FOUNDATION INVESTIGATION AND DESIGN REPORT
for
MCGILLVRAY CREEK TRIBUTARY CULVERT REPLACEMENT
HIGHWAY 11 NORTHBOUND LANES STATION 21+978
SITE NO. 44-366/C1
TOWNSHIP OF SOUTH HIMSWORTH
NORTH BAY AREA, ONTARIO
G.W.P. 323-00-00

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PML Ref.: 10TF013A-C1
Index No.: 318FIR and 319FDR
GEOCRES No.: 31L-165
December 12, 2012

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for
McGillvray Creek Tributary Culvert Replacement (Site No. 44-366/C1)
Highway 11 NBL Sta. 21+978
Township of South Himsworth
North Bay Area, Ontario
GWP 323-00-00

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1. INTRODUCTION

This report summarizes the results of the foundation investigation carried out for the proposed replacement of the Tributary of the McGillvray Creek culvert on the Highway 11 northbound lanes (NBL) 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 investigation 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 21+978, Highway 11 NBL chainage, in the Township of South Himsworth (refer to General Arrangement Drawing 'Highway 11 NBL Box Culvert at Sta. 21+978.49 ' prepared by AECOM dated December 2010, revised February 2012).

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

2. SITE DESCRIPTION AND GEOLOGY

The contemplated replacement culvert will be located on the existing Highway 11 NBL about 70 m south of the existing Highway 11 NBL / Main Street intersection. 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 Highway 11 transportation corridor and farming activities east and west of the highway and scattered residential houses east of the Highway 11 NBL. Locally, the existing Highway 11 median is approximately 50 m wide. The local topography of the site is generally undulating and sloping down to the west and up northerly. The existing Highway 11 NBL embankment is about 6 to 8 m high at the culvert location. The McGillvray Tributary Creek flows approximately in an east to west direction under the Highway 11

NBL. The ground cover includes grasses and bushes near the creek area and scattered stands of trees.

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 plain deposits which overlies Precambrian age monzonitic (granitic) rock formation.

3. INVESTIGATION PROCEDURES

The subsurface investigation was carried out during the period of December 7 to 22, 2011. A total of three boreholes (CN-1 to CN-3) were drilled to 6.7 to 13.4 m at the locations shown on Drawing N-1, appended. A dynamic cone penetration test was conducted about 1.5 m west of borehole CN-1 to 7.5 m and data was compiled in Record of Borehole CN-1.

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 and buried utilities. 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 guidelines 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 (24)
- Atterberg Limits (4)
- Grain size distribution analyses (8)

The laboratory grain size distribution charts are presented in Figures CN-GS-1 to CN-GS-4 and Atterberg Limits results are presented in Figure CN-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 N-1. The boundaries between soil strata are transitional and have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

Three boreholes (CN-1 to CN-3) were drilled along the alignment of this culvert to depths ranging from 6.7 to 13.4 m. The subsurface stratigraphy revealed in the boreholes generally comprised a topsoil or fill underlain by cohesionless deposits of sandy silt /silt over sand/silty sand/gravelly sand. Cobbles and boulders were encountered within the silty sand/gravelly sand deposits in boreholes CN-2 and CN-3. Probable bedrock was inferred by auger refusal in boreholes CN-1 and CN-3 at 6.7 and 7.8 m (elevation 251.2 and 252.0). Groundwater was observed in boreholes CN-1 and CN-3.

4.1 Topsoil

A 300 mm thick topsoil layer was encountered surficially in boreholes CN-1 and CN-3 extending to 0.3 m (elevation 258.4 and 258.7).

4.2 Fill

A 6.8 m thick fill unit was encountered in borehole CN-2 drilled on the existing embankment shoulder. The cohesionless fill unit has two distinct layers which include sand and silty sand. The sand fill layer extended to 3.8 m (elevation 263.1) and silty sand fill layer extended a further 3.0 m to 6.8 m (elevation 260.1) N values varied from 3 to 13 indicating very loose to compact relative density.

The results of the grain size distribution analyses for a silty sand fill sample are included in Figure CN-GS-1. The moisture content determinations ranged from 10 to 19%.

4.3 Sandy Silt / Silt

A cohesionless sandy silt/ silt deposit was encountered below the topsoil layer at 0.3 m (elevation 258.4 and 258.7) in boreholes CN-1 and CN-3 and below the fill unit at 6.8 m (elevation 260.1) in borehole CN-2. The deposit was 3.4 to 5.2 m thick extending to sandy soils at 3.7 to 11.0 m (elevation 253.1 to 255.9). N values ranged from 2 to15 indicating very loose to compact relative density.

The results of grain size distribution analyses for silt / sandy silt samples are included in Figures CN-GS-2 and CN-GS-3. The plasticity chart is presented in Figure CN-PC-1. The liquid and plastic limits of silt samples ranged from 22 to 29 and 17 to 24, respectively with plasticity index values of 3 to 7. The moisture content determinations typically ranged from 18 to 26%, locally with higher values of 37 to 42% due to organics in borehole CN-3.

4.4 Silty Sand / Sand / Gravelly Sand

A cohesionless sand deposit with varying silt and gravel content was encountered below the silt / sandy silt at 3.7 to 11.0 m (elevation 253.1 to 255.9) in all of the boreholes. The unit was 1.9 to 3.0 m thick extending to the underlying probable bedrock at 7.8 and 6.7 m (elevation 251.2 to 252.0) in boreholes CN-1 and CN-3 and to borehole termination depth of 13.4 m (elevation 253.5) in borehole CN-2. Borehole CN-2 was terminated in this deposit at 13.4 m (elevation 253.5). N values ranged from 8 to 25. Two N values of 20 and 30 for 15 and 10 cm sampler penetration were also recorded, indicating sampling refusal on cobbles or bedrock. The stratum was found to have a typically loose to compact relative density.

The results of grain size distribution analysis for a silty sand sample are included in Figure CN-GS-4. The moisture content determinations ranged from 8 to 20%.

4.5 Probable Bedrock

The probable bedrock was inferred by auger refusal in boreholes CN-1 and CN-3 at 7.8 and 6.7 m (elevation 251.2 and 252.0).

4.6 Groundwater

Groundwater was encountered in boreholes CN-1 and CN-3. During augering, groundwater was observed at 0.3 and 1.5 m (elevation 257.2 and 258.7). Upon completion of drilling, groundwater was measured at 1.5 m (elevation 257.2) in borehole CN-3. No water was encountered in borehole CN-2. At the time of the investigation, the water level in the McGillvray Tributary Creek was at about elevation 258.0 and was at about elevation 258.5 in January 2011. The groundwater level is subject to seasonal fluctuations and rainfall patterns.

5. CLOSURE

Mr. F. Portela 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.

