



**FOUNDATION INVESTIGATION AND DESIGN REPORT**

**for**

**HIGHWAY 401 / BROCK STREET UNDERPASS**

**SITE 22-151, W.O. 09-20009**

**WHITBY, ONTARIO**

**REGIONAL MUNICIPALITY OF DURHAM**

PETO MacCALLUM LTD.  
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PML Ref.: 10TF008A-B  
Index No.: 074FIR and 075FDR  
GEOCRES No.: 30M15-271  
January 23, 2018



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**FOUNDATION INVESTIGATION REPORT**

for

Highway 401 / Brock Street Underpass

Site 22-151, W.O. 09-20009

Whitby, Ontario

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**1. INTRODUCTION**

This report summarises the results of a foundation investigation carried out for the proposed replacement of an underpass carrying Brock Street traffic over Highway 401 in Whitby, Ontario. The investigation was conducted for AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

The underpass is at approximate Station 13+858, Highway 401 chainage, in Whitby (ref. General Arrangement (GA) drawing Brock Street Underpass' prepared by AECOM in September 2017). The new GA drawing shows a wider structure than originally planned in 2014.

The report provides subsurface information pertaining to the proposed structure and approaches within about 20 m of the abutments.

All elevations in this report are expressed in meters.

**2. SITE DESCRIPTION AND GEOLOGY**

The existing Brock Street underpass is a two-span rigid frame structure with an approximately 33.5 m width. It is situated north of Lake Ontario in the Town of Whitby providing access to the Whitby Go Station located south of the Highway 401. The highway is oriented in the west-east direction at the underpass location. Site photographs are included in Appendix A.

The performance of the foundations of the existing underpass appears to be adequate based on a visual inspection of the abutments above grade. The approach embankments to the structure show no signs of distress, except for some cracking in the pavement at the abutments.

The topography at the site is irregular in detail and has been modified previously for the construction of the highway. Currently the north approach embankment is up to 8 m high at the



bridge abutments and slopes up northerly. It is inferred that the highway was constructed within an earth cut through the underpass area.

The study area is located in the physiographic region known as the Iroquois Plain ("Physiography of Southern Ontario" by Chapman and Putnam and Map 1050 A of Lindsay-Peterborough Area, for Ontario, published by the Geological Survey of Canada). In general, the plain is a mosaic of lacustrine sandy and clayey deposits with till plains and drumlins. Small drainage courses and creeks currently drain the area southerly towards Lake Ontario.

The site stratigraphy includes mixed soils underlain by bedrock of the Whitby Formation that typically comprises grey and black shale according to the Aggregate Resources Inventory of the Town of Whitby published by the Ontario Geological Survey, Paper 41. The bedrock in the immediate vicinity of the site is less than 15 m deep.

### **3. INVESTIGATION PROCEDURES**

The original field work for this study was carried out during the period of April 28 to May 13, 2014 and comprised 8 boreholes drilled to depths of 7.9 to 17.7 m at the locations shown on Drawing B-1, attached. On March 2, 2017 borehole RW1-2 was drilled to 9.8 m depth near the south abutment for the new retaining wall along the Highway 401 exit ramp. This borehole was included to provide subsurface data for the wider bridge configuration than planned in 2014. It is noted that the choice of locations for some boreholes was affected by accessibility issues and the presence of utilities. Further details are summarised in the following table:

LOCATION	BOREHOLE No.	DEPTH (m)		
		AUGER / CONE	ROCK CORE	TOTAL
North Approach	B-1	9.8	–	9.8
North Abutment	B-2	12.5	–	12.5
	B-3	13.7	4.0	17.7
Pier	B-4	7.9	–	7.9
	B-5	8.6	4.9	13.5
South Abutment	B-6	9.6	–	9.6
	B-7	12.6	–	12.6
	RW1-2	9.8	–	9.8
South Approach	B-8	9.5	–	9.5



The locations of the boreholes in relation to the working points of the structure were established in the field by Peto MacCallum Ltd. The ground surface elevations at the boreholes were provided by J.D. Barnes Ltd.

The boreholes were advanced using continuous flight solid stem augers, powered by a track-mounted CME-55 and truck-mounted CME-75 drill rigs, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff. Two boreholes were extended 4.0 and 4.9 m into bedrock using NQ diamond rock coring equipment supplemented by wash boring techniques. The 4.9 m long rock core was obtained at the pier to provide data for the resistance of caissons socketed into the bedrock.

Representative samples of the soils were recovered at 0.75 m depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. Penetrometer testing was also performed to further assess the shear strength of the cohesive soils encountered. Grinding observed in the process of augering was indicative of larger particle sizes such as cobbles. The results of the field tests and observations are reported on the Record of Borehole sheets.

The groundwater conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in the open borehole. Upon completion of drilling, the boreholes were backfilled with bentonite / cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.

A piezometer consisting of a 19 mm diameter PVC pipe slotted over the bottom 3.0 m was installed in borehole B-3 to monitor groundwater conditions. The annular space around the pipe was backfilled with auger cuttings and a bentonite seal placed as illustrated on the borehole log. The water level in the piezometer was measured on June 18 and July 9, 2014.



A piezometer was also similarly installed in borehole RW1-2 on March 2, 2017 the water level in the piezometer was measured on March 20 and April 16, 2017. The piezometers were abandoned in accordance with MTO guidelines and MOE Regulation 903.

Soils were identified in the field in accordance with the MTO Soil Classification procedures. All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determination. In addition, 13 Atterberg limits tests and 19 grain size distribution analyses were carried out on selected soil samples, with the results presented in Figures B-PC-1 to B-PC-6 and B-GS-1 to B-GS-6 respectively as well as on the corresponding Record of Borehole sheets. The laboratory testing in borehole RW1-2 was reported in the enclosed borehole log and Figures RW-PC-1 (1), RW1-GS-1 (1) and RW1-GS-2 (1).

#### **4. SUMMARISED SUBSURFACE CONDITIONS**

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, bedrock descriptions, inferred stratigraphy, boundary elevations, standard penetration test data, penetrometer undrained shear strength values and groundwater observations. The results of laboratory Atterberg limits testing, grain size distribution analyses, moisture content determination 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 B-1 and B-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 typical subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised surficial fill overlying variable cohesionless till and cohesive clayey silt till / silty clay till mantling bedrock. The glacial till units were locally interbedded by clay and silt deposits in boreholes B-2 and B-5. Cobbles and boulders were encountered in boreholes B2, B3 and B4. The bedrock surface was contacted at 7.0 to 13.7 m (elevation 75.9 to 77.4) in boreholes B-2 to B-5. The piezometric water level measured in borehole B-3 was at 2.3 m (elevation 87.3) on



June 18, 2014 and at 2.5 m (elevation 87.1) on July 9, 2014. In borehole RW1-2, the water level was at 5.1 m (elevation 83.7) on March 20, 2017 and 4.9 m (elevation 83.9) on April 19, 2017.

The strata encountered are summarised below.

#### **4.1 Topsoil**

Surficial topsoil was present in boreholes B-1 and RW1-2. The silty topsoil was 600 and 300 mm in thickness respectively, and 16% and 22% in moisture content. The topsoil was penetrated at elevation 89.1 in borehole B-1 and 88.5 in borehole RW1-2.

#### **4.2 Fill**

Pavement structure consisting of 150 to 200 mm thick asphalt and 300 to 750 mm thick sand and gravel was present in boreholes B-2, B-4 to B-8. The granular material was compact to very dense, typically dense.

Directly beneath the sand and gravel fill at 0.5 to 0.6 m (elevation 83.1 to 88.9) in boreholes B-4 to B-6 and B-8 and interbedded in clayey silt fill in borehole RW1-2 were sand and gravelly sand fill units. This unit was loose to very dense and had a moisture content of 3 to 4%. The sand fill was 300 to 600 mm in thickness and penetrated at 0.9 to 1.2 m (elevation 82.5 to 88.5).

Clayey silt and/or silty clay fill was present surficially in borehole B-3 and under the pavement structure or sand fill at 0.6 to 0.9 m (elevation 88.4 to 89.2) in boreholes B-2, B-6 to B-8. Firm to very stiff in consistency and 12 to 20% in moisture content, the cohesive fill had a thickness of 0.6 to 1.3 m and was penetrated at 1.3 to 2.2 m (elevation 87.2 to 88.3). Clayey silt fill of similar firm to stiff consistency was also found in borehole RW1-2 from 0.3 to 3.0 m depths (elevation 88.5 to 85.8)

The results of Atterberg limits testing and grain size distribution analysis conducted on a cohesive sample of the fill are presented in respective Figures B-PC-1 and B-GS-1. The clayey silt fill had a liquid limit of 27, plastic limit of 13, thus giving the plasticity index of 14.





#### **4.3 Sand Till / Silty Sand Till / Sandy Silt Till**

Overlain by the topsoil, fill or clayey silt till at 0.6 to 2.5 m (elevation 82.5 to 89.1) in all the boreholes was cohesionless till of variable granulometric composition (sand, silty sand, sandy silt). This stratum was loose to very dense (SPT-‘N’ values of 9 to 104), its moisture content ranging from 5 to 14%. The cohesionless till was 2.6 to 4.2 m in thickness and penetrated at 4.3 to 7.3 m (elevation 76.6 to 85.5) in boreholes B-1 to B-5. The remaining boreholes were terminated within the stratum at depths of 9.5 to 12.6 m (elevation 76.7 to 79.9). It is noteworthy that cobbles and boulders were encountered in the sand till in boreholes B-2 to B-4 and should be expected in the stratum across the site.

The results of Atterberg limits testing and grain size distribution analyses performed on 11 samples of the stratum are presented in respective Figures B-PC-2, B-PC-3 and B-GS-2, B-GS-3. The liquid and plastic limits of the cohesionless till ranged from 12 to 15 and from 11 to 13 respectively, with the plasticity index of 1 to 4 (slightly plastic).

#### **4.4 Clayey Silt Till / Silty Clay Till**

Underlying the fill or sand till at depths of 1.3 to 6.4 m (elevation 79.0 to 88.3) in boreholes B-1 to B-4 and B-7 was a deposit of clayey silt till or silty clay till. Very stiff to hard in consistency, this deposit was 1.0 to 7.3 m thick and penetrated at 2.5 to 13.7 m (elevation 75.9 to 87.2). Borehole B-1 was terminated in the silty clay till at 9.8 m (elevation 79.9).

The unit was also encountered in borehole RW1-2 from 3.0 to 9.8 in depth (elevation 85.8 to 79.0). At this borehole location the deposit was interbedded with a dense to very dense silt layer between 6.4 and 7.5 m depth (elevation 82.4 to 81.3).

The results of Atterberg limits testing and grain size distribution analyses conducted on 4 samples of the deposit are presented in respective Figures B-PC-4, B-PC-5 and B-GS-4, B-GS-5. Figures RW1-PC-1 (1), RW1-GS-1 (1), RW1-GS-2 (1), for borehole RW1-2 were also included. The liquid and plastic limits of the clayey silt till were 17 to 18 and 11 to 12 respectively, thus giving the



plasticity index of 6. The silty clay till had a liquid limit of 40 and 48, plastic limit of 18 and 22, its plasticity index being 22 and 26. The moisture content of the deposit varied between 9 and 32%.

#### **4.5 Clay**

A layer of clay was encountered below the clayey silt till at 5.6 m (elevation 84.2) in borehole B-2 and within the sand till at 4.1 m (elevation 79.8) in borehole B-5. This layer was very stiff to hard in consistency and 26 to 32% in moisture content. The clay had a thickness of 6.8 m in borehole B-2 and 2.4 m in borehole B-5 and was penetrated at respective depths of 12.4 and 6.5 m (elevation 77.4).

The results of Atterberg limits testing and grain size distribution analyses performed on 3 samples of the layer are presented in respective Figures B-PC-6 and B-GS-6. The liquid and plastic limits of the clay ranged from 56 to 59 and from 25 to 27 respectively, with the plasticity index of 31 to 32.

#### **4.6 Bedrock**

Bedrock was contacted at 7.0 to 13.7 m (elevation 75.9 to 77.4) in boreholes B-2 to B-5. Near the south abutment, probable bedrock was inferred at 9.8 m (elevation 79.0) in borehole RW1-2. The slightly weathered to unweathered bedrock comprises a dark grey to dark green / light black shale with interbedded limestone of the Whitby Formation.

The measured core recovery typically varied between 79 and 100%. The RQD determined from the rock cores was in a range of 58 to 97%, thus indicating a fair to excellent quality rock. The rock quality was very poor (RQD of 0 and 23%) in the upper 1.3 m core sample in borehole B-3 and 0.7 m core sample at 11.3 m (elevation 72.6) in borehole B-5.

A detailed description of the rock cores retrieved from boreholes B-3 and B-5 is given in Table A, appended. Photographs of the rock cores are shown in Appendix B.



#### **4.7 Groundwater**

In the process of augering, water was detected at 2.1 to 5.8 m (elevation 79.7 to 87.2) in all the boreholes. Upon completion of drilling, groundwater was measured in boreholes B-4 to B-7 at 4.0 to 7.0 m (elevation 79.7 to 83.2). No water was observed in boreholes B-1 to B-3 and B-8 upon completion of drilling.

The piezometric water level measured in borehole B-3 drilled at the north abutment was at 2.3 m (elevation 87.3) on June 18, 2014 and at 2.5 m (elevation 87.1) on July 9, 2014. The high water levels in boreholes B-1 to B-3 drilled north of Highway 401 reflect perched groundwater conditions in the local sand deposits.

At the south abutment the water level in the piezometer installed in borehole RW1-2, was at 5.1 m (elevation 83.7) on March 20, 2017 and at 4.9 m (elevation 83.9) on April 16, 2017.

The groundwater levels at the site are subject to seasonal fluctuations and precipitation patterns.



## 5. CLOSURE

The field work was carried out under the supervision of Mr. F. Portela and Mr. S. Aziz and direction of Mr. A. DeSira, MEng, P.Eng., Project Engineer. The equipment was supplied by Atcost Drilling Inc.

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

Yours very truly,

Peto MacCallum Ltd.



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



Carlos M.P. Nascimento, P.Eng.  
MTO Designated Principal Contact

GD/CN/BRG:gd-jk-nk



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

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	CORE NO.	DEPTH (m)	RECOVERY (%)	RQD (%)	DEPTH (m)	DESCRIPTION
B-3	12	13.7 – 15.0	100	0	13.7 – 17.7	SHALE WITH INTERBEDDED LIMESTONE: Dark grey to dark green, fine grained, with quartz vein and occasional interbedded grey limestone (effervesces freely in dilute (5%) hydrochloric acid), soft to medium strength, bedding in shale horizontal, laminated and fissile, slightly weathered to unweathered, close spaced flat partings, smooth planar, tight, very poor becoming fair to excellent quality.
	13	15.0 – 16.2	100	66		
	14	16.2 – 17.7	100	94		
B-5	9	8.6 – 8.8	89	84	8.6 – 13.5	SHALE WITH INTERBEDDED LIMESTONE: Dark grey to light black, fine grained, with quartz vein and occasional interbedded grey limestone (effervesces freely in dilute (5%) hydrochloric acid), soft to medium strength, bedding in shale horizontal, laminated and fissile, slightly weathered to unweathered, close spaced flat partings, smooth planar, tight, fair to excellent (locally very poor) quality.
	10	8.8 – 9.0	90	65		
	11	9.0 – 9.8	79	58		
	12	9.8 – 10.6	84	76		
	13	10.6 – 11.3	100	97		
	14	11.3 – 12.0	23	23		
	15	12.0 – 13.5	100	93		

NOTE: RQD = Rock Quality Designation

Originated: SA  
 Compiled: JO  
 Checked: SAT

## 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
$\sigma'_{v0}$	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				$i$	1	HYDRAULIC GRADIENT
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	$j$	kN/m <sup>3</sup>	SEEPAGE FORCE
$e$	1, %	VOID RATIO	WTPL		WETTER THAN PLASTIC LIMIT			

**RECORD OF BOREHOLE No. B-1**


1 of 1

**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 690.3 N; 350 075.6 E      **ORIGINATED BY** F.P.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** Continuous Flight Solid Stem Augers      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** May 09, 2014      **CHECKED BY** G.D.

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED      + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE												
89.7	Ground Surface						20	40	60	80	100						GR SA SI CL			
0.0	Topsoil		1	SS	7															
89.1	Sand with silt, some gravel trace to some clay  Loose      Dark      Moist brown shale fragments Compact to Brown very dense  (TILL)  Wet		2	SS	9												14 47 29 10			
0.6			3	SS	23															
			4	SS	39															
			5	SS	50/13cm															
			6	SS	50/10cm															
84.9	Silty clay some sand, trace gravel  Hard      Grey      Moist (TILL)		7	SS	66/25cm												1 12 36 51			
4.8			8	SS	39															
			9	SS	43															
			10	SS	33															
79.9	End of borehole																			
9.8																				

\* 2014 05 09

 Water level observed during drilling

**RECORD OF BOREHOLE No. B-2**

1 of 1

**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 675.0 N; 350 088.9 E      **ORIGINATED BY** F.P.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** Continuous Flight Solid Stem Augers      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** May 09, 2014      **CHECKED BY** G.D.

