



**FOUNDATION INVESTIGATION AND DESIGN REPORT
for
MCGILLVRAY CREEK TRIBUTARY CULVERT
WEST SERVICE ROAD STATION 10+687
SITE NO. 44-508/C
TOWNSHIP OF SOUTH HIMSWORTH
NORTH BAY AREA, ONTARIO
G.W.P. 323-00-00**

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PML Ref.: 10TF013A-C4
Index No.: 324FIR and 325FDR
GEOCRES No.: 31L-164
January 11, 2013



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FOUNDATION INVESTIGATION REPORT

for
McGillvray Creek Tributary Culvert
West Service Road Station 10+687
Site No. 44-508/C
Township of South Himsworth
North Bay Area, Ontario
G.W.P. 323-00-00

1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed West Service Road Culvert for the Tributary of McGillvray Creek as part of the south entrance to Powassan project which extends from 5.7 km south of the Highway 534 northerly 5.0 km. The study was carried out by Peto MacCallum Ltd. (PML) for AECOM Canada Ltd (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

The culvert is at approximate Station 10+687, West Service Road chainage, in the Township of South Himsworth (ref. General Arrangement Drawing 'West Service Road Box Culvert' prepared by AECOM dated December 2010).

The purpose of this report was to summarize the subsurface stratigraphy encountered in the foundation investigation at the proposed culvert.

2. SITE DESCRIPTION AND GEOLOGY

The contemplated new culvert will be located on the proposed West Service Road about 450 m south of the existing Purdon Line and about 500 m west of the existing Highway 11 Southbound Lanes. The site is about 34 km south of the City of North Bay in the Geographic Township of South Himsworth.

Land use in the vicinity of the site includes the existing gravel pit to the west and farming land with livestock to the east. The local topography of the site is generally flat sloping to the west. The Creek flows approximately in an east to west direction at the proposed West Service Road. The ground cover includes grasses and bushes near the creek area and stands of trees elsewhere.



The project site is located within the physiographic region known as the Number 11 Strip. The soil cover at the project site is from sandy glaciolacustrine deposits which overlies Precambrian age monzonitic (granitic) rock formation.

3. INVESTIGATION PROCEDURES

The subsurface investigation was carried out on December 22, 2011 and January 10 and 11, 2012. A total of two boreholes (CW-1 and CW-2) were drilled to 10.4 and 12.5 m at the locations shown on Drawing WS-1, appended. In addition, dynamic cone penetration tests were carried out 3 m east of borehole CW-1 and 2.0 m east of borehole CW-2 to 11.9 and 12.5 m at the locations shown on Drawing WS-1. The planned inlet borehole at the east end of the culvert was located in the area of the private property that was protected by an electric cattle fence and was not carried out due to lack of permission to enter at the time of investigation. Although the results of the investigation are considered representative due to the uniform 11.9 and 12.5 m thick soil cover in the two drilled boreholes, allowances should be made for local variations in subsurface stratigraphy.

The culvert control points were staked in the field by exp Geomatics according to the GA Drawing dated December 2010 prepared by AECOM. The positions of the boreholes relative to the culvert control points were selected by PML allowing for drill rig accessibility. The ground surface elevations at the borehole locations were established by PML using the ground surface elevations at the control points. All elevations in this report are expressed in metres.

The boreholes were advanced using continuous flight hollow stem augers through the soil cover with a track-mounted D-120 drill rig, 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 intervals of depth 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 holes.

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

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 (16)
- Atterberg Limits (1)
- Grain size distribution analyses (6)

The laboratory grain size distribution charts are presented in Figures CW-GS-1 to CW-GS-3. The Atterberg Limits results are presented in Figure CW-PC-1. All of the test results are summarized 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, bedrock description, inferred stratigraphy, standard penetration test results and groundwater observations. The results of laboratory grain size distributions, Atterberg Limits and moisture content determinations are also shown on the Record of Borehole Sheets.

The borehole locations, stratigraphic profile and cross-sections prepared from the borehole data are shown on Drawing WS-1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.



The subsurface stratigraphy revealed in the boreholes generally comprised of a topsoil layer underlain by cohesionless deposits of sand and silt over cohesionless silty sand /sand deposits. A 1.1 m thick clayey silt was encountered in borehole CW-2 below the topsoil. Cobbles and boulders were encountered within the sand deposit. Probable bedrock was inferred by refusal at 11.9 and 12.5 m (elevation 244.7 and 244.1). Groundwater was observed in both boreholes.

4.1 Topsoil

A 300 and 600 mm thick topsoil was encountered surficially in both boreholes extending to 0.3 and 0.6 m (elevation 256.0 and 256.3). The sandy topsoil had a moisture content of 25%. Organic soils (alluvium) should also be anticipated within the McGillivray Creek Tributary.

4.2 Clayey Silt

Below topsoil in borehole CW-2, a 1.1 m thick clayey silt layer was locally encountered at 0.3 m (elevation 256.3) extending to 1.4 m (elevation 255.2). N values of 2 and 3 were recorded indicating soft consistency.

4.3 Sand and Silt

A cohesionless sand and silt deposit containing organics was encountered at 0.6 m (elevation 256.0) in borehole CW-1. The unit was 1.6 m thick extending to silty sand at 2.2 m (elevation 254.4). N value of 2 was recorded indicating very loose relative density.

The results of grain size distribution analysis for a sand and silt sample are included in Figure CW-GS-1. The plasticity charts are presented in Figure CW-PC-1. The liquid and plastic limits were 22 and 19, respectively, with the corresponding plasticity index value of 3. The moisture content determinations were 31 and 46% due to organics.



4.4 Silty Sand / Sand

A cohesionless silty sand stratum was encountered at 2.2 and 1.4 m (elevation 254.4 and 255.2) in boreholes CW-1 and CW-2, respectively. The unit was 3.8 and 7.2 m thick extending to 6.0 and 8.6 m (elevation 248.0 and 250.6). N values ranged from 3 to 21, typically in the range of 7 to 14 indicating loose to compact relative density. Low N value of 3 may have been affected by hydraulic disturbance during drilling.

Below the silty sand in both boreholes, a cohesionless sand deposit was encountered at 6.0 and 8.6 m (elevation 248.0 and 250.6). The unit was 3.9 and 4.4 m thick, probably 5.9 m thick in borehole CW-1 extending to the underlying bedrock at 11.9 and 12.5 m (elevation 244.1 and 244.7). N values ranged from 2 to 24 and 50 for 3 cm sampler penetration. The stratum was found to be very loose to compact relative density. Low N value of 2 may have been affected by hydraulic disturbance during drilling.

