



**FOUNDATION INVESTIGATION AND DESIGN REPORT  
for  
INTERCHANGE CROSSING ROAD OVERPASS  
HIGHWAY 11 SOUTHBOUND LANES  
SITE NO. 44-505/2  
TOWNSHIP OF SOUTH HIMSWORTH  
NORTH BAY AREA, ONTARIO  
G.W.P. 323-00-00**

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PML Ref.: 10TF013A-S3  
Index No.: 355FIR and 356FDR  
GEOCRES No.: 31L-170  
April 5, 2013



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**FOUNDATION INVESTIGATION REPORT**

for  
Interchange Crossing Road Overpass, Highway 11 Southbound Lanes  
Site No. 44-505/2  
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 Interchange Crossing Road Southbound Lanes (SBL) Overpass at the existing Highway 11 SBL. The overpass is part of the new interchange at the south entrance to Powassan project extending 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).

Based on the information provided by AECOM, it is understood that the proposed new bridge will be constructed along the Southbound Lanes of Highway 11 approximately between Sta. 20+825.3 and 20+857.3 Geographic Township of South Himsworth in the North Bay Area (refer to AECOM Drawing No. 1, dated April 2011, Highway 11 Interchange Crossing Road Overpass SBL General Arrangement-Alt. 2A).

The purpose of this report was to summarize the subsurface stratigraphy encountered at the proposed structure and approaches within about 20 m of the abutments.

**2. SITE DESCRIPTION AND GEOLOGY**

The contemplated structure is proposed for the Highway 11 SBL about 1.2 km south of the existing Purdon Line and Highway 11 SBL at-grade intersection in Powassan. The site is about 35 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, farming activity east and west of the Highway 11 and scattered residential houses east of



Highway 11 NBL. Gravel pits are present both east and west of the highway. The existing Highway 11 median is approximately 50 m at the proposed structure site. The local topography of the site is generally flat in the median area and flat to rolling to the west of Highway 11 SBL. A TransCanada Pipelines Ltd. facility crosses Highway 11 approximately 300 m south of the proposed overpass location. The ground cover includes grasses in the Highway 11 SBL area and bushes 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 glaciofluvial outwash deposits of kame formation comprising sand and gravel soils locally with cobbles and boulders which overlies Precambrian age monzonitic (granitic) rock formation.

### **3. INVESTIGATION PROCEDURES**

The field work for this study was carried out during the period of November 14 to December 6, 2011. A total of seven boreholes and one auger probe (ICS-1 to ICS-8) were drilled to 1.2 to 12.5 m at the locations shown on Drawing IS-1, appended. Two boreholes ICS-2 and ICS-5 were added to investigate sloping bedrock conditions for deep foundations. In addition, auger probe ICS-7 and borehole ICS-8 were added to further assess subsurface conditions for the south approach embankment.

The structure control points were staked in the field by exp Geomatics according to the GA Drawing dated April 2011 prepared by AECOM. The positions of the boreholes relative to the structure control points were selected at each foundation unit by PML allowing for drill rig accessibility, underground utilities and to minimize interference with the Highway 11 traffic lanes. Consequently, the approach boreholes ICS-1 and ICS-6 drilled approximately 10 m away from the proposed new centreline of the platform. The ground surface elevations at the borehole locations were established by PML using the ground surface elevations at the structure control points as provided by exp Geomatics. All elevations in this report are expressed in metres.



The boreholes were advanced using continuous flight hollow stem augers and 'N' casing 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. Two boreholes, ICS-2 and ICS-5 were extended 2.6 to 3.3 m into bedrock to 10.1 and 15.1 m using NQ diamond rock coring equipment. In addition, boulder coring was carried out through a layer of boulders in borehole ICS-4 from 5.0 to 5.8 m.

Soil samples were recovered from the boreholes at regular 0.75 and 1.5 m depth intervals using the standard penetration test method. Standard penetration tests and field vane tests were conducted to assess the strength characteristics of the substrata. Because the presence of silt seams in the clayey deposits exceeded the field vane capacity, only a limited number of field vane tests were carried out. Pocket penetrometer tests were carried out in the clayey soil seams to obtain representative test results on the in-situ shear strength. 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 (48)
- Atterberg limits (6)
- Grain size distribution analyses (14)



The laboratory grain size distribution charts are presented in Figures ICS-GS-1 to ICS-GS-6 and Atterberg Limits results are presented in Figures ICS-PC-1 and ICS-PC-2. All of the test results are summarized on the Record of Borehole sheets.

We also refer to the results of consolidation testing carried out on representative samples of cohesive clayey soils obtained for the design of sections of the embankments of the proposed N-E/W and E/W-S ramps. These results were reported in the Foundation Investigation and Design report prepared by PML, Reference No. 10TF013A-H1.

#### **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 Drawings IS-1 and IS-2. 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 drilled at the site generally comprised fill or surficial topsoil overlying cohesionless silt to sand underlain by clayey silt over cohesionless sand which in turn was underlain by silty sand till to sand till mantling bedrock. Cobbles and boulders were encountered within the till deposit. The bedrock surface was contacted / inferred at 6.8 to 12.5 m (elevation 268.1 to 273.4) in five boreholes (ICS-2 to ICS-5 and ICS-8). The remaining two boreholes (ICS-1 and ICS-6) and an auger probe (ICS-7) were terminated by refusal on probable boulders at 1.2 to 6.1 m (elevation 278.3 to 279.2).

A summary of the findings is given below.



#### **4.1 Fill**

A 300 mm thick fill unit was encountered surficially in boreholes ICS-1, ICS-2, ICS-4, ICS-6 and ICS-7. The unit extended to 0.3 m (elevation 279.9 to 280.3). The fill unit includes cohesionless silty sand. N values ranged from 3 to 8 indicating very loose to loose relative density.

#### **4.2 Topsoil**

A 300 mm thick topsoil layer was encountered surficially in boreholes ICS-3, ICS-5 and ICS-8 extending to 0.3 m (elevation 279.9 and 280.2).

#### **4.3 Silt to Sand**

A deposit of cohesionless silt to sand was encountered below the fill or topsoil at 0.3 m (elevation 279.9 to 280.3) in all of the boreholes. The deposit was 0.6 to 1.4 m thick extending to 0.9 to 1.7 m (elevation 278.5 to 279.6). Borehole ICS-7 was terminated in this unit at 1.2 m, elevation 279.2. N values ranged from 3 to 20, locally 1 in borehole ICS-3 indicating a variable very loose to compact relative density.

The results of grain size distribution analysis for a silty sand sample are included in Figure ICS-GS-1. The moisture content determinations ranged from 18 to 23%.

#### **4.4 Clayey Silt**

A deposit of cohesive clayey silt containing silty clay, silt and sandy silt layers was encountered below the cohesionless soils at 0.9 to 1.7 m (elevation 278.5 to 279.6) in boreholes ICS-2 to ICS-6 and ICS-8. The stratum was 0.8 to 3.8 m thick extending to 2.2 to 5.6 m (elevation 274.6 to 278.0). N values ranged from 4 to 14. Pocket penetrometer test results varied from 62 to 138 kPa. In situ vane testing conducted in borehole ICS-4 within the clayey silt deposit indicated shear strength value greater than 100 kPa. The stratum exhibited typically a firm to very stiff consistency.





The results of grain size distribution analysis for clayey silt and silty clay samples are included in Figures ICS-GS-2 and ICS-GS-3. The plasticity charts are presented in Figures ICS-PC-1 and ICS-PC-2. The liquid and plastic limits of clayey silt samples ranged from 28 to 33 and 18 to 21, respectively with plasticity index values of 7 to 13. The liquid and plastic limits of a silty clay sample were 39 and 21 with the corresponding plasticity index value of 18. The moisture content determinations ranged from 26 to 42%.

Based on consolidation test results carried out for the N-E/W and E/W-S ramps of the interchange on cohesive soils with similar characteristics (Reference PML Report No. 10TF013A-H1), it is inferred that the underlying native cohesive clayey soils were subjected to preconsolidation pressures of 570 to 1,000 kPa. The measured initial void ratios ( $e_o$ ) were 0.82 and 1.3, compression index ( $C_c$ ) were 0.4 and 0.7 and coefficient of consolidation ( $C_v$ ) were 1.7 and 1.9 m<sup>2</sup>/month. These values were utilized for estimating the settlement of the cohesive soils at the bridge approaches.

#### **4.5 Sand**

A cohesionless sand deposit was encountered below the cohesive clayey silt deposit at 3.4 to 4.9 m (elevation 275.7 to 277.1) in boreholes ICS-2, ICS-3 and ICS-8. The deposit was 1.1 to 2.6 m thick extending to 5.2 and 6.0 m (elevation 274.5 to 275.3). N values ranged from 17 to 25 indicating a compact relative density.

The results of grain size distribution analysis for a sand sample are included in Figure ICS-GS-4. The moisture content determinations ranged from 1 to 3%.

#### **4.6 Silty Sand Till / Sand Till**

A till deposit with varying silt composition was encountered in all of the boreholes except in auger probe ICS-7 at 1.2 to 6.0 m (elevation 274.5 to 279.2). The till deposit was at least 0.5 to 7.1 m thick extending to 2.1 to 12.5 m (elevation 268.1 to 278.3). Cobbles and boulders were encountered within this deposit. N values ranged from 16 to 86 and 50 to 62 for 8 to 15 cm



sampler penetration. The relative density of this deposit was typically found to be dense to very dense with a local compact condition.

The results of grain size distribution analysis for silty sand till / sand till samples are included in Figures ICS-GS-5 and ICS-GS-6. The moisture content determinations ranged from 8 to 18%.

#### **4.7 Cobbles and Boulders**

A 0.8 to 3.6 m thick layer of cobbles and boulders was encountered in boreholes ICS-2, ICS-4 and ICS-5 at 2.2 to 6.0 m (elevation 274.6 to 278.0). Boulder coring was carried in borehole ICS-4 from 5.0 to 5.8 m (elevation 274.1 to 274.6) to advance the borehole. The layer extended to 5.8 and 9.0 m (elevation 271.6 to 274.6). Boreholes ICS-1 and ICS-6 were terminated by refusal on boulders at 2.1 and 6.1 m (elevation 274.1 and 278.3). Auger probe ICS-7 was also terminated by refusal on a boulder at 1.2 m (elevation 279.2).

#### **4.8 Bedrock**

The bedrock surface was contacted / inferred at 7.8 and 12.5 m (elevation 268.1 and 272.7) at the south abutment boreholes ICS-2 and ICS-3. At the north abutment boreholes ICS-4 and ICS-5, the bedrock was contacted / inferred at 6.8 and 12.0 m (elevation 268.4 and 273.4).

The bedrock surface between borehole locations slopes at angles of 12.1° (boreholes ICS-2 and ICS-3) and 13.1° (boreholes ICS-4 and ICS-5) on the transverse direction at the abutments. On a north-south direction, the bedrock slopes at approximately of 0.5° (boreholes ICS-2 and ICS-4) and 0.3° (boreholes ICS-3 and ICS-5). Steeper angles may occur along the bedrock surface slopes between borehole locations.



The summary of depth to bedrock and elevations is provided in following table:

Foundation Element	Borehole	Depth to Bedrock (m)	Bedrock Elevation	
			Refusal	Cored
South Approach	ICS-1	>2.1	<278.3	
	ICS-7	>1.2	<279.2	
	ICS-8	7.7	272.8	
South Abutment	ICS-2	12.5		268.1
	ICS-3	7.8	272.7	
North Abutment	ICS-4	12.0	268.4	
	ICS-5	6.8		273.4
North Approach	ICS-6	<6.1	<274.1	

The measured core recovery varied between 93 and 100%. The RQD determined from the rock cores was in a range of 80 to 100%, indicating a good to excellent quality rock.

The granite bedrock exhibited high strength and was typically found to be weathered to unweathered.

A detailed description of the rock cores retrieved from boreholes ICS-2 and ICS-5 is given in Table A, appended. Photographs of the rock cores are shown in Appendix A.

#### 4.9 Groundwater

During augering, groundwater was observed at 0.3 to 4.3 m (elevation 275.9 to 280.2) in boreholes ICS-4, ICS-6 and ICS-8. Upon completion of drilling, groundwater was measured at 2.4 m (elevation 278.0) in borehole ICS-4. No water was encountered in boreholes ICS-1 and ICS-3 and auger probe ICS-7. The remaining boreholes ICS-2 and ICS-5 were charged with drilling water for rock coring. Based on the natural water content profile in the boreholes, it is estimated that the water level at the site is in the range of 1.8 to 4.8 m (elevation 275.4 to 278.7). The groundwater level is subject to seasonal fluctuation 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.



