FOUNDATION INVESTIGATION AND DESIGN REPORT

for

HIGHWAY 401 / HENRY STREET UNDERPASS
SITE 22-152, W.O. 09-20009
WHITBY, ONTARIO

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PML Ref.: 10TF008A-H
Index No.: 014FIR and 015FDR
GEOCRES No.: 30M15-199
September 5, 2014

Peto MacCallum Ltd.
CONSULTING ENGINEERS

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for
Highway 401 / Henry Street Underpass
Site 22-152, 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 Henry Street traffic over Highway 401 in Whitby, Ontario. The project is administered as a Design Build. 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+415, Highway 401 chainage, in Whitby (ref. General Arrangement Drawing 1 'Highway 401 / Henry Street Underpass' prepared by AECOM in September 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 site is situated north of Lake Ontario between Salem Road and Brock Street in the Town of Whitby. Highway 401 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 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,

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BARRIE, HAMILTON, KITCHENER, TORONTO

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 topography at the site is irregular in detail, with 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.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out during the period of April 25 to May 7, 2014 and comprised 8 boreholes drilled to 6.5 to 18.2 m at the locations shown on Drawing H-1, attached. It is noted that the choice of locations for some boreholes was affected by accessibility issues and the presence of utilities. The criterion of 100 blows per 0.3 m penetration was met for all the boreholes, with one borehole cored into bedrock to give information for possible construction of deep foundations (caissons) with embedment into the bedrock at the pier location due to construction constraints at the ground surface. Further details are summarised in the following table:

LOCATION	BOREHOLE No.	DEPTH (m)		
		AUGER / CONE	ROCK CORE	TOTAL
North Approach	H-1	6.5	–	6.5
North Abutment	H-2	7.8	–	7.8
	H-3	9.3	–	9.3
Pier	H-4	12.4	–	12.4
	H-5	13.2	5.0	18.2
South Abutment	H-6	10.8	–	10.8
	H-7	9.5	–	9.5
South Approach	H-8	9.2	–	9.2

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. One borehole was extended 5.0 m into bedrock using NQ diamond rock coring equipment supplemented by wash boring techniques.

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. 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, cave-in occurred in boreholes H-3, H-6 and H-8 at respective depths of 8.5, 9.7 and 6.7 m (elevation 83.0, 80.7 and 85.6). The boreholes were backfilled with bentonite/cement grout in accordance with the MTO guidelines and MOE Regulation 903 for borehole abandonment procedures.

Two piezometers consisting of 19 and 50 mm PVC pipes slotted over the bottom 3.0 m were installed to monitor groundwater conditions – one in borehole H-3 and another in borehole H-6. The annular space around the pipe was backfilled with auger cuttings and a bentonite seal placed as illustrated on the respective borehole logs. The water level in the piezometers was measured on June 18 and July 9, 2014.

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, 5 Atterberg limits tests and 16 grain size distribution analyses were carried out on selected soil samples, with the results presented in Figures H-PC-1 to H-PC-3 and Figures H-GS-1 to H-GS-6 respectively as well as on the corresponding Record of Borehole sheets.

Two rock core specimens from borehole H-5 were tested for unconfined compressive strength, with the results summarised in section 4.5 of this report and shown on the borehole log.

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 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 H-1 and H-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 surficial fill overlying clayey silt till or silty clay till underlain by cohesionless till mantling bedrock. Cobbles were encountered in 4 boreholes. Boulders are also likely to be encountered in the glacial till (borehole H-2 was terminated on a probable boulder). The bedrock surface was contacted at 11.9 and 13.2 m (elevation 75.7 and 74.4) at the pier location. The piezometric water level was at 3.1 and 3.8 m (elevation 88.4 and 86.6) on June 18, 2014 and at 3.4 and 4.6 m (elevation 88.1 and 85.8) on July 9, 2014.

The strata encountered are summarised below.

4.1 Fill

Pavement structure consisting of 125 to 225 mm thick asphalt and 200 to 500 mm thick sand and gravel was present in boreholes H-2, H-4, H-5 and H-7. The sand and gravel was present at the surface in borehole H-1 as well. The granular material was loose to compact in relative density and 5 to 9% in moisture content.

Clayey silt and/or silty clay fill was present surficially in boreholes H-3, H-6, H-8 and under the pavement structure at 0.3 to 0.5 m (elevation 91.7 to 92.7) in boreholes H-1, H-2, H-7. Soft to stiff in consistency and 9 to 31% in moisture content, the cohesive fill had a thickness of 0.8 to 3.2 m and was penetrated at 0.8 to 3.7 m (elevation 89.3 to 91.0).

Directly beneath the sand and gravel fill at 0.6 m (elevation 87.0) in borehole H-5 was sand fill. This unit was compact to dense and had a moisture content of about 9%. The sand fill was 600 mm in thickness and penetrated at 1.2 m (elevation 86.4).