The results of grain size distribution analysis for silty sand / sand samples are included in Figures CW-GS-2 and CW-GS-3. The moisture content determinations ranged from 12 to 24%.

4.5 Bedrock

Probable bedrock was inferred by auger and/or dynamic cone refusal in both boreholes at 11.9 and 12.5 m (elevation 244.7 and 244.1).

4.6 Groundwater

Groundwater was encountered in both boreholes. During augering, groundwater was observed at 2.1 and 1.5 m (elevation 254.5 and 255.1) in boreholes CW-1 and CW-2, respectively. The groundwater levels at the site is governed by the water level in the McGillvray Creek tributary in view of the typical relatively pervious soil stratigraphy. At the time of the investigation, the water level in the McGillvray Creek Tributary was at about elevation 255.0 and was about elevation 255.5 in January 2011. The groundwater level is subject to seasonal fluctuations and rainfall patterns.



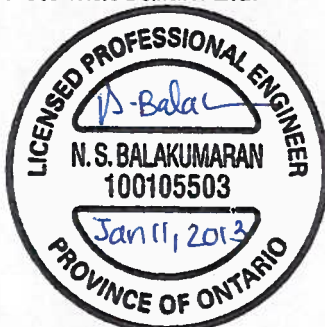
5. CLOSURE

Mr. A. Djirdeh carried out the field investigation for this study under the supervision of Mrs. N.S. Balakumaran, P. Eng., and Mr. C. M. P. Nascimento, P. Eng., Project Manager. Walker 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.

This Foundation Investigation Report was prepared by Mrs. N. S. Balakumaran, P. Eng., and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C. M. P. Nascimento, P. Eng., Project Manager conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