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

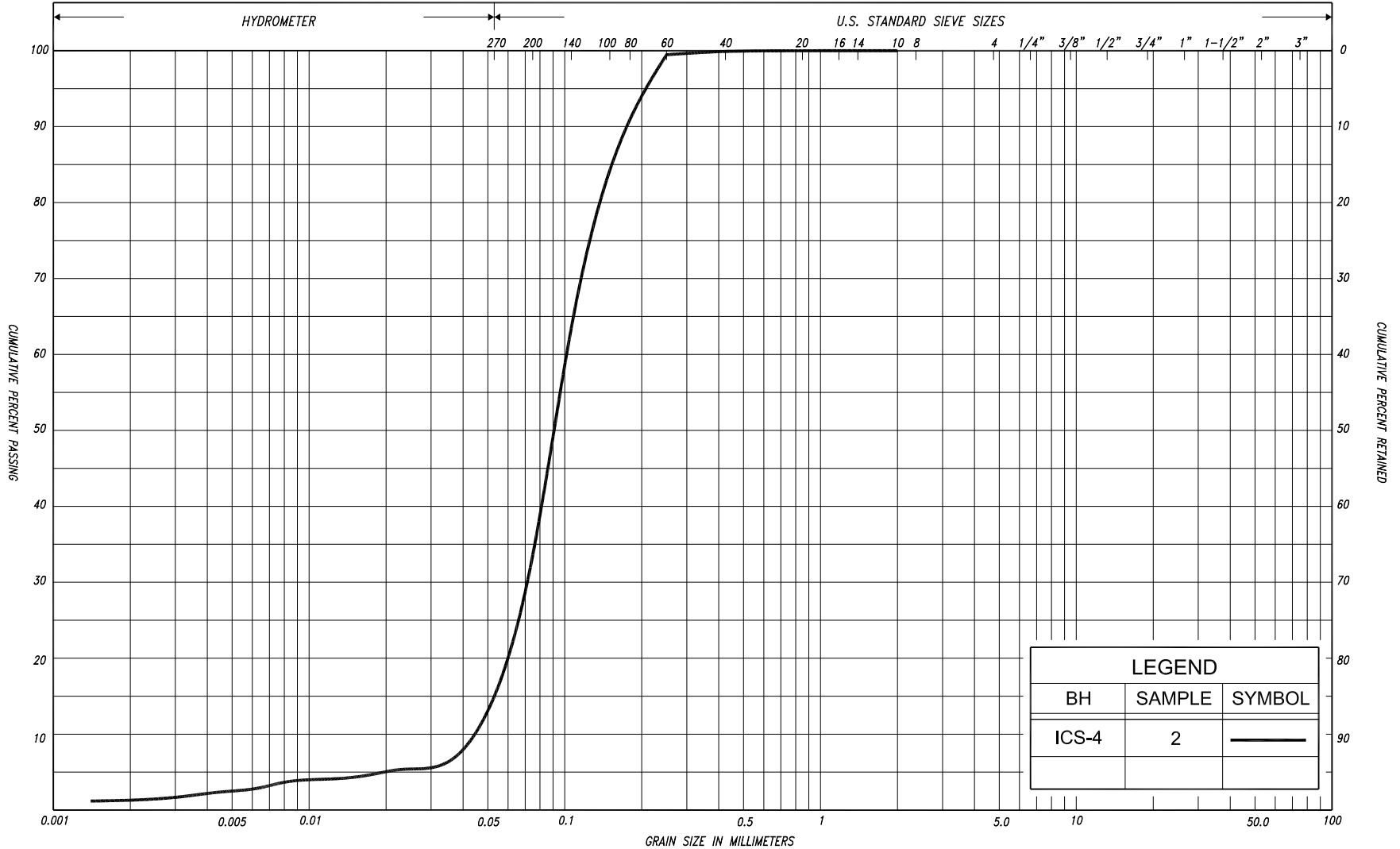


**TABLE A**  
**ROCK CORE DESCRIPTIONS**

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
ICS-2	16	12.5 – 12.8	100	92	12.5 – 15.1	GRANITE: Pink, medium crystalline, garnetiferous, high strength, slightly weathered to unweathered, with 100 mm thick layer of pink pegmatite, coarse crystalline, close to moderate spaced flat to dipping cross joints, rough planar, tight to slightly altered with red iron oxide on partings, excellent quality.
	17	12.8 – 13.6	93	93		
	18	13.6 – 15.1	98	98		
ICS-5	10	6.8 – 7.5	100	100	6.8 – 10.1	GRANITE: Pink, medium crystalline, garnetiferous, high strength, slightly weathered, close to moderate spaced flat to dipping (locally vertical) cross joints, rough planar, tight to slightly altered with red iron oxide and/or silt on partings, good to excellent quality.
	11	7.5 – 9.1	97	80		
	12	9.1 – 10.1	97	97		

NOTE: RQD = Rock Quality Designation

Originated: JFW  
Compiled: FP  
Checked: NB / CN



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

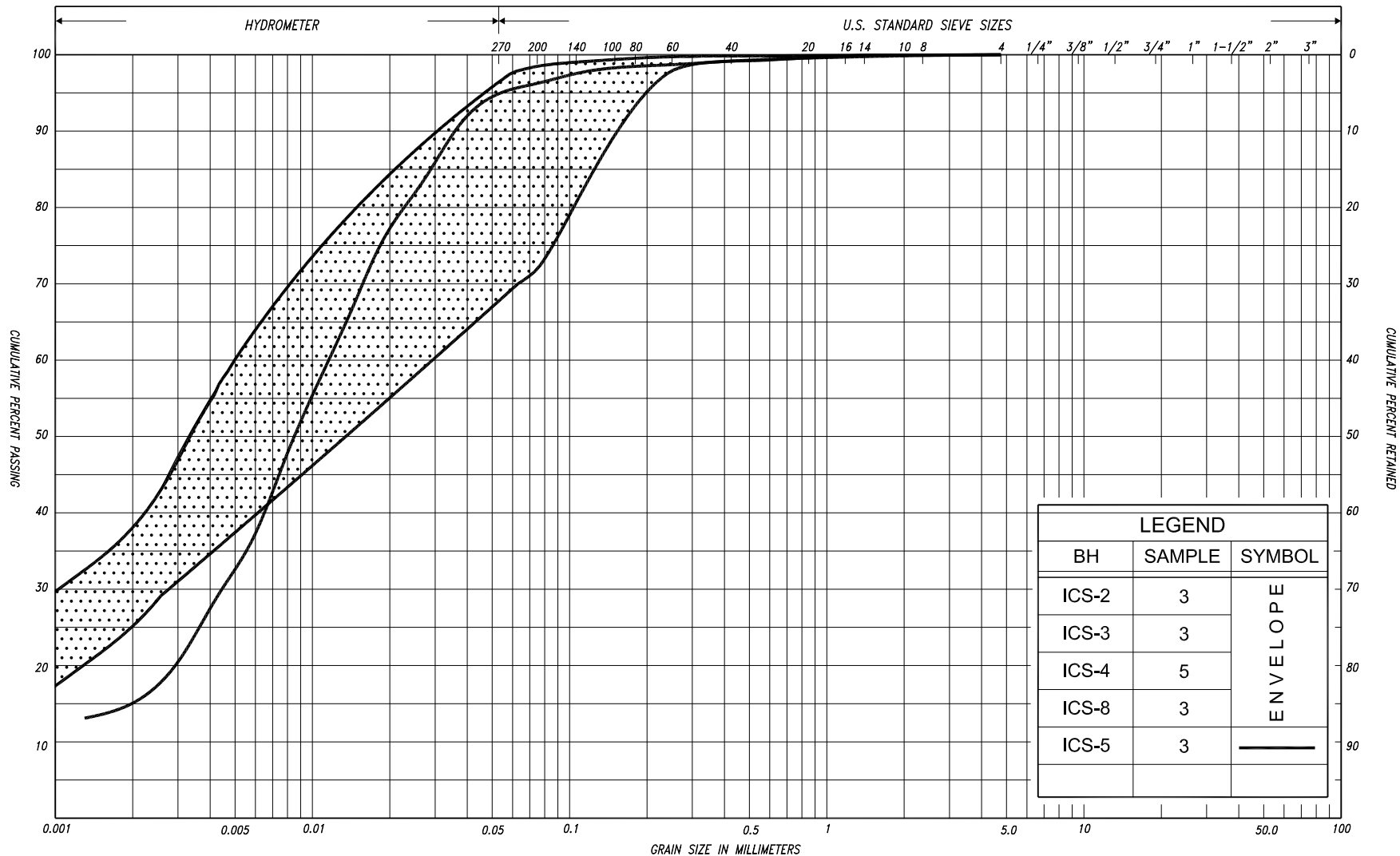
SILTY SAND, trace clay

FIG No. ICS-GS-1

HWY: 11

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





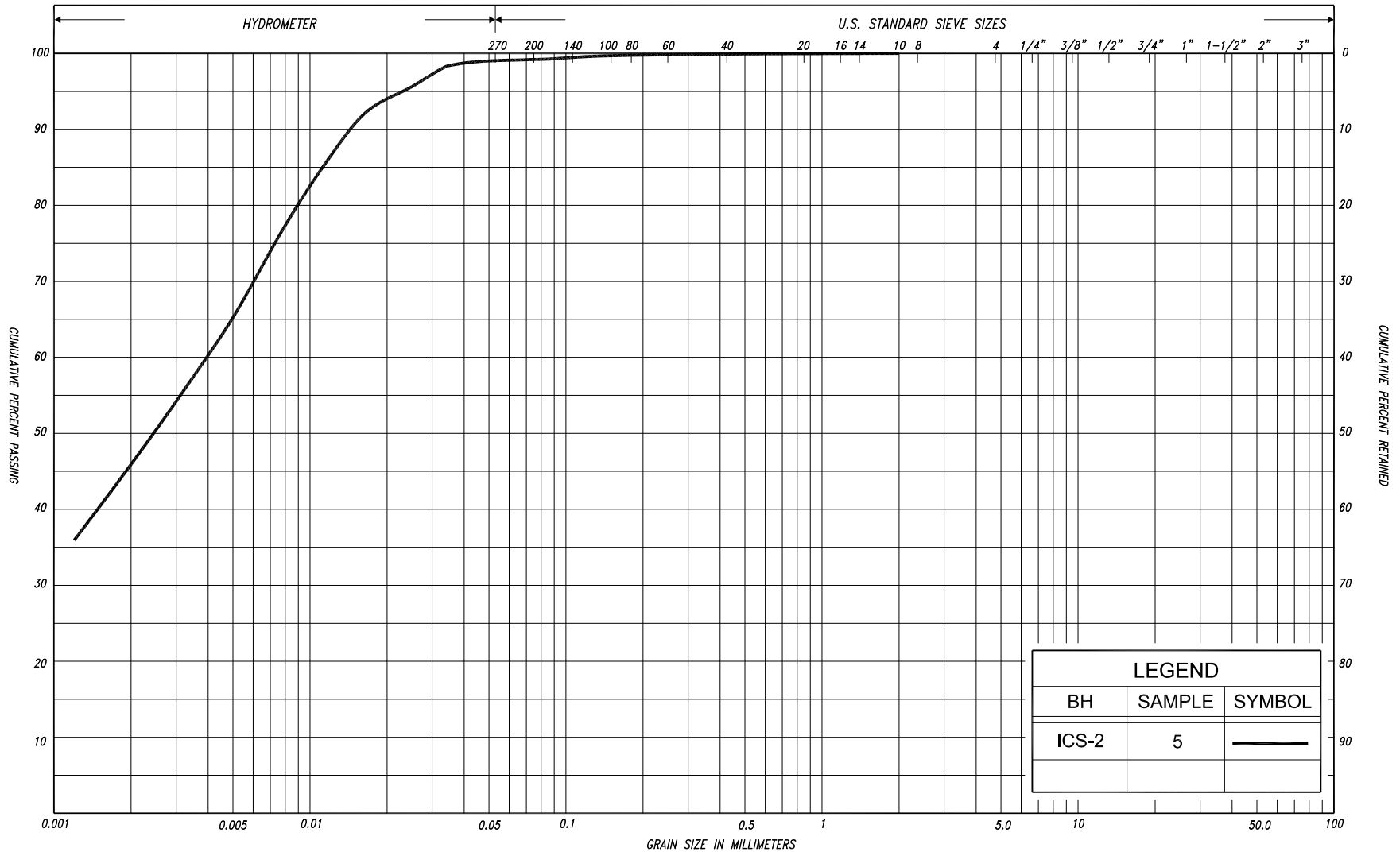
SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.
	SILT												GRAVEL			COBBLES	U.S. BUREAU
CLAY		SILT		V. FINE		FINE		MED.		COARSE							
SAND																	