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									
								○ UNCONFINED      + FIELD VANE									
								● QUICK TRIAXIAL      × LAB VANE									
							WATER CONTENT (%)										
89.8	Ground Surface						20	40	60	80	100						
0.0	175mm asphalt over sand and gravel (PAVEMENT FILL)		1	SS	42												
89.2	Silty clay, trace sand organic inclusions		2	SS	11												
0.6	Dark brown Clayey silt trace sand, trace gravel																
88.1	Stiff Brown Moist (FILL)		3	SS	17												
1.7	Sand with silt, with gravel some clay		4	SS	38												
	Compact to Brown Moist dense (TILL)		5	SS	40												
	cobbles																
	Grey		6	SS	22												
85.5	Clayey silt, sandy some gravel		7	SS	29												
4.3	Very stiff Grey Moist (TILL)																
84.2	Clay some silt, trace sand		8	SS	25												
5.6	Very stiff Grey Moist to hard																
			9	SS	18												
			10	SS	18												
			11	SS	28												
77.4	Shale bedrock		12	SS	100/15cm												
12.4	End of borehole																
77.3																	
12.5																	
	* 2014 05 09																
	▽ Water level observed during drilling																
	■ Penetrometer test																

\* 2014 05 09

▽ Water level observed during drilling

■ Penetrometer test



**RECORD OF BOREHOLE No. B-3**

1 of 2

**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 669.6 N; 350 110.2 E      **ORIGINATED BY** S.A.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** C.F.S.S.A. and Rock Coring      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** May 12 and 13, 2014      **CHECKED BY** G.D.

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									
								20 40 60 80 100									
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE									
					WATER CONTENT (%)												
89.6	Ground Surface																
0.0	Clayey silt rootlets, organics and topsoil inclusions		1	SS	5												
	Firm to Dark Moist stiff brown																
88.3	(FILL)		2	SS	10												
1.3	Clayey silt trace sand, trace gravel																
	Very stiff Brown/ Moist grey		3	SS	19												
	(TILL)																
87.2	Sand, with silt some gravel, trace clay		4	SS	42												
2.4	Dense to Grey Moist very dense																
	(TILL)		5	SS	44												
			6	SS	62												
	cobbles and boulders																
83.2	Wet		7	SS	27												
6.4	Silty clay, trace gravel																
	Very stiff Grey Moist to hard																
	seams of clayey silt till		8	SS	48												
	Moist to wet																
	(TILL)																
			9	SS	50												
			10	SS	38												
	shale fragments		11	SS	80/8cm												
75.9	Shale bedrock with interbedded limestone																
13.7	Unweathered to slightly weathered		12	RC	REC 100%												
	Soft to medium strength Very poor quality																
	Cont'd																

**RECORD OF BOREHOLE No. B-3**

2 of 2

**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 669.6 N; 350 110.2 E      **ORIGINATED BY** S.A.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** C.F.S.S.A. and Rock Coring      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** May 12 and 13, 2014      **CHECKED BY** G.D.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT   NATURAL MOISTURE CONTENT   LIQUID LIMIT			UNIT WEIGHT  γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)						
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W <sub>p</sub> W                      W <sub>L</sub>										
								○ UNCONFINED                      + FIELD VANE ● QUICK TRIAXIAL                      × LAB VANE					WATER CONTENT (%)										
74.6							20	40	60	80	100	20	40	60	kN/m <sup>3</sup>	GR   SA   SI   CL							
	Shale bedrock with interbedded limestone  Unweathered to slightly weathered  Soft to medium strength  Fair to excellent quality  (Cont'd.)		13	RC	REC 100%	74										RQD 66%							
			14	RC	REC 100%	73									RQD 94%								
			71.9						72														
17.7	End of borehole																						
	<div><div>*      2014   05   12</div><div>      Water level observed during drilling</div><div>*      Borehole charged with drilling water</div><div>NOTE:      Piezometer was installed 1.0 m north of borehole at location with the same ground surface elevation</div><div><div>Piezometer Readings:</div><table><tr><td>Date</td><td>Depth (m)</td><td>Elev.</td></tr><tr><td>06/18/2014</td><td>2.3</td><td>87.3</td></tr><tr><td>07/09/2014</td><td>2.5</td><td>87.1</td></tr></table><div>Piezometer Legend:</div><div><div></div> Bentonite seal</div><div><div></div> Native</div><div><div></div> Filter sand</div><div><div></div> Screen</div></div></div>	Date	Depth (m)	Elev.	06/18/2014	2.3	87.3	07/09/2014	2.5	87.1													
Date	Depth (m)	Elev.																					
06/18/2014	2.3	87.3																					
07/09/2014	2.5	87.1																					

**RECORD OF BOREHOLE No. B-4**


1 of 1


**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 623.2 N; 350 126.8 E      **ORIGINATED BY** F.P.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** Continuous Flight Solid Stem Augers      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** April 29, 30 & May 2, 2014      **CHECKED BY** G.D.

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		SHEAR STRENGTH kPa					W <sub>p</sub>	W	W <sub>L</sub>		
83.7	Ground Surface					20	40	60	80	100					
0.0	200mm asphalt over sand and gravel (PAVEMENT FILL)														
83.1	Sand, trace gravel		1	SS											
0.6	Dense to Brown Moist compact (FILL)		2	SS											
82.5	Sand with silt, some gravel trace to some clay		3	SS											18 49 23 10
1.2	Dense to Grey Moist very dense (TILL)		4	SS											
			5	SS											20 48 23 9
	cobbles and boulders														
79.0	Wet														
4.7	Silty clay, trace sand		6	SS											0 8 32 60
	Hard Grey Moist (TILL)														
			7	SS											
	shale fragments														
76.7	Shale bedrock														
7.0															
75.8			8	SS											
7.9	End of borehole														

\* 2014 04 29 &amp; 05 02

 Water level observed during drilling

 Water level measured after drilling

**RECORD OF BOREHOLE No. B-5**

1 of 1

**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 625.0 N; 350 099.5 E      **ORIGINATED BY** F.P.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** C.F.S.S.A. and Rock Coring      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** April 28 & 29, and May 01, 2014      **CHECKED BY** G.D.

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE		20	40	60	80	100					
83.9	Ground Surface														
0.0	200mm asphalt over sand and gravel (PAVEMENT FILL)														
83.3	Sand, trace gravel		1	SS											
0.6															
83.0	Dense Brown Moist (FILL)		2	SS											
0.9	Sand, with silt some gravel, trace clay		3	SS											
	Dense to Grey Moist very dense (TILL)		4	SS											
			5	SS											
79.8	Wet														
4.1	Clay with silt, trace sand		6	SS											
	Very stiff Grey Moist to hard														
			7	SS											
77.4	Sand, with silt some gravel, trace clay shale fragments														
6.5	Dense Grey Moist to wet (TILL)		8	SS											
76.6	Shale bedrock with interbedded limestone														
7.3	Unweathered to slightly weathered														
	Soft to medium strength		9	RC											
	Fair to excellent, locally very poor quality		10	RC											
			11	RC											
			12	RC											
			13	RC											
			14	RC											
			15	RC											
70.4	End of borehole														
13.5	* 2014 04 28 and 29														
	Water level observed during drilling														
	Water level measured after drilling														

**RECORD OF BOREHOLE No. B-6**

1 of 1

**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 581.4 N; 350 118.3 E      **ORIGINATED BY** F.P.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** Continuous Flight Solid Stem Augers      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** May 06, 2014      **CHECKED BY** G.D.

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	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		SHEAR STRENGTH kPa										WATER CONTENT (%)
							○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE										
89.3	Ground Surface						20	40	60	80	100					GR SA SI CL	
0.0	175mm asphalt over sand and gravel																
88.8	(PAVEMENT FILL)		1	SS	36												
0.5	Sand, trace gravel																
88.4	Dense to Brown Moist loose		2	SS	8												
0.9	Silty clay trace sand, trace gravel organic inclusions		3	SS	3												
87.2	Stiff to Brown Moist firm																
2.1	(FILL)																
	Sand with silt to silty some gravel, some clay		4	SS	45											18 45 25 12	
	Dense to Brown Moist very dense to wet trace clay		5	SS	101/20cm												
	Grey		6	SS	43												
	(TILL)		7	SS	75											19 39 33 9	
			8	SS	104											18 45 29 8	
			9	SS	101/25cm												

\* 2014 05 06

▽ Water level observed during drilling

▼ Water level measured after drilling

**RECORD OF BOREHOLE No. B-7**

1 of 1

**METRIC**

**W.O.** 09-20009      **LOCATION** Co-ord: 4 858 580.6 N; 350 132.0 E      **ORIGINATED BY** F.P.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** Continuous Flight Solid Stem Augers      **COMPILED BY** A.D.  
**DATUM** Geodetic      **DATE** May 05, 2014      **CHECKED BY** G.D.

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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED + FIELD VANE												
								● QUICK TRIAXIAL × LAB VANE												
89.3	Ground Surface						20	40	60	80	100									
0.0	150mm asphalt over sand and gravel		1	SS	37															
88.4	Dense to Brown Moist compact (PAVEMENT FILL)		2	SS	24															
0.9	Silty clay trace sand, trace gravel organics																			
87.8	Very stiff Brown Moist (FILL)		3	SS	21															
1.5	Clayey silt some sand to sandy trace to some gravel																			
86.8	Very stiff Brown Moist (TILL)		4	SS	71															
2.5	Sand, with silt some gravel, some clay																			
	Dense to Grey Moist very dense (TILL)		5	SS	63															
			6	SS	41															
	shale fragments		7	SS	55	▽*														
	Wet																			
			8	SS	40															
	sandy silt seams					▽*														
			9	SS	61															
81.1																				
8.2	Silty sand some gravel, some clay																			
	Dense to Grey Moist very dense (TILL)		10	SS	43															
			11	SS	100/13cm															
			12	SS	100/23cm															
76.7																				
12.6	End of borehole																			

\* 2014 05 05

▽ Water level observed during drilling

▽ Water level measured after drilling

# RECORD OF BOREHOLE No. B-8

1 of 1

## METRIC

W.O.	09-20009	LOCATION	Co-ord: 4 858 564.2 N; 350 123.2 E	ORIGINATED BY	F.P.
DIST	Durham	HWY	401	BOREHOLE TYPE	Continuous Flight Solid Stem Augers
DATUM	Geodetic	DATE	May 06, 2014	CHECKED BY	G.D.

[illegible]

**RECORD OF BOREHOLE No RW1-2**

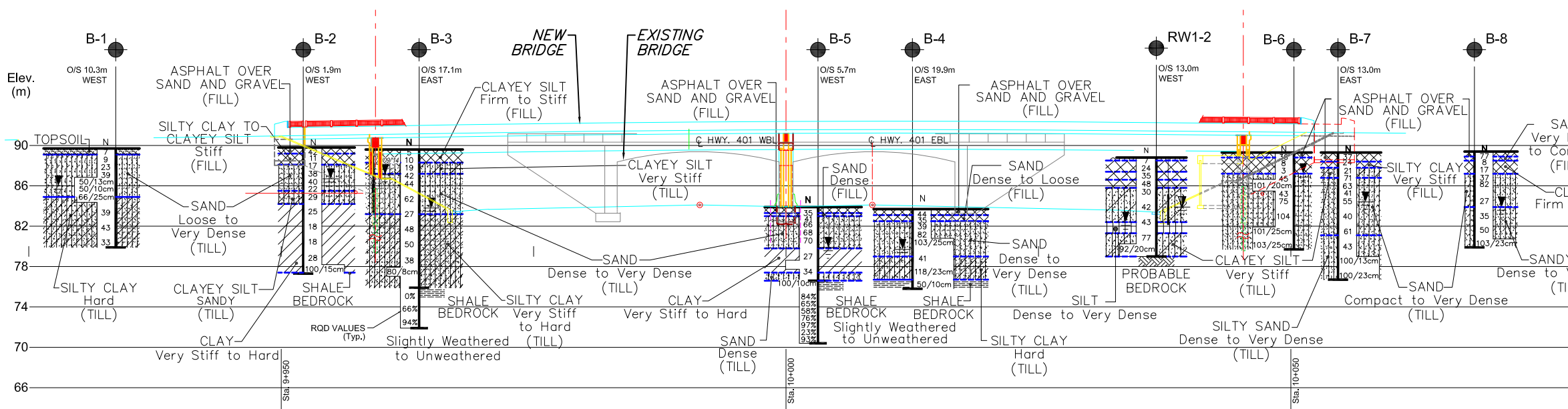
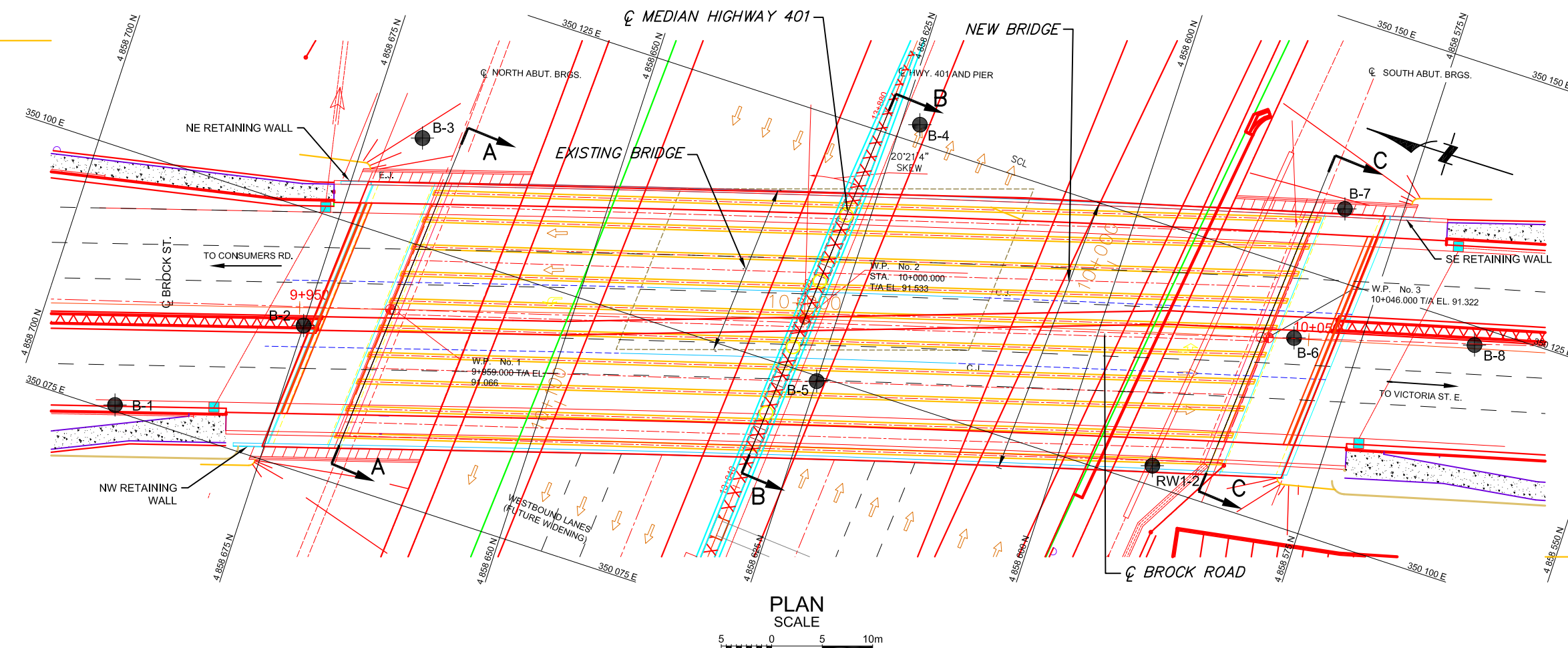
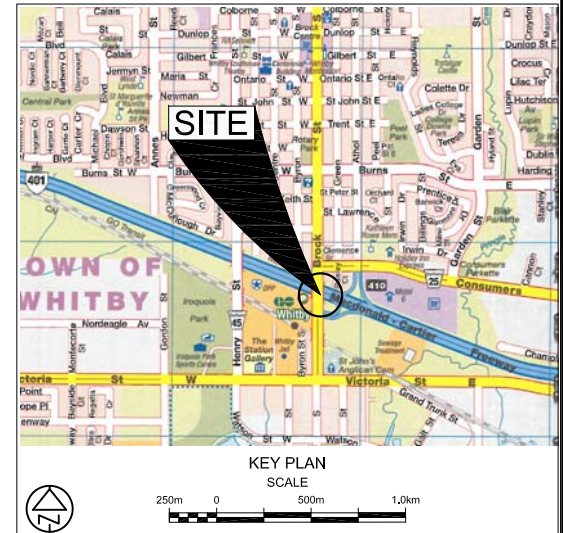
1 of 1

**METRIC**

**TASK No.** 2009-E-0038      **LOCATION** Coords: 4 858 590.8 N ; 350 101.9 E      **ORIGINATED BY** S.A.  
**DIST** Durham      **HWY** 401      **BOREHOLE TYPE** Continuous Flight Solid Stem Augers      **COMPILED BY** G.D.  
**DATUM** Geodetic      **DATE** March 02, 2017      **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		
88.8	Ground Surface						20	40	60	80	100									
0.0 88.5 0.3	Topsoil		1	SS	7								○							
	Clayey silt rootlets, organic inclusions cobbles																			
	Firm to Brown/ Moist very stiff dark brown		2	SS	24								○							
	Gravelly sand																			
	Compact to dense		3	SS	35															
	Clayey silt, sand seams																			
	Very stiff Dark Moist to hard brown/grey (FILL)		4	SS	48								○							
85.8 3.0	Clayey silt, sandy trace gravel		5	SS	30								○H							
	Very stiff Brown/ Moist to hard grey																			
	(TILL)																			
			6	SS	42								○							
82.4 6.4	Silt, some sand trace clay, trace gravel		7	SS	43								○							
	Dense to Grey Wet very dense																			
81.3 7.5	Clayey silt, sandy trace to some gravel		8	SS	77								○H							
	Hard Brown/ Moist grey																			
	(TILL)																			
	wet sand seams																			
			9	SS	92/20cm								○							
79.0 9.8	End of borehole																			
	Refusal on probable bedrock																			
	* 2017 03 02																			
	▽ Water level observed during drilling																			
	▽ Water level measured in Monitoring Well																			
	Water Level Readings:																			
	Date	Depth (m)	Elev.																	
	Mar.20/'17	5.1	83.7																	
	Apr.16/'17	4.9	83.9																	
	Monitoring Well Legend:																			
	[Symbol] Flush cover and concrete																			
	[Symbol] Bentonite seal																			
	[Symbol] Filter sand																			
	[Symbol] Screen																			