The results of Atterberg limits testing and grain size distribution analyses performed on 2 cohesive samples of the fill are presented in respective Figures H-PC-1 and H-GS-1. The silty clay fill had a liquid limit of 43 and 49, plastic limit of 23, with the plasticity index of 20 and 26.

4.2 Silty Clay

A layer of silty clay was encountered at 1.6 m (elevation 91.0) in borehole H-2. This layer was firm in consistency, with a moisture content of 12%. The silty clay was 300 mm thick and penetrated at 1.9 m (elevation 90.7).

4.3 Clayey Silt Till / Silty Clay Till

Overlain by the fill or silty clay at 0.8 to 3.0 m (elevation 89.3 to 90.7) in boreholes H-1 to H-3, H-6 and H-8 was a deposit of clayey silt till or silty clay till. Stiff to hard in consistency, this deposit had a thickness of 0.8 to 1.7 m and was penetrated at 1.8 to 4.7 m (elevation 87.6 to 89.5). It is noteworthy that cobbles were contained in the clayey silt till in boreholes H-2 and H-8, though cobbles and boulders should be expected within the deposit at the site.

The results of Atterberg limits testing and grain size distribution analyses conducted on 2 samples of the deposit are presented in respective Figures H-PC-2 and H-GS-2. The clayey silt till had a liquid limit of 21, plastic limit of 12 and 13, its plasticity index being 8 and 9. The moisture content of the deposit varied between 9 and 29%.

4.4 Sandy Silt Till / Sand and Silt Till / Silty Sand Till / Sand Till

Underlying the fill or clayey silt till / silty clay till at 0.7 to 4.7 m (elevation 86.4 to 89.5) in all the boreholes was cohesionless till of variable granulometric composition (sandy silt, sand and silt, silty sand, sand). This stratum was compact to very dense (SPT-‘N’ values of 11 to 115), its moisture content ranging from 6 to 19%. The cohesionless till had a thickness of 11.2 m in borehole H-4 and 12.0 m in borehole H-5 and was penetrated at respective 11.9 and 13.2 m (elevation 75.7 and 74.4). The remaining boreholes were terminated within the stratum at 6.5 to 10.8 m (elevation 79.6 to 85.5). It is worth noting that cobbles were encountered in the sand and silt till / sandy silt till in boreholes H-3 and H-6. It is also noted that borehole H-2 was terminated at 7.8 m (elevation 84.8) on a possible boulder. Cobbles and boulders should be expected in the cohesionless till across the site.

The results of Atterberg limits testing and grain size distribution analyses performed on 12 samples of the stratum are presented in respective Figures H-PC-3 and H-GS-3 to H-GS-6. The liquid and plastic limits of the silty sand till were 13 and 11 respectively, thus giving the plasticity index of 2.

4.5 Bedrock

Bedrock was contacted in boreholes H-4 and H-5 at respective 11.9 and 13.2 m (elevation 75.7 and 74.4). The unweathered bedrock comprises a dark grey to light black shale with interbedded limestone.

The measured core recovery typically varied between 90 and 99%. The RQD determined from the rock cores was in a range of 74 to 88%, thus indicating a fair to good quality rock. The upper 0.4 m core sample in borehole H-5 had poor rock quality (RQD of 44%) and a measured core recovery of 66%.

The unconfined compressive strength determined on two rock core specimens from borehole H-5 was 40.1 and 62.8 MPa.

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

4.6 Groundwater

In the process of augering, water was detected at 2.1 to 7.6 m (elevation 83.9 to 87.3) in boreholes H-1, H-3 to H-6 and H-8. Upon completion of drilling, groundwater was measured in boreholes H-1, H-3, H-4, H-6 and H-8 to be at 4.6 to 7.9 m (elevation 82.5 to 87.1). No water was observed in boreholes H-2 and H-7 during or upon completion of drilling.