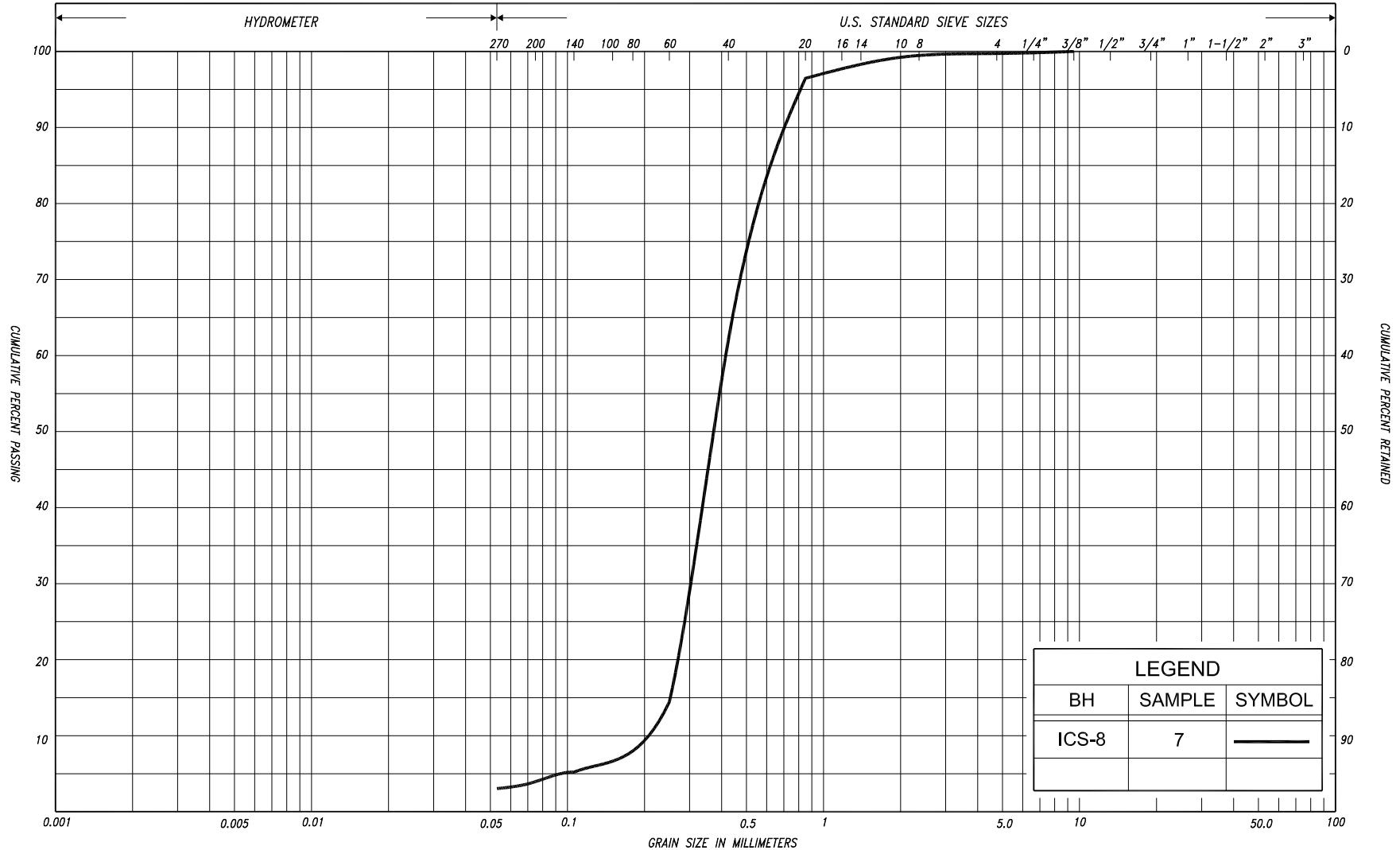
## GRAIN SIZE DISTRIBUTION

CLAYEY SILT, trace to with sand (CL)