- NOTES:
- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
  - REFER TO DRAWING B-2 FOR SECTIONS A-A, B-B AND C-C.
  - THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
  - DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

LEGEND			
	Borehole		
	Borehole and Cone		
N	Blows/0.3m (Std. Pen Test, 475 J/blow)		
CONE	Blows/0.3m (60 Cone, 475 J/blow)		
	WL at time of investigation April - July 2014 and March - April 2017 (RW1-2)		
WH	Penetration due to weight of hammer		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		
BH No	ELEVATION	NORTHINGS	EASTINGS
B-1	89.7	4 858 690.3	350 075.6
B-2	89.8	4 858 675.0	350 088.9
B-3	89.6	4 858 669.6	350 110.2
B-4	83.7	4 858 623.2	350 126.8
B-5	83.9	4 858 625.0	350 099.5
B-6	89.3	4 858 581.4	350 118.3
B-7	89.3	4 858 580.6	350 132.0
B-8	89.4	4 858 564.2	350 123.2
RW1-2	88.8	4 858 290.8	350 101.9

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



Reference AECOM Drawing: 60154317-BROCK-GA-01 dated Jan. 2018

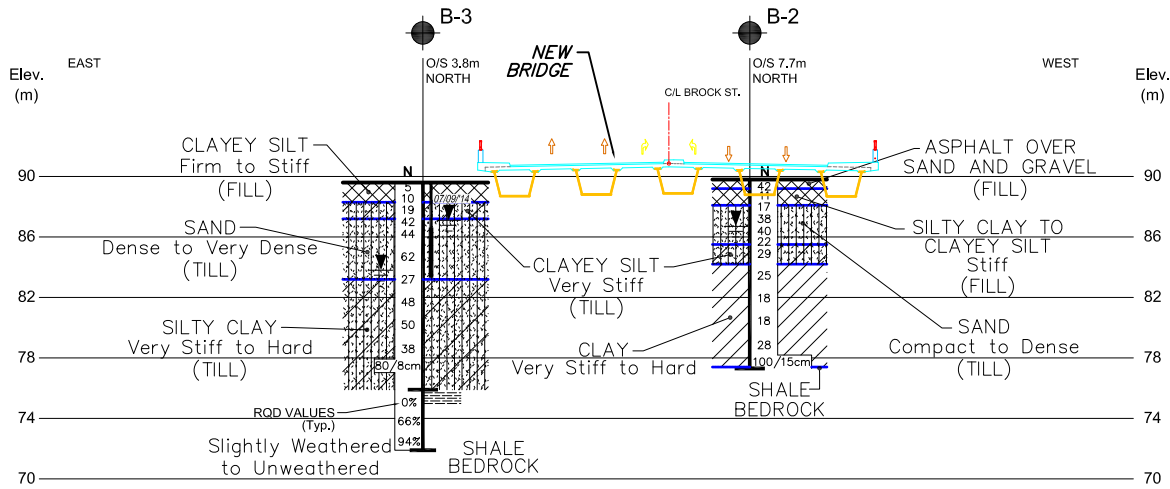
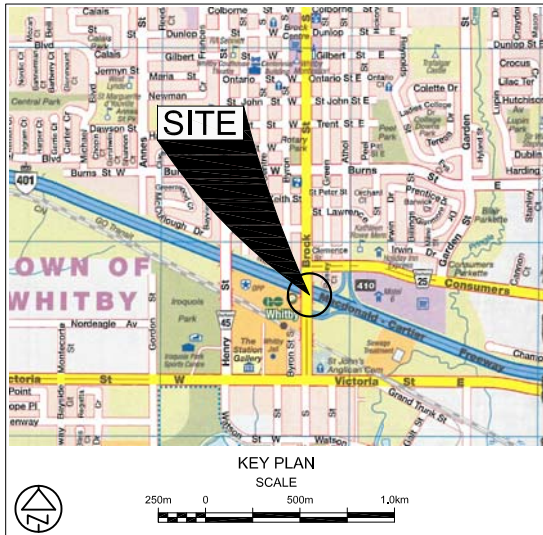
REVISIONS			
DATE	BY	DESCRIPTION	
01/22/2018	C.N.	SHEET No. WAS ADDED AND DRAWING No. WAS ALTERED AS PER AECOM'S EMAIL DATED JAN. 19, 2018	
Geocres No. 30M15-271			
HWY No	401	CHECKED	GD
SUBM'D	NA	DATE JAN. 23, 2018	DIST Central
DRAWN	NA	APPROVED CN	SITE 22-151
			DWG B-1

CONT No 2017-2038  
P.O. No 09-20009  
W.P. No 2410-13-00

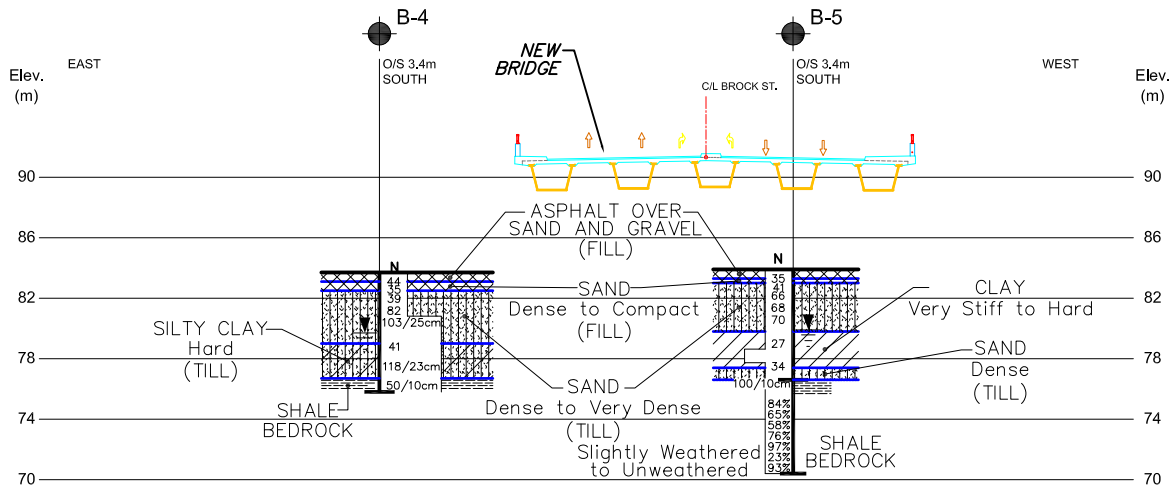
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TOWN OF WHITBY  
SOIL STRATA

SHEET

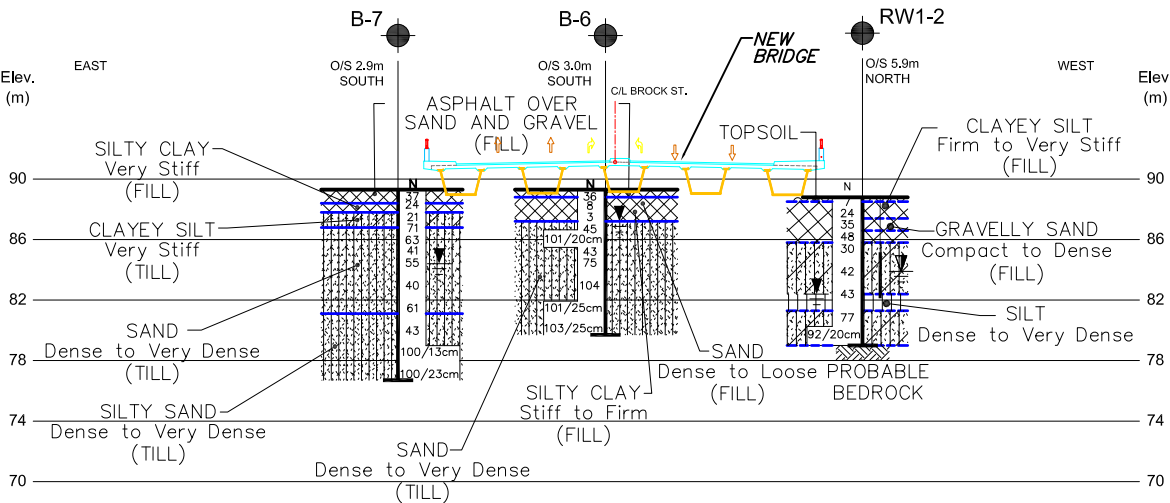
**PML** **Peto MacCallum Ltd.**  
CONSULTING ENGINEERS



SECTION A-A



SECTION B-B



SECTION C-C

SCALE



NOTES:

- THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
- REFER TO DRAWING B-1 FOR BOREHOLE LOCATIONS AND CENTRE LINE PROFILE.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
- DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

LEGEND

- Borehole
- Borehole and Cone
- N Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE Blows/0.3m (60 Cone, 475 J/blow)
- WL at time of investigation April - July 2014 and March - April 2017 (RW1-2)
- WH Penetration due to weight of hammer
- Head
- ARTESIAN WATER
- Encountered
- PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
For details, refer to Drawing B-1			

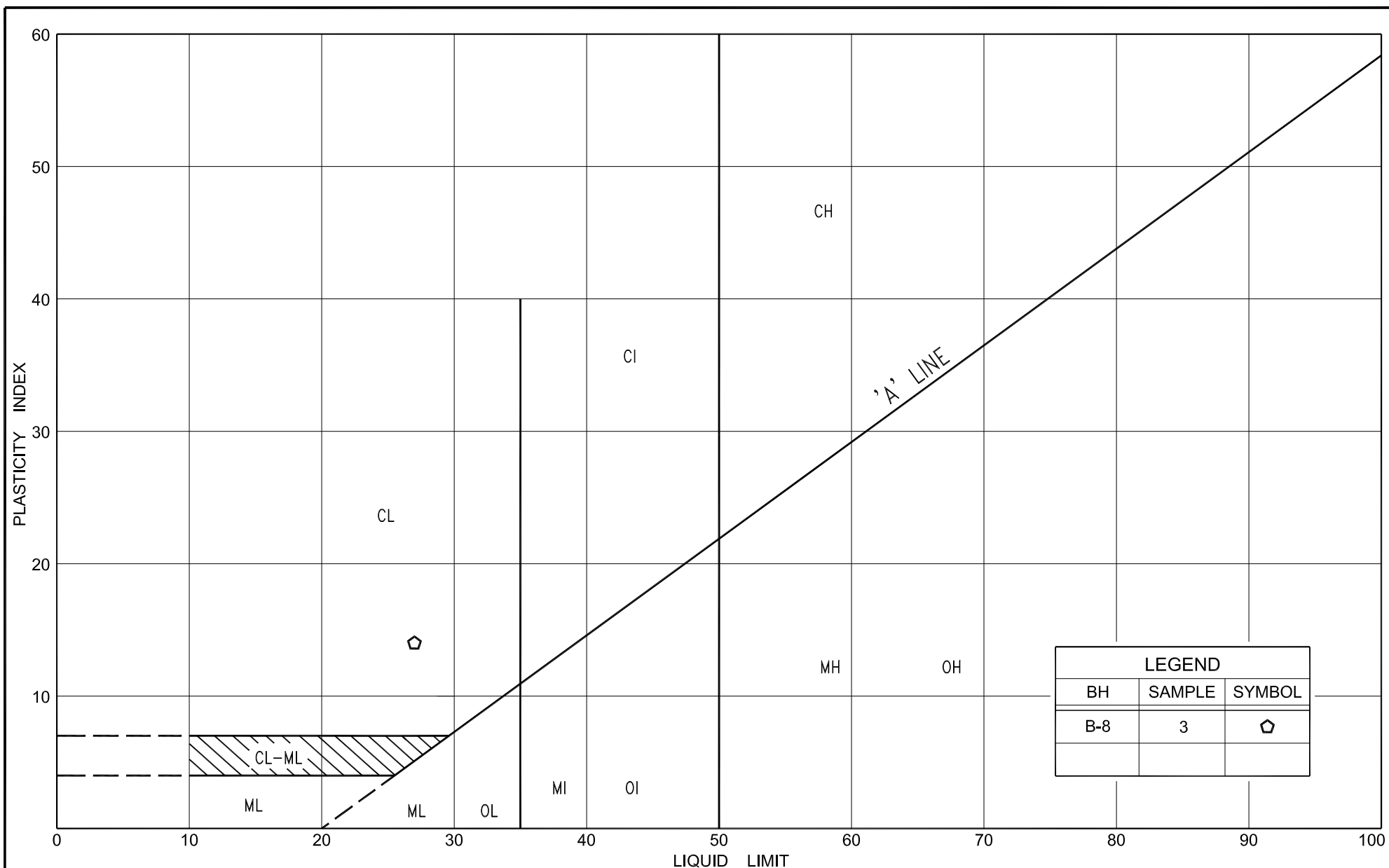
NOTE

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

REVISIONS	DATE	BY	DESCRIPTION
01/22/2018	C.N.		SHEET No. WAS ADDED AND DRAWING No. WAS ALTERED AS PER AECOM'S EMAIL DATED JAN, 19, 2018
Geocres No. 30M15-271			
HWY No	401	DIST	Central
SUBM'D	NA	CHECKED	GD
DRAWN	NA	CHECKED	CN
DATE	JAN, 23, 2018	APPROVED	CN
SITE	22-151	DWG	B-2