The piezometric water level measured in boreholes H-3 and H-6 was at 3.1 and 3.8 m (elevation 88.4 and 86.6) on June 18, 2014 and at 3.4 and 4.6 m (elevation 88.1 and 85.8) on July 9, 2014, respectively. 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., 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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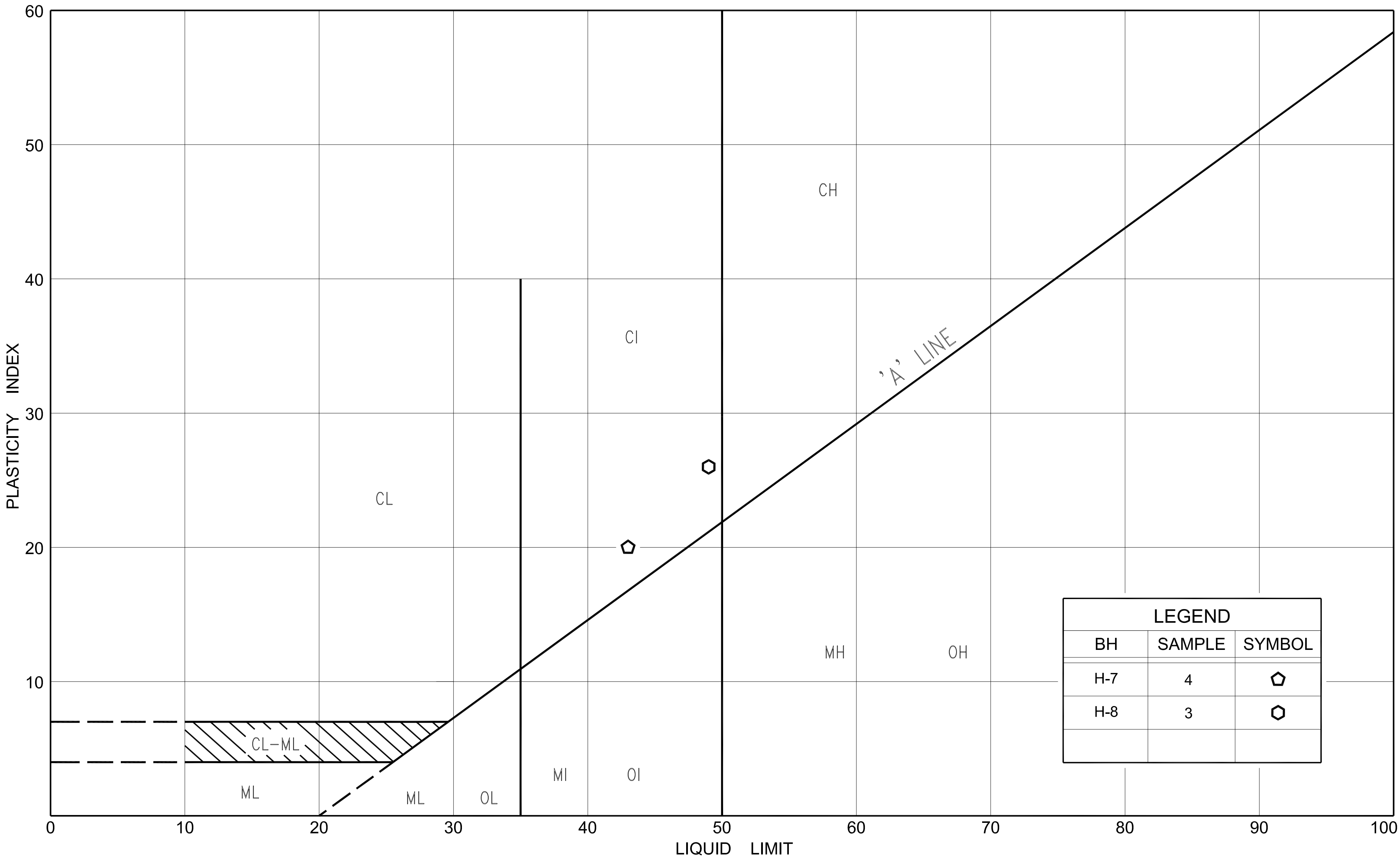
GD/CN/BRG:gd-mi

TABLE A
ROCK CORE DESCRIPTIONS

CORE RECOVERY					CORE DESCRIPTION	
HOLE NO.	**CORE NO.**	**DEPTH (m)**	**RECOVERY (%)**	**RQD (%)**	**DEPTH (m)**	**DESCRIPTION**
H-5	7 8 9 10	13.2 – 13.6 13.6 – 15.1 15.1 – 16.7 16.7 – 18.2	66 93 90 99	44 88 74 86	13.2 – 18.2	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, unweathered, close spaced flat partings, smooth planar, tight, poor becoming fair to good quality.