FIG No. ICS-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

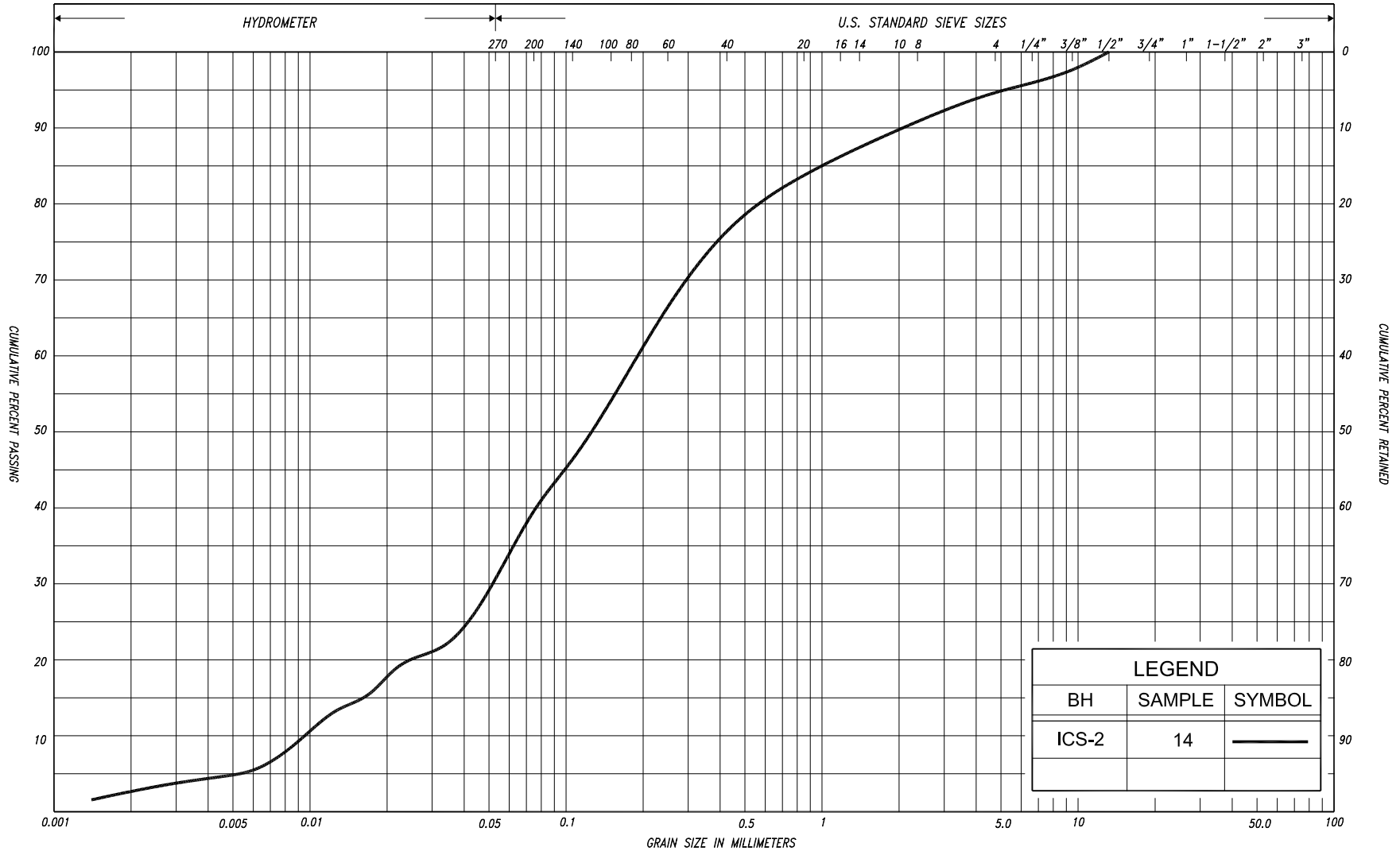
SAND, trace silt

FIG No. ICS-GS-4

HWY: 11

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





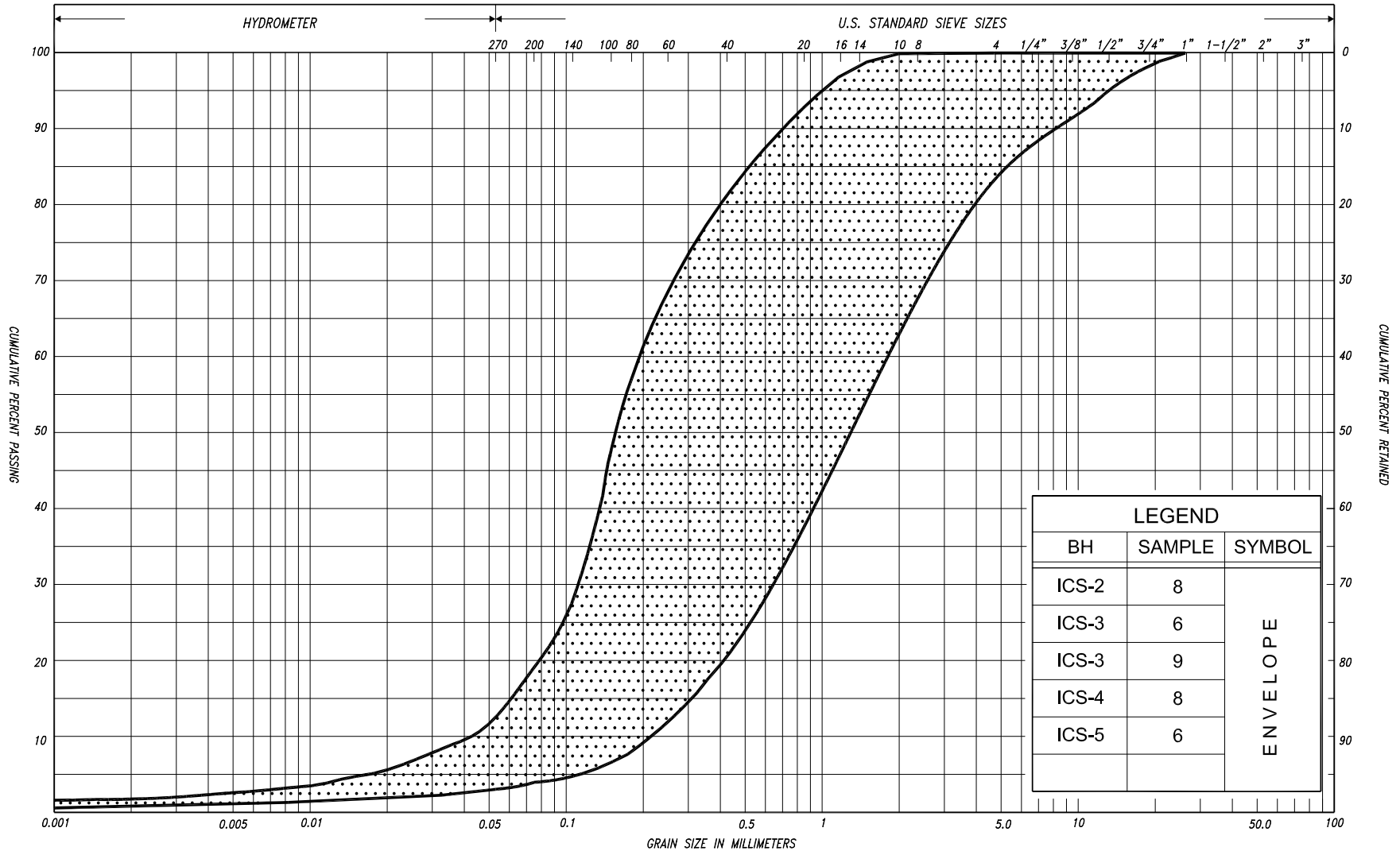
LEGEND		
BH	SAMPLE	SYMBOL
ICS-2	14	—

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												

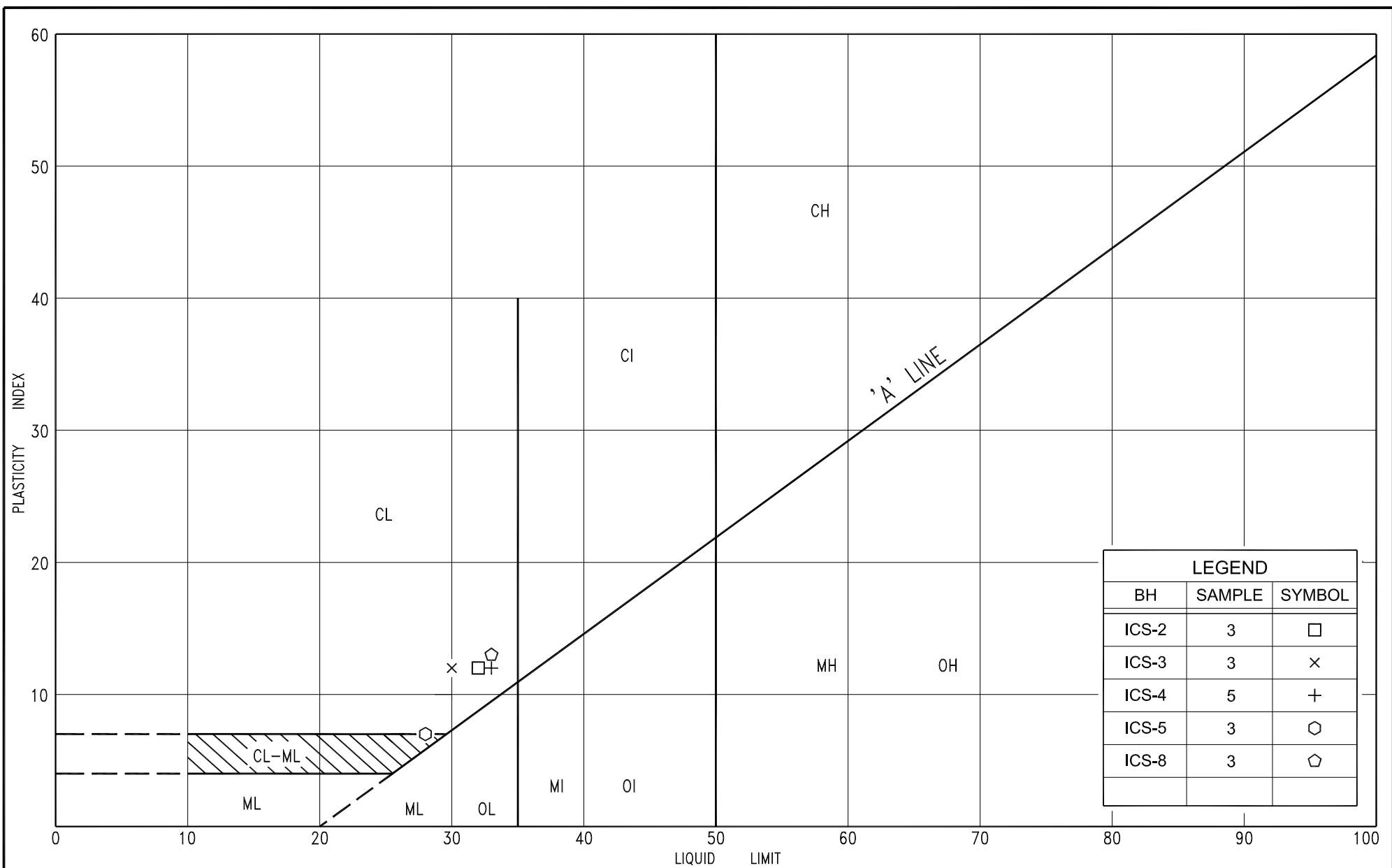


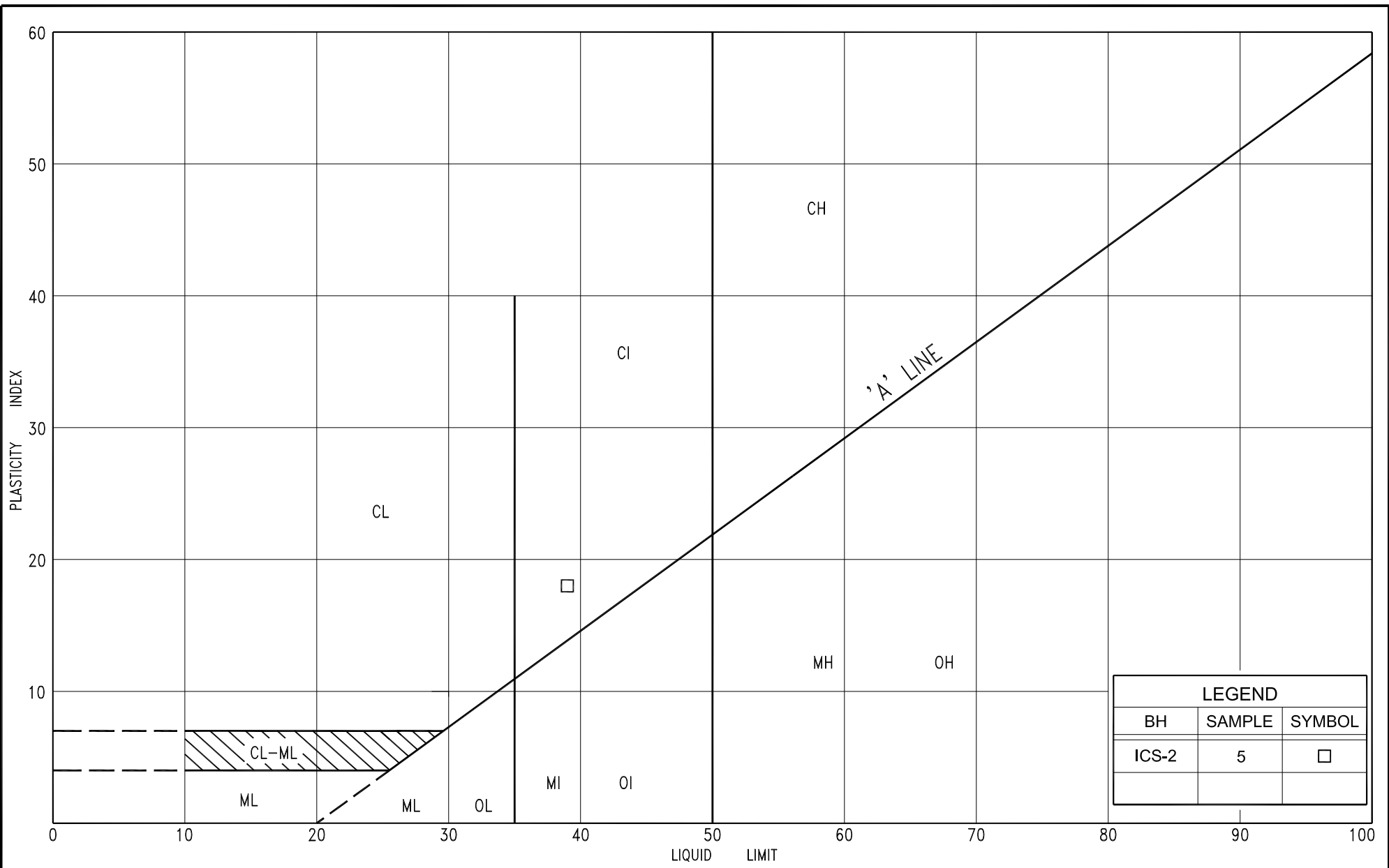
**GRAIN SIZE DISTRIBUTION**  
 SILTY SAND, trace clay, trace gravel  
 (TILL)

FIG No.	ICS-GS-5
HWY:	11
G.W.P. No.	323-00-00



SILT & CLAY				FINE		MEDIUM		COARSE		GRAVEL			COBBLES	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																





# PLASTICITY CHART SILTY CLAY, trace sand (CI)

FIG No. ICS-PC-2

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
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_i$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

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

**RECORD OF BOREHOLE No ICS-1**

1 of 1

**METRIC**

**G.W.P.** 323-00-00      **LOCATION** Co-ords: 5 101 903.8 N ; 316 485.4 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** November 17, 2011      **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								20 40 60 80 100										20 40 60		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
280.4	Ground Surface																			
0.0 280.1 0.3	Silty sand		1	SS	6		280													
	Loose Dark brown Moist (FILL)																			
279.2	Sandy silt, trace gravel organics		2	SS	20															
1.2	Loose to Brown Moist compact						279													
278.3	Sand trace silt, trace gravel		3	SS	53															
2.1	Compact to Brown Moist very dense (TILL)																			
	End of borehole																			
	Refusal on probable boulder																			
	* Borehole dry																			

**RECORD OF BOREHOLE No ICS-2**

1 of 2

**METRIC**

**G.W.P.** 323-00-00      **LOCATION** Co-ords: 5 101 922.4 N ; 316 478.0 E      **ORIGINATED BY** F.P.  
**DIST** North Bay HWY 11      **BOREHOLE TYPE** C.F.H.S.A. and NQ Diamond Coring      **COMPILED BY** N.S.B.  
**DATUM** Geodetic      **DATE** November 14 to 16, 2011      **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED + FIELD VANE									
								● QUICK TRIAXIAL × LAB VANE									
							WATER CONTENT (%)										
							20 40 60 80 100					20 40 60			GR SA SI CL		
280.6	Ground Surface																
0.0 280.3 0.3	Silty sand																
	Loose Dark Moist brown (FILL)		1	SS	8								○				
279.5	Sand, trace silt organics		2	SS	15								○				
1.1	Loose to Brown Wet compact																
	Clayey silt, trace sand		3	SS	14						138		○			0 8 56 36	
	Very stiff Mottled Moist to firm grey/ brown silt layers		4	SS	7								○				
	silty clay layers		5	SS	6								○			0 1 53 46	
	sandy silt layers		6	SS	8								○				
275.7	Sand, trace gravel		7	SS	17								○				
4.9	Compact Reddish Moist brown		8	SS	25								○			7 86 (7)	
274.6	Sand, some gravel cobbles and boulders to 9m		9	SS	86												
6.0	Very dense Reddish Moist brown		10	SS	50/5cm												
	(TILL)		11	SS	50/8cm												
			12	SS	50/8cm												
			13	SS	63												
270.2	Silty sand trace clay, trace gravel																
10.4	Dense to Brown Moist very dense		14	SS	35								○			5 56 36 3	
	(TILL)																
268.1			15	SS	67								○				
12.5	Granite bedrock		16	RC NQ	REC 100%											RQD 92%	
	Slightly weathered to unweathered		17	RC NQ	REC 93%											RQD 93%	
	High strength																
	Excellent quality		18	RC NQ	REC 98%											RQD 98%	



## RECORD OF BOREHOLE No ICS-2

2 of 2

METRIC

<b>G.W.P.</b> 323-00-00	<b>LOCATION</b>	Co-ords: 5 101 922.4 N ; 316 478.0 E	<b>ORIGINATED BY</b> F.P.
-------------------------	-----------------	--------------------------------------	---------------------------

<b>DIST</b>	North Bay	<b>HWY</b>	11	<b>BOREHOLE TYPE</b>	C.F.H.S.A. and NQ Diamond Coring	<b>COMPILED BY</b>	N.S.B.
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**DATUM** Geodetic **DATE** November 14 to 16, 2011 **CHECKED BY** C.N.

[illegible]

# RECORD OF BOREHOLE No ICS-3

1 of 1

**METRIC**

<b>G.W.P.</b>	<u>323-00-00</u>	<b>LOCATION</b>	<u>Co-ords: 5 101 930.7 N ; 316 499.1 E</u>	<b>ORIGINATED BY</b>	<u>F.P.</u>
<b>DIST</b>	<u>North Bay</u>	<b>HWY</b>	<u>11</u>	<b>BOREHOLE TYPE</b>	<u>C.F.H.S.A. and 'N' Casing</u>
<b>COMPILED BY</b>	<u>N.S.B.</u>				
<b>DATUM</b>	<u>Geodetic</u>	<b>DATE</b>	<u>December 02, 2011</u>	<b>CHECKED BY</b>	<u>C.N.</u>

SOIL PROFILE			SAMPLES		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES
280.5	Ground Surface				
0.0	Topsoil				
280.2	Silt, trace sand sandy silt seams		1	SS	1
0.3	Very loose Grey Moist to compact		2	SS	18
279.1	Clayey silt, with sand		3	SS	10
1.4	Stiff Grey Wet to firm		4	SS	5
277.1			5	SS	21
3.4	Sand trace silt, trace clay		6	SS	18
	Compact Reddish brown Moist		7	SS	17
274.5			8	SS	18
6.0	Sand some gravel, trace silt		9	SS	45
	Dense Reddish brown Moist (TILL)				
272.7			10	SS	50/8cm
7.8	End of borehole				
	Refusal on probable bedrock				
	Sample 10: Sampler bouncing				
	* Borehole dry				
	C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers				
	'N' casing advanced from 6.0m depth				

## RECORD OF BOREHOLE No ICS-4

1 of 1

METRIC

<b>G.W.P.</b> 323-00-00	<b>LOCATION</b>	Co-ords: 5 101 952.2 N ; 316 466.3 E	<b>ORIGINATED BY</b> F.P.
-------------------------	-----------------	--------------------------------------	---------------------------

<b>DIST</b>	North Bay	<b>HWY</b>	11	<b>BOREHOLE TYPE</b>	C.F.H.S.A. and NQ Diamond Coring	<b>COMPILED BY</b>	N.S.B.
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**DATUM** Geodetic **DATE** November 16 to 18, 2011 **CHECKED BY** C.N.

[illegible]

**RECORD OF BOREHOLE No ICS-5**

1 of 1

**METRIC**

**G.W.P.** 323-00-00      **LOCATION** Co-ords: 5 101 960.5 N ; 316 487.4 E      **ORIGINATED BY** F.P.  
**DIST** North Bay HWY 11      **BOREHOLE TYPE** C.F.H.S.A. and NQ Diamond Coring      **COMPILED BY** N.S.B.  
**DATUM** Geodetic      **DATE** December 06, 2011      **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE		● QUICK TRIAXIAL						× LAB VANE		
280.2	Ground Surface						20	40	60	80	100									
0.0 279.9	Topsoil																			
0.3	Sandy silt		1	SS	4	▽*														
279.4	Loose Brown Moist																			
0.8	Sand		2	SS	12															
278.8	Compact Brown Wet																			
1.4	Clayey silt, trace sand																			
	Firm Brown Wet		3	SS	8															
278.0																				
2.2	Sand, some silt, trace clay some gravel to 3.0m cobbles and boulders to 5.8m depth		4	SS	63															
	Compact to Reddish Moist very dense brown		5	SS	24															
	(TILL)																			
	trace gravel		6	SS	36															
			7	SS	50/15cm															
	some gravel		8	SS	62/15cm															
			9	SS	74															
273.4																				
6.8	Granite bedrock																			
	Slightly weathered		10	RC NQ	REC 100%															
	High strength																			
	Excellent quality		11	RC NQ	REC 97%															
			12	RC NQ	REC 97%															
270.1																				
10.1	End of borehole																			
	Samples 7 and 8: Sampler bouncing																			
	* Borehole charged with drilling water																			
	C.F.H.S.A. denotes Continuous Flight Hollow Stem Augers																			

**RECORD OF BOREHOLE No ICS-6**

1 of 1

**METRIC**

**G.W.P.** 323-00-00      **LOCATION** Co-ords: 5 101 966.2 N ; 316 460.8 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** November 17, 2011      **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
280.2	Ground Surface						20	40	60	80	100									
0.0 279.9	Silty sand																			
0.3	Very loose Dark brown (FILL) Moist		1	SS	3															
	Sand, trace silt		2	SS	8															
278.5	Very loose Brown to loose Wet																			
	Clayey silt silt seams		3	SS	4															
1.7	Firm to stiff Brown Wet		4	SS	6															
			5	SS	6															
			6	SS	8															
			7	SS	6															
274.6																				
5.6	Sand, some gravel		8	SS	37															
274.1	Dense Reddish brown (TILL) Moist																			
6.1	End of borehole																			
	Refusal on probable boulder																			