Nesam S. Balakumaran, P.Eng.
Project Engineer

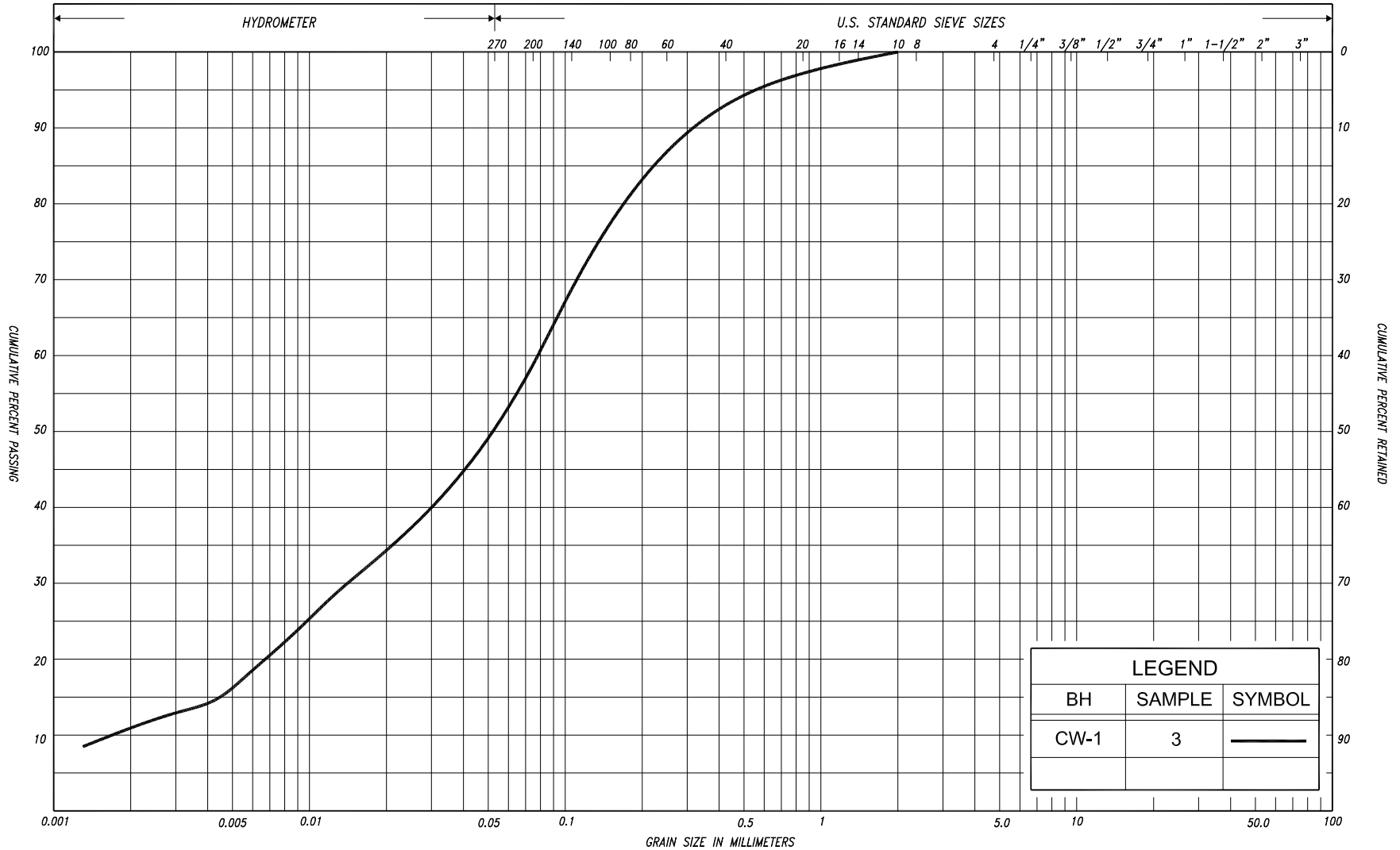


Carlos M.P. Nascimento, P.Eng.
Project Manager

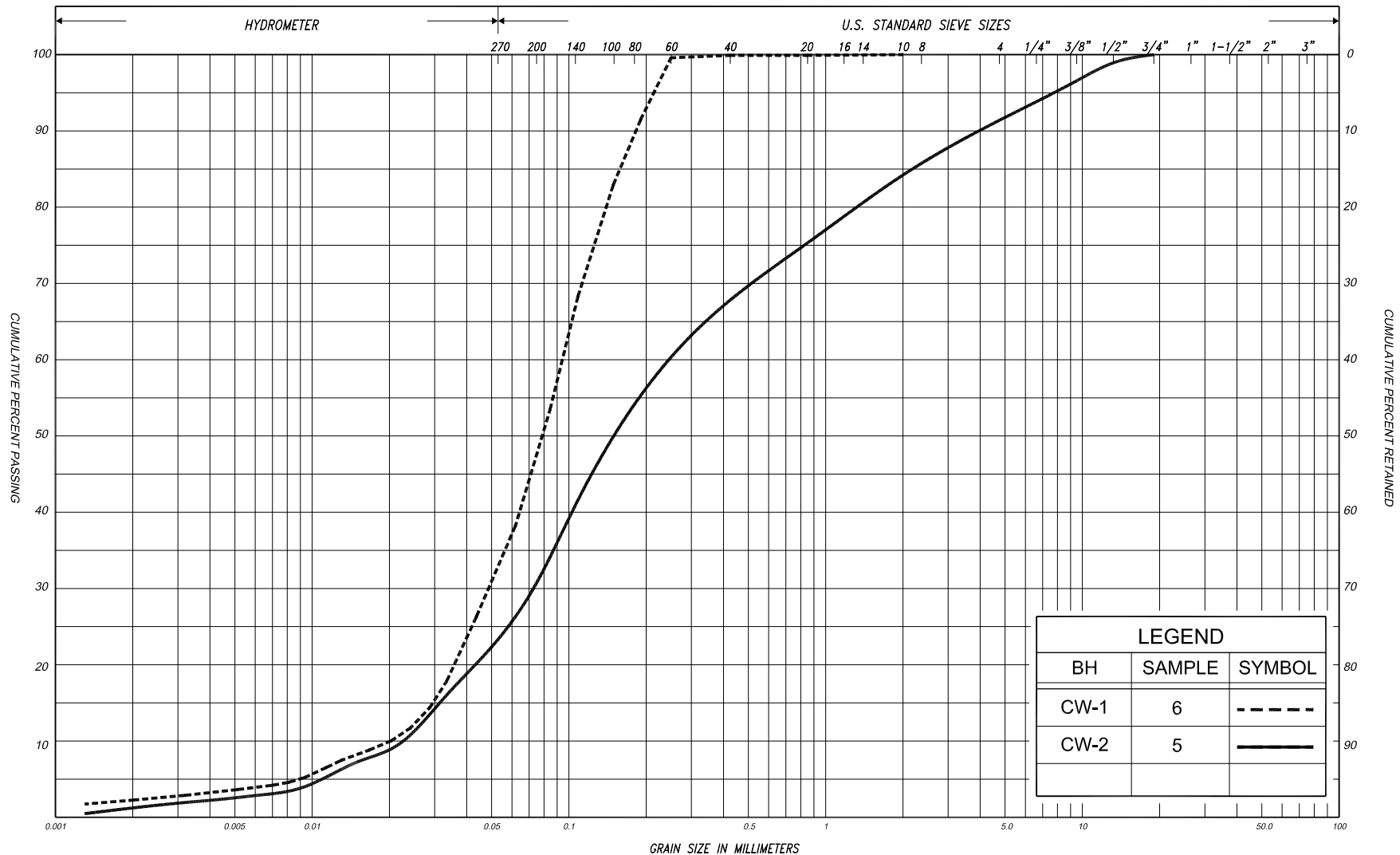


Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

NB/CN/BRG:nb-mi-nk



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



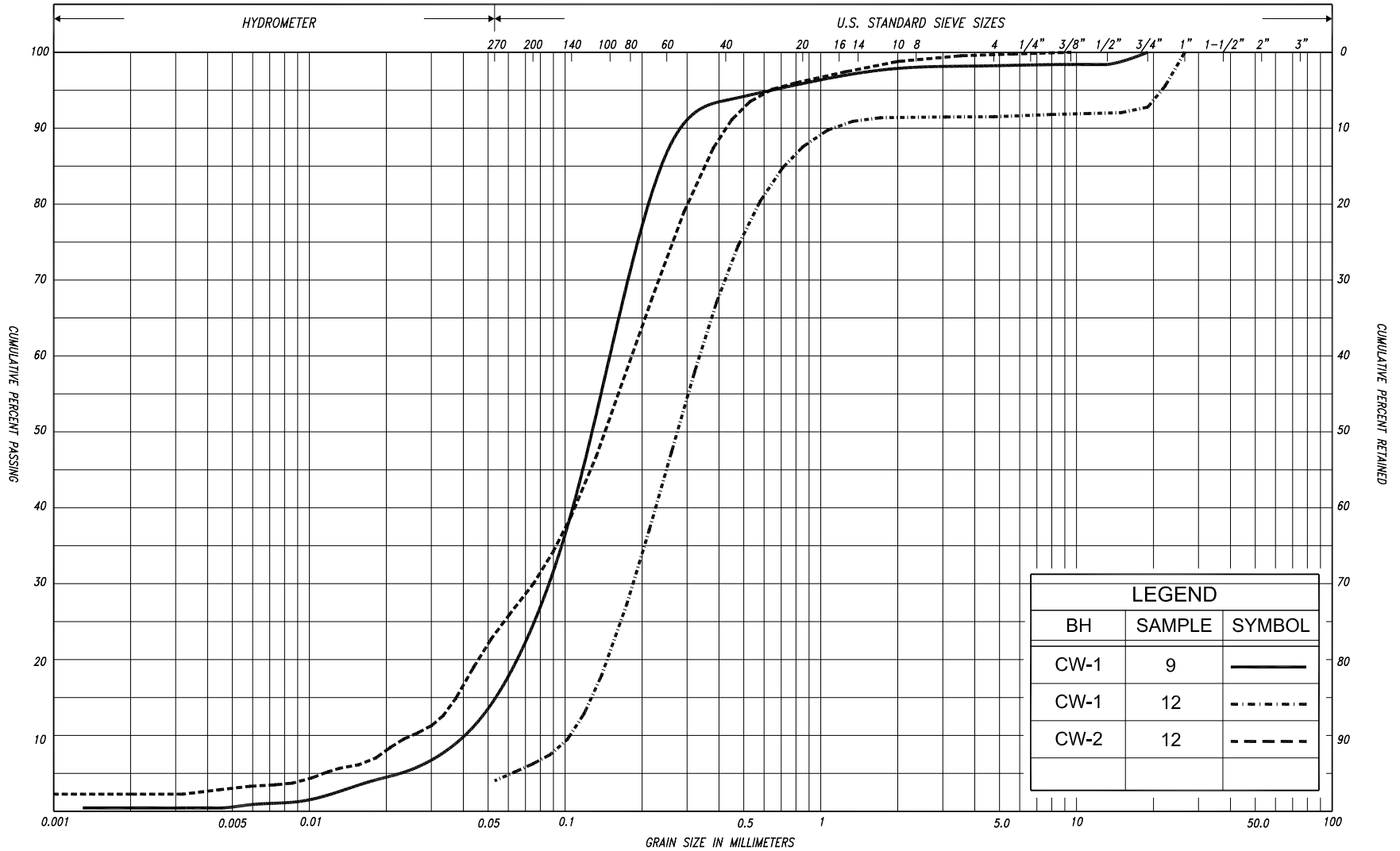
SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL				COB BLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL				COBBLES	M.I.T.
	SILT																
CLAY		SILT			V. FINE	FINE	MED.	COARSE	GRAVEL							U.S. BUREAU	
					SAND												