NOTE: RQD = Rock Quality Designation

Originated: SA
Compiled: JO
Checked: SAT

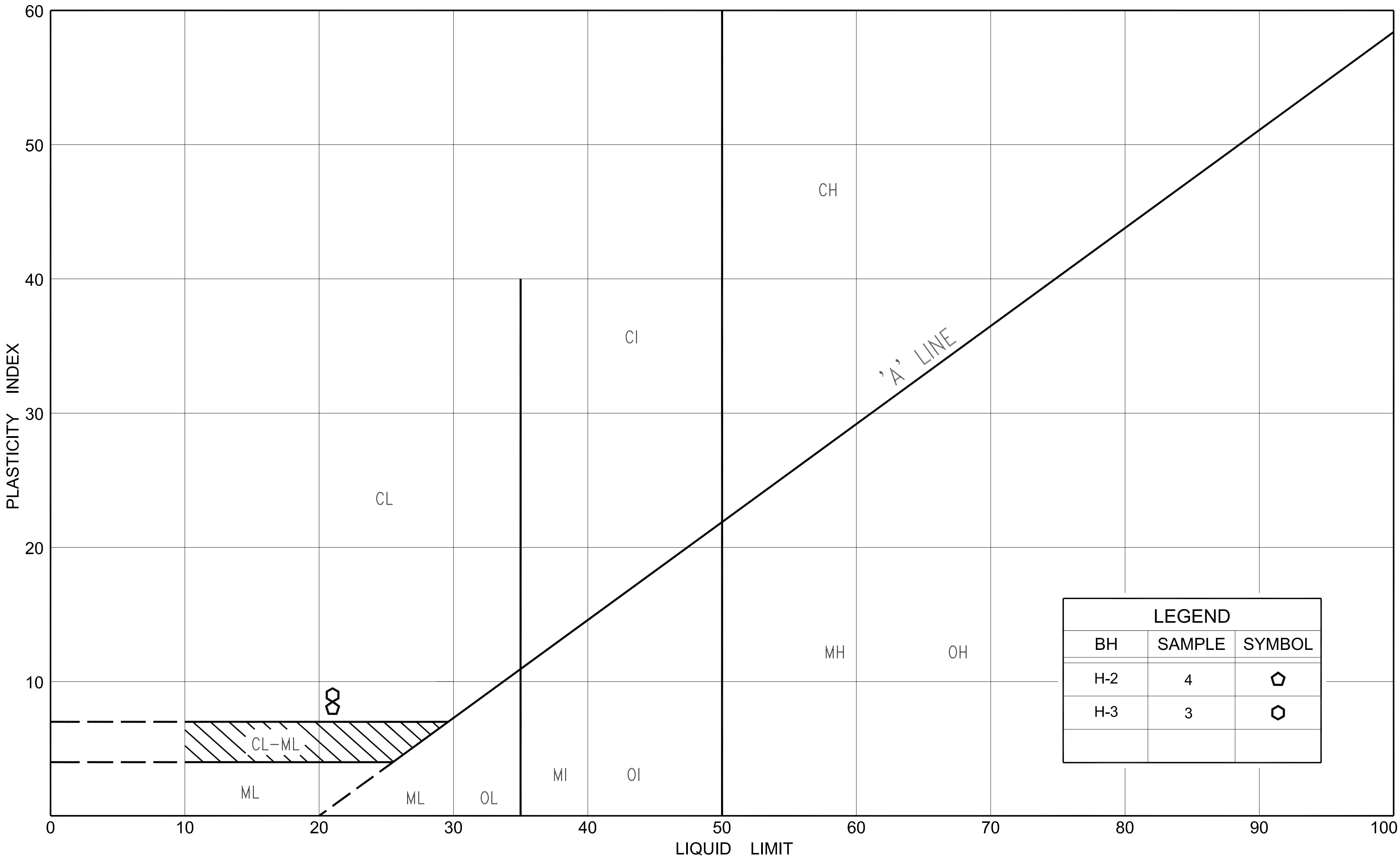


Ontario

PLASTICITY CHART

SILTY CLAY, some to with sand, trace gravel
(FILL)