\* 2011 11 17

▽ Water level observed during drilling

■ Penetrometer test

**RECORD OF BOREHOLE No ICS-7**

1 of 1

**METRIC**

**G.W.P.** 323-00-00      **LOCATION** Co-ords: 5 101 902.4 N ; 316 485.9 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** November 17, 2011      **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE												
280.4	Ground Surface							20	40	60	80	100								
0.0 280.1	Silty sand	△																		
0.3	Loose Dark brown Moist (FILL)	•					280													
279.2	Sandy silt, trace gravel organics	•																		
1.2	Loose to compact Brown Moist	•																		
	End of auger probe																			
	Refusal on probable boulder																			
	* Auger probe dry																			

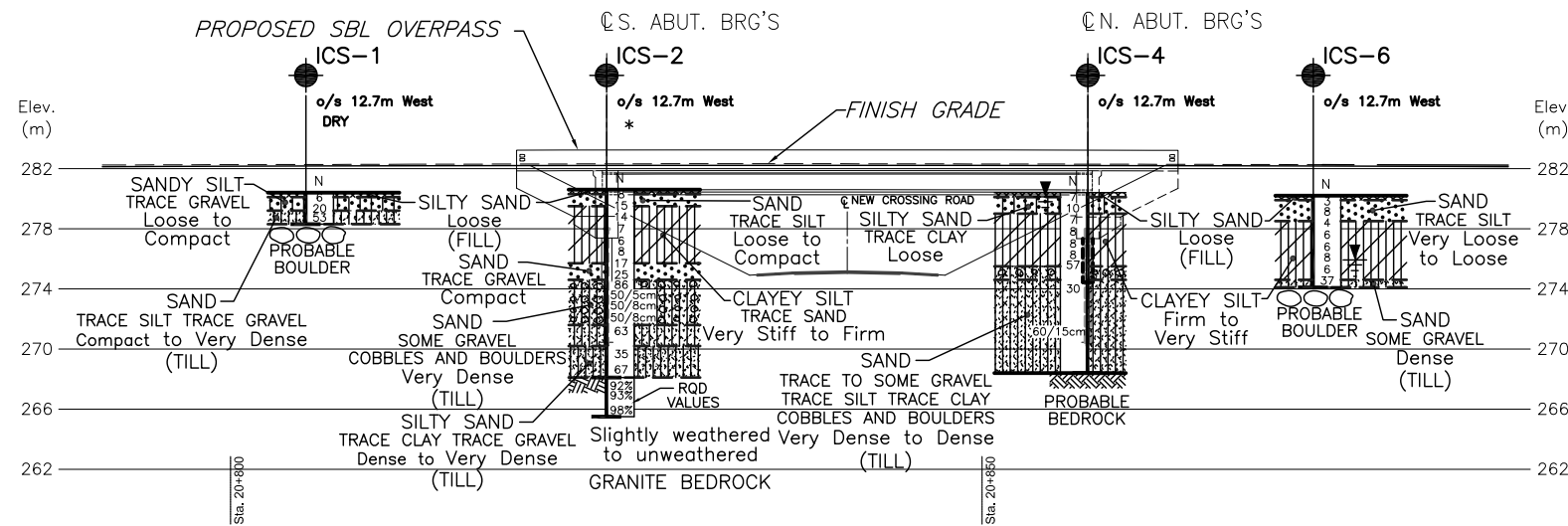
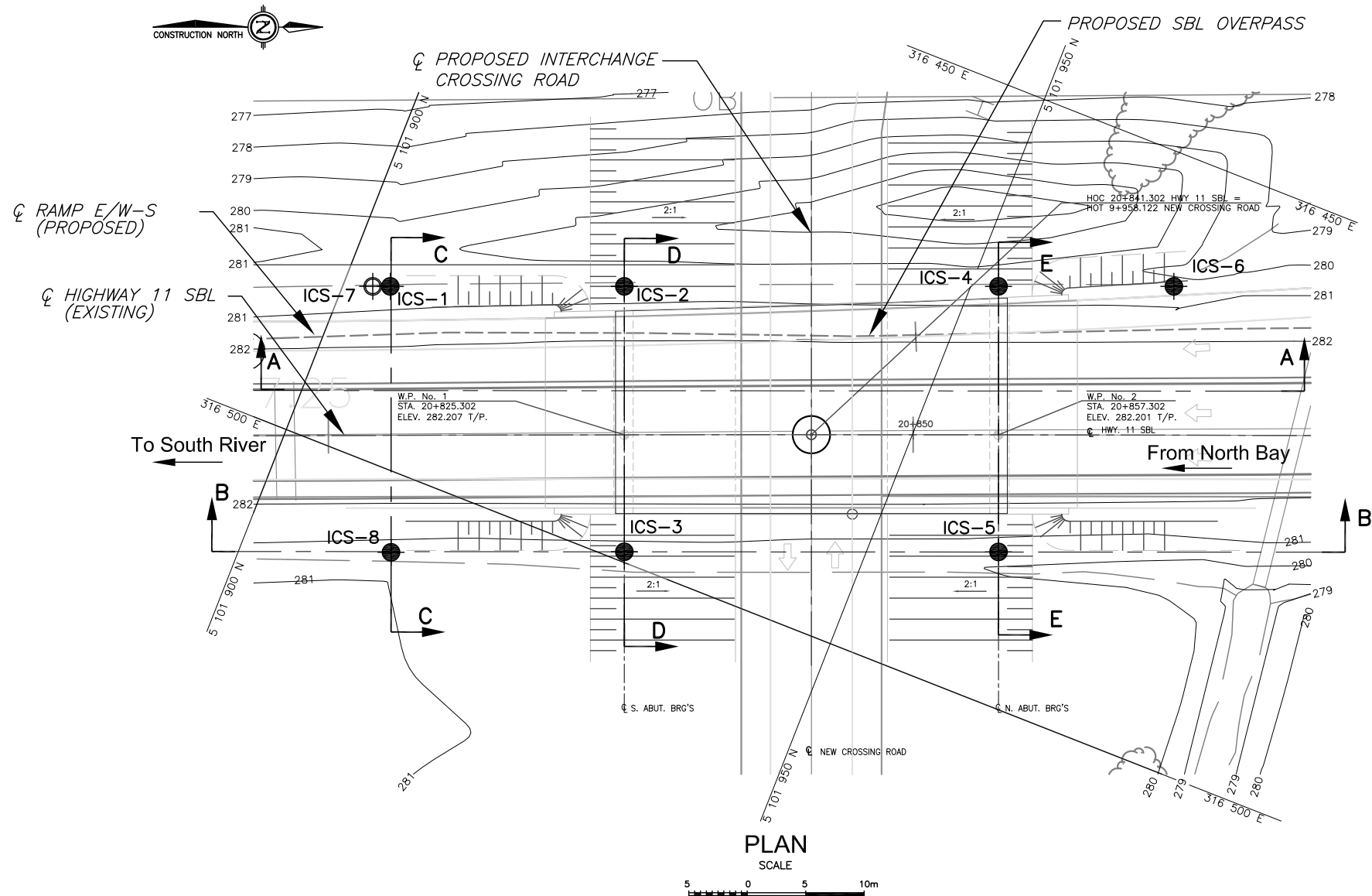
**RECORD OF BOREHOLE No ICS-8**

1 of 1

**METRIC**

**G.W.P.** 323-00-00      **LOCATION** Co-ords: 5 101 912.2 N ; 316 506.5 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 05, 2011      **CHECKED BY** C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa										WATER CONTENT (%)		
280.5	Ground Surface																		
0.0 280.2	Topsoil																		
0.3	Silty sand		1	SS	4	▽*													
279.6	Loose      Brown      Wet																		
0.9	Clayey silt, some sand		2	SS	11														
	Stiff to      Mottled      Wet very stiff grey																		
			3	SS	10						138						0 12 52 36		
			4	SS	6														
	Silt layers																		
			5	SS	11														
276.7																			
3.8	Sand, trace silt																		
	Compact      Reddish      Dry brown		6	SS	22														
			7	SS	20											0 96      (4)			
275.3																			
5.2	Sand, some gravel																		
	Compact      Reddish      Moist brown		8	SS	24														
	cobbles																		



NOTES:

1. DRAWINGS IS-1 AND IS-2 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. SEE DRAWING IS-2 FOR PROFILE B-B AND SECTIONS C-C, D-D AND E-E
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.

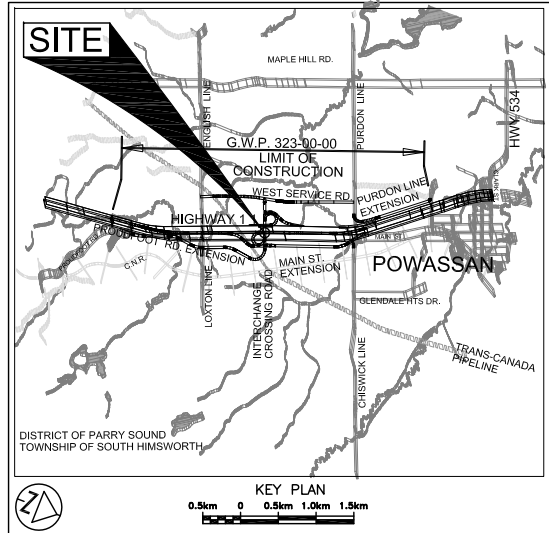
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GWP No 323-00-00  
WP No 5214-10-01

INTERCHANGE CROSSING ROAD OVERPASS  
HIGHWAY 11 SOUTHBOUND LANES  
BOREHOLE LOCATIONS AND SOIL STRATA



SHEET  
1

**PML Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



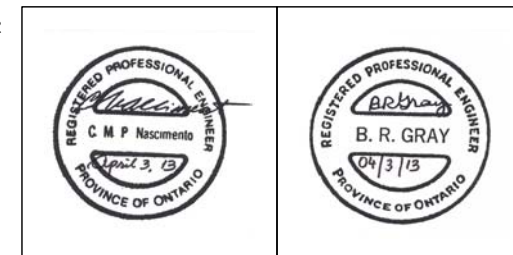
LEGEND

- Borehole
- Auger Probe
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL at time of investigation Nov. and Dec. 2011
- \* Water level not established
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
ICS-1	280.4	5 101 903.8	316 485.4
ICS-2	280.6	5 101 922.4	316 478.0
ICS-3	280.5	5 101 930.7	316 499.1
ICS-4	280.4	5 101 952.2	316 466.3
ICS-5	280.2	5 101 960.5	316 487.4
ICS-6	280.2	5 101 966.2	316 460.8
ICS-7	280.4	5 101 902.4	316 485.9
ICS-8	280.5	5 101 912.2	316 506.5

NOTE

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



REF AECOM Drawings: 60157537 Interchange Crossing Road  
Overpass\_SBL\_GA\_ALT2A.dwg dated April 2011; Hwy 11 Base-Trow.dwg;  
Hwy 11-Design.dwg; X-Hwy11-CONTOURS.dwg

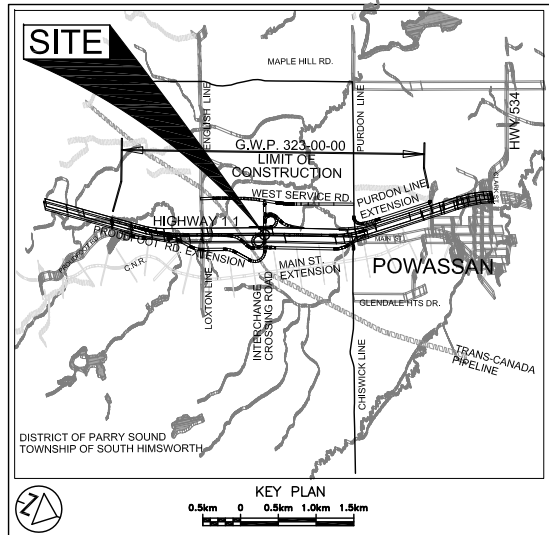
REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 31L-170

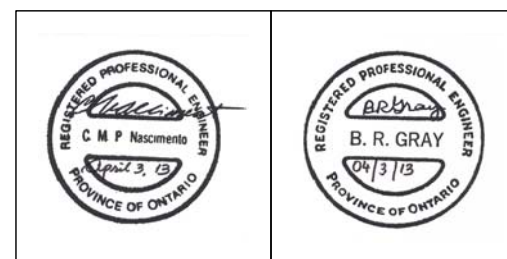
HWY No	11	CHECKED	NSB	DATE	APR. 03, 2013	DIST	North Bay
SUBM'D	NA	CHECKED	BRG	APPROVED	CN	SITE	44-505/2
DRAWN	NA	CHECKED	BRG	APPROVED	CN	DWG	IS-1



INTERCHANGE CROSSING ROAD OVERPASS HIGHWAY 11 SOUTHBOUND LANES SOIL STRATA	SHEET  2
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1. DRAWINGS IS-1 AND IS-2 SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. SEE DRAWING IS-1 FOR BOREHOLE LOCATIONS AND PROFILE A-A.
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.