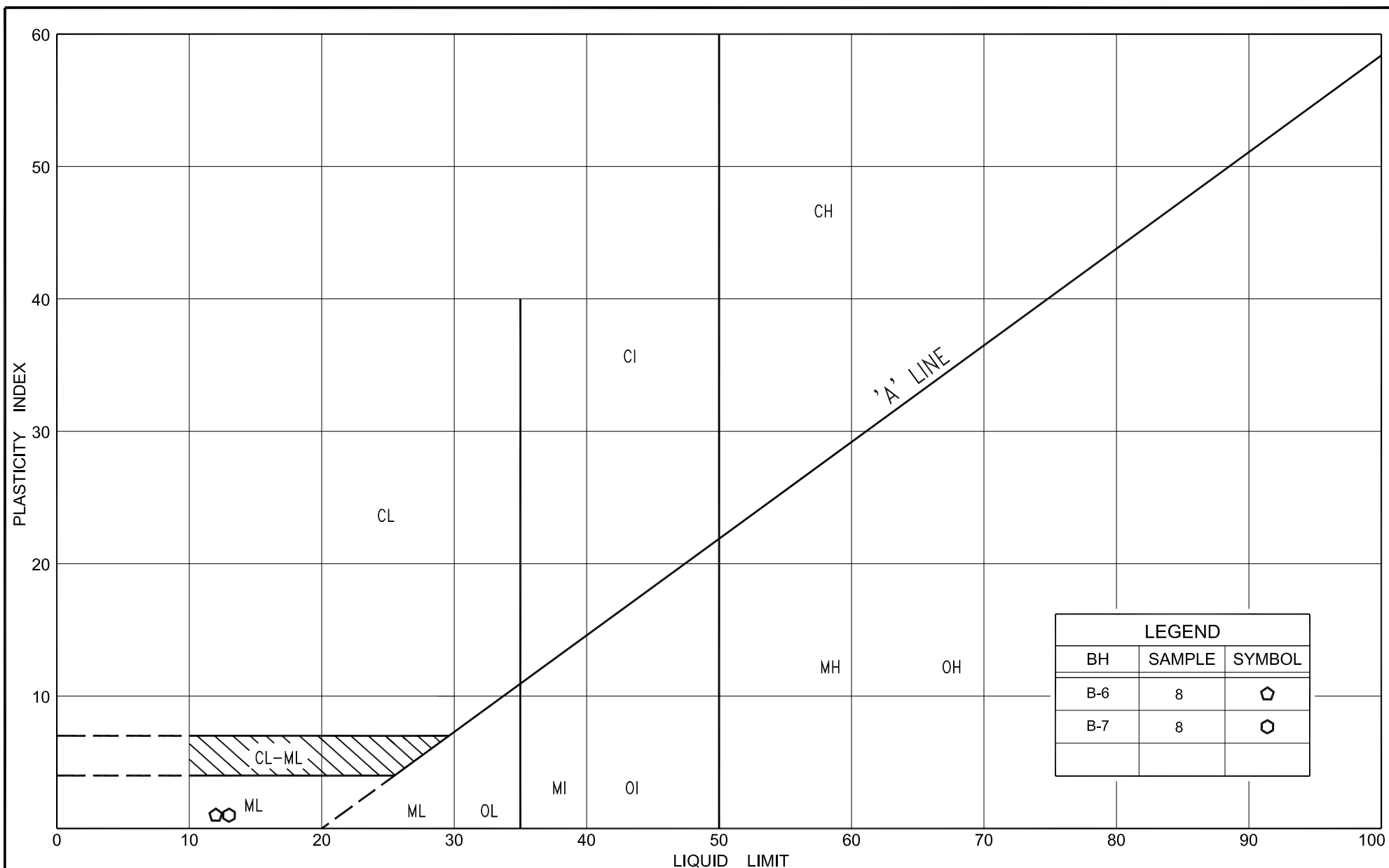


Reference AECOM Drawing: 60154317-BROCK-GA-op 1 dated Sept. 2014



**PLASTICITY CHART**  
CLAYEY SILT, with sand, trace gravel  
(FILL)

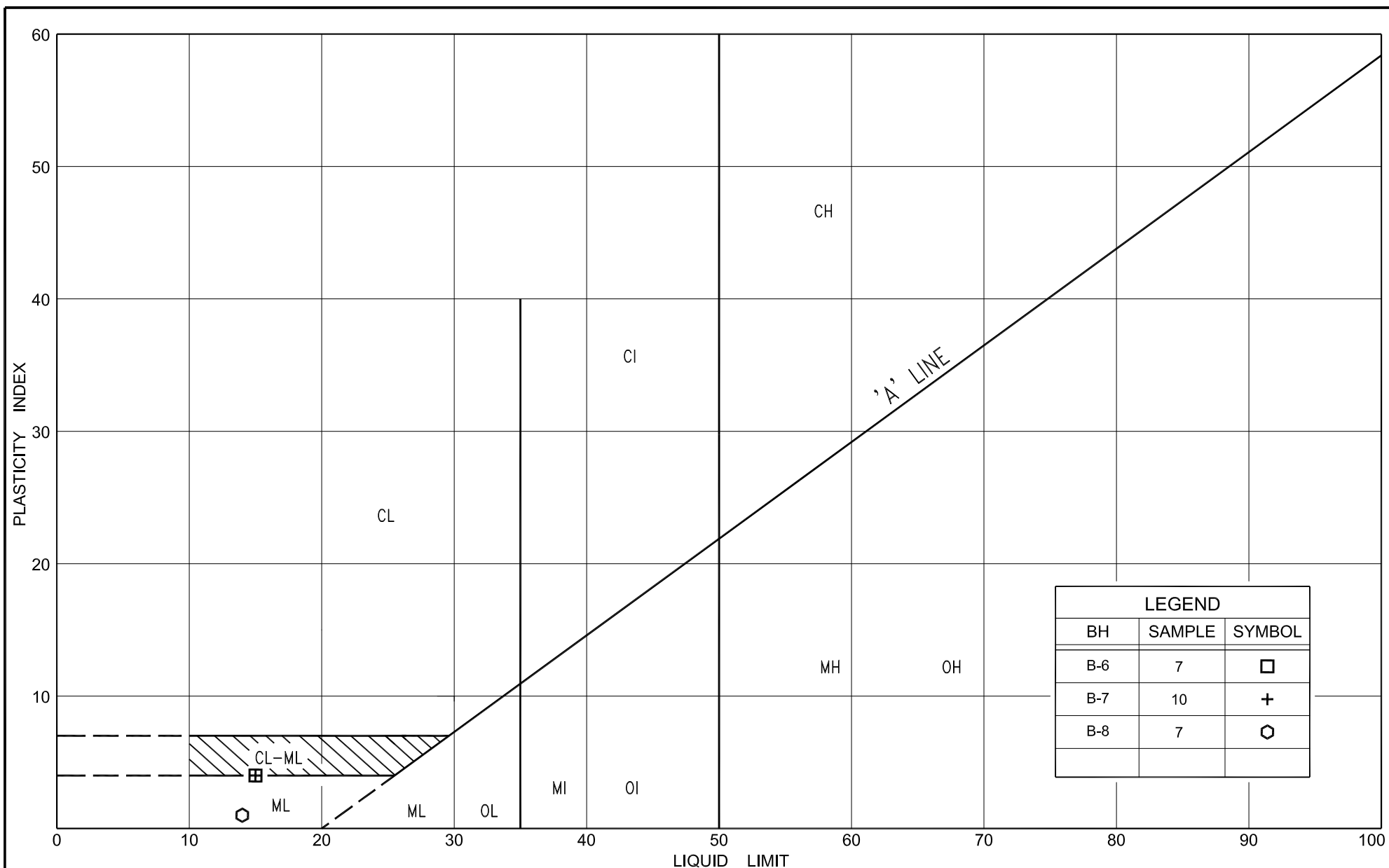
FIG No.	B-PC-1
HWY:	401
P.O. No.	09-20009



