actual design requirements concerning the length and width of aprons at the inlet/outlet of the culvert as well as the rock size, apron thickness, height of erosion protection on the embankment slope and type of material (clay seals at the inlet, drainage and/or filter blankets at the outlet) 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 minimize the potential for erosion of fine particles from below the treatment.

Any newly constructed embankment slopes and retained soils behind the headwalls and wing walls should be covered with topsoil or suitable excess earth material and seeded in accordance with OPSS 802 and 804, 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 803) may be placed instead of erosion control blankets. Additional appropriate erosion control measures for the project should be assessed using the following erodibility K factor.

SOIL TYPE	K FACTOR
Sand and gravel fill	0.18
Silty Clay fill with organics	0.22
Clayey Silt fill	0.35
Native Silty Clay	0.38



7. CLOSURE

This Foundation Investigation Report was prepared by Mr. H. Gharegrat, P.Eng., and reviewed by Mr. G. Degil, PhD, P.Eng., Senior Foundation Engineer. Mr. C. M. P. Nascimento, P. Eng., Project Manager and MTO Designated Principal Contact, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



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HG/GD/CN:hg-nk



TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 422	Construction Specification for Precast Reinforced Concrete Box Culverts and Box Sewers in Open Cut
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS 539	Construction Specification for Temporary Protection Systems
OPSS 802	Construction Specification for Topsoil
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Excavation and Backfilling of Structures
OPSS 1004	Material Specification for Aggregates - Miscellaneous
OPSS 1860	Material Specification for Geotextiles
SP 422S01	Construction Specification for Precast Concrete Box Culvert
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets
OPSD 3090.101	Foundation Frost Depth for Southern Ontario
OPSD 3121.150	Minimum Granular Backfill Requirements - Walls Retaining
MTOD 803.021	Bedding and Backfill for Precast Concrete Box Culverts