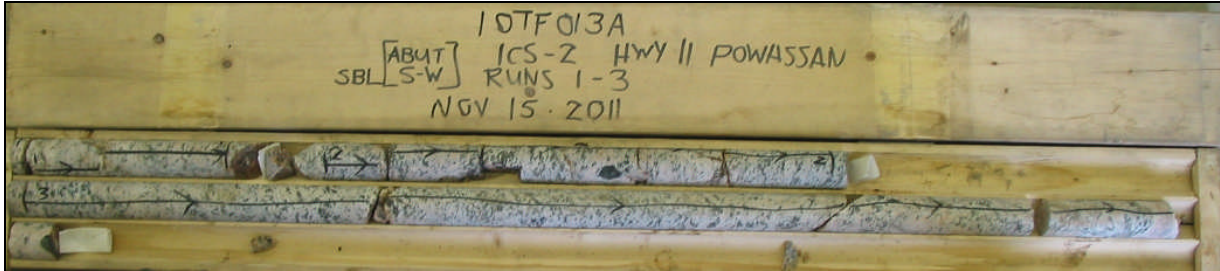
REVISIONS			
	DATE	BY	DESCRIPTION

HWY No	11			DIST	North Bay
SUBM'D	NA	CHECKED	NSB	DATE	APR. 03, 2013
DRAWN	NA	CHECKED	BRG	APPROVED	CN
				DWG	IS-2



## **APPENDIX A**

### Rock Core Photographs



**Photograph 1:** Cores retrieved from borehole ICS-2. Cores 16 to 18 from 12.5 to 15.1 m depth. RQD values ranged from 92 to 98%, indicating excellent rock quality.



**Photograph 2:** Cores retrieved from borehole ICS-5. Cores 10 to 12 from 6.8 to 10.1 m depth. RQD values ranged from 80 to 100%, indicating good to excellent rock quality.



**FOUNDATION DESIGN REPORT  
for  
INTERCHANGE CROSSING ROAD OVERPASS  
HIGHWAY 11 SOUTHBOUND LANES  
SITE NO. 44-505/2  
TOWNSHIP OF SOUTH HIMSWORTH  
NORTH BAY AREA, ONTARIO  
G.W.P. 323-00-00**

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## **FOUNDATION DESIGN REPORT**

for

Interchange Crossing Road Overpass, Highway 11 Southbound Lanes

Site No. 44-505/2

Township of South Himsworth

North Bay Area, Ontario

GWP 323-00-00

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### **1. INTRODUCTION**

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and approach embankments for the proposed construction of an overpass to carry traffic on the Highway 11 Southbound Lanes (SBL) and interchange E/W-S ramp over the proposed unnamed Interchange Crossing Road in Powassan, Ontario. This report was prepared for AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation (MTO).

The proposed overpass over the proposed Interchange Crossing Road will be constructed along the existing Southbound Lanes of Highway 11 approximately between Sta. 20+825.3 and 20+857.3, Highway 11 SBL chainage, in the Township of South Himsworth. The overpass is proposed as a single span structure with approximate length of 32 m between abutments and width of 17.4 and 18.5 m at the south and north abutments, respectively (refer to AECOM Drawing No. 1, dated April 2011, Highway 11 Interchange Crossing Road Overpass SBL, General Arrangement–ALT 2A).

The road grade of Highway 11 SBL at the overpass location is planned at elevation 282.2 at the abutments. The approach embankments to the structure at the existing highway centreline are envisaged to be rehabilitated with less than 0.1 m grade changes. The proposed E/W-S ramp widening will extend over the bridge and require a raise in the grade at the south and north approach embankments of approximately 1.3 and 0.3 m above the existing grades. The road grade of the proposed Interchange Crossing Road is planned to be at approximate elevation 275.0 that will require a 7.2 m cut below the existing highway grade level.

In summary, the surficial soil cover included 0.3 m thick fill units or topsoil. This surficial soils overlay loose to compact cohesionless silt to sand deposits (0.6 to 1.4 m thick) which in turn is underlain by continuous cohesive typically stiff to very stiff clayey silt with silty clay layers (0.8 to 3.8 m thick) followed by a compact cohesionless sand deposit (1.1 to 2.6 m thick). Below the sand, the stratigraphy included



a 0.5 to 7.1 m thick typically dense sand till / silty sand till. At the north abutment and west side of the south abutment locations, a 0.8 to 3.6 m thick layer of cobbles and boulders was encountered below the sand / clayey silt deposit within the upper zone of till deposit. Granite bedrock of high strength was contacted/inferred below the native soils at 6.8 to 12.5 m depths (elevation 268.1 to 273.4) in five boreholes (ICS-2 to ICS-5 and ICS-8). Two boreholes (ICS-1 and ICS-6) and one auger probe (ICS-7) drilled for the approach embankments were terminated by refusal on probable boulders at 1.2 to 6.1 m depths (elevation 274.1 to 279.2).

The groundwater levels were observed and estimated in the boreholes at 1.8 to 4.8 m depths (elevation 275.4 to 278.7) and may vary due to seasonal fluctuations and rainfall patterns.

Based on the encountered subsurface conditions and the height of the embankments over the existing ground surface, perched spread footings placed on the native clayey soils would require relatively large footings in view of the low load carrying capacity of these soils. Since the existing ground surface will be lowered by about 7.2 m depth to elevation 275.0 for the proposed Interchange Crossing Road as indicated previously, the proposed overpass may be founded on spread footings placed at deeper levels on the dense to very dense sand till deposit allowing for a foundation frost depth of 1.9 m. The upper 0.6 m zone of the bedrock encountered 1.6 m below the level of the Interchange Crossing Road level in borehole ICS-5 should be excavated and filled below the footing level with OPSS Granular B Type II material to avoid point loading and differential settlement of the north abutment spread footing. This alternative would require a wide excavation to construct the footings some 9.0 m below the existing highway pavement or shoring below the proposed Interchange Crossing Road level.

Because of the potential delays and installation difficulties caused by the high water level and the layers of cobbles and boulders present within the till deposits, the use of drilled cast-in-place concrete caissons is not considered suitable for this site.

The overpass may also be founded on deep foundations using steel H-piles driven to refusal on bedrock. It is noted that the presence of layers of cobbles and boulders will likely require pre-drilling at the pile locations or heavy pile driving to advance the piles to the bedrock levels. The steel HP-section piles should be equipped with driving shoes due to the presence of boulders within the till soils found above the bedrock.



To obtain an adequate long term factor of safety of 1.5, the slopes for the new Interchange Crossing Road should be cut at 2.5H:1V slope or flatter in view of the existing subgrade conditions.

Potential seepage from sandy layers encountered at the level of the Interchange Crossing Road pavement should be mitigated. We refer to the Pavement Design Report prepared by PML, Reference No. 13TF013B for design recommendations.

The "red flag" issues outlined in the preceding paragraph 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. These comments and recommendations are based on the conditions revealed during the investigations and no responsibility is assumed by the consultants or the MTO for alerting the contractor to all critical issues for each foundation alternative. The requirements to deliver acceptable construction quality remain the responsibility of the contractor.

The elevations referred in this report are expressed in meters. A list of the Ontario Provincial Standard documents referenced in this report is enclosed in Table 1.

## **2. FOUNDATIONS**

### **2.1 General**

It is considered that the fill and organic soils are highly compressible, therefore are not considered to be suitable to support the overpass foundations.

Spread footings placed on the stiff to very stiff cohesive native clayey soils may be used for semi-integral or conventional abutment design, however the relatively low geotechnical resistances of these soils will require wide spread footings. In view of the proposed Interchange Crossing Road that is planned at about elevation 275.0, up to 7.2 m below existing highway grade level, the overpass spread footings foundations may be placed on the underlying very dense to dense till deposit. These cohesionless deposits have a higher geotechnical resistance that is considered to be suitable for a conventional abutment design (rigid frame structure).

The assessment of the feasibility of using cast-in-place concrete drilled caissons bearing on the glacial till or on the bedrock to support the overpass should consider the difficulties and potential





delays caused by the presence of cobbles and boulders in the till and compact cohesionless sand as well as the groundwater control requirements.

Founding the proposed overpass on steel H-piles driven to refusal on the underlying granite bedrock is considered feasible for the south and north abutments.

Conventional, semi-integral and integral abutments are considered feasible at this site based on the foregoing considerations. The type of foundation employed to support the foundation loads of the proposed structure and the system of bridge design will be dictated by structural considerations, economic considerations and construction constraints. From a foundations engineering perspective, use of integral abutments supported on piles driven to refusal on bedrock is the preferred type of abutment foundation.

The seismic site coefficient for the conditions at this site is 1.0 (Type I soil profile as per clause 4.4.6 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00). The zonal acceleration ratio is 0.05. The bridge site is located in a Seismic Performance Zone 1.

All footings subject to frost action should be provided with 1.9 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the CHBDC.

## **2.2 Spread Footings on Native Soils**

### **2.2.1 Perched Spread Footings**

Perched spread footings that should be founded above the elevation 275.0 level of the Interchange Crossing Road pavement may be used for conventional or semi-integral abutments. The reference founding levels for these spread footings placed on 1.9 m below the final grade of the foreslope for frost protection approximate elevation 277.0 on the native clayey soils or



cohesionless soils encountered at the proposed south and north abutments are provided in the following table.

FOUNDATION UNIT	SUBGRADE TYPE	BOREHOLE	REFERENCE LEVELS	
			ELEVATIONS	DEPTHS* (m)
South Abutment	Stiff to Very Stiff Clayey Soils	ICS-2	277.0	3.6
	Compact Sand	ICS-3	277.0	3.5
North Abutment	Stiff to Very Stiff Clayey Soils	ICS-4	277.0	3.4
	Compact to Very Dense Sand Till	ICS-5	277.0	3.2

\* Depth from the existing ground surface

The following uniform geotechnical resistances should be used for design of the spread footings:

FOUNDATION UNIT	SUBGRADE TYPE	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL REACTION AT SLS (kPa)
South Abutment	Stiff to Very Stiff Clayey Soils or Compact to Very Dense Cohesionless Sandy Soils	250	150
North Abutment			

For the purpose of the computations for geotechnical resistances a 2.0 m wide footing was assumed and the stabilized water level was assessed to be at least 1.0 m below the footing subgrade level. The geotechnical reaction at SLS normally allows for 25 mm of settlement.

This relatively low value of geotechnical resistance may require relatively large abutment footings and make the spread footing alternative impractical for structural design. A more feasible spread footing alternative is outlined in the following section of this report.

Construction of the footings should be performed and monitored in accordance with OPSS 902 to verify the competency of the founding surfaces.



### 2.2.2 Deep Level Spread Footings

Since the Interchange Crossing Road will require an excavation to elevation 275.0, spread footings placed at elevation 273.1 that is 1.9 m below the proposed road level for frost protection is considered a feasible alternative. These deep level footings should be founded on the dense to very dense sand till encountered in the boreholes and may be designed for higher geotechnical bearing resistance than the perched footings outlined in the previous section of this report.

For these deeper level footings, the proposed overpass may be designed as a semi-integral abutment or as a rigid frame structure founded on spread footings.

The reference founding levels for spread footings placed on the native sand till deposit and/or bedrock at the south and north abutments are provided in the following table:

FOUNDATION UNIT	SUBGRADE TYPE	BOREHOLE	REFERENCE LEVELS	
			ELEVATIONS	DEPTHS (*) (m)
South Abutment	Very Dense Sand Till	ICS-2	273.1	7.5
	Dense Sand Till	ICS-3	273.1	7.4
North Abutment	Dense to Very Dense Sand Till	ICS-4	273.1	7.3
	Bedrock (***)	ICS-5	273.4	6.8

(\*) Depth from the existing ground surface.

(\*\*) Bedrock should be locally excavated to avoid point loads.

The upper 0.6 m zone of the bedrock encountered 1.6 m below the level of the Interchange Crossing Road level in borehole ICS-5 should be excavated and filled below the footing level with OPSS Granular B Type II material to avoid point loading and differential settlement of the north abutment spread footing. This alternative would require a wide excavation to construct the footings some 9.0 m below the existing highway pavement or shoring below the proposed Interchange Crossing Road level. The areal extent of the bedrock excavation will be determined once the excavation is carried out to the foundation level and the eastern one third of the footing area should be considered for tendering purposes. The OPSS Granular B Type II fill material under the footing should be compacted according to OPSS 501 to 95% of its maximum dry density.



Footings founded directly on bedrock where applicable do not require foundation frost protection.

The following uniform geotechnical resistances should be used for design of the spread footings:

FOUNDATION UNIT	SUBGRADE TYPE	FACTORED GEOTECHNICAL RESISTANCE AT ULS (kPa)	GEOTECHNICAL REACTION AT SLS (kPa)
South Abutment	Dense to Very Dense Sand Till	700	450
North Abutment			
North Abutment	Bedrock	5,000 (*)	N/A

(\*) Not recommended to avoid point loading under the footing.

A 2.0 m wide footing was assumed and the stabilized water level was assessed to be at least 1.0 m above the footing subgrade levels. The geotechnical reaction at SLS normally allows for 25 mm of settlement since bedrock is considered an unyielding founding medium, placing only part of the footing directly on the bedrock is not recommended, as outlined previously in this report.

Construction of the footings should be performed and monitored in accordance with OPSS 902 to verify the competency of the founding surfaces.

Considering that the excavations for these footings will be some 9.0 m below the existing highway pavement, consideration should be given to the use of an earth supporting system below the level of the Interchange Crossing Road, elevation 275.0 to minimize the extent of the excavations. The selection and detail design of temporary earth supporting systems should be carried out by the contractor.

### 2.2.3 Sliding Resistance

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the founding soils. The following parameters should be used for sliding resistance of cast-in-place concrete spread footings placed on native soils.