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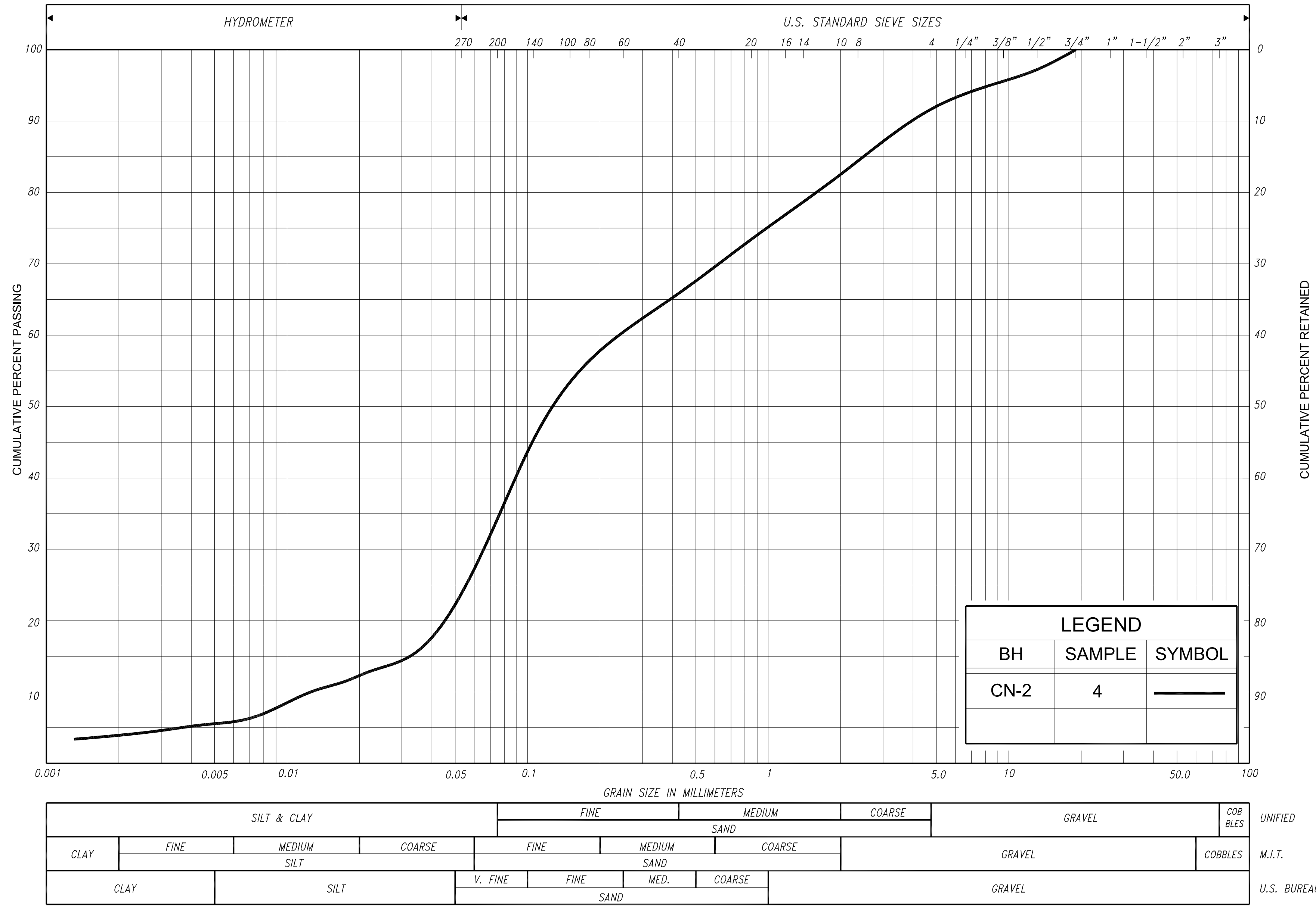
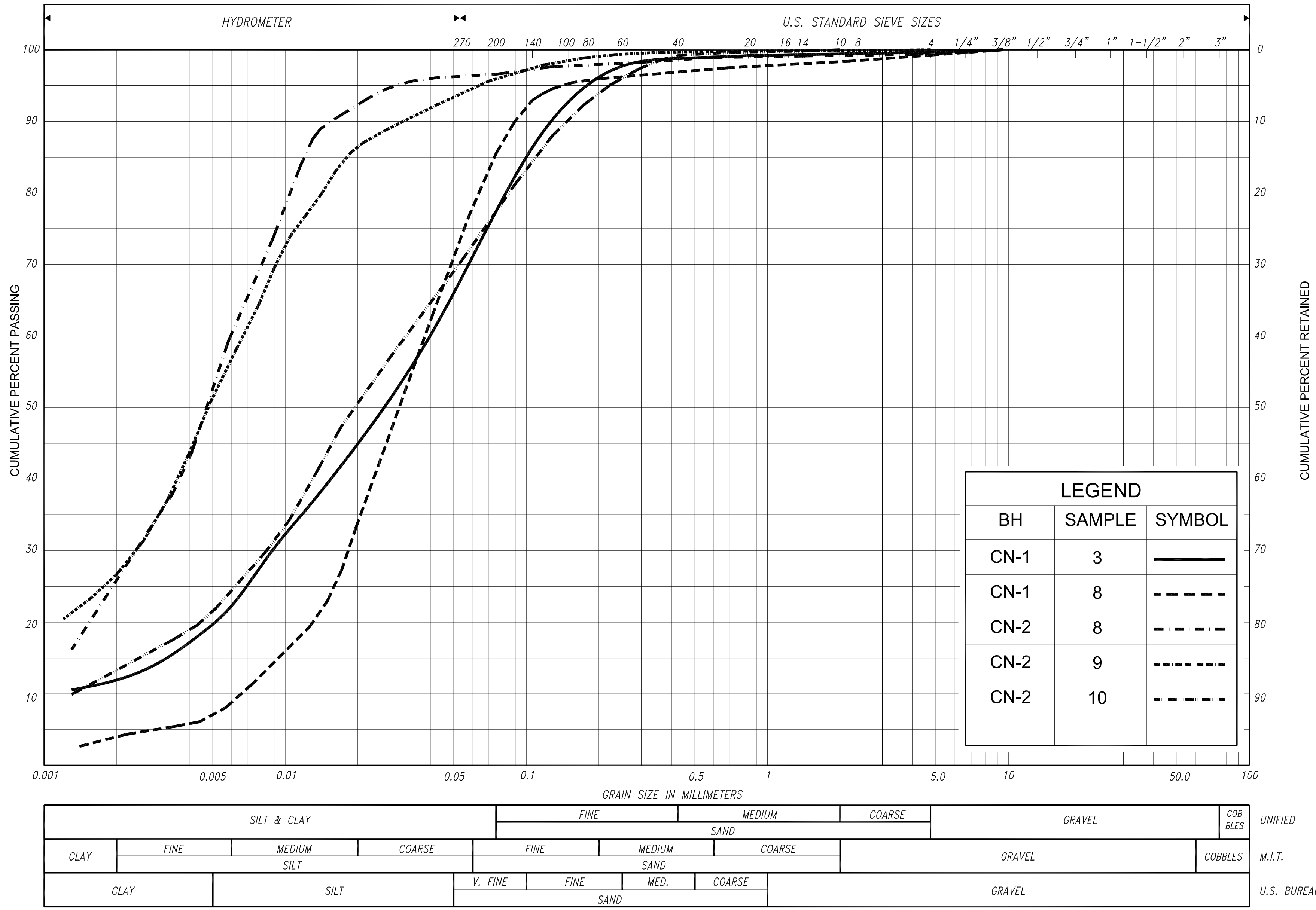
NB/CN/BRG:nb-mi-nk