GRAIN SIZE DISTRIBUTION

SILTY SAND, trace clay, trace gravel

FIG No. CW-GS-2
 HWY: 11
 G.W.P. No. 323-00-00



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



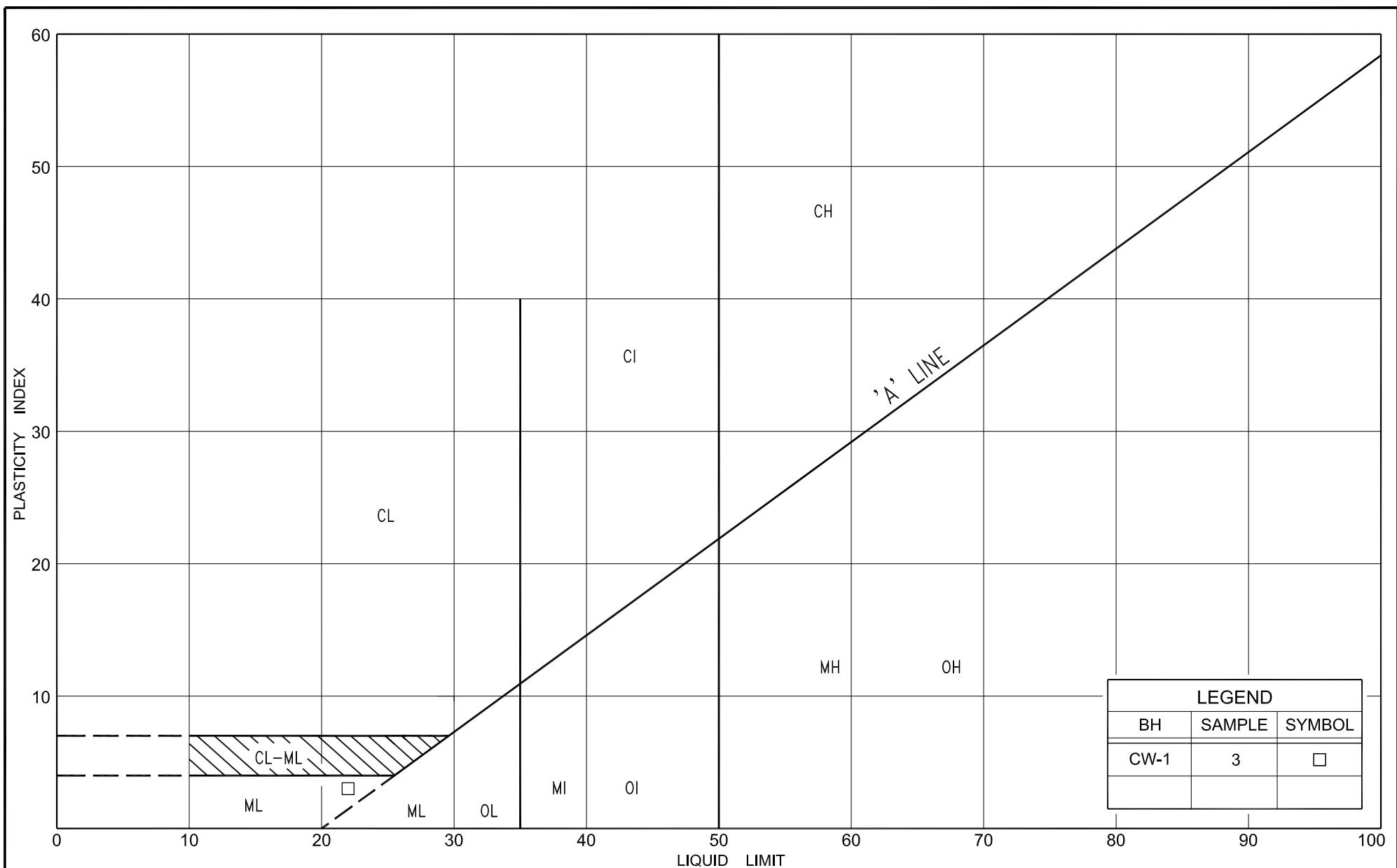
GRAIN SIZE DISTRIBUTION

SAND, trace to with silt, trace clay, trace gravel

FIG No. CW-GS-3

HWY: 11

G.W.P. No. 323-00-00



PLASTICITY CHART
SAND AND SILT, some clay, trace gravel

FIG No. CW-PC-1
HWY: 11
G.W.P. No. 323-00-00

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 (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
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'_{v0}	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	WTP		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No CW-1

1 of 1

METRIC

G.W.P. 323-00-00 **LOCATION** Co-ords: 5 102 463.0 N ; 315 939.5 E **ORIGINATED BY** A.K.
DIST North Bay HWY 11 **BOREHOLE TYPE** C.F.H.S.A. and 'N' Casing + D.C.P.T. **COMPILED BY** N.S.B.
DATUM Geodetic **DATE** December 22, 2011 and January 10, 2012 **CHECKED BY** B.R.G.