PARAMETER	STIFF TO VERY STIFF CLAYEY SILT	COMPACT SAND / SAND TILL	DENSE / VERY DENSE SAND TILL
Friction angle, degrees	0	30	35
Cohesion, kPa	50	0	0
Unit weight, kN/m <sup>3</sup>	19.0	20.0	23.0

### 2.3 Pile Foundation

Steel H-piles could be used to support the foundation loads at both abutments. The piles should be driven to refusal on bedrock anticipated at depths from the existing ground surface of 7.8 and 12.5 m (elevation 268.1 and 272.7) at the south abutment and 6.8 and 12.0 m (elevation 268.4 and 273.4) at the north abutment.

The following factored geotechnical axial resistance at ULS for the following sections of steel piles is considered to be appropriate (refer to notes 5 and 6 in Section 3.3.3 of the Pile Driving Notes in the Structural Manual, June 2011):

PILE SECTION	FACTORED GEOTECHNICAL AXIAL RESISTANCE AT ULS (kN)
HP 310 x 110	2000
HP 360 x 152	2800

The geotechnical reaction at SLS allows for 25 mm compression of the founding medium. Considering the bedrock to be non-yielding, the design is not expected to be governed by settlement criteria since the loading required to produce 25 mm deformation of the bedrock would be larger than the factored geotechnical resistance at ULS.

As indicated previously, cobbles and boulders were encountered in the boreholes. A NSSP should be prepared to advise the contractor of the potential presence of boulders at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in OPSS 903 and that heavy pile driving with pre-augering may be locally required.

It is anticipated that the working platforms to drive the piles will be cut into the existing embankment. Any additional fill that may be required at these locations should comprise OPSS Granular A material to allow installation of the piles without damage. Alternative granular



materials such as Granular B Type II could be employed provided the maximum particle size does not exceed 75 mm.

The piles will be driven through variable compact to very dense soils containing cobbles and boulders and set on or into bedrock that exhibits a surface inclination varying from 0.3 to 13.1° between borehole locations. It is considered that the very dense soils above the bedrock will provide adequate pile tip fixity at this site. OPSS 903 calls for the use of OPSD 3000.100 (Steel HP310 Driving Shoe) or Titus H Bearing Pile Points Standard Model on piles driven to bedrock under these conditions.

The piles should be installed and monitored in accordance with the requirements of OPSS 903. This should involve confirmation of the founding elevation, alignment, plumbness, uniformity of set and quality of splices and should be done on a full-time basis by experienced geotechnical personnel.

Pile caps should be provided with at least 1.9 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

#### 2.3.1 Integral Abutment Considerations

For the integral abutment design, the H-piles should be also driven to refusal on bedrock anticipated at the depths/elevations and axial resistance that are indicated in the previous section. The minimum 5.0 m long free pile length below the abutment stem which should be incorporated in the design will not be a concern at this site.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type I. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.



### 2.3.2 Lateral Resistance

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The recommended lateral resistance is as follows:

	NATIVE CLAYEY SOILS		GRANULAR BACKFILL OR DENSE TO VERY DENSE COHESIONLESS SOILS		COMPACT COHESIONLESS SOILS	
Pile Section	HP 310	HP 360	HP 310	HP 360	HP 310	HP 360
Factored Lateral Resistance at ULS, kN	160	160	120	170	110	150
Lateral Resistance at SLS, kN	65	80	50	70	40	50

If greater resistance is required, batter piles should be installed.

The coefficient of horizontal subgrade reaction,  $k_s$ , should be computed using the following equation to evaluate the point of contraflexure:

$$k_s = n_h z/b$$

where  $n_h$  = coefficient related to soil density,  $\text{kN/m}^3$   
= 10,000 for granular backfill  
 $z$  = depth, m  
 $b$  = pile width, m

The coefficient of horizontal subgrade reaction,  $k_s$ , for the native clayey silt units should be taken as  $28,000 \text{ kN/m}^3$ .

Group action for lateral loading should be considered when the pile spacing in the direction of the loading is less than eight pile diameters/widths. Group action can be evaluated by reducing the coefficient of horizontal subgrade reaction in the direction of loading by a reduction factor,  $R$ , as follows:

PILE SPACING IN DIRECTION OF LOADING $d$ = PILE DIAMETER OR WIDTH	SUBGRADE REACTION REDUCTION FACTOR, $R$
8d	1.00
6d	0.70
4d	0.40
3d	0.25



## 2.4 Comparison of Foundation Alternatives

Caisson foundations were not considered to be practical in view of installation difficulties due to high groundwater table with water bearing stratum and presence of layers of cobbles and boulders in the glacial till stratum. Spread footings placed on structural fill were not considered due to the presence of the proposed Interchange Crossing Road which is planned to be cut below the existing grade.

A comparison of the relative advantages and disadvantages related to each of the feasible foundation alternatives discussed in the preceding paragraph is presented below.

ADVANTAGES	DISADVANTAGES
<b>Perched Spread Footings on Native Soils</b>	
<ul style="list-style-type: none"> <li>• Lower cost than deep foundations</li> <li>• Allows use of semi-integral or conventional abutment designs</li> </ul>	<ul style="list-style-type: none"> <li>• Relatively low geotechnical resistances will require wide footing and may render this alternative structurally impractical</li> </ul>
<b>Deep Level Spread Footings</b>	
<ul style="list-style-type: none"> <li>• Allows use of semi-integral or rigid frame abutments</li> <li>• Higher bearing resistance than perched footings</li> <li>• Feasible and practical alternative due to cut required for Interchange Crossing Road construction</li> </ul>	<ul style="list-style-type: none"> <li>• More costly than perched footings</li> <li>• Groundwater control will be required for footing construction</li> <li>• May require temporary earth cut support below the Interchange Crossing Road level to minimize extent of excavation and groundwater control</li> </ul>
<b>Driven Piles</b>	
<ul style="list-style-type: none"> <li>• Allows use of integral and semi-integral abutments</li> <li>• Lower long-term deck maintenance costs with integral or semi-integral abutment design</li> <li>• Negligible settlements of foundations</li> </ul>	<ul style="list-style-type: none"> <li>• More costly than shallow foundation alternatives</li> <li>• Heavy equipment for pile driving is required.</li> <li>• May require pre-augering through layers of boulders</li> </ul>





### 3. ABUTMENT WALLS

The abutment walls should be designed to resist the unbalanced lateral earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure may be computed using the equivalent fluid pressure diagrams presented in Section 6.9 of the CHBDC or employing the following equation, assuming a triangular pressure distribution:

$$p = K (\gamma h + q) + C_p + C_s$$

where  $K$  = coefficient of lateral earth pressure (dimensionless)  
 $\gamma$  = unit weight of free-draining granular material,  $\text{kN/m}^3$   
 $h$  = depth below final grade, m  
 $q$  = surcharge load, kPa, if present  
 $C_p$  = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)  
 $C_s$  = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)  
where  $\phi$  = angle of internal friction of retained soil ( $35^\circ$  for Granular B Type II)  
 $\delta$  = angle of friction between the soil and wall ( $23.5^\circ$  for Granular B Type II)

The seismic site coefficient and zonal acceleration ratio for the conditions at this site were provided in section 2.1 of this report.

Free-draining granular material or rockfill should be used as backfill behind the walls. The following parameters are recommended for design:

PARAMETERS	Granular A or Granular B Type II	ROCKFILL
Angle of Internal Friction, degrees	35	42
Unit Weight, $\text{kN/m}^3$	22.8	18.0
Coefficient of Active Earth Pressure $K_a$	0.27	0.20
Coefficient of Earth Pressure At-Rest $K_o$	0.43	0.33
Coefficient of Passive Earth Pressure $K_p$	3.69	5.04

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



A weeping tile system (MTO SP 405F03 and OPSD 3190.100) should be installed to minimize the build-up of hydrostatic pressure behind the walls. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system.

Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standards Drawings for granular or rock backfill at abutments (OPSD 3101.150 and 3101.200), as applicable.

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to OPSS 501 for additional information in this regard.

#### **4. APPROACH EMBANKMENTS**

The level of the approach embankments will be typically the existing grades at the Highway 11 SBL centreline and no new fill placement is anticipated at the south and north approach embankments of the existing Highway 11 SBL centreline. The proposed E/W-S ramp widening for south and north approach embankments will be about 0.3 to 1.3 m above the existing grades.

It is anticipated that the new embankments will be constructed with earth fill and/or granular material similar to the existing embankment or rockfill, if available. Construction of the embankment fill for the abutments on the existing soils is considered to be feasible.

The embankment widening for the E/W-S ramp should be carried out in general accordance with OPSD 203.020. The loose fill identified in the boreholes located on the west side (ICS-2, ICS-4, ICS-6 and ICS-7) of the abutment locations and present along the alignment of the E/W-S ramp approach fills within 20 m of the abutments should be stripped prior to placement of the embankment fill. Backfilling adjacent to the structure abutments should be carried out in conformance to Ontario Provincial Standards Drawings for granular or rock backfill at abutments (OPSD 3101.150 or 3101.200), as applicable. Granular A or Granular B Type II (maximum particle size of 75 mm) should be employed within the embankment fill where piles will be driven, if applicable.



The embankments should be constructed in accordance with OPSD 200.020, 201.020 (if applicable), 202.010, 208.010 and OPSS 206. The side slopes of the approach embankments should be inclined no steeper than 2H: 1V for earth fill and 1.25H: 1V for rockfill, if utilized.

It is considered that the approach embankments constructed in accordance with the foregoing recommendations will be stable.

Since no new fill placement anticipated at the existing centerline, no additional settlement of the approach embankments at the existing Highway 11 SBL centreline is expected.

As indicated previously, the grade raise of about 1.3 m is anticipated at the E/W-S ramp widening portion. Settlement of the road surface within the approach embankments to the E/W-S ramp widening portion should be expected as a result of consolidation of the new embankment fill. Since it is inferred from the existing topography that the existing grade at the E/W-S ramp widening portion was loaded with approximately 1.5 m thick native soils before construction of the existing SBL embankment, settlements of the road surface due to consolidation of the underlying native cohesive clayey silt and compact sand and the compact to very dense glacial till and the bedrock at both embankments will be negligible.

The estimated magnitude of settlement of new granular material is in the order of 10 mm at the at the E/W-S ramp widening near the abutments. These granular materials will settle during construction and/or within about one month of fill placement.

It is considered that earth fill utilizing local native soils will be susceptible to surface erosion, in view of the silty nature of soils. Earth fill slopes should be protected against surface erosion by sodding (OPSS 803) and suitable vegetation. Refer to OPSS 804 for time constraints and type of seed and mulch required. Local areas of concentrated surface water flow should be protected with additional measures, such as rip-rap, rock protection or granular sheeting (OPSS 511).

## **5. INTERCHANGE CROSSING ROAD**

As indicated previously, the proposed Interchange Crossing Road is planned about 7.2 m below the existing grade at the overpass location. In addition, at the abutment locations the existing grade will be raised about some 1.3 m above existing grade at the E/W-S ramp widening portion.



Slope stability analyses were carried out for the proposed Interchange Crossing Road cut slope for long-term condition (effective stress analysis which governs for the cut slope stability). Based on the soil data and laboratory tests conducted on selected samples, the table below summarizes the soil parameters applied to the analyses:

SOIL TYPE	UNIT WEIGHT (kN/m <sup>3</sup> )	LONG-TERM ANALYSIS	
		EFFECTIVE COHESION (kPa)	EFFECTIVE FRICTION ANGLE (Degrees)
Granular Fill	22.5	0	35
Silty Sand to Sand	19	0	30
Clayey Soils	19	8	28
Sand to Silty Sand Till	23	0	35

The stability of the proposed Interchange Crossing Road cut section was analysed using the limit equilibrium methods and the SLOPE/W software developed by Geo-Slope International Ltd. The software analyses numerous potential failure surfaces and establishes a minimum safety factor aided by user input.

The slope stability analyses were carried out for cut slope angles of 2H: 1V, 2.25H:1V and 2.5H:1V. The results of the slope stability analyses are provided in Figures A-1 to A-3 attached in Appendix FDR-1 and listed below:

LOCATION	CUT SLOPE	LONG-TERM CONDITION FACTOR OF SAFETY	FIGURE NO.
Interchange Crossing Road	2H:1V	1.38	A-1
	2.25H:1V	1.42	A-2
	2.5H:1V	1.49	A-3

A factor of safety of 1.5 or greater is generally considered geotechnically adequate for a long-term stable slope. Based on analyses results, Interchange Crossing Road cut slope should be 2.5H:1V or flatter to obtain the adequate long-term stable slope.

The proposed Interchange Crossing Road will likely intercept wet-sand layers that will provide a persistent groundwater flow into the road cut. We refer to the Pavement Design Report prepared by PML, Reference No. 10TF013B for recommendation on mitigation measures of potential problems that may occur due to these potential groundwater flows.



## **6. CONSTRUCTION CONSIDERATIONS**

### **6.1 Excavation**

All excavation at the structure foundation sites should be carried out in accordance with the Occupational Health and Safety Act (OHSA), local and MTO regulations. For this purpose, the upper typically loose to compact silt to sand and firm to very stiff clayey silt are classified as Type 3 soil according to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Any cobbles or boulders exposed on the excavation slope faces must be removed.

### **6.2 Groundwater Control**

In the boreholes drilled at the abutments, groundwater was observed during drilling and was measured upon completion of drilling. The observed groundwater depths typically ranged from 0.3 to 4.3 m (elevation 275.9 to 280.2) in boreholes ICS-4, ICS-6 and ICS-8. The long term groundwater levels were estimated to be at 1.8 to 4.8 m depths (elevation 275.4 to 278.7) and may vary due to seasonal fluctuations and rainfall patterns.