Ontario	**GRAIN SIZE DISTRIBUTION** SILTY SAND, trace clay, trace gravel (FILL)	FIG No. CN-GS-1 HWY: 11 G.W.P. No. 323-00-00

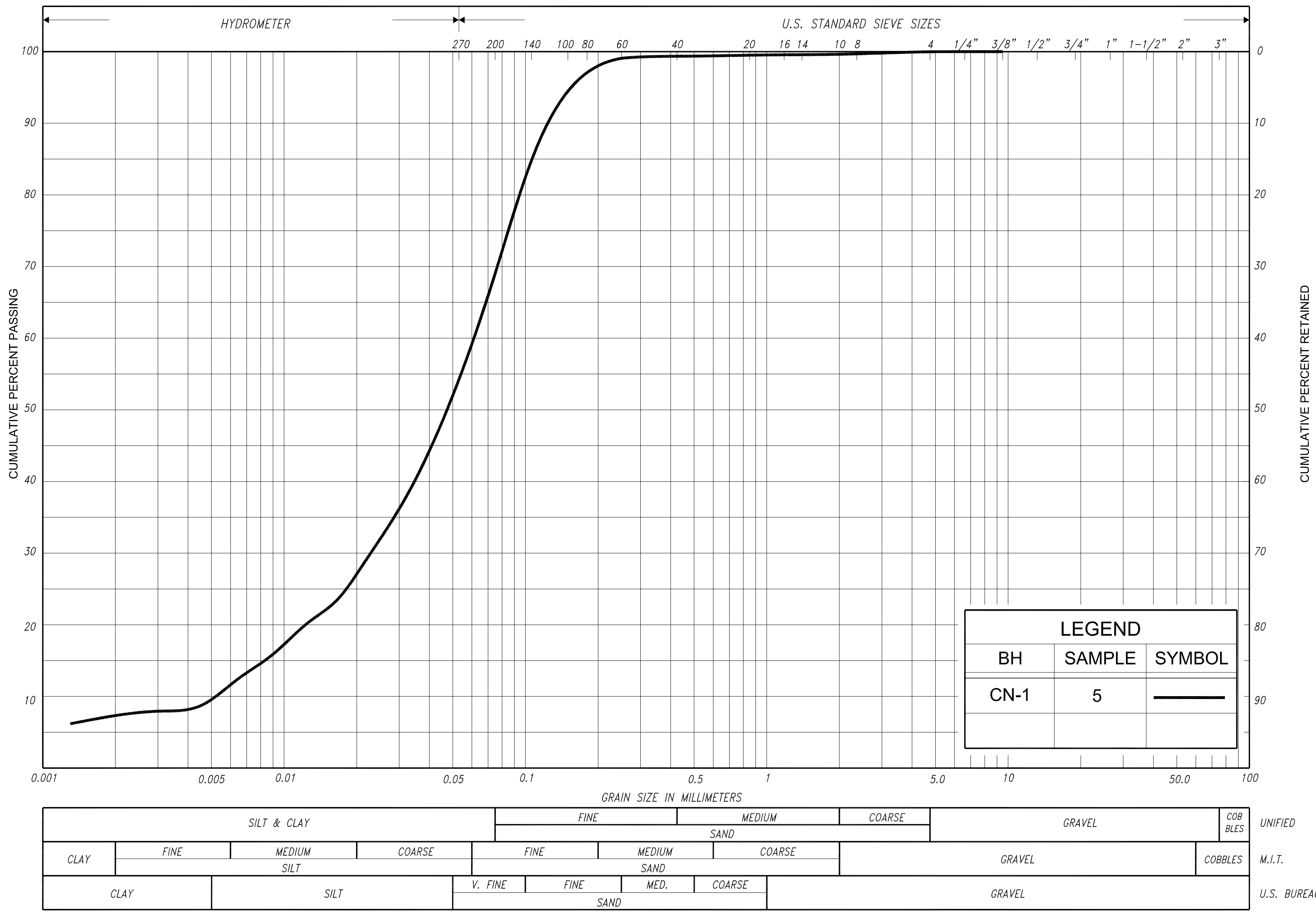


Ontario

GRAIN SIZE DISTRIBUTION

SILT, trace to with clay, trace to with sand, trace gravel

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

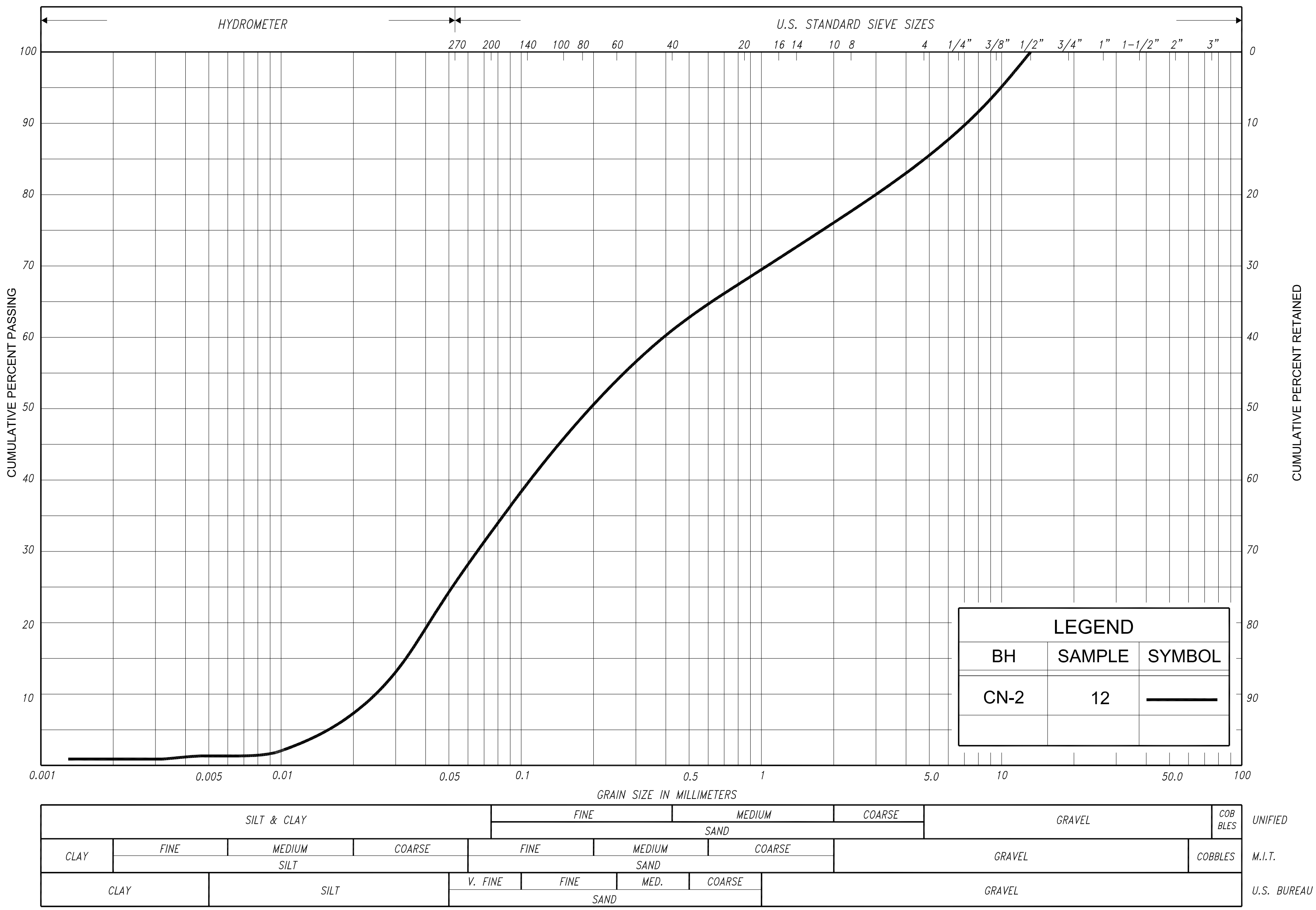


GRAIN SIZE DISTRIBUTION

SANDY SILT, trace clay

FIG No.	CN-GS-3