**PLASTICITY CHART**  
 SAND, with silt, some gravel, trace to some clay  
 (TILL)

FIG No. B-PC-2  
 HWY: 401  
 P.O. No. 09-20009



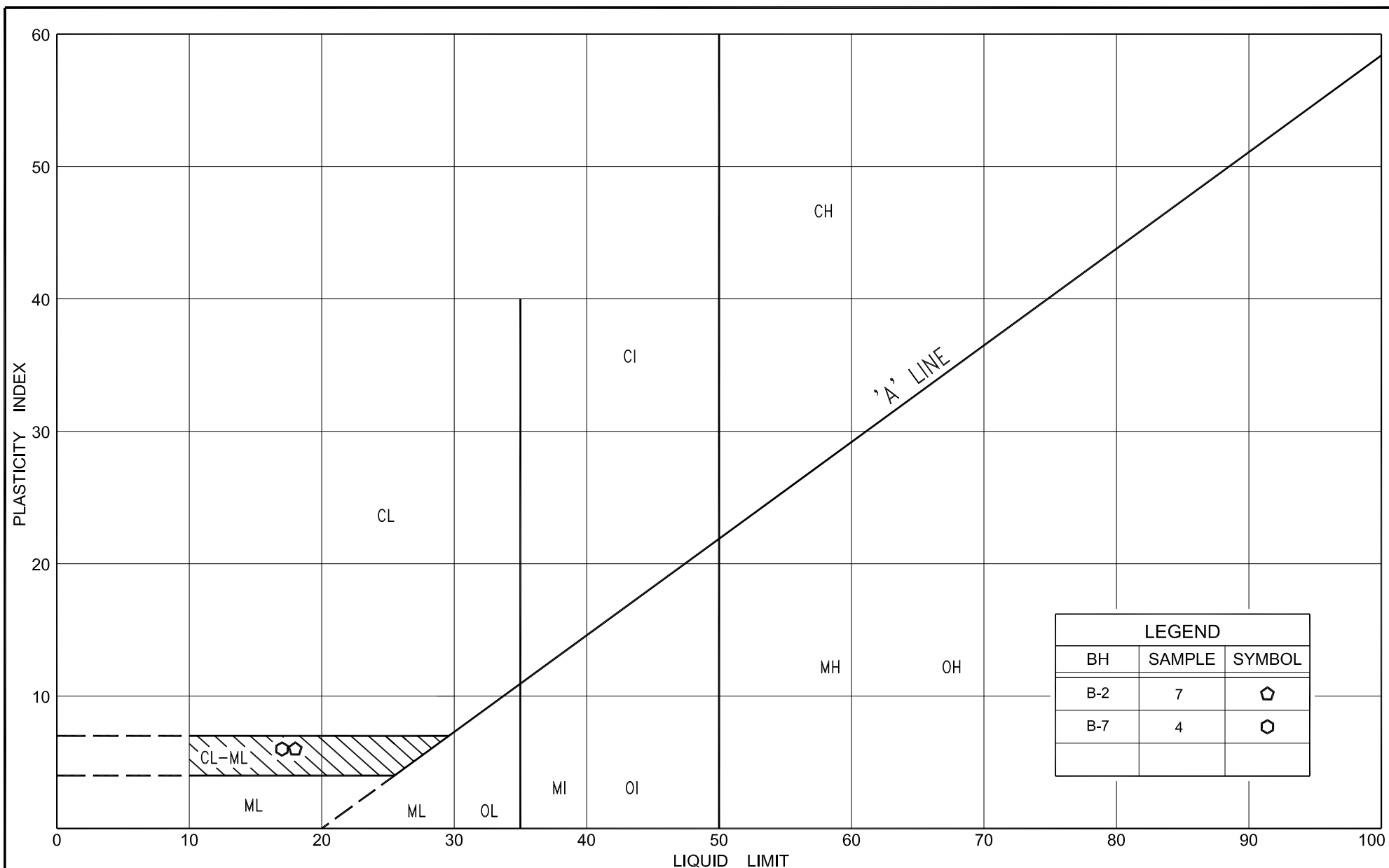


**PLASTICITY CHART**  
**SILTY SAND / SANDY SILT**  
 trace to some gravel, trace to some clay  
 (TILL)

FIG No. B-PC-3

HWY: 401

P.O. No. 09-20009

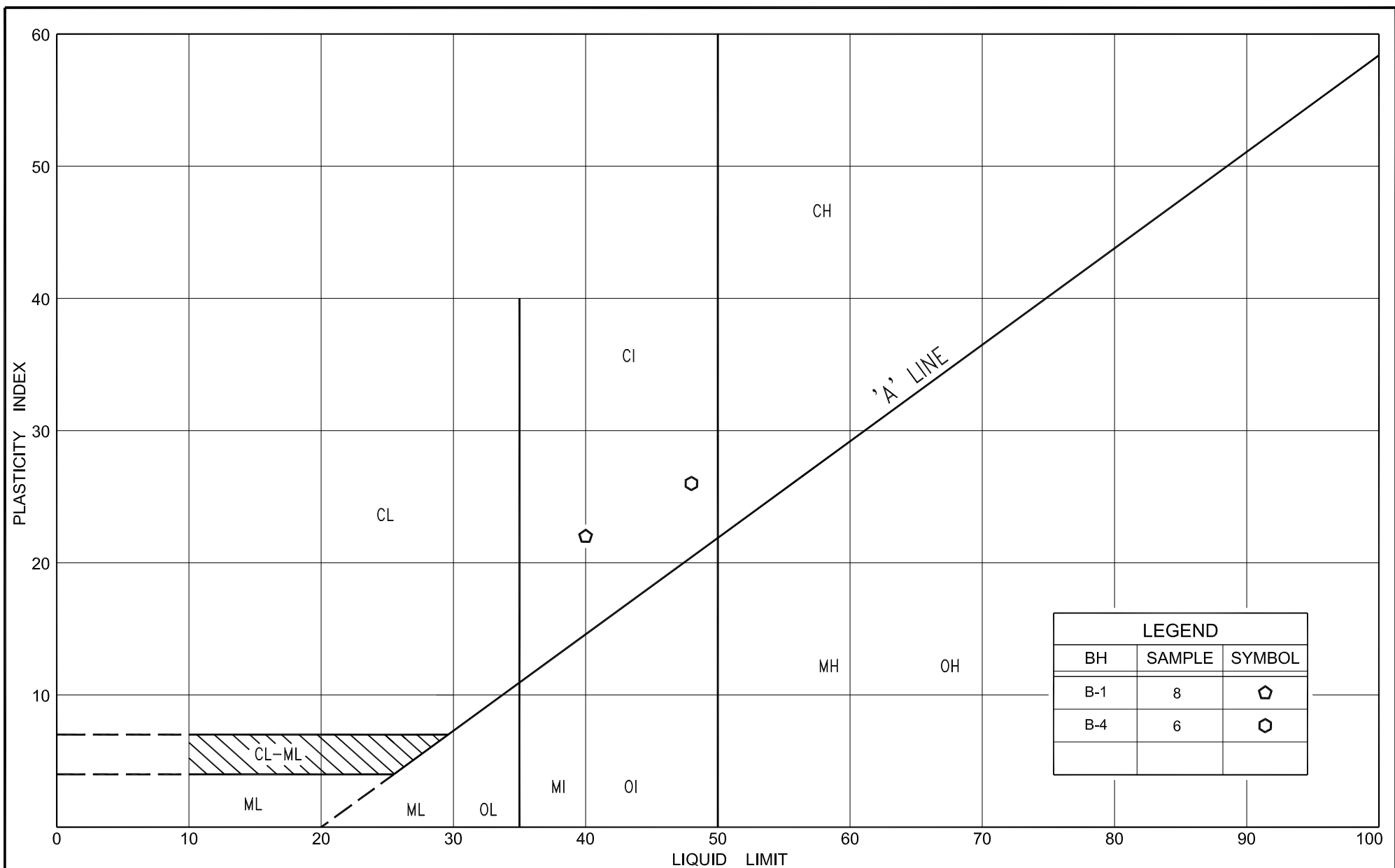


**PLASTICITY CHART**  
CLAYEY SILT, sandy, some gravel  
(TILL)

FIG No. B-PC-4

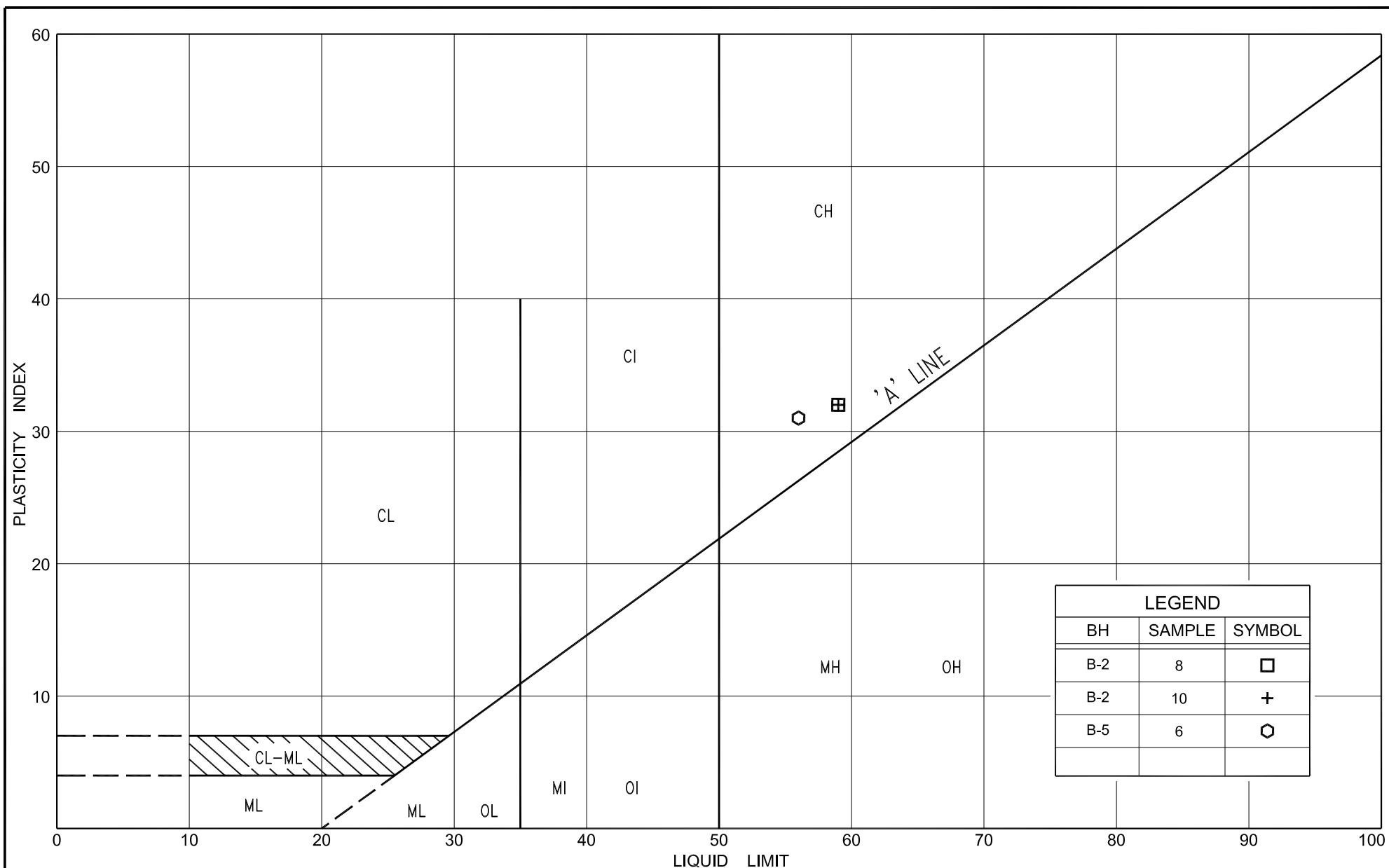
HWY: 401

P.O. No. 09-20009



**PLASTICITY CHART**  
 SILTY CLAY, trace to some sand, trace gravel  
 (TILL)

FIG No. B-PC-5  
 HWY: 401  
 P.O. No. 09-20009



## PLASTICITY CHART

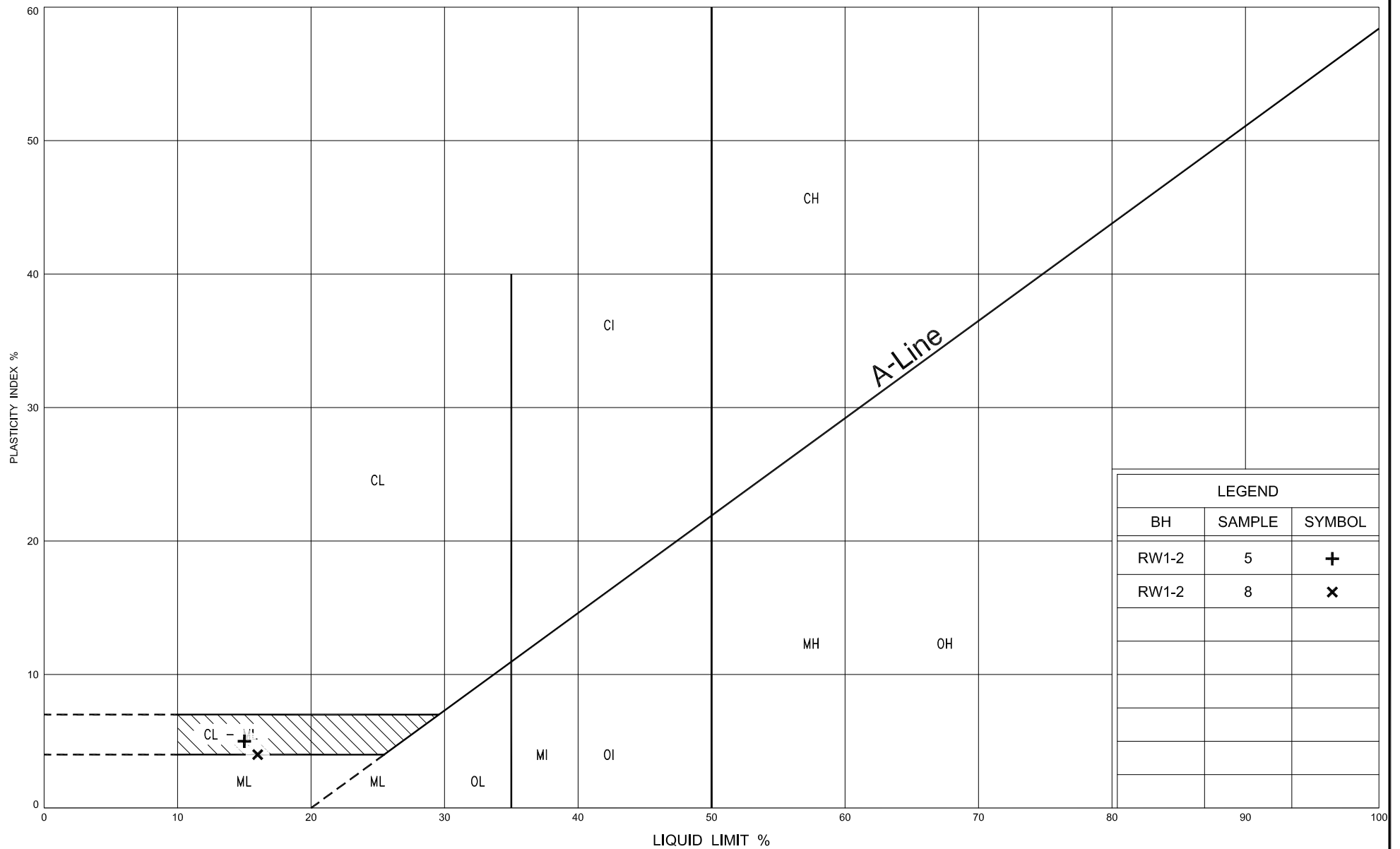
CLAY, some to with silt, trace sand

FIG No. B-PC-6

HWY: 401

P.O. No. 09-20009





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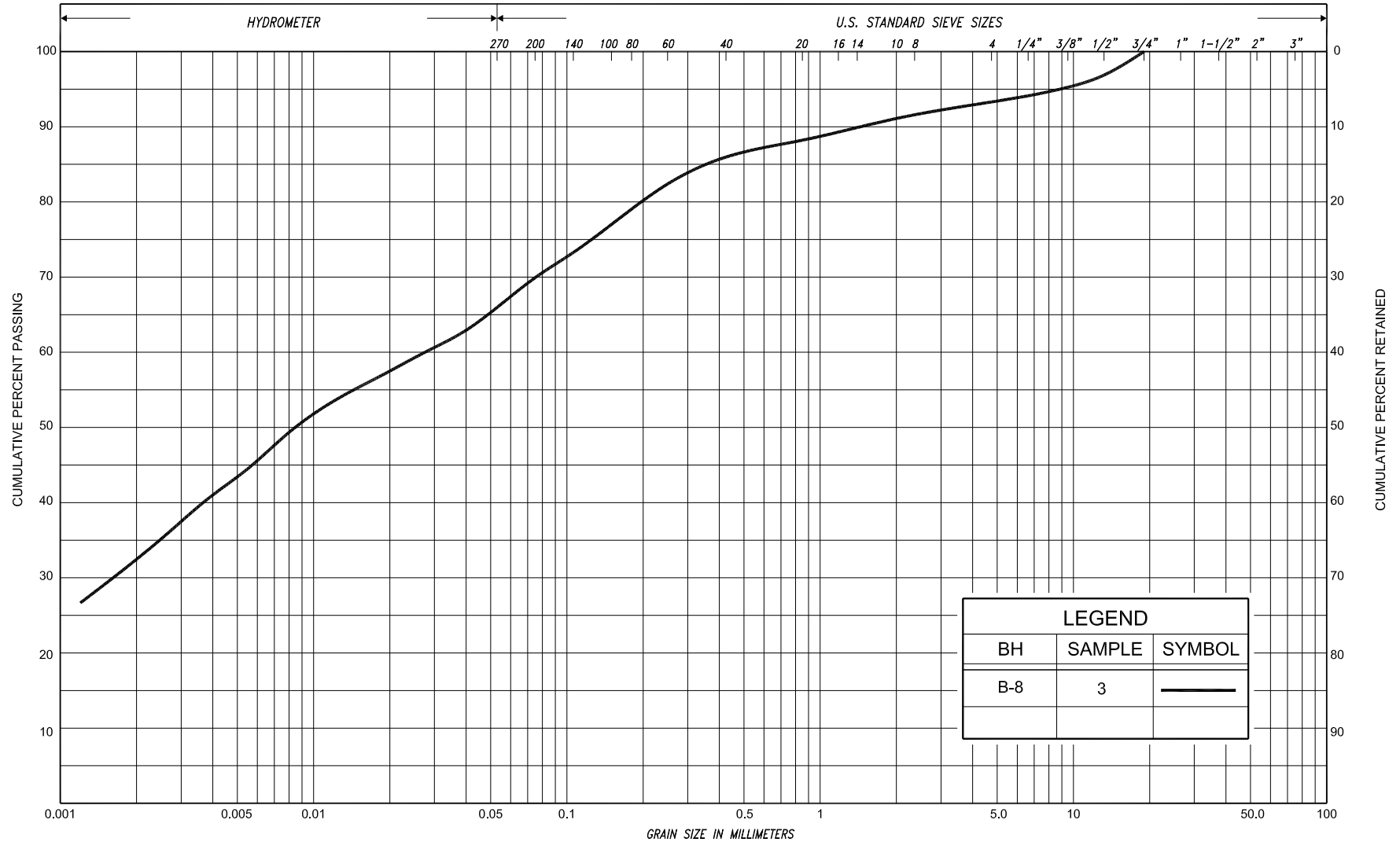
## PLASTICITY CHART

CLAYEY SILT, with sand to sandy, trace to some gravel (CL-ML)  
(TILL)

FIG No. RW1-PC-1(1)

HWY 401

P.O. No. 09-20009



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													

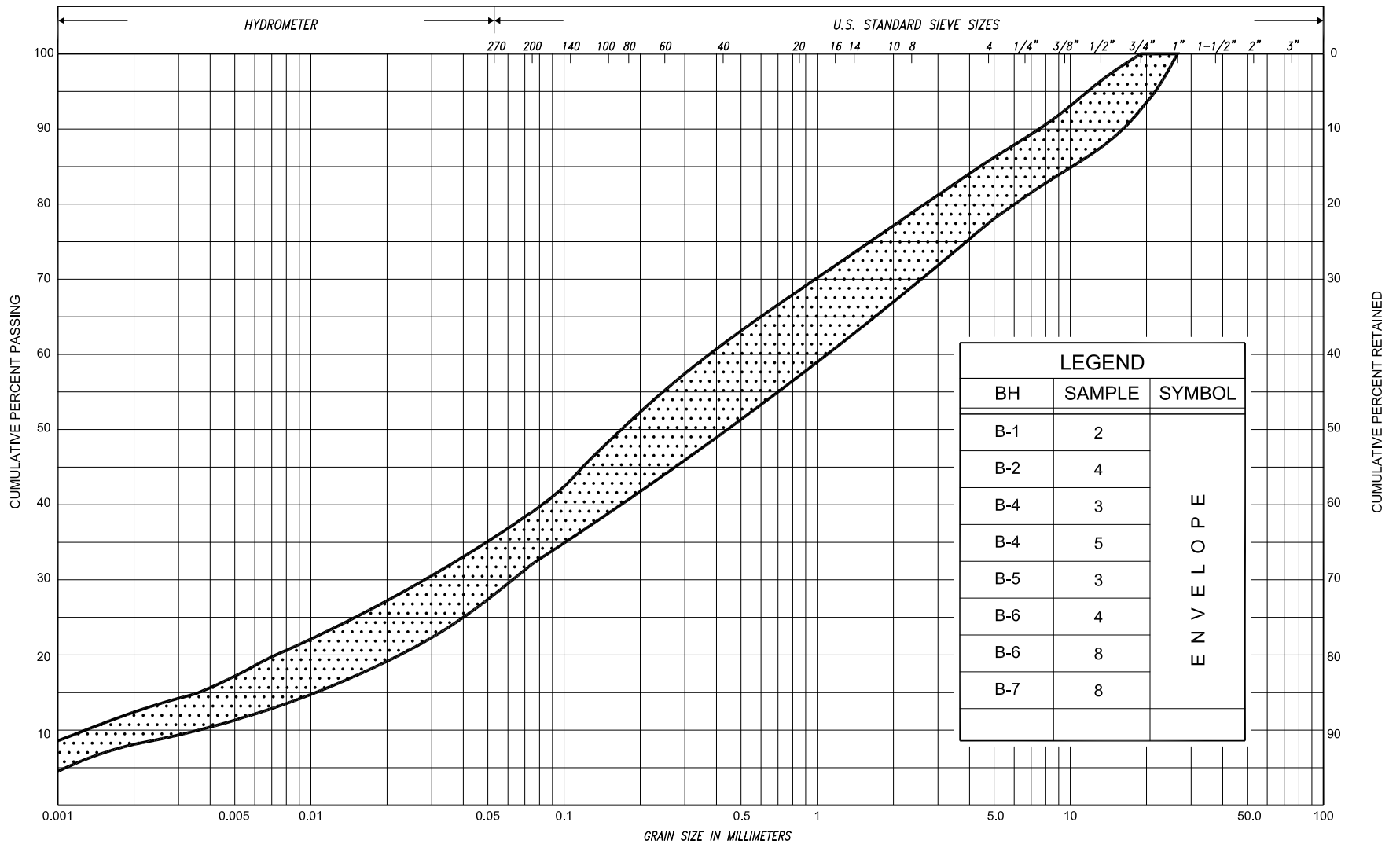


# GRAIN SIZE DISTRIBUTION CLAYEY SILT, with sand, trace gravel (FILL)

FIG No. B-GS-1

HWY: 401

P.O. No. 09-20009



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



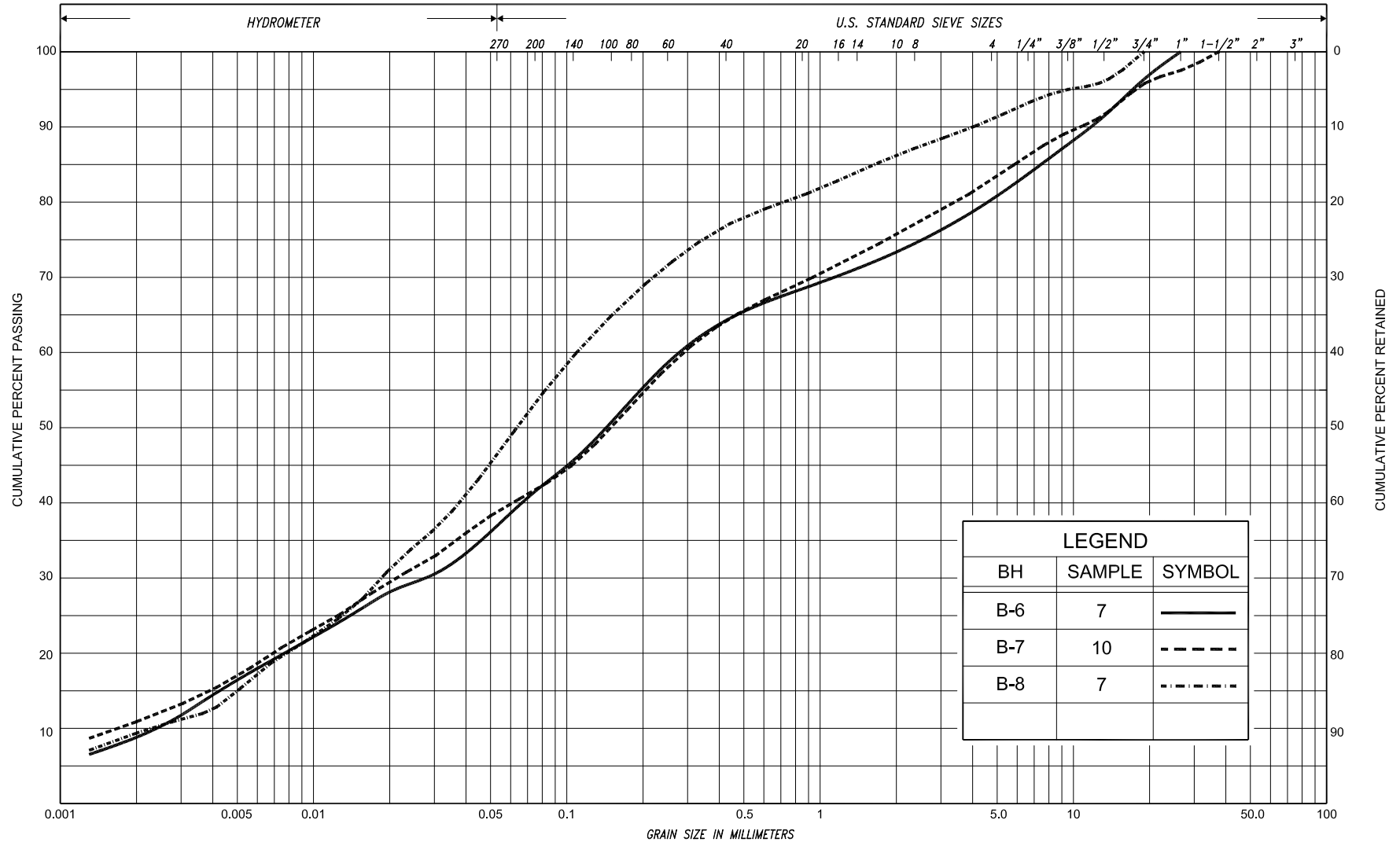
## GRAIN SIZE DISTRIBUTION

SAND, with silt, some to with gravel, trace to some clay  
(TILL)

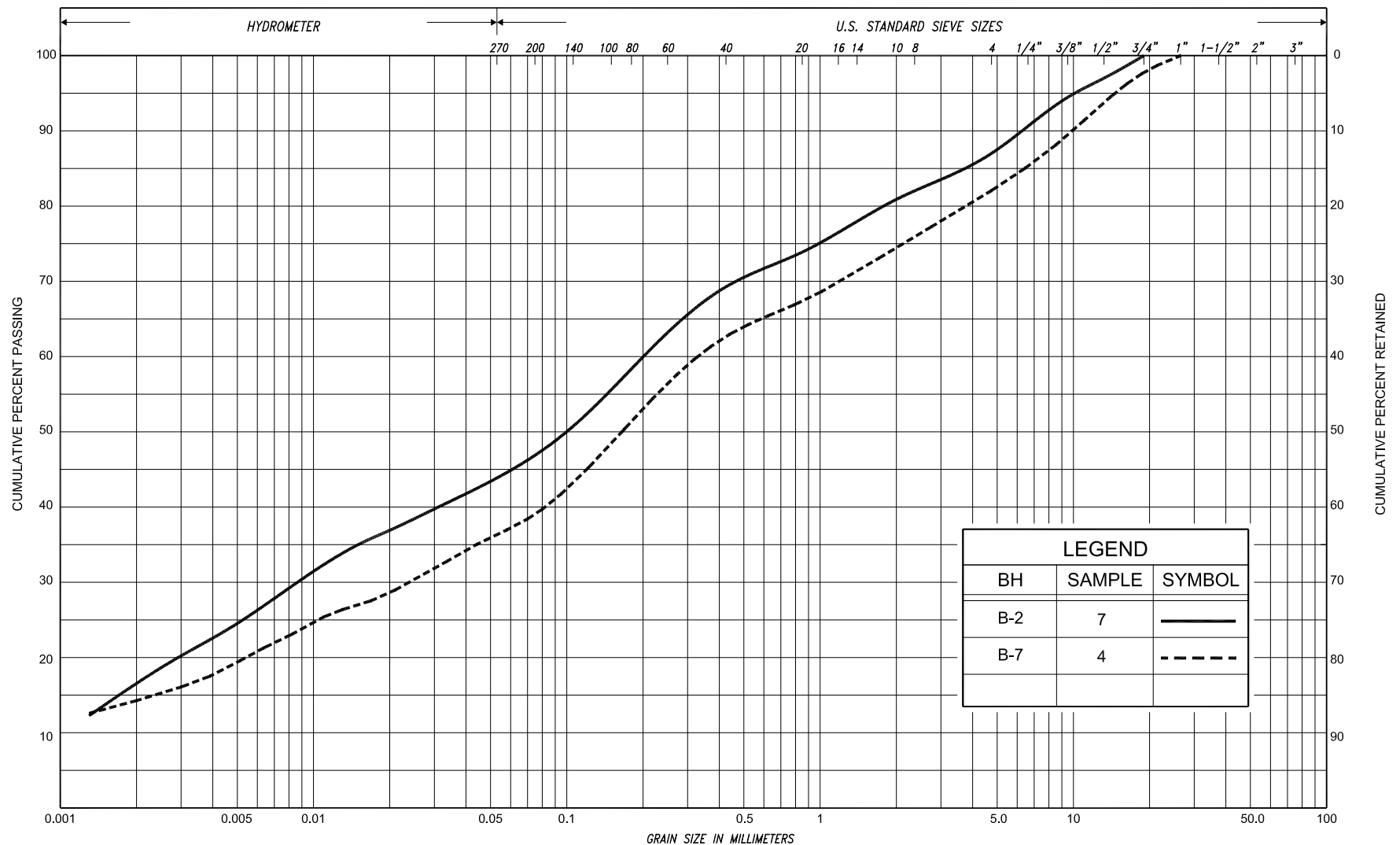
FIG No. B-GS-2

HWY: 401

P.O. No. 09-20009



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



SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED	
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										