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									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
					WATER CONTENT (%)												
256.6	Ground Surface																
0.0	Topsoil		1	SS	4												
256.0	Sand and silt, some clay organics		2	SS	2												
0.6	Very loose Grey Wet		3	SS	2												
254.4	Silty sand, trace clay		4	SS	7												
2.2	Loose to Grey Wet compact		5	SS	10												
	trace gravel, organics		6	SS	10												
	sand seams		7	SS	10												
	Very loose		8	SS	3												
250.6	Sand with silt, trace gravel		9	SS	2												
6.0	Very loose Brown Wet		10	SS	2												
	trace silt		11	SS	21												
	Compact		12	SS	20												
	cobbles and boulders		13	SS	24												
246.2	End of borehole																
10.4	Probable sand																
244.7	End of dynamic cone penetration test																
11.9	Refusal on probable bedrock																
	* 2011 12 22																
	▽ Water level observed during drilling																
	D.C.P.T. denotes dynamic cone penetration test																
	Dynamic cone penetration test was carried-out 3m east of borehole CW-1																

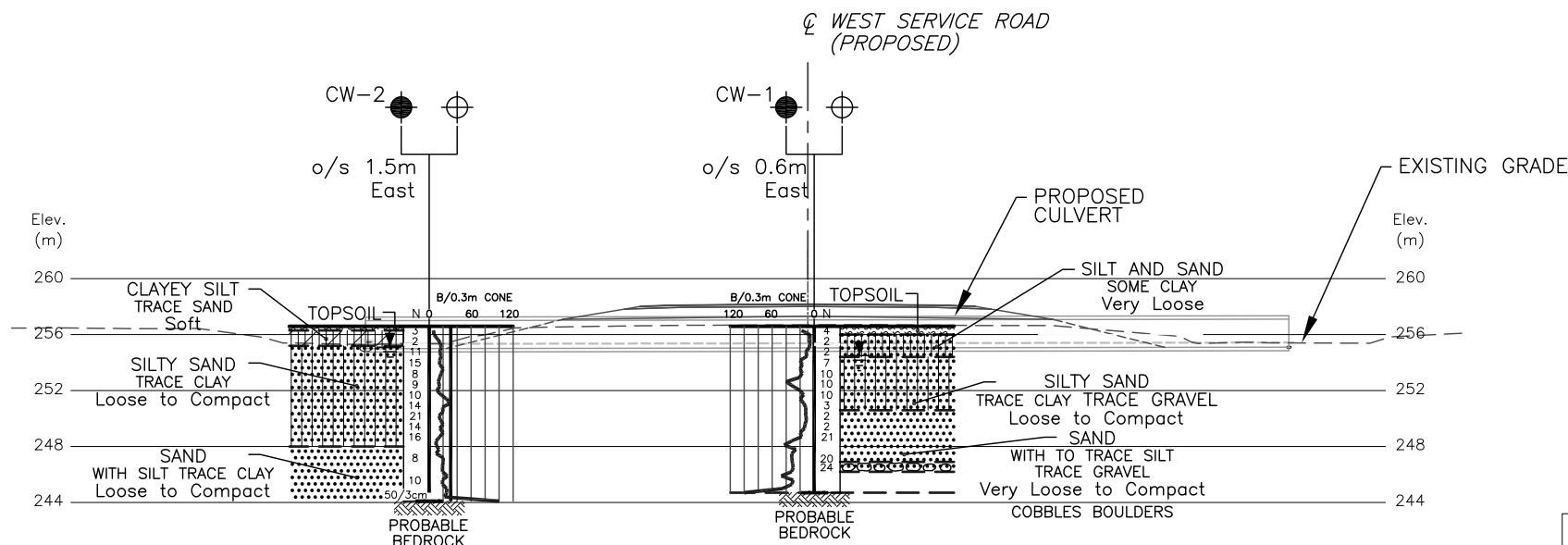
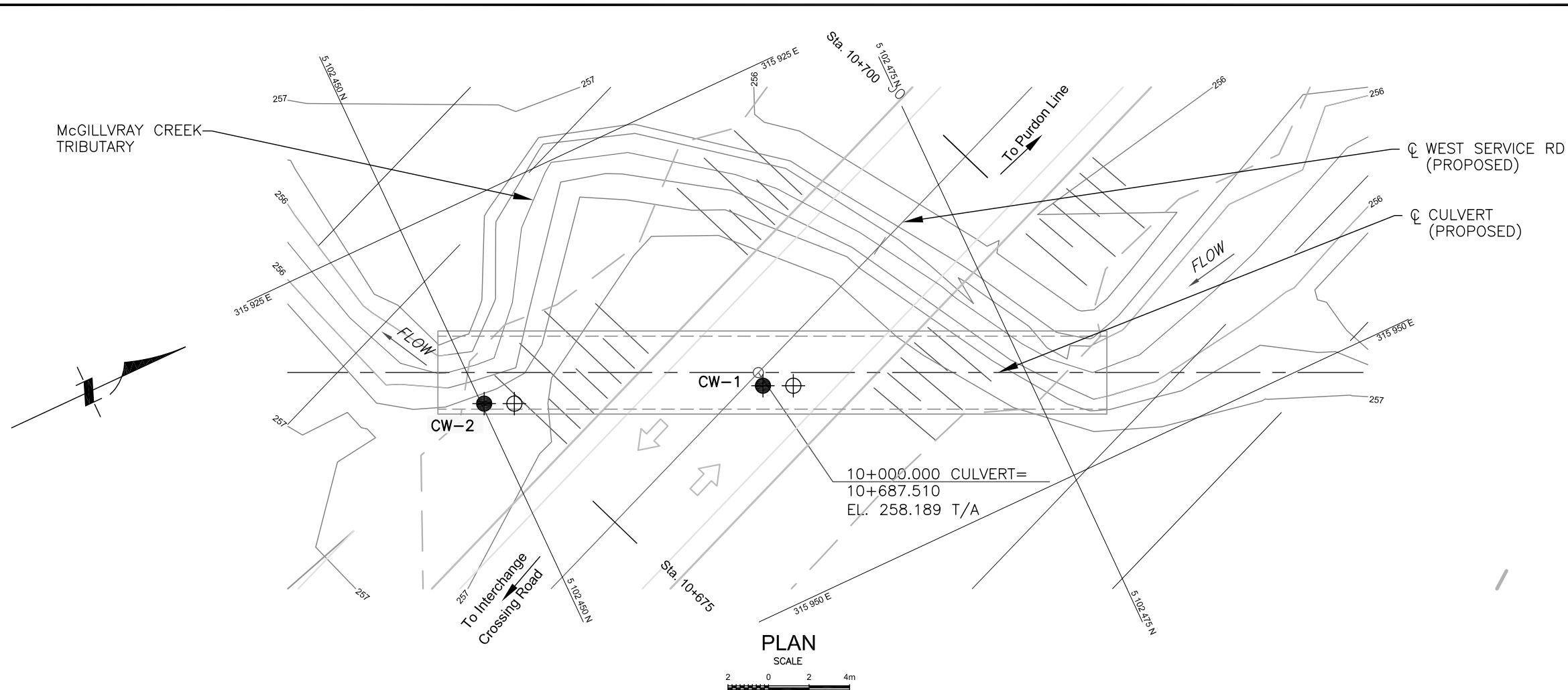
RECORD OF BOREHOLE No CW-2