HWY:	11
G.W.P. No.	323-00-00

Ontario

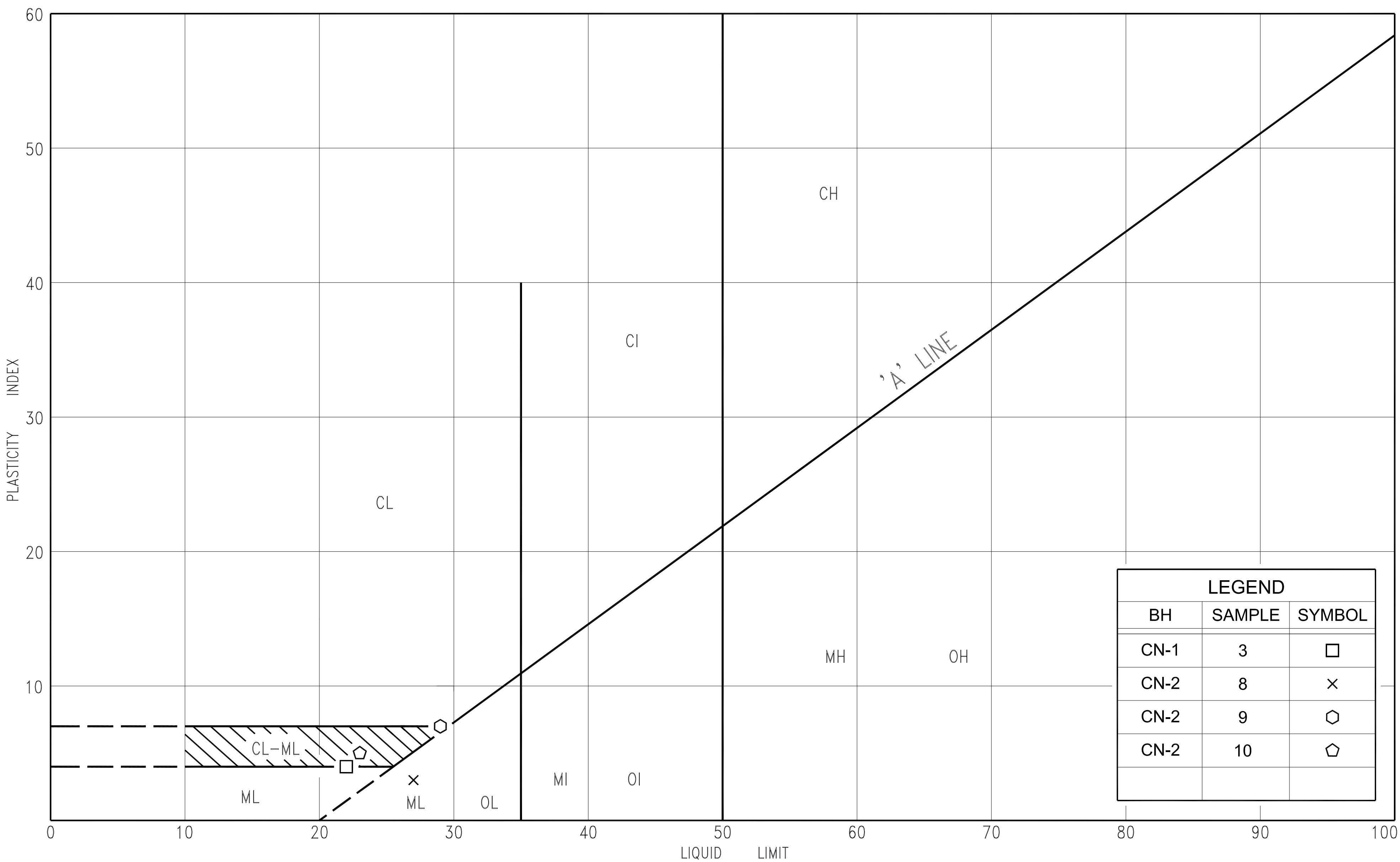


Ontario

GRAIN SIZE DISTRIBUTION

SILTY SAND, some gravel, trace clay

FIG No.	CN-GS-4
HWY:	11
G.W.P. No.	323-00-00



Ontario

PLASTICITY CHART

SILT, some to with clay, trace to with sand, trace gravel

FIG No.	CN-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/0.3 m)	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:

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
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

RECORD OF BOREHOLE No CN-1

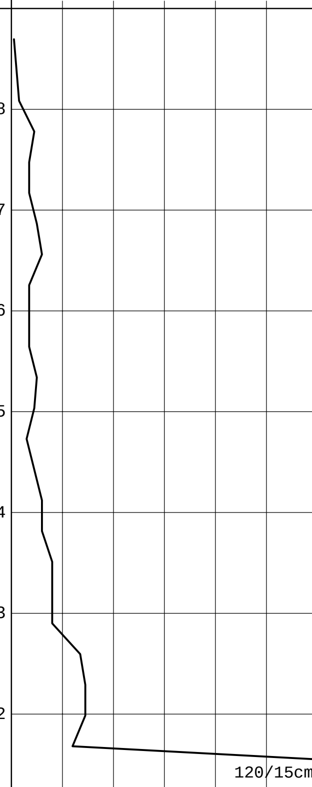
METRIC

G.W.P. 323-00-00	**LOCATION** Co-ords: 5 102 986.7 N ; 316 120.8 E		**ORIGINATED BY** F.P.
DIST North Bay **HWY** 11	**BOREHOLE TYPE** C.F.H.S.A.+ D.C.P.T. + 'N' Casing		**COMPILED BY** N.S.B.
DATUM Geodetic	**DATE** December 07, 2011		**CHECKED BY** B.R.G.

ELEV / DEPTH (m)	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	NATURAL MOISTURE CONTENT w (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
259.0	Ground Surface										
258.7 / 0.0	Topsoil				∇ *						
0.3	Sandy silt, organic inclusions; Loose, Brown, Wet; some clay	1	SS	5		○					
		2	SS	5		○					
257.6 / 1.4	Silt, some clay, some sand, trace gravel; Loose, Grey, Wet	3	SS	8		○		1	22	65	12
256.8 / 2.2	Sandy silt, trace clay, sand layers; Compact to very loose, Grey, Wet	4	SS	12		○					
		5	SS	4		○		0	31	62	7
		6	SS	2		○					
254.5 / 4.5	Silt, some sand, trace clay, trace gravel; Loose, Grey, Wet	7	SS	7		○					
		8	SS	7		○		1	13	82	4
253.1 / 5.9	Gravelly sand; Compact, Reddish brown, Wet	9	SS	14							
		10	SS	25							
		11	SS	20/15cm							
251.2 / 7.8	End of borehole. Refusal on probable bedrock. Sample 11: Sampler bouncing										

DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100); SHEAR STRENGTH kPa (20 40 60 80 100): ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE

ELEVATION SCALE: 258, 257, 256, 255, 254, 253, 252



PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

REMARKS & GRAIN SIZE DISTRIBUTION (%)

* 2011 12 07

∇ Water level observed during drilling

NOTE: Dynamic cone was carried out 1.5m west of borehole CN-1

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

D. C. P. T. denotes Dynamic Cone Penetration Test

5 Numbers refer to Sensitivity

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

7 , :

Ontario

Peto MacCallum Ltd.
CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No CN-2

METRIC

G.W.P. 323-00-00 **LOCATION** Co-ords: 5 103 029.6 N ; 316 155.4 E **ORIGINATED BY** F.P.

DIST North Bay **HWY** 11 **BOREHOLE TYPE** C.F.H.S.A. and 'N' Casing **COMPILED BY** N.S.B.

DATUM Geodetic **DATE** December 19, 2011 **CHECKED BY** B.R.G.

SOIL PROFILE | SAMPLES | GROUND WATER CONDITIONS | ELEVATION SCALE | 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

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	GR	SA	SI	CL
266.9	Ground Surface								
0.0	Sand, trace gravel Very loose to compact, Brown, Moist to wet (FILL)	1	SS	3					
					266				
		2	SS	4	265				
					264				
		3	SS	9					
	Silty sand trace clay, trace gravel				263				
		4	SS	8	262	8	58	30	4
		5	SS	10					
					261				
		6	SS	13					
260.1 6.8	Silt, trace sand trace clay, trace gravel organics Loose to compact, Grey, Wet				260				
		7	SS	6					
	with clay	8	SS	10	259	1	3	70	26
		9	SS	15		0	4	69	27
					258				
	some clay, with sand	10	SS	7		0	23	64	13
					257				
255.9 11.0	Silty sand some gravel, trace clay Loose to compact, Reddish brown, Wet	11	SS	9	256				
					255				
		12	SS	16		15	52	32	1
	Casing advanced through cobbles/boulders from 12.8 to 13.4m depth				254				
253.5 13.4	End of borehole								

* Borehole dry

C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers

5 Numbers refer to Sensitivity

20 15 5 10 (%) STRAIN AT FAILURE

7

Ontario

Peto MacCallum Ltd.
CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No CN-3

METRIC

G.W.P. 323-00-00 **LOCATION** Co-ords: 5 103 043.9 N ; 316 185.3 E **ORIGINATED BY** F.P.

DIST North Bay **HWY** 11 **BOREHOLE TYPE** Continuous Flight Hollow Stem Augers **COMPILED BY** N.S.B.

DATUM Geodetic **DATE** December 22, 2011 **CHECKED BY** B.R.G.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS
258.7	Ground Surface				
0.0 / 258.4	Topsoil	1	SS	3	
0.3	Silt, some clay, trace sand; Very loose Brown Wet; trace clay, organics Grey	2	SS	3	
257.3 / 1.4	Sandy silt, organics; Very loose to loose Grey Wet	3	SS	4	▽* ▼*
		4	SS	4	
		5	SS	3	
255.0 / 3.7	Sand, trace silt, trace gravel; Compact Grey Wet	6	SS	12	
254.2 / 4.5	Gravelly sand; Loose to dense Reddish brown Wet	7	SS	17	
		8	SS	8	
	cobbles and boulders	9	SS	30/10cm	
252.0 / 6.7	End of borehole; Refusal on probable bedrock				

Elevation scale: 258, 257, 256, 255, 254, 253, 252

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

* 2011 12 22

▽ Water level observed during drilling

▼ Water level measured after drilling

5 Numbers refer to Sensitivity

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

FOUNDATION DESIGN REPORT
for
MCGILLVRAY CREEK TRIBUTARY CULVERT REPLACEMENT
HIGHWAY 11 NORTHBOUND LANES STATION 21+978
SITE NO. 44-366/C1
TOWNSHIP OF SOUTH HIMSWORTH
NORTH BAY AREA, ONTARIO
G.W.P. 323-00-00

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Index No.: 319FDR
GEOCRES No.: 31L-165
December 12, 2012

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FOUNDATION DESIGN REPORT
for
McGillvray Creek Tributary Culvert Replacement (Site No. 44-366/C1)
Highway 11 NBL Sta. 21+978
Township of South Himsworth
North Bay Area, Ontario
GWP 323-00-00

1. INTRODUCTION

It is planned to replace the existing culvert for the Tributary to the McGillvray Creek on the Highway 11 Northbound Lanes (NBL) as part of the south entrance to 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, revised February 2012, Highway 11 NBL replacement culvert will be constructed at the same location as the existing culvert, Sta. 21+978 (Highway 11 NBL chainage) in the Township of South Himsworth. The proposed culvert will be a cast-in-place 3.0 × 3.0 m concrete box culvert with a total length of 51 m. The existing Highway 11 NBL will be realigned at the culvert location. The proposed centreline will be located about 8.0 m west of the existing centreline. The new road grade is proposed at elevation 262.7 and about 2.5 to 7.2 m above existing grades at the culvert location.