Subject to the ground water level at the time of construction, it is anticipated that conventional sump pumping techniques may be adequate to control seepage of groundwater into the perched foundation excavations for spread footing (elevation 277.0) or pile cap (elevation 277.5 to 279.0). However, groundwater control including dewatering will likely be required for excavations to construct deep spread footing at about elevation 273.1. The contractor is responsible for the selection, performance and detailed design of the dewatering system. Any seepage water that enters the excavation during construction from the excavation slope may be handled by sump 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.

Surface water run-off should be diverted away from the excavations to ensure that the foundations are constructed in the dry.

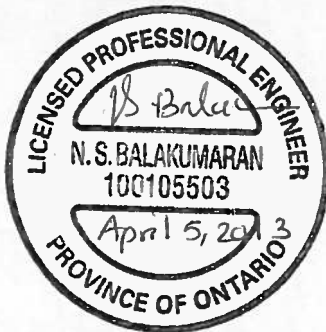


## 7. CLOSURE

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



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



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

NB/CN/BRG:nb-mi



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

<b>DOCUMENT</b>	<b>TITLE</b>
OPSS 206	Construction Specification for Grading
OPSS 501	Construction Specification for Compacting
OPSS 511	Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting
OPSS 803	Construction Specification for Sodding
OPSS 804	Construction Specification for Seed and Cover
OPSS 902	Construction Specification for Excavation and Backfilling - Structures
OPSS 903	Construction Specification for Deep Foundations
SP 405F03	Construction Specification for Pipe Subdrains
OPSD 200.020	Earth/Shale Grading-Divided Rural
OPSD 201.020	Rock Grading-Divided Rural
OPSD 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD 203.020	Embankments Over Swamp – Existing Slope Excavated to 1H:1V
OPSD 208.010	Benching of Earth Slopes
OPSD 3000.100	Foundation Piles – Steel H-Pile Driving Shoe
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD 3101.200	Rock Backfill - Walls Abutment
OPSD 3190.100	Retaining Wall and Abutment Wall Drain Detail



**TABLE 2**  
**GRADATION SPECIFICATION FOR SAND FILL IN**  
**PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS**

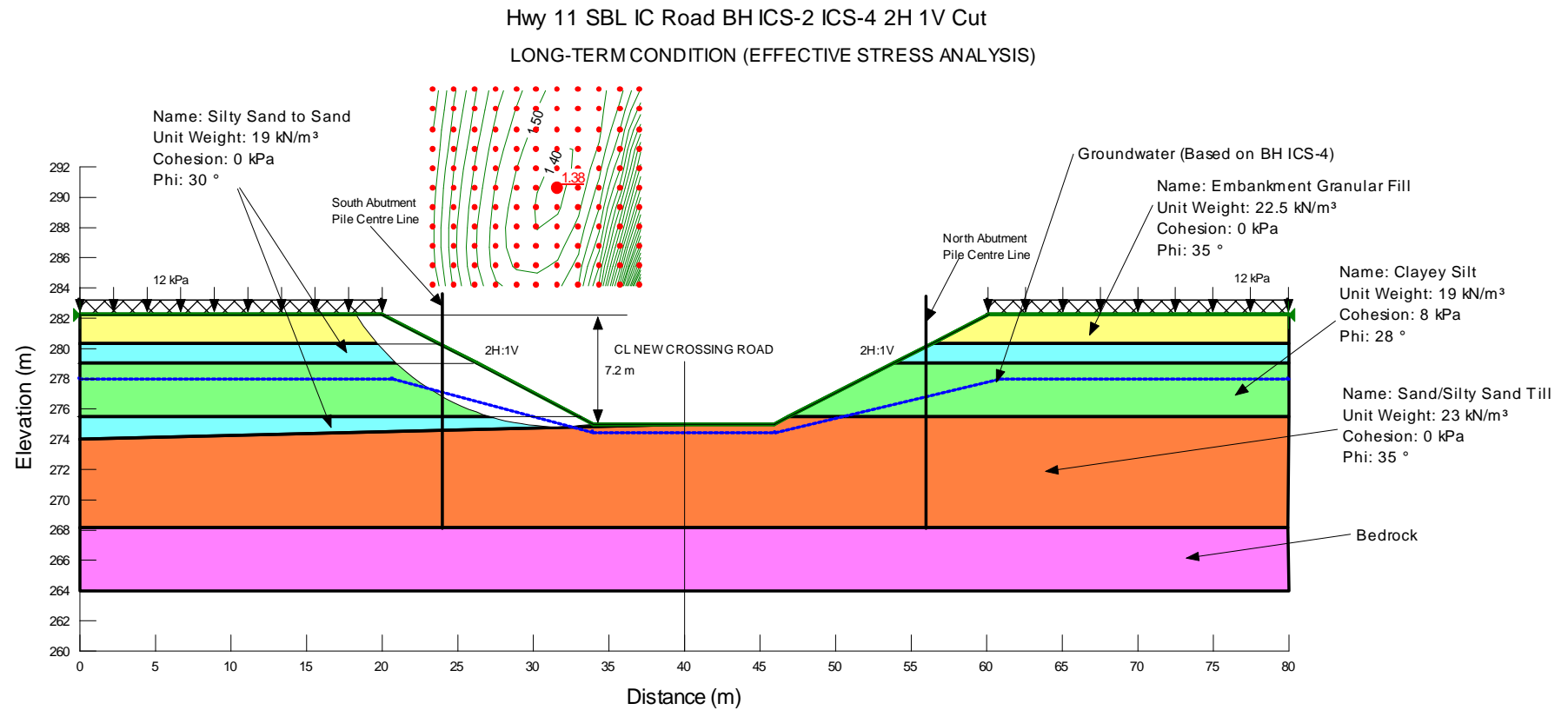
MTO SIEVE DESIGNATION		PERCENTAGE PASSING BY MASS
2 mm	#10	100
600 µm	#30	80 – 100
425 µm	#40	40 – 80
250 µm	#60	5 – 25
150 µm	#100	0 – 6





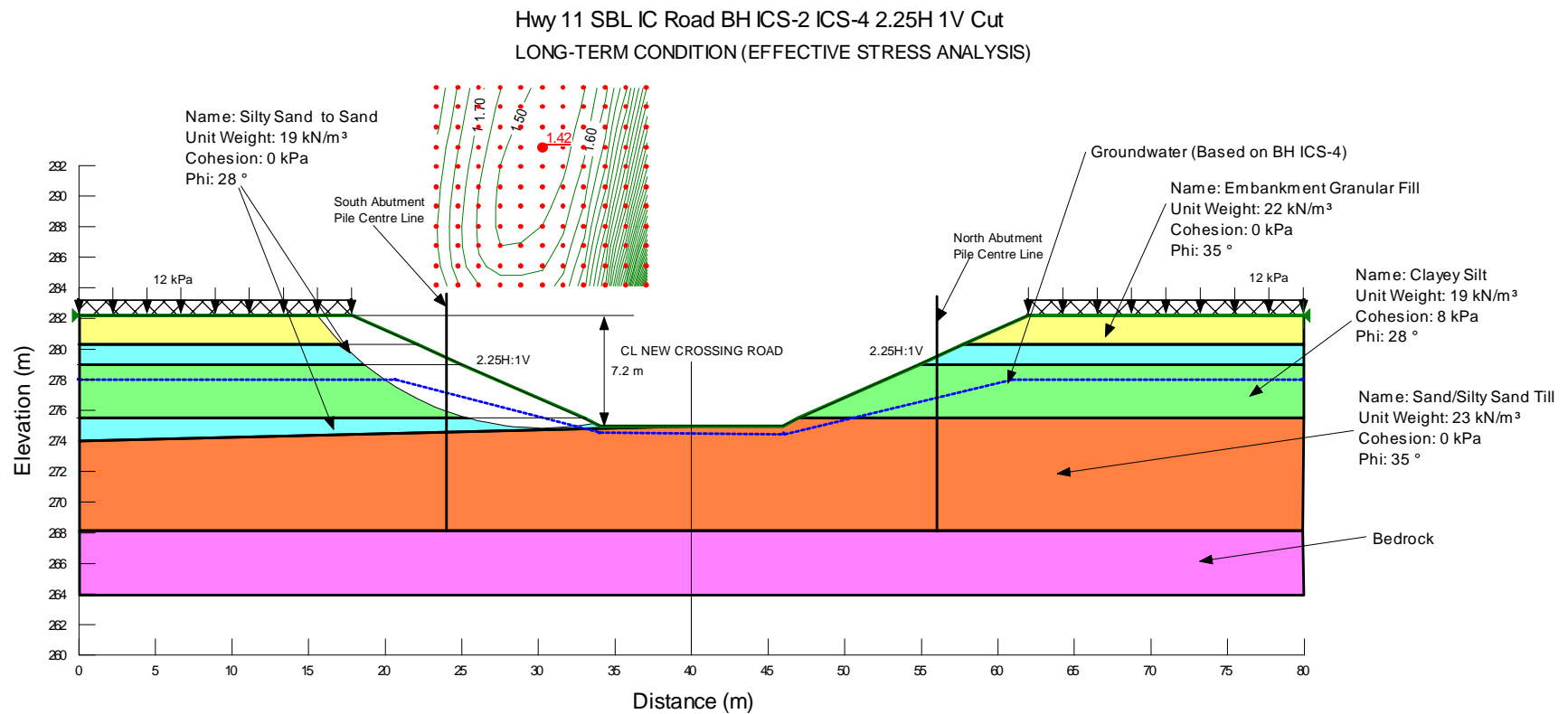
## **APPENDIX FDR-1**

### Slope Stability Diagrams



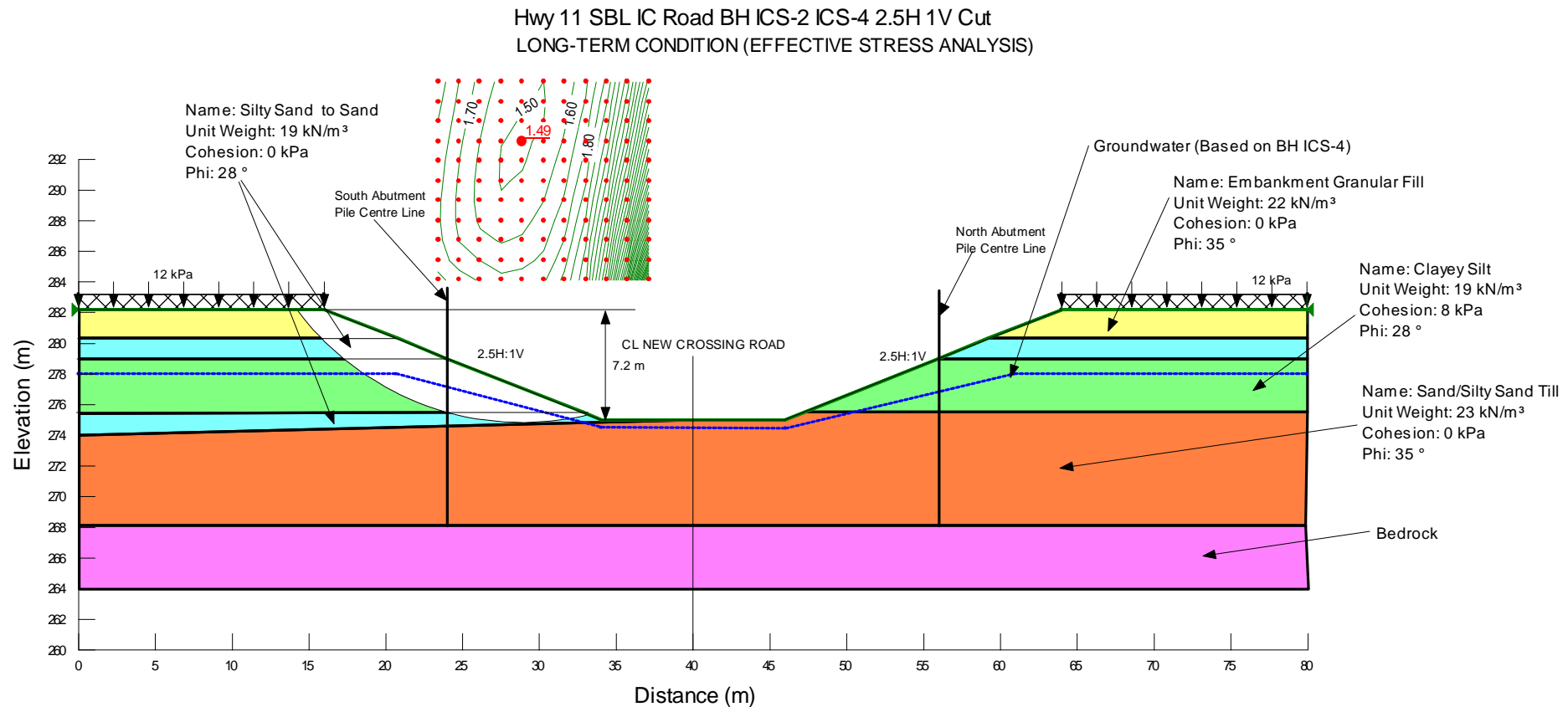
Note: FOS = 1.38, the minimum FOS of 1.5 was not met.

**FIGURE A-1**



Note: FOS = 1.42, the minimum FOS of 1.5 was not met.

**FIGURE A-2**



Note: FOS = 1.49, the minimum FOS of 1.5 was met.

**FIGURE A-3**