1 of 1

METRIC

G.W.P. 323-00-00 **LOCATION** Co-ords: 5 102 450.1 N ; 315 934.5 E **ORIGINATED BY** A.K.
DIST North Bay HWY 11 **BOREHOLE TYPE** C.F.H.S.A. and Dynamic Cone Penetration Test **COMPILED BY** N.S.B.
DATUM Geodetic **DATE** January 10 and 11, 2012 **CHECKED BY** B.R.G.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa										WATER CONTENT (%)				
							○ UNCONFINED		+ FIELD VANE												
							● QUICK TRIAXIAL		× LAB VANE												
256.6	Ground Surface						20	40	60	80	100										
0.0	Topsoil						20	40	60	80	100										
256.3	Clayey silt, trace sand		1	SS	3								○								
0.3	Soft Brown Moist		2	SS	2																
255.2	Silty sand, trace clay					▽*															
1.4	Compact Grey Wet to loose		3	SS	11								○								
			4	SS	15								○								
			5	SS	8								○			0 53 45 2					
			6	SS	9								○								
	Compact		7	SS	10																
			8	SS	14																
			9	SS	21																
			10	SS	14								○								
			11	SS	16																
248.0	Sand with silt, trace clay																				
8.6	Loose to Grey Wet compact		12	SS	8								○			0 70 28 2					
			13	SS	10																
244.1	End of borehole		14	SS	50/3cm																
12.5	Refusal on probable bedrock																				
	Sample 14: Sampler bouncing																				
	* 2012 01 10																				
	▽ Water level observed during drilling																				
	Dynamic cone penetration test was carried-out 2m east of borehole CW-2															C.F.H.S.A. denotes continuous flight hollow stem augers					

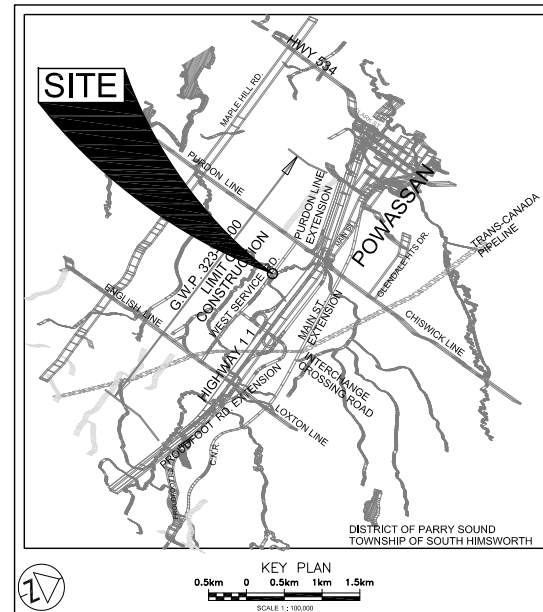


NOTES:

1. INLET BOREHOLE WAS NOT CARRIED OUT DUE TO LACK OF PERMISSION TO ENTER. AN ALLOWANCE SHOULD BE MADE FOR VARIATIONS IN SUBSURFACE STRATIGRAPHY.
2. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
3. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
4. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

CONT No		
GWP No	323-00-00	
WP No	5217-10-01	
CULVERT AT STA. 10+687		SHEET
WEST SERVICE ROAD		
BOREHOLE LOCATIONS AND SOIL STRATA		

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CONSULTING ENGINEERS



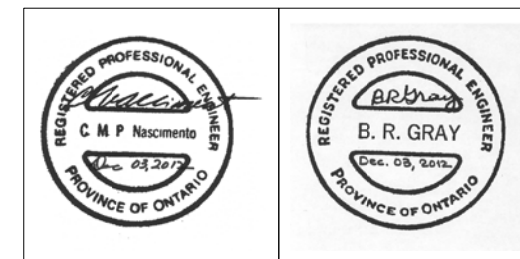
LEGEND			
	Borehole		
	Cone		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60 Cone, 475 J/blow)		
	WL at time of investigation Dec. 2011 and Jan. 2012		
*	Water level not established		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		
BH No	ELEVATION	NORTHINGS	EASTINGS
CW-1	256.6	5 102 463.0	315 939.5
CW-2	256.6	5 102 450.1	315 934.5

— 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. 31L-164

HWY No	11	DIST	North Bay
SUBM'D	NA	CHECKED	NSB
DATE	DEC. 03, 2012	SITE	44-506/C
DRAWN	NA	CHECKED	BRG
APPROVED	CN	DWG	WS-1



REF AECOM Drawings:
60157537 McGillvary_Creek_West_Service
Rd_Culvert-10+687_1_GA.dwg dated Dec., 2010;
Hwy 11-Design.dwg; Hwy11_MTO_InRoads_Contours.dwg



**FOUNDATION DESIGN REPORT
for
MCGILLVRAY CREEK TRIBUTARY CULVERT
WEST SERVICE ROAD STATION 10+687
SITE NO. 44-508/C
TOWNSHIP OF SOUTH HIMSWORTH
NORTH BAY AREA, ONTARIO
G.W.P. 323-00-00**

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Table 1 – List of Standard Specifications Referenced in Report

FOUNDATION DESIGN REPORT
for
McGillvray Creek Tributary Culvert
West Service Road Station 10+687
Site No. 44-508/C
Township of South Himsworth
North Bay Area, Ontario
G.W.P. 323-00-00

1. INTRODUCTION

The installation of the proposed West Service Road culvert for the Tributary of McGillvray Creek is planned as part of the south entrance of the Powassan project which extends from 5.7 km south of the Highway 534 northerly 5.0 km. This report was prepared for AECOM Canada Ltd (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

This foundation design report provides foundation engineering comments and recommendations for the proposed culvert design and construction.