In summary, the subsurface stratigraphy revealed in the boreholes generally included a 0.3 m thick topsoil or 6.8 m thick embankment fill overlying 3.4 to 5.2 m thick cohesionless sandy silt / silt underlain by 1.9 to 3.0 m thick cohesionless silty sand / sand / gravelly sand mantling probable bedrock. Cobbles and boulders were encountered within silty sand/ gravelly sand deposits. The probable bedrock was inferred by auger refusal at 7.8 and 6.7 m depths (elevation 251.2 and 252.0) in boreholes CN-1 and CN-3. The groundwater level measured during the field investigation was at 0.3 and 1.5 m depths (elevation 257.3 and 258.7) in boreholes CN-1 and CN-3.

From a foundation perspective, the proposed cast-in-place culvert is feasible at the culvert location. However, cast-in-place culverts can typically tolerate a maximum of 25 mm of differential settlement, after which, cracking may appear within the culvert. This culvert may experience estimated 0 to 65 mm of total settlement with maximum differential settlements in the 25 to 30 mm range, as outlined in Section 2 of this Report. Consequently, consideration should be given at the design engineer's discretion, to introduce joints in the culvert to accommodate the differential settlements which are larger than 25 mm.

Due to the realignment of the existing Highway NBL and the grade raise change at the culvert location, the underlying soils at the culvert location will settle unevenly and create differential settlement which may be excessive for a cast-in-place concrete culvert. Mitigation measures to reduce the magnitude of differential settlements are discussed in this report.

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

In view of the deep excavation up to 11 m depth (including grade raise changes) required for the installation of this culvert and the proposed snowmobile culvert proposed 30 m to the north approximately at Sta. 22+009 (NBL), Site No. 44-371/C1, this culvert should be constructed before the snowmobile culvert for temporary slope stability considerations.

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 257.7 at the east end (inlet) and elevation 257.5 at the west end (outlet). The subgrade level for a concrete box culvert is interpreted to be about 0.7 m below the proposed invert levels at elevation 257.0 and 256.8 allowing for the thickness of concrete base of the culvert and for the bedding and levelling course. The proposed road grade at the proposed culvert will be about elevation 268.5 indicating that the soil cover above the culvert will be approximately up to 7.5 m.

In summary, the subgrade soils revealed in the boreholes below the anticipated culvert subgrade level (elevation 256.8 to 257.0) comprised of 1.8 m very loose to loose sandy silt over 3.0 m thick loose to compact sand/gravelly sand over probable bedrock (borehole CN-3) at the east end of culvert. In the middle section, 1.0 m thick loose silt over the 2.4 m thick silty sand (borehole CN-2) was encountered. At the west end of culvert, 3.7 m thick of compact silt / sandy silt over 1.8 m thick gravelly sand over probable bedrock (borehole CN-1) was encountered. It is noted that cobbles and boulders were encountered within the sandy soil deposits.

At the time of the field investigation in December, 2011 groundwater was contacted in two boreholes CN-1 and CN-3 at elevation 257.3 and 258.7, respectively. These groundwater levels were about 0.3 and 1.9 m above the inferred subgrade level. The water level in the creek was at elevation 258.0 in January 2011, 1.0 to 1.2 m above the subgrade levels.

It is estimated that about 0.3 m thick of topsoil and 1.9 m thick cohesionless sandy silt / silt will be excavated to achieve the anticipated culvert subgrade level at the outlet location (borehole CN-1). In addition, about 5.5 to 6.8 m thick sand/sandy silt embankment fill and 2.3 m thick silt is to be excavated to reach the subgrade level in the section from the middle area through the east end (inlet) of the culvert.

The 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. Granular B Type II is considered to be preferred in the expected wet conditions.

As indicated previously, due to the realignment of the existing Highway NBL and grade raise change, the native soils under the culvert will undergo variable differential settlement along its alignment. For the ease of reference, four zones were identified from east end (inlet) to west end (outlet) and are described in the following table. The table provides the estimated settlements for the culvert assuming that it is constructed concurrently with the realigned embankment construction.

ZONE	ZONE DESCRIPTIONS (*)		ESTIMATED TOTAL SETTLEMENT (mm)	ESTIMATED DIFFERENTIAL SETTLEMENT (mm)
	FROM	TO		
1	East end of the culvert (inlet)	About 8 m east of proposed highway centreline	0 – 20	0 – 20
2	About 8 m east of proposed highway centreline	About 9 m west of proposed highway centreline	20 – 45	5 – 20
3	About 9 m west of proposed highway centreline	About 16.5 m west of proposed highway centreline	40 – 65	20 – 25
4	About 16.5 m west of proposed highway centreline	West end of the culvert (outlet)	10 – 40	30

(*) Zone descriptions are based on the cross-section in the GA Drawing received February 23, 2012.

The estimated total settlement of the culvert at the east end (inlet) Zone 1 is 0 to 20 mm with 20 to 45 mm at Zone 2, 40 to 65 mm at Zone 3 and 10 to 40 mm in Zone 4. It is estimated that the resulting differential post-construction settlement will vary between 0 and 30 mm should the culvert be constructed concurrently with the embankment construction. A graphic representation of the total and differential settlements estimated to occur along the culvert is provided below.

West End (Outlet) | (New) Centreline Highway 11 NBL | East End (Inlet)

Distance from CL (m)	25.2	16.5	12.0	9.0	8.0	13.0	25.8	
Total Settlement (mm)	10	40	65	45	25	20	0	0
Differential Settlement (mm)	30	25	20	20	5	20	0	
Zone	4	3		2		1		

Given the estimated magnitude of differential settlements under the culvert ranging from 0 to 30 mm, it is considered that the proposed cast-in-place concrete culvert will be suitable for the site. Construction joints will be required to accommodate up to 30 mm of differential settlement, if the culvert is constructed concurrently with the embankment construction.