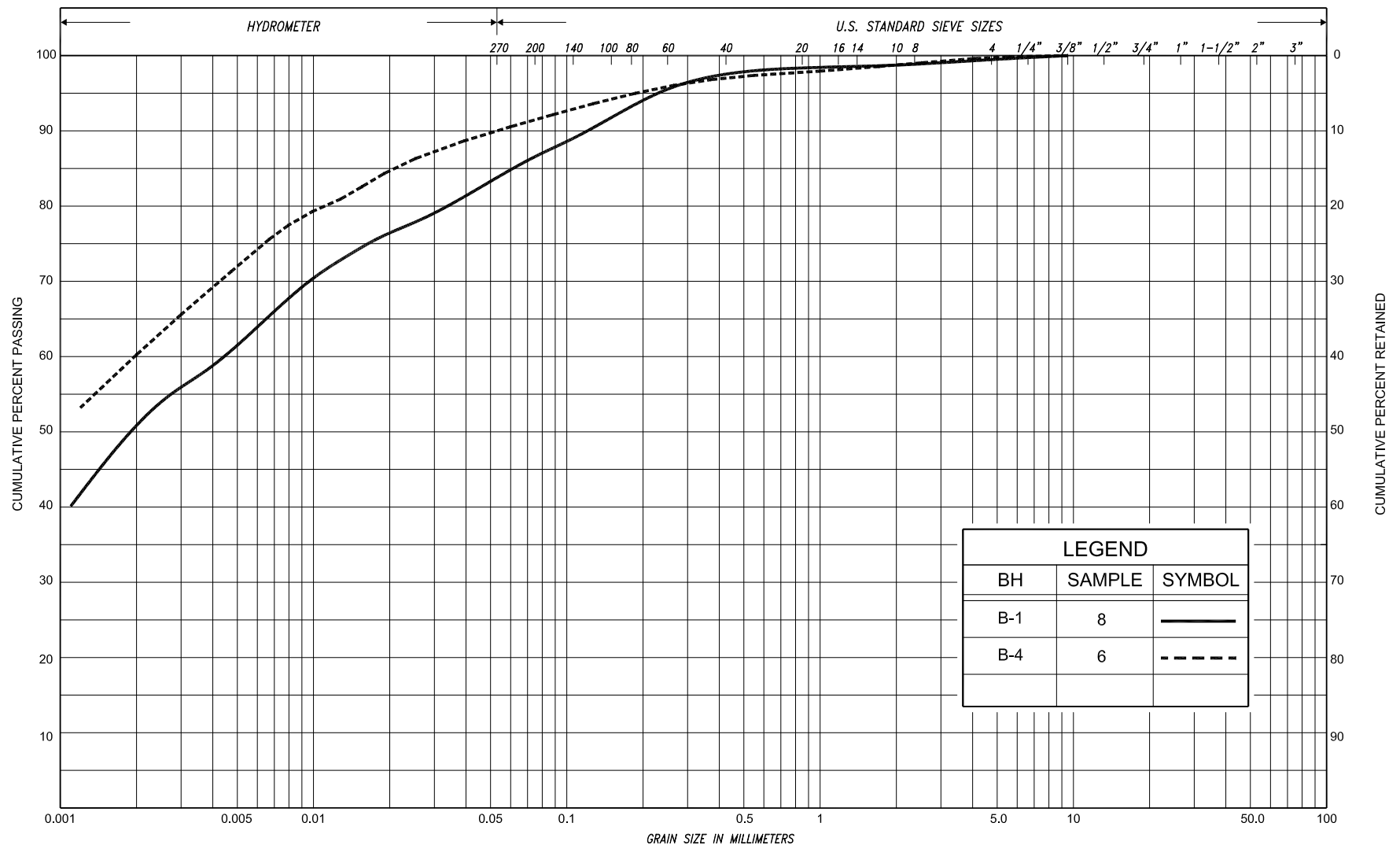


# GRAIN SIZE DISTRIBUTION CLAYEY SILT, sandy, some gravel (TILL)

FIG No. B-GS-4

HWY: 401

P.O. No. 09-20009



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

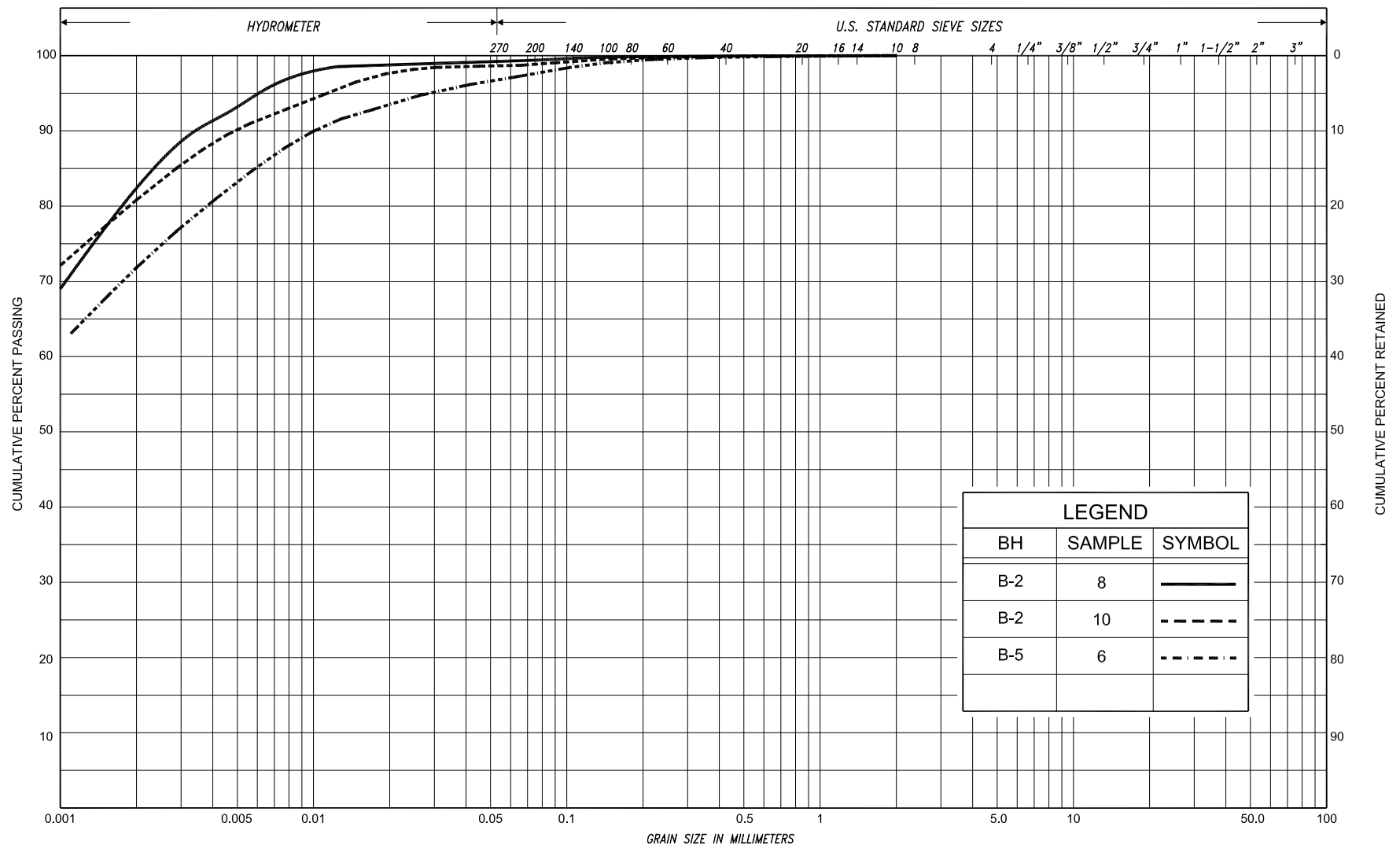


# GRAIN SIZE DISTRIBUTION SILTY CLAY, trace to some sand, trace gravel (TILL)

FIG No. B-GS-5

HWY: 401

P.O. No. 09-20009



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



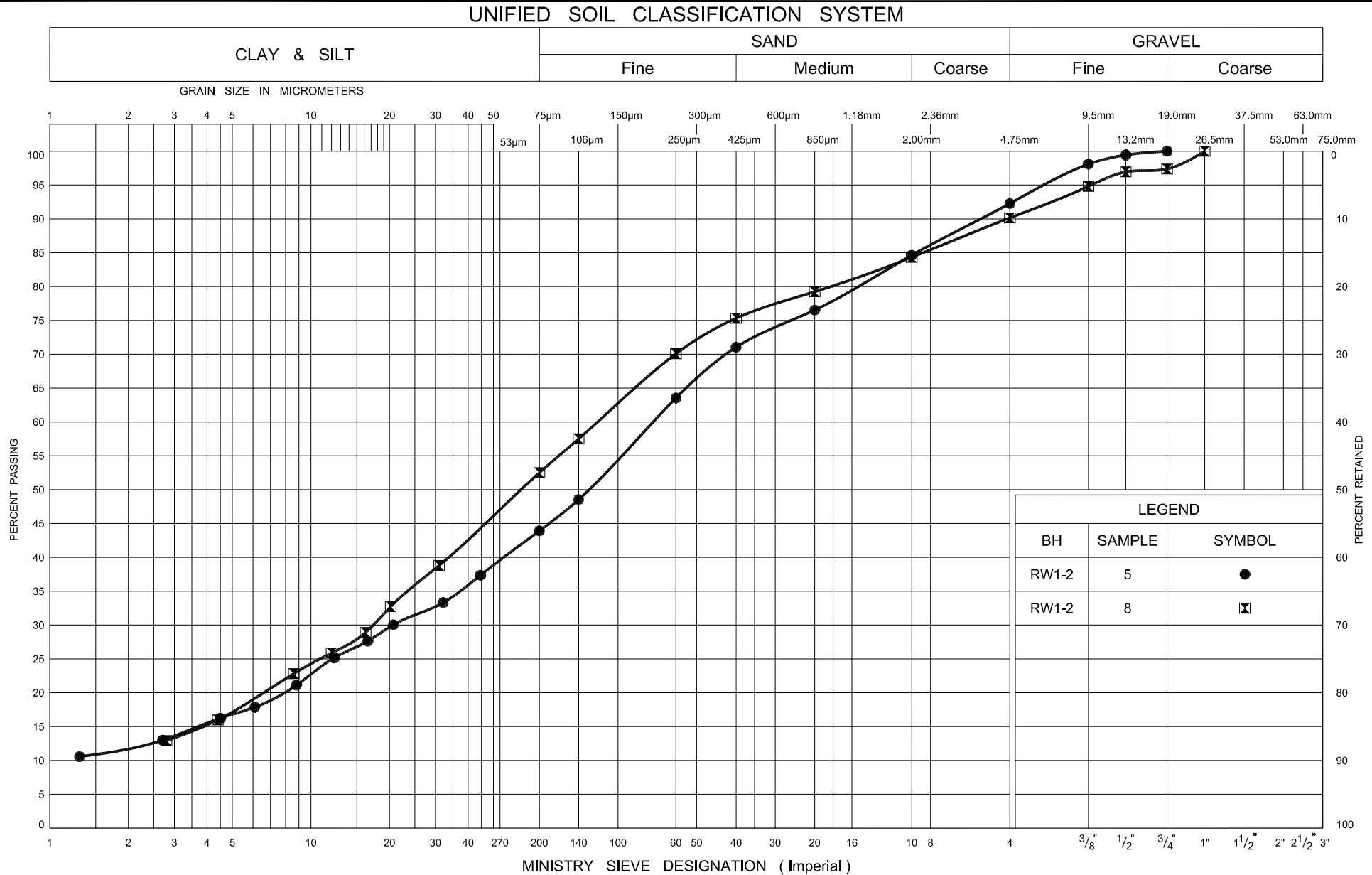
## GRAIN SIZE DISTRIBUTION

CLAY, some to with silt, trace sand

FIG No. B-GS-6

HWY: 401

P.O. No. 09-20009



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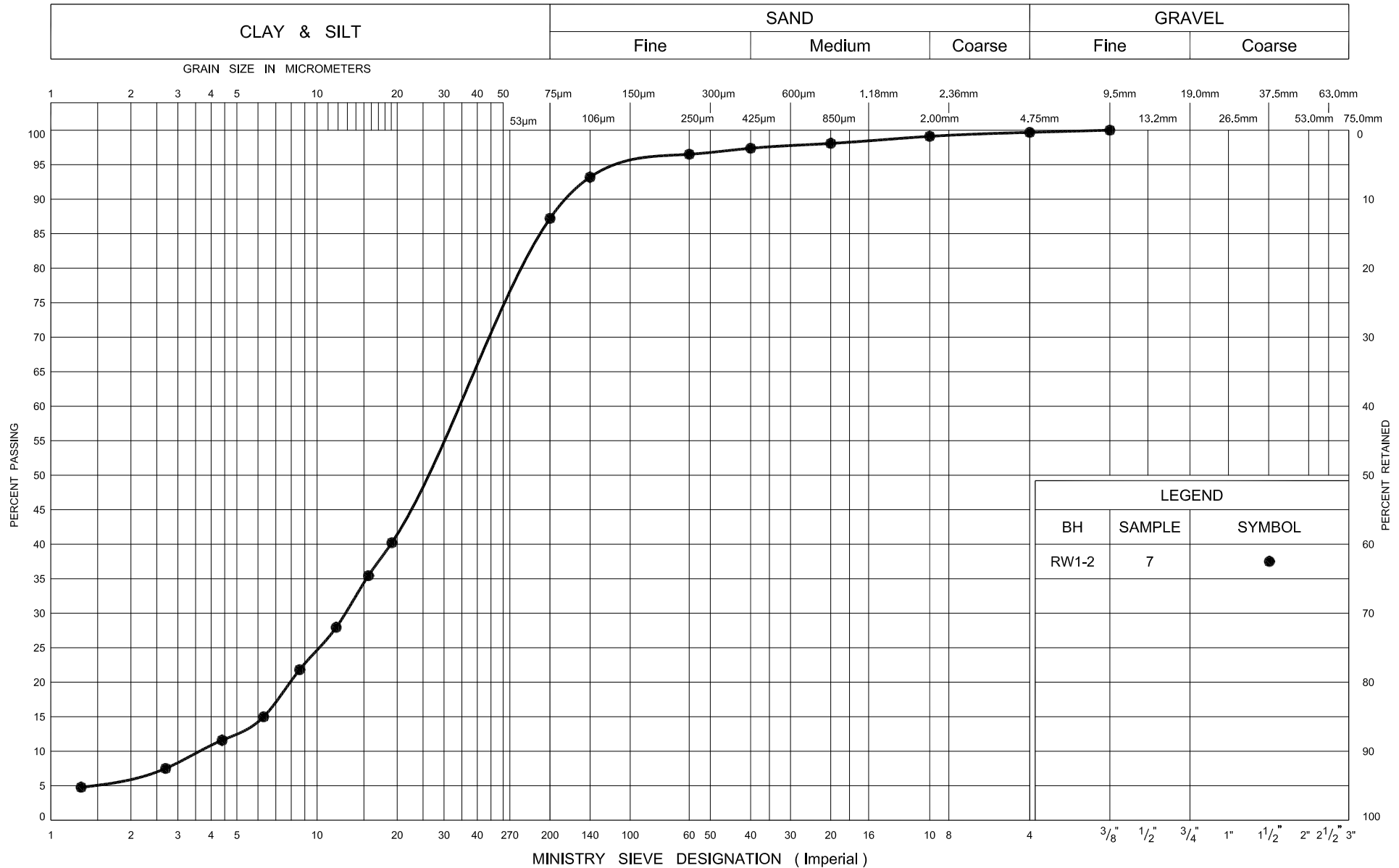
Ontario

**GRAIN SIZE DISTRIBUTION**  
CLAYEY SILT, with sand to sandy, trace to some gravel (CL-ML)  
(TILL)

FIG No.	RW1-GS-1(1)
HWY	401
P.O. No.	09-20009



## UNIFIED SOIL CLASSIFICATION SYSTEM



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Transportation  
Ontario

## GRAIN SIZE DISTRIBUTION

SILT, some sand, trace clay, trace gravel

FIG No. RW1-GS-2(1)

HWY 401

P.O. No. 09-20009



## **APPENDIX A**

### Site Photographs



**Photograph 1:** Facing north at the south approach to the Brock Street underpass. (April 7, 2014)



**Photograph 2:** Facing south at the south abutment, west side. (April 7, 2014)





**Photograph 3:** East side of the south approach, facing north. (April 7, 2014)



**Photograph 4:** Facing south at the north abutment, east side. (April 7, 2014)

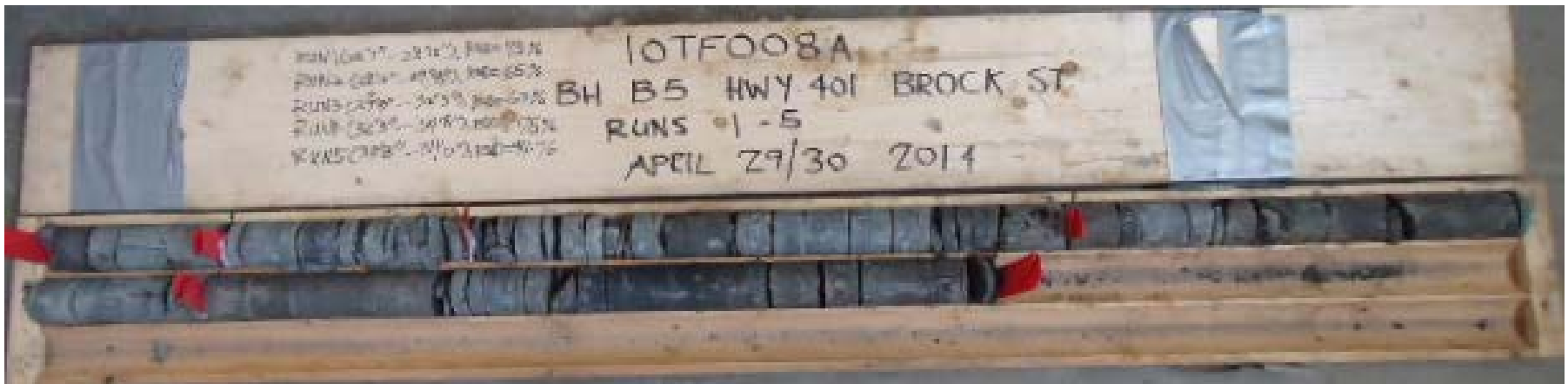


## **APPENDIX B**

Rock Core Photographs



**Photograph 1:** Cores retrieved from borehole B-3. Rock Cores 12 to 14 from 13.7 to 17.7 m depth. RQD values ranged from 0 to 94%, indicating very poor becoming fair to excellent rock quality.