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



LEGEND

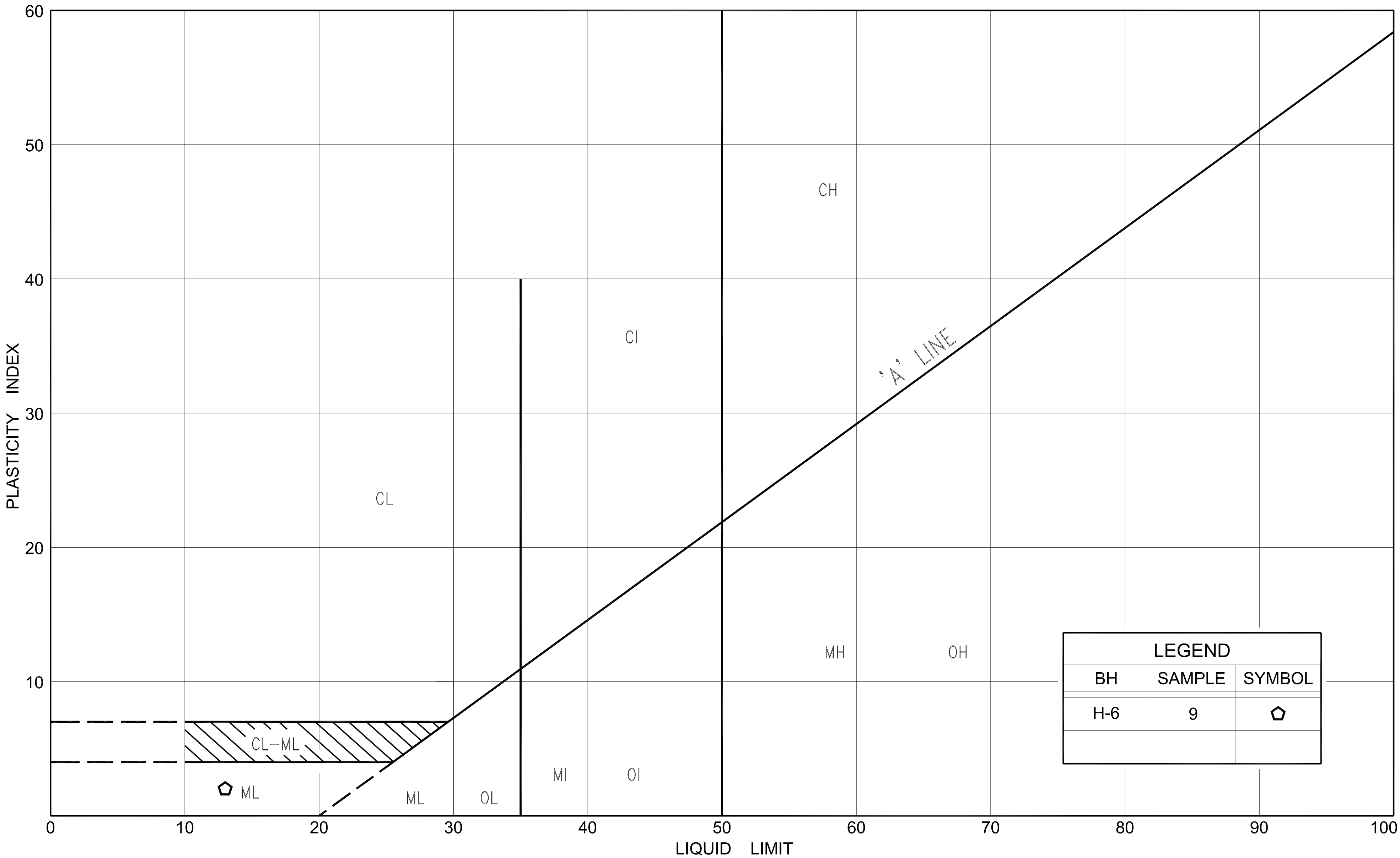
BH	SAMPLE	SYMBOL
H-2	4	⬠
H-3	3	⬡

Ontario

PLASTICITY CHART

CLAYEY SILT, with sand to sandy, trace gravel
(TILL)