To reduce the magnitude of post construction settlements under the culvert, it is recommended that the culvert be constructed after the fill for the proposed embankment realignment has been in place at the culvert location for a period of two months. After this preloading stage, estimated differential settlements of about 10 to 15 mm are anticipated between Zones 2, 3 and 4 of the culvert and negligible settlements would remain between the east end of the culvert and Zone 2.

The recommended geotechnical bearing resistance at ultimate limit states (ULS) and the geotechnical reaction at serviceability limit states (SLS) for the 3.0 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	Loose to Compact Cohesionless Sandy Silt / Silt	200	125

The geotechnical resistance at SLS normally allows for 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 up to about 1.9 m above the level of the culvert subgrade level were assumed for computation of the geotechnical resistances.

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 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 95% 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 proposed cast-in-place box culvert may be carried out in general accordance with MTOD 803.21, OPSS 422 and MTO SP 422S01 although these specifications are applicable to precast box culverts. Reference is made to SP 109S31 if proposed cast-in-place culvert is substituted by a precast culvert.

Topsoil and any other deleterious soils revealed during the preparation at or below the subgrade should be excavated prior to placement of the granular base below the box culvert and replaced with compacted Granular A or Granular B Type II. Granular B Type II should be preferred for construction under wet conditions. In addition, any other deleterious soils encountered below the existing culvert at the subgrade level should be removed.

Wet cohesionless silty / sandy soils are anticipated at the subgrade final excavation level elevation 256.8 to 257.0. Site conditions may be unfavourable due to 0.3 to 1.9 m higher water level above the subgrade final excavation level and the existing pervious wet sandy/silty soils. Groundwater control will be required for construction of the proposed culvert as outlined 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 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 underwater, 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 box culvert constructed on undisturbed soils or compacted materials are as follows.

SOIL TYPE	MODULUS OF SUBGRADE REACTION (MN/m^{3})
Granular A or Granular B Type II	45
Silty /Sandy Soils	10

2.2.3 Sliding Resistance

The following parameters should be used to compute sliding resistance of cast-in-place box culvert, headwalls and wingwalls 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
Silty /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 McGillvray Creek Tributary 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 and MTO SP 422S01. 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 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/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)

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

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

δ = angle of friction between soil and wall (23.5º 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 MATERIAL (*)
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

For headwalls and wingwalls 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 provided with 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 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 Excavation

Excavation to the anticipated founding level of the proposed Highway 11 NBL McGillvray Creek Tributary culvert is expected to extend through the topsoil and into the native deposits of cohesionless silty / sandy soils. 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 minimum recommended factor of safety of 1.3 is considered to be acceptable for earth cut slopes during construction. The stability of the anticipated temporary 9.2 m deep cut slope (including the existing embankment but excluding the proposed grade raise) was analysed using the limit equilibrium methods and the SLOPE/W software developed by Geo-Slope International Ltd. A summary of the soil engineering properties assumed for the calculations is as follows.

SOIL TYPE	UNIT WEIGHT (kN/m^{3})	SHEAR STRENGTH (kPa)	INTERNAL ANGLE FRICTION (Degrees)
Embankment Fill	21.5	0	33
Silt / Sandy Silt	20.0	0	30
Silty Sand to Gravelly Sand	20.0	0	30

The results of the slope stability analyses are provided in Figures A-1 and A-2 attached in Appendix FDR-1 and listed below.

ANALYSIS NO.	TEMPORARY CUT DESCRIPTION	FACTOR OF SAFETY (FOS)	FIGURE NO.	MEETS FOS <1.3?
1	2.5H:1V side slope from bottom of excavation (unconfigured)	1.2	A-1	No
2	2.6H:1V side slope from bottom of excavation (unconfigured)	1.3	A-2	Yes

The analyses indicated that the temporary slopes cut into the existing embankment at 2.5H:1V (Analysis No. 1) from the base of excavation will not be adequately stable.

To maintain temporary side slope stability at the subgrade level, a temporary cut slope at 2.6H:1V from the base of excavation (Analysis No. 2) should be constructed. The minimum acceptable factor of safety of 1.3 was obtained for this temporary cut slope.

Because it is planned to move the Highway 11 NBL traffic to the SBL while the culvert is constructed, it is anticipated that temporary protection system following OPSS 539 will not be required. Due to the 7 to 8 m thick sand / silty sand embankment, the embankment cut slopes for the culvert replacement should be sloped at 2.6H:1V during construction.

Notwithstanding the results of the foregoing slope stability analysis, the slopes should follow the Occupational Health and Safety Act (OHSA). The existing cohesionless native soils and fill units are considered Type 3 soils according to OHSA. Where these soils are below the groundwater table these soils are considered to be Type 4 soils in the OHSA.

In view of the deep excavation up to 11 m depth (including grade raise changes) required for the installation of this culvert and the proposed snowmobile culvert proposed 30 m to the north approximately at Sta. 22+009 (NBL), Site No. 44-371/C1, this culvert should be constructed before the snowmobile culvert for temporary slope stability considerations.

5.2 Groundwater Control

The stabilized groundwater level is expected to be consistent with the water level in the McGillvray Creek Tributary, near elevation 258.0 that is about 1.0 to 1.2 m above the inferred subgrade level. It will be necessary to implement measures to temporarily lower the groundwater table and to permit construction of cast-in-place culvert and 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 dewatering system utilizing dam and pumping techniques may not be sufficient due to the pervious subgrade soils. Consequently, cofferdams will likely be required for the installation of the culvert for the construction in the dry. The dewatering system should be placed at the each end. The contractor is responsible for the selection, performance and detailed design of the dewatering system. Any seepage water enter in excavation from the excavation slope may be handled by sump and pumping techniques.

An underwater construction alternative to prepare the culvert subgrade and bedding layer is provided in Section 2.2.1. For construction of a cast-in-place culvert, however, groundwater control will likely be required to establish dry conditions, as outlined above.