According to the General Arrangement drawing dated December 2010, the proposed West Service Road culvert will be located at approximate Sta. 10+687 (West Service Road chainage) in the Township of South Himsworth. The proposed culvert will be a cast-in-place 3.6 x 2.0 m concrete box culvert with a total length of 33 m. This culvert is proposed at a skew angle of 46.2° towards the north at the proposed West Service Road centreline. The proposed road grade is at elevation 258.2 at the culvert location which is about 1.0 to 3.2 m above existing grade.

In summary, the subsurface stratigraphy revealed in the boreholes generally comprised a 0.3 to 0.6 m thick topsoil overlying 1.1 m thick clayey silt or 1.6 m thick sand and silt underlain by 8.2 and 11.1 m thick cohesionless silty sand / sand deposit mantling probable bedrock. Cobbles and boulders were encountered within sand deposit. The groundwater level observed during the field investigation was at 1.5 and 2.1 m depths (elevation 254.5 and 255.1).

From a foundation perspective, the construction of the proposed cast-in-place culvert is feasible at the culvert location, subject to the following comments.



It is understood that a cast-in-place concrete box culvert will be installed at the proposed culvert location. Therefore, the recommendations for precast culvert and a discussion of the advantages and disadvantages of the two culvert options are not included in this report.

It is considered that a cast-in-place culvert can typically tolerate a maximum of 25 mm of differential settlement, after which, cracking may appear within the culvert. The expected differential settlements for this culvert between the centreline and the west and east ends (inlet and outlet) are in the 25 to 30 mm range as outlined in Section 2 of this Report. Consequently, consideration should be given by the design engineer to introduce joints in the culvert to accommodate differential settlements which are larger than 25 mm.

The construction of the culvert will need to consider groundwater control measures for construction under wet or dry conditions as outlined in the report. These are required because of the pervious subgrade soils and high water levels at the site.

It is noted that the inlet borehole (east end) was not drilled due to lack of permission to drill where the area was protected by an electric fence for cattles. An allowance should be made for any variations in assumed soil stratigraphy at the inlet location.

The "red flag" issues outlined in the preceding paragraphs and the recommended methods of overcoming these issues noted in the following sections of the report are intended to alert and aid the designer and the contractor. It is noted that no responsibility or liability is assumed by the consultants or the MTO for alerting the contractor to all critical issues. The requirement to deliver acceptable construction quality remains the responsibility of the contractor.

The foundation frost penetration depth at the site is 1.9 m according to OPSD 3090.100.

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

The invert levels of the proposed box culvert are specified to be near elevation 255.1 at the east end (inlet) and elevation 255.0 at the west end (outlet). The subgrade level for a concrete box culvert is interpreted to be about 0.6 m below the proposed invert levels at elevation 254.5 and 254.4 allowing for the thickness of concrete base of the culvert and for the bedding and levelling courses. The proposed road grade at the proposed culvert will be at about elevation 258.2.

In summary, the subgrade soils revealed in the boreholes below the anticipated culvert subgrade level (elevation 254.4) comprised of 10.3 m loose to compact silty sand/ sand over probable bedrock (borehole CW-2) at the west end of culvert. In the middle section, 9.7 m thick loose to compact silty sand/sand (borehole CW-1) was encountered over the probable bedrock. At the east end of culvert, it was inferred that the subsurface conditions will comprise about 9.7 to 10.4 m loose to compact silty sand/sand over probable bedrock.

Groundwater was contacted in two boreholes at elevation 254.5 to 255.1. The groundwater level at the time of the field investigation in December 2011 and January 2012 is about 0.1 to 0.6 m above the inferred subgrade level.

It is estimated that about 0.3 to 0.6 m thick topsoil and 1.1 m thick cohesive clayey silt and 0.8 to 1.6 m thick cohesionless silty sand / sand and silt excavation will be required to achieve the anticipated culvert subgrade level at the outlet and middle locations (boreholes CW-2 and CW-1). It is anticipated that up to 2.0 m thick native soils excavation will be required at the inlet location.

The Granular A / Granular B Type II bedding materials and the underlying cohesionless soils that are in the zone of influence below the design subgrade level are considered capable of adequately supporting the stress imposed by the concrete box culvert.

The culvert bedding should be 300 mm thick and comprise Granular A or Granular B Type II with a maximum particle size of 37.5 mm.



The estimated total settlement of the culvert at the west end (outlet) is about 5 to 10 mm and 35 mm at the centreline. It is anticipated that total settlement at the east end (inlet) is about 5 to 10 mm. It is estimated that the resulting differential post-construction settlement between the west end and centreline of the culvert will be in the order of 25 to 30 mm and between the centreline and east end of the culvert will be 25 to 30 mm should the culvert be constructed concurrently with the new embankment construction .

The estimated magnitude of differential settlements under the culvert of 25 to 30 mm is not considered tolerable for the proposed cast-in-place concrete culvert. Construction joints should be added to accommodate the estimated 25 to 30 mm of differential settlement.

The recommended factored geotechnical bearing resistances at ultimate limit states (ULS) and the geotechnical reaction at serviceability limit states (SLS) for the 3.6 m wide concrete box culvert constructed on the native cohesionless soils below the granular bedding layer are as follows:

CULVERT SECTION	SUBGRADE SOIL TYPE	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL REACTION AT SLS (kPa)
Entire Length	Typically compact Cohesionless Silty Sand / Sand	225	150

The geotechnical reaction at SLS normally allows for a 25 mm compression of the founding medium. In addition, the cohesionless soil settlements under the proposed culvert discussed previously in this section should be considered. A foundation embedment depth of 2.0 m and groundwater at about 1.0 m above the level of the culvert invert were assumed for computation of the geotechnical resistance.



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. All the cobbles and boulders should be removed from the subgrade level. A site review should be conducted by qualified geotechnical personnel during preparation of the subgrade and compaction of the granular fill.

For the box culvert, it is recommended to provide a 300 mm thick granular bedding below the culvert. The bedding material should comprise Granular A, satisfying the specifications within OPSS 1010, 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 and frost taper treatment for the box culverts should be carried out in accordance with OPSS 422 and MTO SP 109S31.

Topsoil/ organic silt and any other deleterious soils revealed during the subgrade preparation should be excavated prior to placement of the granular base below the box culvert and the materials replaced with compacted Granular A or Granular B Type II. Granular B Type II should be preferred for construction under wet conditions.