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



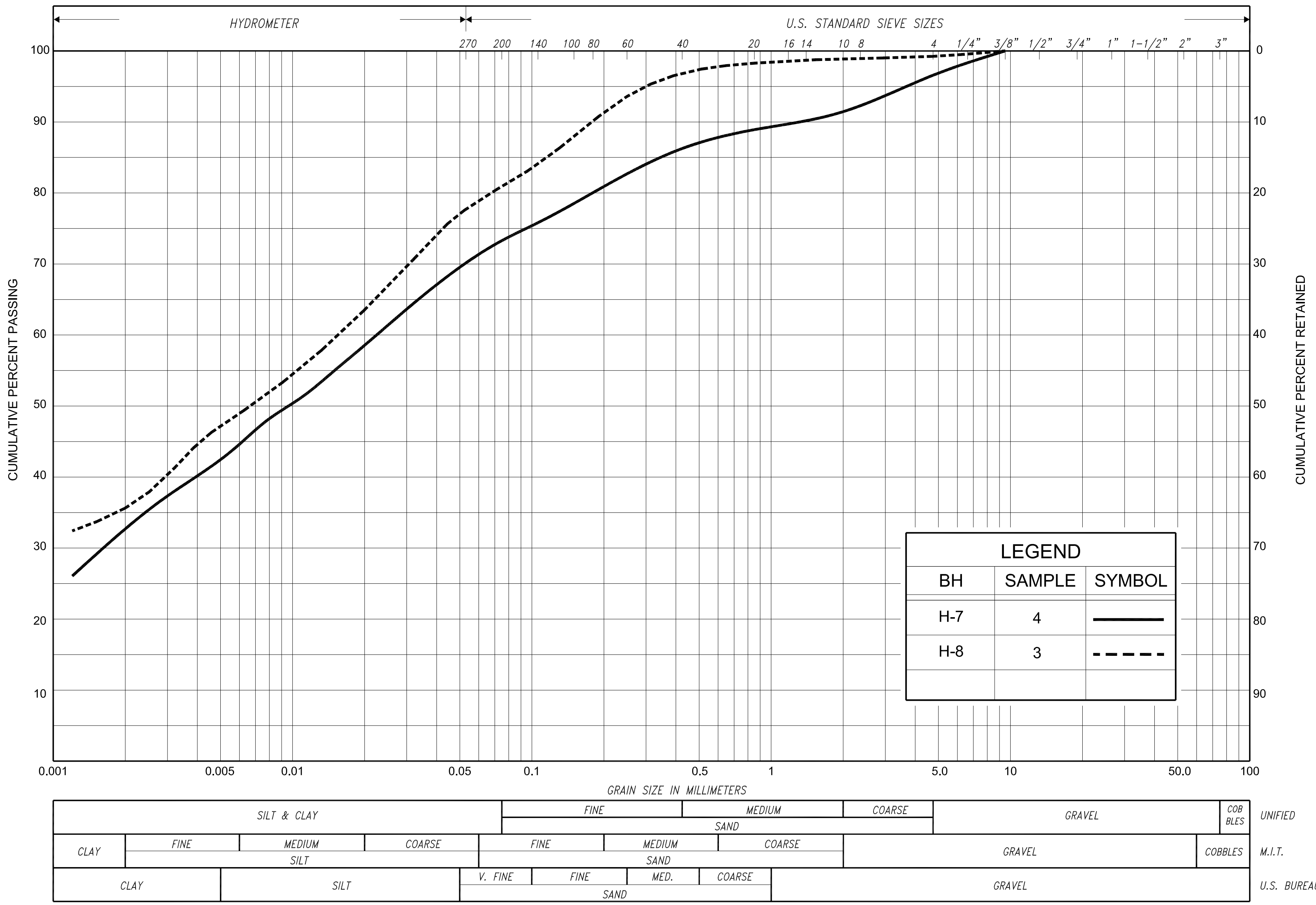
LEGEND

BH	SAMPLE	SYMBOL
H-6	9	⬠

PLASTICITY CHART

SILTY SAND, trace clay, trace gravel
(TILL)