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 258.5 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. At this site, side slopes of 2.5H:1V will likely be required in view of the highly erodible soils as per past performances. 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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NB/CN/BRG:nb-mi-nk

TABLE 1

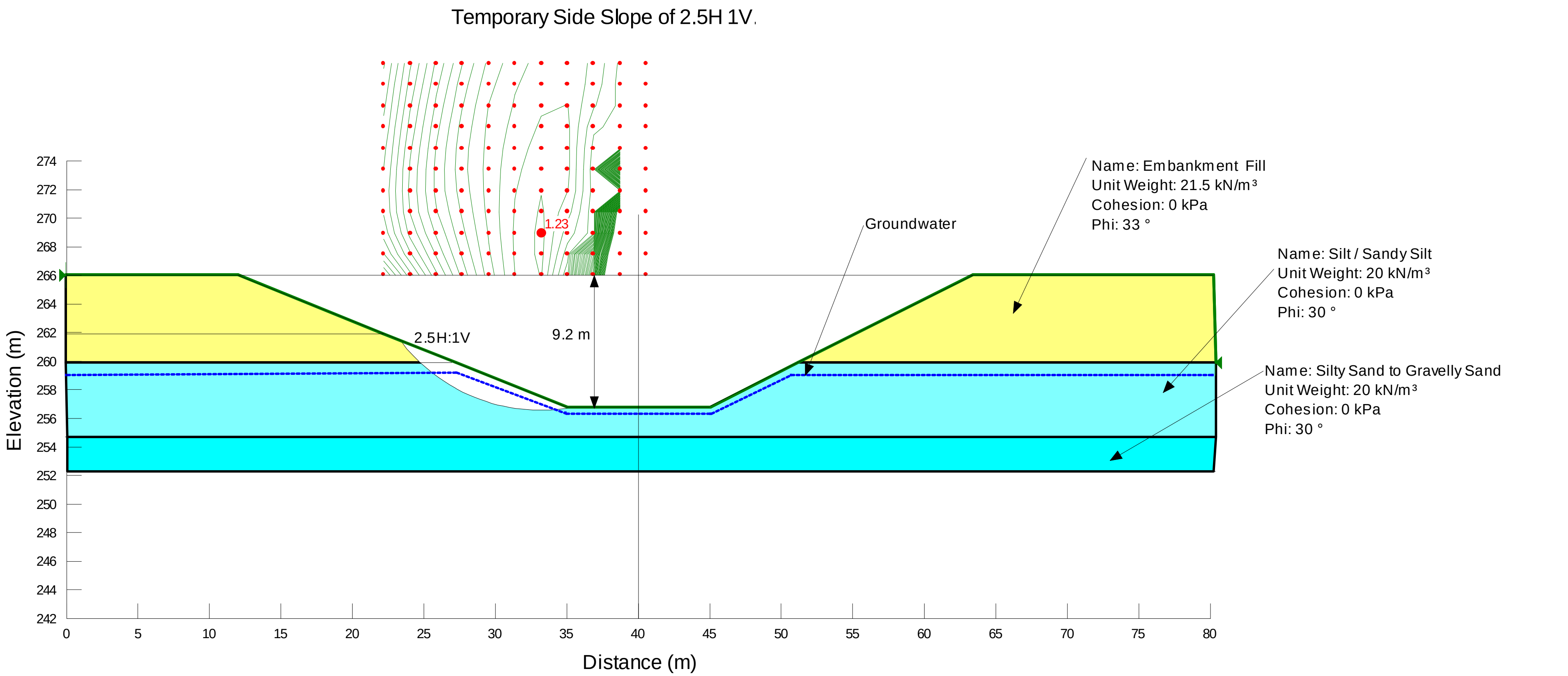
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

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 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
OPSS1010	Material Specification for Aggregates, Base, Subbase, Select Subgrade and Backfill Material
OPSS 1860	Material Specification for Geotextiles
SP 109S31	Substitution of Precast Concrete Box Culverts for Cast-in-Place Concrete Box Culverts
SP 422S01	Construction Specification for Precast Concrete Box Culvert
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 803.21	Bedding and Backfill for Precast Concrete Box Culverts

APPENDIX FDR-1

Slope Stability Diagrams

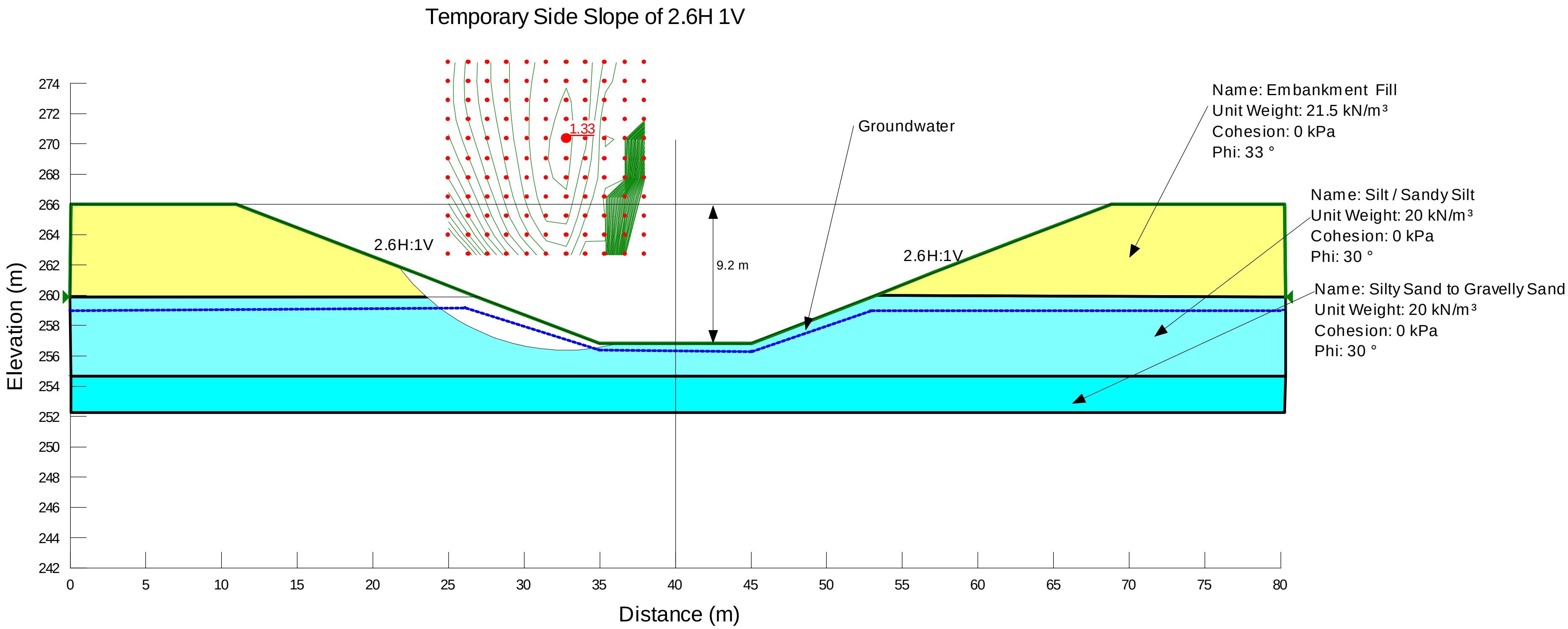
SLOPE STABILITY DIAGRAMS



Note: The criterion for the minimum FOS of 1.3 is not met.

FIGURE A-1

SLOPE STABILITY DIAGRAMS



Note: The criterion for the minimum FOS of 1.3 is met.

FIGURE A-2