Wet cohesionless silty / sandy soils are anticipated at the subgrade final excavation level elevation 254.4. Site conditions may be unfavourable due to 0.6 m higher water level above the subgrade final excavation level and the existing pervious wet sandy soils. Groundwater control will be required for construction of the proposed culvert as outlined in Section 5.2 of this report.

As an alternative to facilitate underwater construction of the subgrade and bedding and minimize disturbance of the cohesionless native soils, the approved subgrade should be immediately covered with a layer of biaxial geogrid (25 by 35 mm maximum aperture / 1.2 to 2.0 kN/m minimum peak tensile strength). Granular B Type II should be used to backfill over the geogrid



where required to bring the grade up to the bottom of the granular bedding. To obtain adequate compaction of the granular material under water conditions, the granular bedding may be placed to a level about 300 mm above the design grade to be above the estimated groundwater level and be compacted from the surface. The grade should then be lowered to the design level of the bedding by carefully excavating the excess bedding material. Under the anticipated wet conditions the levelling course may not be required. It is recommended to provide the minimum 300 mm thickness of granular bedding above the geogrid.

Upon establishing the granular bedding level, the groundwater should be lowered sufficiently below the level of the base of the culvert (at least 200 to 300 mm) to allow for the placement of the cast-in-place concrete in the dry.

2.2.2 Modulus of Subgrade Reaction

The estimated values of the modulus of subgrade reaction for a box culvert constructed on various compacted base materials over the native soils are as follows.

SOIL TYPE	MODULUS OF SUBGRADE REACTION (MN/m ³)
Granular A or Granular B Type II	45
Sandy soils	10

2.2.3 Sliding Resistance

The following parameters should be used to compute sliding resistance of cast-in-place box culvert or cast-in-place headwall and wingwall foundations.



SOIL TYPE	FOUNDATION FRICTION ANGLE (DEGREES)	COHESION (kPa)	UNIT WEIGHT (kN/m ³)
Granular A or Granular B Type II	35	0	22.8
Sandy Soils	30	0	20.0

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

2.2.4 Seismic Site Coefficient

The seismic site coefficient for the conditions at the proposed West Service Road culvert site is 1.0 –Type I soil profile as per clause 4.4.6 of the CHBDC.

3. CULVERT BACKFILL

Backfill adjacent to the culverts should be placed in accordance with OPSD 3121.150, OPSS 422 MTOD 803.021 and MTO SP 422S01. The requirements for frost tapers are 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 minimize the potential for movement and/or damage of the culvert due to the lateral earth pressure induced by compaction. Refer to OPSS 902 for additional comments.

The proposed 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)
 $\quad = \gamma - \gamma_w$
 γ_w = unit weight of water
 $\quad = 9.8 \text{ kN/m}^3$
 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)
 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 design:

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

(*) Assumes that excavated native soils used for backfill are inorganic cohesionless soils.



The design should consider both the maximum water level in the stream and the stabilized 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 WINGWALLS

The previous recommendations and geotechnical parameters for culvert foundations and backfill should be utilized for the design of the foundations for headwalls and wingwalls. 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 to provide 1.9 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 minimize the build-up of hydrostatic pressure behind the headwalls and wingwalls. The weeping tiles should be surrounded by a properly designed granular filter or non-woven Class II geotextile (with an FOS of 75-150 μ m according to OPSS 1860) placed to prevent migration of fines into the system.

5. CONSTRUCTION CONSIDERATIONS

5.1 Excavation

Excavation to the anticipated founding level of the proposed West Service Road culvert is expected to extend through the topsoil into the native deposits of cohesionless silty / sandy soils and cohesive clayey silt. 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.



The native silty/sandy soils above the water table are classified as Type 3 soils necessitating temporary cut slopes to be inclined at 1H:1V. Below the water table, cut slopes should be shaped at 3H:1V or flatter. Where composite soil types exist, the excavation slopes should be cut to the requirements of the soil type with the highest number that is present in the slope according to OHSA.

5.2 Groundwater Control

The stabilized groundwater level is expected to be consistent with the water level in the McGillvray Creek Tributary, near elevation 255.0 that is about 0.6 m above the inferred subgrade level. If construction under wet condition as outlined in Section 2.2.1 is not implemented, it will be necessary to implement measures to temporarily lower groundwater table and to permit construction of cast-in-place culvert, headwalls and wingwalls. The groundwater level should be lowered to a minimum of 0.5 m below the proposed founding levels.

For construction in the dry, it is considered that a dewatering system involving dam and pumping techniques may not be sufficient due to pervious subgrade soils. Consequently, cofferdams will likely be required for the installation of the culvert for construction in the dry. The contractor is responsible for the selection, performance and detailed design of the dewatering system. Any seepage water that enters into the excavation from the excavation slope may be handled by sump and pumping techniques.

From the Foundations standpoint, the requirement for a permit to take water (PTTW) will depend on the water tightness of the contractor's selected type of dewatering system. The PTTW requirement will also depend on the groundwater levels at the time of construction since these are subject to seasonal fluctuations and precipitation patterns. A PTTW may be required to address other engineering facets, such as those of Hydrology engineering.

It should be noted that a water level at elevation 255.5 was measured in January 2011, and the construction should be scheduled during summer months to reduce the requirements for groundwater control. The flow of surface water should be diverted away from the excavations.



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 rip-rap 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 wingwalls 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) shall be placed. Additional appropriate erosion control measures for the project should be assessed using the following erodibility K factor.

SOIL TYPE	K FACTOR
Sand	0.2
Silt	0.3



7. CLOSURE

This Foundation Design Report was prepared by Mrs. N. S. Balakumaran, P. Eng., and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C. M. P. Nascimento, P. Eng., Project Manager conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.



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Project Engineer



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Project Manager



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

NB/CN/BRG:nb-mi-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 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
OPSS1010	Material Specification for Aggregates, Base, Subbase, Select Subgrade and Backfill Material
OPSS 1860	Material Specification for Geotextiles
SP 422S01	Construction Specification for Precast Concrete Box Culvert
SP 109S31	Substitution of Precast Concrete Box Culverts for Cast-In-Place Concrete Box Culverts
OPSD 810.010	Rip-Rap Treatment for Sewer and Culvert Outlets
OPSD 3090.100	Foundation Frost Depth for Northern Ontario
OPSD 3121.150	Minimum Granular Backfill Requirements - Walls Retaining
MTOD 803.021	Bedding and Backfill for Precast Concrete Box Culverts