**Photograph 2:** Cores retrieved from borehole B-5. Rock cores 9 to 13 from 8.6 to 11.3 m depth. RQD values ranged from 58% to 97%, indicating fair to excellent rock quality.



**FOUNDATION DESIGN REPORT**

**for**

**HIGHWAY 401 / BROCK STREET UNDERPASS**

**SITE 22-151, W.O. 09-20009**

**WHITBY, ONTARIO**

**REGIONAL MUNICIPALITY OF DURHAM**

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January 23, 2018



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Table 1 – List of Standard Specifications Referenced in Report

Table 2 – Gradation Specification for Sand Fill in Pre-augered Holes at Integral Abutments

Table 3 – Reduction Factors Due to Caisson Spacing

Figure 1 – Abutment on Compacted Fill Showing Granular A Core

Appendix A – Non-Standard Special Provisions (NSSP)



**FOUNDATION DESIGN REPORT**  
for  
Highway 401 / Brock Street Underpass  
Site 22-151, W.O. 09-20009  
Whitby, Ontario

---

**1. INTRODUCTION**

This report provides foundation engineering comments and recommendations regarding design and construction of the foundations and approach embankments for the proposed replacement of an underpass carrying Brock Street traffic over Highway 401 in Whitby, Ontario. The report was prepared for AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

This report is intended for the use of AECOM on behalf of the Ministry of Transportation of Ontario (MTO) for AECOM. It shall not be used or relied upon for any other purposes, or by any other parties including construction or design-build contractors. Where comments are made in this report on construction, they are provided only to highlight aspects, which could affect the design of the project. Contractors must make their own interpretation of the subsurface information based on the data provided in the Foundation Investigation Report as it may affect equipment selection, construction methods and scheduling.

The underpass is at approximate Station 13+858, Highway 401 chainage, in Whitby. The underpass is proposed to be a 2-span structure with a total length of 87.0 m and width of 26.5 m (ref. Brock Street Underpass General Arrangement drawing prepared by AECOM in September 2017). Retaining walls approximately 3.0 to 4.0 m long were designed at the ends of the north-east, north-west and south-east wing walls of the proposed bridge.

The road grade on Brock Street at the underpass location is planned to be at elevation 91.1 at the north abutment, elevation 91.5 at the pier and elevation 91.3 at the south abutment. The approach embankments to the structure are envisaged to be 6 to 8 m high at the north and south abutments (interpolated from ground surface elevations and the road grade on Brock Street). The approach embankments will be widened on the west side to accommodate the proposed shift of the centreline of the underpass by some 3.5 m. The road grade on Highway 401 is planned to be unchanged at approximate elevation 84.0.



In summary, the subsurface stratigraphy revealed in the boreholes was variable and generally comprised surficial sand and gravel, sand and/or clayey silt / silty clay fill overlying cohesionless till and clayey silt till / silty clay till mantling shale bedrock. A deposit of clay was present below or within the glacial till in boreholes B-2 and B-5. Cobbles and boulders were encountered in 3 boreholes. The bedrock surface was contacted at 7.0 to 13.7 m (elevation 75.9 to 77.4) in boreholes B-2 to B-5. The shallow fill depths in the boreholes indicate that the highway was constructed in an earth cut at the bridge location. The depth of the cut above the Highway 401 pavement level is estimated at some 3.5 m for the south approach and 4.2 m for the north approach.

High piezometric water level was measured in borehole B-3 (north abutment) at 2.3 m (elevation 87.3) on June 18, 2014 and at 2.5 m (elevation 87.1) on July 9, 2014. This high water level was likely due to locally perched groundwater in the sand deposits north of Highway 401. Elsewhere, groundwater was measured at depths of 4.0 to 7.0 m (elevation 79.7 to 83.2) upon completion of drilling and in a piezometer installed in boreholes RW1-2 where the piezometer was at elevation 83.9 on April 16, 2017.

The presence of hard / very dense till at shallow depths at both abutments and the pier of the underpass makes it feasible to use conventional spread footings bearing on the glacial till. The structure may also be supported on end-bearing piles driven to the hard / very dense till or shale bedrock or on caissons founded in the glacial till or socketed into shale. It is noteworthy that cobbles and boulders present in the till should be expected during foundation construction at the site and the contractor should allow for these obstructions during the design and implementation of the project.

The replacement of the underpass will require staged construction and use of roadway protection systems to maintain traffic over Highway 401. The roadway protection system should consider the high perched groundwater condition encountered in the sand deposits at the existing approach embankments. Refer to section 6 of this report for a detailed discussion.

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. These comments and recommendations are based on the conditions revealed during the investigation 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.

## **2. BRIDGE FOUNDATIONS**

### **2.1 General Discussion**

The design road grade at the underpass location is at elevation 91.1 to 91.5, about 7.5 m above the road grade on Highway 401. The grade is anticipated to remain unchanged on the existing north and south approach embankments. The proposed widening of both embankments on the west side will require a grade raise.

Based on the available information, design and construction of foundations to support the replacement underpass is considered feasible at the site.

A summary of advantages and disadvantages of feasible foundation alternatives is given below:

<b>FOUNDATION TYPE</b>	<b>ADVANTAGES</b>	<b>DISADVANTAGES</b>	<b>RISKS / CONSEQUENCES</b>
Spread footings on native soils	<ul style="list-style-type: none"><li>• Ease of construction</li></ul>	<ul style="list-style-type: none"><li>• Does not allow integral abutment construction</li><li>• Lower bearing resistance than for driven piles or caissons</li><li>• Need for groundwater control to lower perched groundwater in approach embankments</li><li>• Reduced capacity due to high groundwater level</li></ul>	<ul style="list-style-type: none"><li>• Limited support for increase in loading</li></ul>
Spread footings on engineered fill pads	<ul style="list-style-type: none"><li>• Ease of construction</li></ul>	<ul style="list-style-type: none"><li>• Does not allow integral abutment construction</li><li>• Lower bearing resistance than for driven piles or caissons</li><li>• Need to provide erosion protection</li><li>• Need for groundwater control to lower perched groundwater in approach embankments</li><li>• Reduced capacity due to high groundwater level</li></ul>	<ul style="list-style-type: none"><li>• Consolidation settlements expected</li></ul>



FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES
Driven piles	<ul style="list-style-type: none"> <li>• High bearing resistance</li> <li>• Allow integral abutment construction</li> </ul>	<ul style="list-style-type: none"> <li>• High cost relative to footings</li> <li>• Vibration induced during driving</li> <li>• Pilot holes required due to presence of cobbles and boulders</li> </ul>	<ul style="list-style-type: none"> <li>• Pile driving next to existing bridge may damage foundations or superstructure</li> </ul>
Caissons	<ul style="list-style-type: none"> <li>• Higher bearing resistance than for other options</li> <li>• May address space limitations at the median for pier construction</li> </ul>	<ul style="list-style-type: none"> <li>• High cost relative to footings</li> <li>• Potential for difficulties to be encountered during drilling through the hard glacial till due to cobbles and boulders</li> </ul>	<ul style="list-style-type: none"> <li>• Augering difficulties could result in construction delays and cost overruns</li> </ul>

Spread footings bearing on the hard / very dense till in combination with engineered fill pads as outlined in this report are considered to be a feasible less means of supporting the foundation loads at the abutments and pier from a foundation engineering perspective provided that the perched groundwater is effectively controlled during construction staging. The use of steel H-piles driven to the hard / very dense till or shale bedrock or caissons founded in the glacial till or socketed into shale are suitable at the site and may be necessary at the pier location due to construction constraints resulting from the narrow median. For the structure to be designed with integral abutments, the use of steel H-piles will be required at the abutments.

Further comments and recommendations for design of the foundations are provided in the following sections. The standard specifications referenced in this report are listed in Appendix A.

#### 2.1.1 Seismic Conditions

The reference Peak Ground Acceleration ( $PGA_{ref}$ ) is 0.075 for the Town of Whitby, Ontario (National Building Code of Canada, 2015). The soil at the project site for seismic design purposes is classified as Type C in accordance with clause 4.4.3.2 of the CHBDC, 2014.

#### 2.1.2 Foundation Frost Protection

All footings and pile or caisson caps subject to frost action should be provided with 1.2 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.



## 2.2 Spread Footings

As discussed in Section 2.1, the foundations for the abutments and pier may be constructed as footings bearing on the glacial till. Spread footings placed on the native soils may be used for the design and construction of semi-integral abutments. Construction of the pier on spread footing may not be possible due to space limitations caused by the need to maintain traffic on Highway 401.

Subject to the composition of the soils and groundwater conditions at the actual foundation locations, the founding levels for 2.5 m wide spread footings bearing on the glacial till and corresponding values of factored geotechnical bearing resistance at ultimate limit states (ULS) and a geotechnical reaction at serviceability limit states (SLS) are given below:

FACTORED GEOTECHNICAL BEARING RESISTANCE AT ULS (kPa)	GEOTECHNICAL REACTION AT SLS (kPa)	NORTH ABUTMENT	PIER	SOUTH ABUTMENT
		BOREHOLES B-2 AND B-3	BOREHOLES B-4 AND B-5	BOREHOLES B-6 AND B-7
400	250	Elev. 87.5	—	—
550	350	Note 1	Elev. 82.5	Elev. 87.0

Note 1: To limit differential settlements along the north abutment due to the presence of a clay deposit in borehole B-2, it is not recommended to lower the founding levels for spread footings.

The geotechnical reaction at SLS allows for 25 mm compression of the founding medium. The geotechnical bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.2 of the CHBDC.

The horizontal force imposed on the foundations will be resisted in part by the friction forces developed between the underside of the footing and the native till soil. An unfactored friction factor of 0.7 is recommended for footings constructed on the glacial till.

### 2.2.1 Footings Constructed on Structural Fill

Footings constructed on structural fill placed in the approach embankments could also be employed to support the foundation loads. Footings on structural fill pads may be used for semi-integral abutments. Construction of the pier on spread footing may not be possible due to space limitations caused by the need to maintain traffic on Highway 401 during construction.



The structural fill should comprise OPSS Granular A material placed and compacted according to OPSS 501 in maximum 200 mm thick lifts, compacted to 100% standard Proctor maximum dry density and extended laterally to a line inclined downwards at 45° to the horizontal originating at least 1 m from the top of the footing. This scheme is illustrated in Figure 1, appended.

The recommended bearing resistance for 2.5 m wide footings constructed on 1 m thick structural fill placed at approximate elevation 87 is as follows:

Factored Bearing Resistance at ULS	= 650 kPa
Bearing Resistance at SLS	= 250 kPa

The resistance at SLS normally allows for 25 mm of compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistance.

The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.2 of the CHBDC.

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 structural fill. An unfactored friction factor of 0.7 is recommended for footings on granular fill.

### **2.3 Driven Piles**

Steel H-piles driven to bedrock at the north abutment and pier and into the very dense cohesionless till at the south abutment is a suitable method of supporting the abutments and pier foundations of the underpass. Further, construction of abutments with battered piles, semi-integral abutment design or construction of integral abutments supported on steel H-piles is considered to be feasible at the site. Section 2.3.1 of this report provides details on integral abutment design and construction.

The bedrock surface was contacted at 7.0 to 13.7 m (elevation 75.9 to 77.4) at the north abutment and pier. In boreholes B-3 and B-5, the bedrock comprised a slightly weathered to unweathered soft to medium strength shale with interbedded limestone and was typically classified as fair to excellent quality (RQD of 58 to 97%) with a core recovery of 79 to 100%. Very poor quality rock



was identified in the upper 1.3 m core sample in borehole B-3 and for 0.7 m length at 11.3 m (elevation 72.6) in borehole B-5. It is considered that the rock is capable of adequately supporting the foundation loads.

At the north abutment and pier, the H-piles should be driven to refusal into the bedrock encountered at 7.0 to 13.7 m below existing grade (elevation 75.9 to 77.4). It is anticipated that the piles will penetrate approximately 1.0 to 1.5 m into the bedrock to reach practical refusal. Cobbles and boulders should be expected during construction of deep foundations at the site.

The recommended factored axial resistance at ULS for the HP 310x110 pile section driven to bedrock is 2000 kN. The resistance at SLS normally allows for 25 mm compression of the pile and founding medium. Considering the bedrock to be non-yielding and the pile length required, the design is not expected to be governed by settlement criteria since the loading necessary to produce 25 mm axial deformation of the pile and bedrock would be larger than the factored resistance at ULS. For design purposes, however, the resistance at SLS may be taken equal to the factored axial resistance at ULS.

Although probable bedrock was encountered in borehole RW1-2 at elevation 79.0, it is recommended that the piles be uniformly designed for bearing on the very dense glacial till overlying the bedrock. For steel HP 310 x 110 piles driven into the very dense glacial till below elevation 77.0 to 81.0 at the south abutment location, the factored axial resistance at ULS of 1600 kN and axial resistance at SLS of 1400 kN are recommended. The founding elevation for the H-piles at the south abutment should be selected for embedment of 1.5 m into the glacial till and/or bedrock, where bedrock is encountered.

A summary of the recommended axial resistance for the H-piles is as follows:

LOCATION	MATERIAL	AXIAL RESISTANCE, kN		REFERENCE BOREHOLE / (ESTIMATED TIP ELEVATION)
		FACTORED ULS	SLS	
North Abutment	Bedrock	2000	2000	B-2 (76.0); B-3 (74.5)
Pier	Bedrock	2000	2000	B-4 (75.5); B-5 (75.5)
South Abutment	Very dense sand till/ silty sand till	1600	1400	B-6 (81.0); B-7 (77.0) RW1-2 (79.0)



At the abutments, piles will be driven through the native soils containing relatively low compressibility clayey silt till / silty clay till. Taking into account that the abutment locations have been preloaded by the existing approach embankments for a considerable period of time, no settlements are anticipated under the existing fill if the road grade will be maintained. Under the widened section of the embankment, however, 15 to 20 mm settlements are estimated. Taking into account that these settlements will occur during construction of the approach embankment, it is considered that no allowance for negative skin friction will be required for the design.

The piles should be installed and monitored in accordance with the requirements of MTO OPSS 903. The presence of the very dense glacial till deposit will cause pile driving operation difficulties. An NSSP prepared to alert the Contractor to the presence of cobbles and boulders at the site is attached to this report in Appendix A.

The approach embankment fill within the limits of the pile foundation including fill placed below grade to replace any excavated unsuitable/compressible soils should comprise Granular A or Granular B Type II with a maximum nominal size of 75 mm to enable driving of the piles and minimise the potential for damage during pile installation. Granular B Type II is recommended below the water table if required.

#### 2.3.1 Integral Abutment Considerations

For the integral abutment design, the H-piles at both abutments should be driven to practical refusal as recommended in the previous section. The minimum 5 m long pile length below the abutment stem to be incorporated in the design is not anticipated to be a concern at this site.

The soil adjacent to the upper portion of the abutment piles is expected to comprise typically compact to very dense sandy/silty soils. To accommodate movement of the integral abutment system, it is recommended that two concentric CSPs extending at least 3 m below the bottom of the abutment be placed around the pile to create an annular space. The inner CSP of 600 mm diameter should be filled with sand meeting the gradation requirements of Granular B Type I with a maximum particle size of 37.5 mm. Alternatively, a single CSP filled with loose uniform sand meeting the requirements given in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.





## 2.4 Caissons

Supporting the structure on caissons founded in the hard / very dense till or, alternatively, socketed into the shale bedrock is considered feasible. This type of foundation is considered to be useful to reduce the space required at the pier location for the construction of the foundation.

### 2.4.1 South Abutment

For the south abutment, the factored axial resistance for caissons founded in the hard / very dense till for various diameters and embedment ratios is as follows:

CAISSON DIAMETER, m	FACTORED AXIAL RESISTANCE, kN, FOR RATIO OF DEPTH BELOW SURFACE OF HARD / VERY DENSE TILL TO CAISSON DIAMETER		
	1	2	3
0.9	400	550	700
1.2	720	970	1220
1.5	1130	1520	1910
1.8	1620	2185	2750

In the table above, the reference surface of the hard / very dense till should be considered at the following levels:

SOUTH ABUTMENT	
BOREHOLE	ELEVATION
B-6	85.0
B-7	85.0
RW1-2	81.3

The resistance at SLS normally allows for 25 mm compression of the element and founding material. Taking into account the denseness of the glacial till at the site, settlements of the caissons are estimated to be within 25 mm under the above loading. The geotechnical resistance at SLS is therefore not expected to govern the design of the caissons.