FIG No.	H-PC-3
HWY:	401
P.O. No.	09-20009

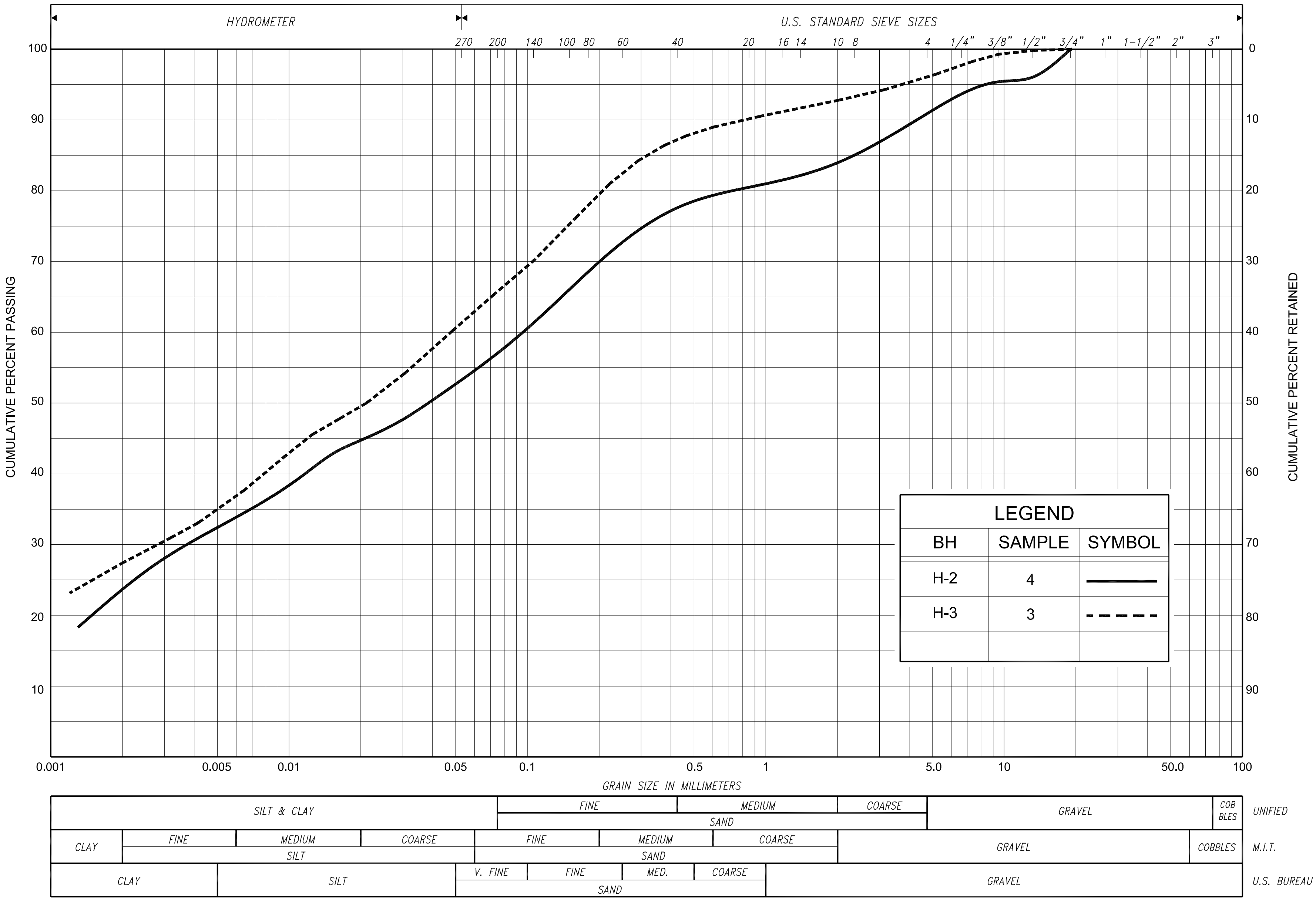


Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY, some to with sand, trace gravel
(FILL)

FIG No.	H-GS-1
HWY:	401
P.O. No.	09-20009

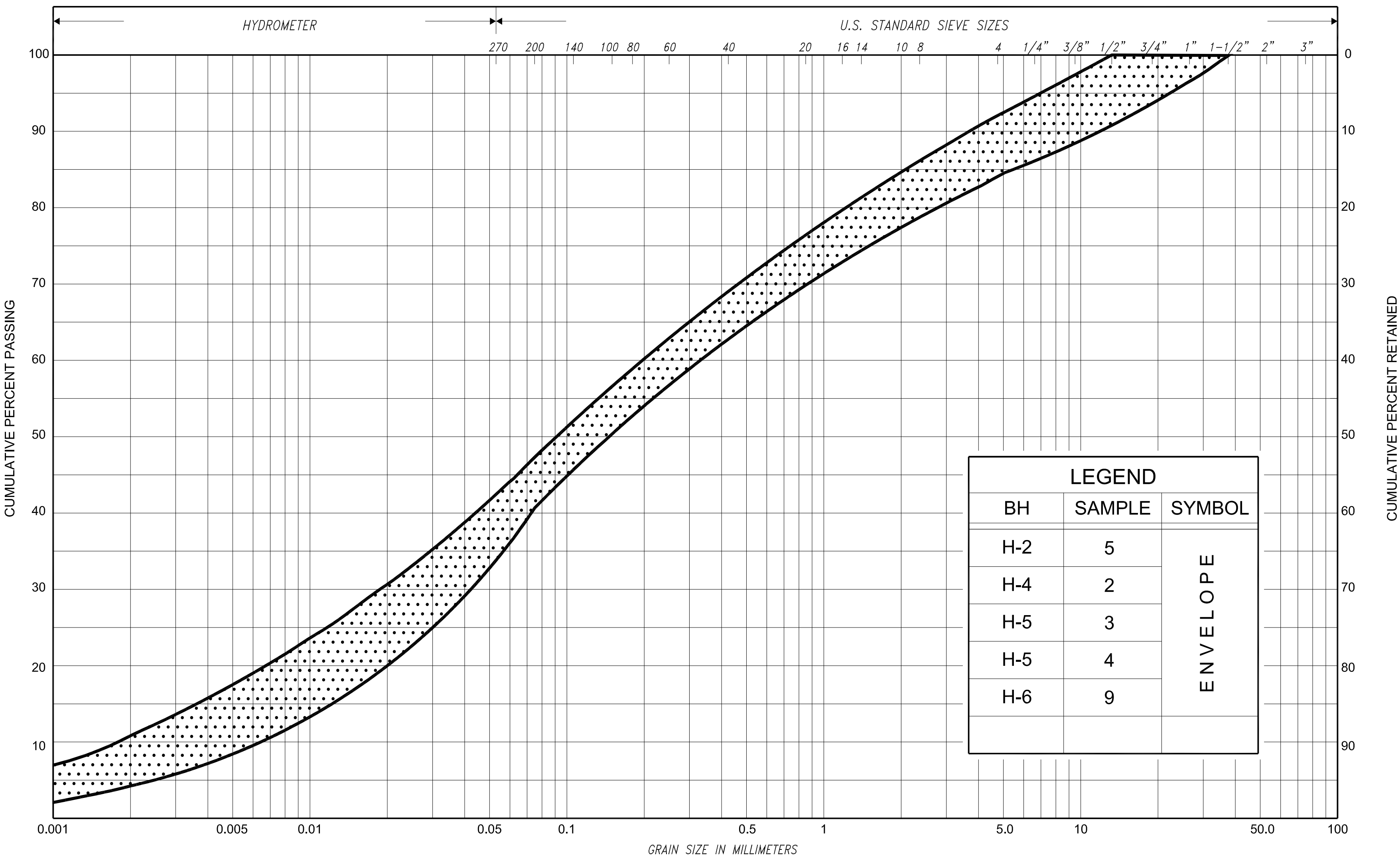


Ontario

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, with sand to sandy, trace gravel
(TILL)