#### 2.4.2 North Abutment and Pier

In view of the variable denseness/consistency of the subsoil at the north abutment and pier, founding these elements on caissons constructed within the soil cover is not recommended. The north abutment and pier may be founded on caissons rocketed into the bedrock. The factored axial resistance for caissons socketed at least 1 m into sound shale, based on the rock found in boreholes B-3 and B-5 at respective elevation 74.6 and 75.3 is given below:

FOUNDATION ELEMENT	CAISSON DIAMETER, m	FACTORED AXIAL RESISTANCE, kN, FOR SOCKET / DIAMETER RATIO (CAISSON BASE ELEVATION)		
		1	2	3
NORTH ABUTMENT	0.9	1500 (el. 73.7)	1900 (el. 72.8)	2250 (el. 71.9)
	1.2	2550 (el. 73.4)	3150 (el. 72.2)	3750 (el. 71.0)
	1.5	3875 (el. 73.1)	4800 (el. 71.6)	5725 (el. 70.1)
	1.8	5450 (el. 72.8)	6800 (el. 71.0)	8150 (el. 69.2)
PIER	0.9	1500 (el. 74.3)	1900 (el. 73.5)	2250 (el. 72.6)
	1.2	2550 (el. 74.0)	3150 (el. 72.9)	3750 (el. 71.7)
	1.5	3875 (el. 73.7)	4800 (el. 72.3)	5725 (el. 70.8)
	1.8	5450 (el. 73.4)	6800 (el. 71.7)	8150 (el. 69.9)

Considering that the bedrock is non-yielding and the caissons are relatively short, the load required for 25 mm compression would exceed the structural capacity of the founding element. The geotechnical reaction at SLS should therefore be assumed equal to the factored geotechnical resistance at ULS for design purposes.

All caisson bases should be installed in accordance with OPSS 903 and inspected by qualified geotechnical personnel to verify the competency of the founding surface. A steel liner and air quality monitoring must be provided by the contractor to permit down-hole entry for cleaning and/or inspection of the caisson bases, if required.

It is noted that the presence of cobbles and boulders in the glacial till mantling the bedrock may present difficulties during the installation. The caissons should be installed using temporary steel liners and, depending on the groundwater conditions at the time of installation, drilling mud or vibration methods may need to be used to advance the caisson drill holes below the glacial till.



Once the caissons are socketed into the shale bedrock, casting of concrete should be possible with only limited pumping requirements.

Resistance of the caissons to lateral loads may be assessed using horizontal subgrade reaction soil parameters. Group action for lateral loading should be considered based on the reduction factors given in the attached Table 3.

The modulus of horizontal subgrade reaction  $k_s$ ,  $\text{MN/m}^3$ , may be estimated using the following equation:

$$k_s = N_h \frac{z}{d}$$

where  $N_h$  = coefficient related to soil density  
= 12  $\text{MN/m}^3$  for dense to very dense cohesionless till  
= 25  $\text{MN/m}^3$  for shale bedrock  
 $z$  = depth, m  
 $d$  = pile diameter, m

### **3. RETAINING WALL FOUNDATIONS**

Three cast-in-place concrete walls were added to the structure at the north-west, north-east and south-east wing walls. The inverted T retaining walls are planned to be 3.0 m high above the foundation level and 3.0 m long on the north side of the bridge and 4.0 long on the south-east corner.

At the designed founding level of El. 87.0 m, the retaining walls will be placed on the native dense to very dense sand till deposit. At this level, the retaining wall foundations may be designed for the following Resistances:

Factored Bearing Resistance at ULS = 650 kPa  
Bearing Resistance at SLS = 250 kPa

The resistance at SLS normally allows for 25 mm of compression of the founding medium. Differential settlement is expected to be less than 75% of this value. A footing embedment depth of 1.2 m was assumed for computation of the ULS resistance.



The bearing resistance for inclined loads should be reduced in accordance with the requirements of clause 6.10.2 of the CHBDC.

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 structural fill. An unfactored friction factor of 0.7 is recommended for footings on granular fill.

Note that engineered fill may be required where the subgrade may have been disturbed for the placement of sewers such as near the NW corner of the bridge. Groundwater was measured near elevation 87.0 m at the retaining walls located north of the bridge. The groundwater should be well drained from behind and inside bottom of the retaining walls since the sand subgrade tends to allow progressive outward movement of the walls due to the wetting cycles of the sandy subsoil.

#### **4. LATERAL EARTH PRESSURE**

The abutment walls, retaining walls and wing walls, if incorporated, 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 6.12.3 of CHBDC)

$C_s$  = earth pressure induced by seismic events, kPa (refer to clause 4.6.5 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 2.1.1.



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

PARAMETERS	GRANULAR A	GRANULAR B TYPE II
Angle of Internal Friction, degrees	35	35
Unit Weight, kN/m <sup>3</sup>	22.8	22.8
Coefficient of Active Earth Pressure $K_a$	0.27	0.27
Coefficient of Earth Pressure At-Rest $K_o$	0.43	0.43
Coefficient of Passive Earth Pressure $K_p$	3.69	3.69

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° 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 (OPSS 405 and OPSD 3190.100) 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 geotextile to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade.

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

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.12.3 of the CHBDC. Refer to OPSS 501 for additional information in this regard.

## 5. APPROACH EMBANKMENTS

The height of fill embankment widenings will be 6 to 8 m high at the north and south approaches to the structure. It is anticipated that the approach embankment widenings will be constructed with earth borrow or granular material.



Any peat or topsoil identified at the abutment locations and along the alignment of the approach fills within 20 m of the abutments should be stripped prior to placement of the embankment fill.

The embankments should be constructed in accordance with OPSD 201.020, 202.010, OPSS 902 and SP 206S03. The side slopes of the approach embankments should be inclined no steeper than 2H:1V for earth fill. A 2 m wide mid-height berm should be provided only if the height of uninterrupted slopes is in excess of 8 m for earth fill embankments (OPSD 202.010).

It is considered that the approach embankments constructed in accordance with these recommendations will be stable. Settlement of the road surface within the footprint of existing embankment will only be governed by 'consolidation' of the newly placed fill. In the widened section along the alignment, an additional settlement will take place due to consolidation of the subsurface soils.

The backfill placed adjacent to the abutments will be about 6 to 8 m thick. The magnitude of 'consolidation' of this fill will be dependent on the workmanship employed by the contractor and, if placed in 200 to 300 mm thick lifts compacted to 100% of the standard Proctor maximum dry density in accordance with the requirements of OPSS 902 and OPSS 501 (Method A), should be some 15 to 20 mm. Within the widened section, settlement of the subsurface soils under the weight of newly placed fill is expected to be in the order of 40 mm, for a total settlement of 55 to 60 mm. The settlement of the approach fill surface near the abutments should be essentially complete within 3 to 4 months after placement of the fill.

Earth fill slopes where employed should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 803 or 804 for time constraints and the type of seed and mulch required.

## **6. EXCAVATION AND GROUNDWATER CONTROL**

It is expected that excavation for construction of spread footings founded on the hard / very dense till will extend through the fill and native soils to some 2 to 3 m depth at both abutments and the pier.

The typically firm to stiff / loose to compact soils are classified as Type 3 soils according to the Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes in



earth over the full depth of excavation should therefore be inclined at an angle of 45° to the horizontal. The need to excavate flatter sideslopes if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction should be considered.

The piezometric water level that was measured in borehole B-3 at 2.3 m (elevation 87.3) on June 18, 2014 and at 2.5 m (elevation 87.1) on July 9, 2014 characterized a perched groundwater condition at the north abutment location. It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the foundation excavations. Groundwater levels are subject to seasonal fluctuations and precipitation patterns.

All work should be carried out in accordance with the Occupational Health and Safety Act (Ontario Regulation 213/91) and with local/MTO regulations.

## **7. CONSTRUCTION STAGING AND ROADWAY PROTECTION**

The current plans call for the centreline of the proposed underpass to be some 3.5 m to the west of the existing centreline. After one half of the new structure is constructed immediately west of the existing underpass, traffic will be diverted to it and the existing structure will be demolished. The east half of the new underpass will then be constructed.

It is anticipated that a suitable roadway protection scheme following OPSS 539 will be necessary behind the abutments to support the walls of excavation and adjacent traffic lanes during staged construction.

Several alternative protection schemes such as sheet piling, sheeting supported by rakers or bracing, cantilever or anchored soldier piles and lagging may be considered. It is noted, however, that a soldier pile and lagging scheme is not considered adequate where the excavation will extend through sandy soils, particularly below the water table due to potentially excessive loss of the retained soils during installation. A road protection scheme designed for an OPSS 539 performance level 2 system is recommended to prevent excessive movement of the existing embankment. The contractor is responsible for the selection, preparation and performance of a detailed design for the roadway protection system.

## 8. CLOSURE

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

Yours very truly,

Peto MacCallum Ltd.



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Senior Foundation Engineer



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GD/CN/BRG:gd-nk





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

<b>DOCUMENT</b>	<b>TITLE</b>
OPSS 405	Construction Specification for Pipe Subdrains
OPSS 501	Construction Specification for Compacting
OPSS 539	Construction Specification For Temporary Protection Systems
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 Piling
SP 206S03	Construction Specification for Grading
OPSD 201.020	Rock Grading-Divided Rural
OPSD 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments
OPSD 3101.200	Walls, Abutment, Backfill Rock
OPSD 3190.100	Retaining Wall and Abutment Wall Drain Detail



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

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

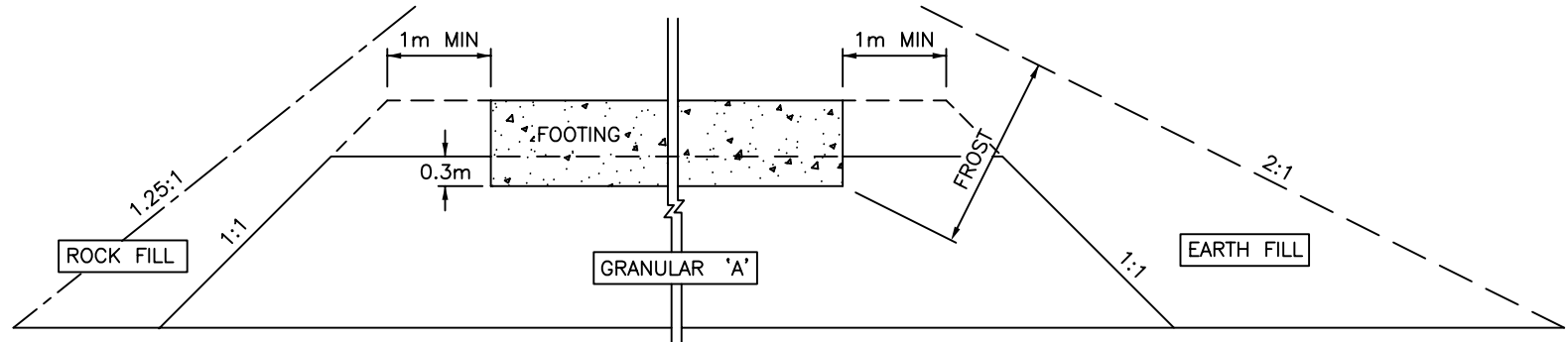
**Note:** From MTO Report S0-96-01, Revision 1 – July 1996.



**TABLE 3**  
**REDUCTION FACTORS DUE TO CAISSON SPACING**

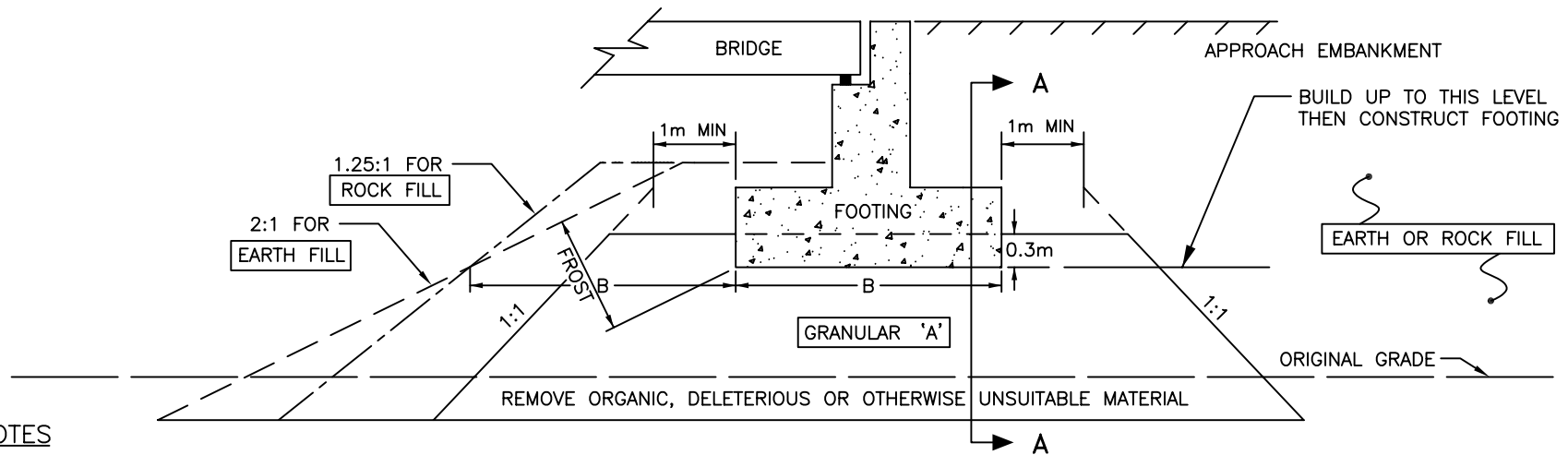
CAISSON SPACING PERPENDICULAR TO THE LOAD	SUBGRADE REACTION REDUCTION FACTOR
$\geq 4D$	1.00
3D	0.83
2D	0.67
1D	0.50

- Notes:**
- 1) D is caisson diameter
  - 2) Spacing is considered the distance from centre to centre



**CROSS SECTION A-A**

NOT TO SCALE



**LONGITUDINAL SECTION**

NOT TO SCALE

**NOTES**

1. CONCEPT SHOWN DOES NOT INCLUDE A MIDHEIGHT BERM.
2. LIMITS OF GRANULAR 'A' CORE TO BE DEFINED BY A SITE SPECIFIC SURVEY.
3. REMOVE ORGANIC, DELETERIOUS OR OTHERWISE UNSUITABLE MATERIAL UNDER AREA OF COMPACTED GRANULAR 'A' AND EARTH OR ROCK FILL AS NOTED IN TEXT OF REPORT.
4. PLACE GRANULAR 'A' AND EARTH OR ROCK FILL ON APPROVED SUBGRADE TO BOTTOM OF FOOTING LEVEL, COMPACTED ACCORDING TO CURRENT M.T.O. STANDARDS.
5. CONSTRUCT CONCRETE FOOTING.
6. PLACE REMAINDER OF GRANULAR 'A' AND EARTH OR ROCK FILL INCLUDING MIDHEIGHT BENCHES, AS REQUIRED.
7. REFER TO TEXT OF REPORT FOR FROST DEPTH.

**FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR 'A' CORE**



## **APPENDIX A**

### Non-Standard Special Provisions (NSSP)



## **NON-STANDARD SPECIAL PROVISIONS (NSSP)**

### **NSSP – Potential for Cobbles and Boulders During Pile Driving (Addition to OPSS 903)**

The Contractor shall be advised that cobbles and boulders were identified within the glacial till at the site and should be considered. The contractor shall ensure that more comprehensive engineering supervision is required than is called for in OPSS 903.

If during pile driving there is evidence that a pile meets refusal on a cobble or boulder, the contractor shall inform the Contract Administrator (CA). The contractor shall be advised that piles meeting refusal on a cobble or boulder may need to be relocated.

### **NSSP– Vibration Monitoring During Pile Driving**

In addition to the requirements in OPSS 903, the Contractor shall monitor vibration at two (2) points on each abutment of the travelled portions of the bridge and two (2) on central pier, during pile driving operations as specified below.

Vibration monitoring shall be carried out on a daily basis, from 2 days prior to the commencement of caisson installation and pile driving operations to establish a baseline. The Contractor shall document at the completion of each day during the construction operations, a cumulative tabular report indicating the current and previous readings to date of the vibration measurements. If the vibration measurements reach the review level of 50 mm/sec, the operation should be reviewed with the CA. Should the vibration measurement exceeds the alarm level of 100 mm/sec, operations shall be stopped immediately and the issue shall be immediately referred to the CA and design team for further evaluation.

### **NSSP – Temporary Roadway Protection (Addition to OPSS 539)**

The possibility of existing granular fill migrating through the temporary roadway protection and difficulties associated with the presence of cobbles and boulders within the glacial till revealed at the site should be considered by the contractor during the selection and installation of the temporary protection system.



#### NSSP – Settlement Monitoring During Structure Removal and Caisson Installation

In addition to the requirements in OPSS 903, settlement monitoring shall be carried out at two (2) points on each abutment and at the center pier of the travelled portions of the bridge during removal operations and during installation of Caissons. Two (2) points shall also be monitored for settlement at each foundation element during the removal operations as specified below.

The settlement monitoring shall consist of survey monitoring with minimum measurement precision of 1 mm.

Settlement monitoring shall be carried out on a daily basis, from 2 days prior to the commencement of removal operations or caisson installation (to establish a baseline) to the completion of the operations. The Contractor shall compile, at the completion of each day during the specified period, a cumulative tabular report indicating the current and previous readings to date of the total amount of settlement change from the base line readings. If the total amount of cumulative settlement change exceeds the review level of 7 mm, the operation should be reviewed with the CA. If the cumulative amount of settlement exceeds the alarm level of 10 mm, the operations shall be stopped immediately and the issue shall be immediately referred to the CA and design team for further instructions.