FIG No.	H-GS-2
HWY:	401
P.O. No.	09-20009



LEGEND

BH	SAMPLE	SYMBOL
H-2	5	ENVELOPE
H-4	2	
H-5	3	
H-5	4	
H-6	9	

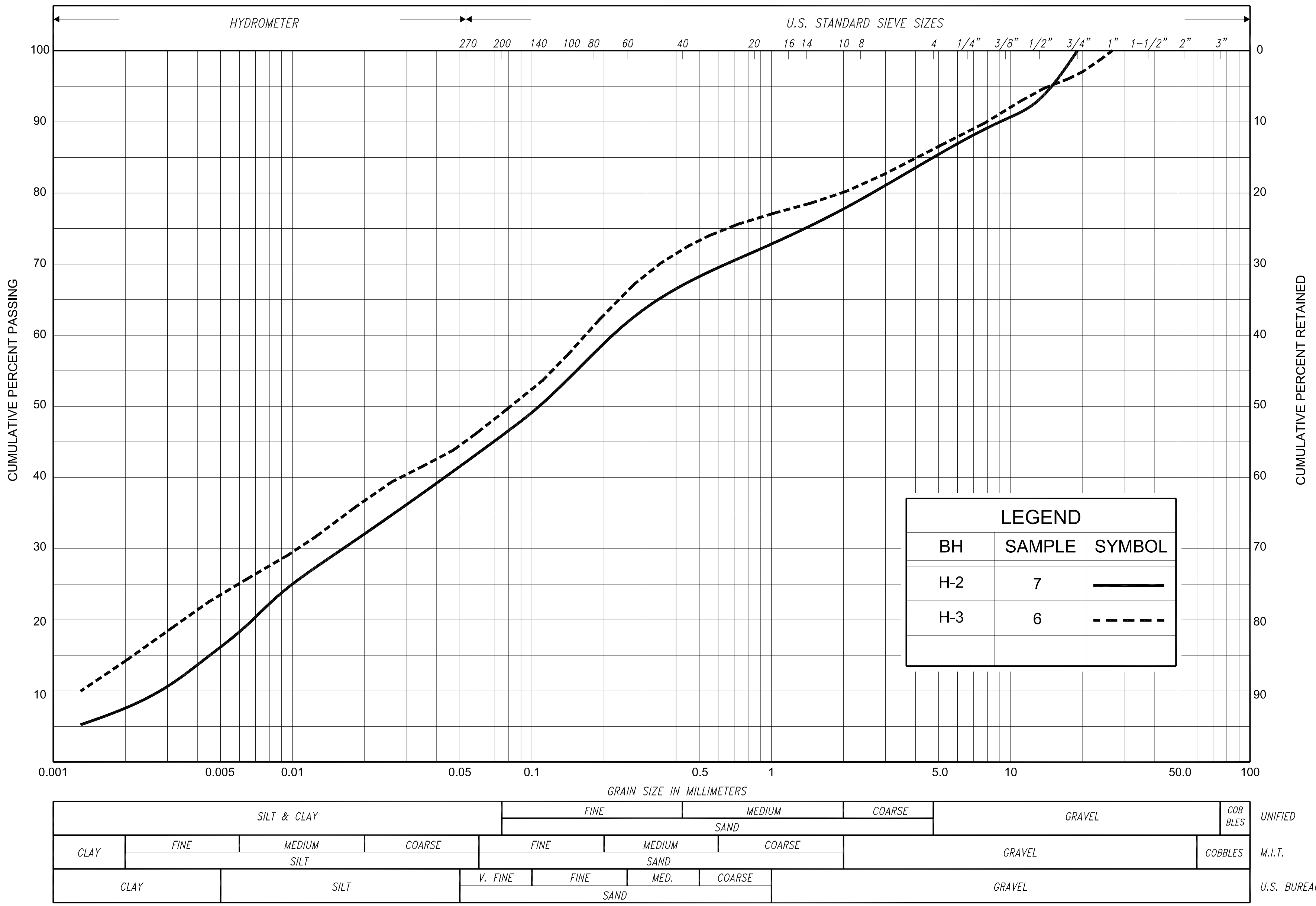
SILT & CLAY			SAND: FINE	SAND: MEDIUM	SAND: COARSE	GRAVEL	COBBLES	UNIFIED
CLAY	SILT: FINE	SILT: MEDIUM	SILT: COARSE	SAND: FINE	SAND: MEDIUM	SAND: COARSE	GRAVEL	COBBLES	M.I.T.
CLAY	SILT	SAND: V. FINE	SAND: FINE	SAND: MED.	SAND: COARSE	GRAVEL	U.S. BUREAU

Ontario

GRAIN SIZE DISTRIBUTION

SILTY SAND, trace to some clay, trace to some gravel (TILL)

FIG No.	H-GS-3
HWY:	401
P.O. No.	09-20009

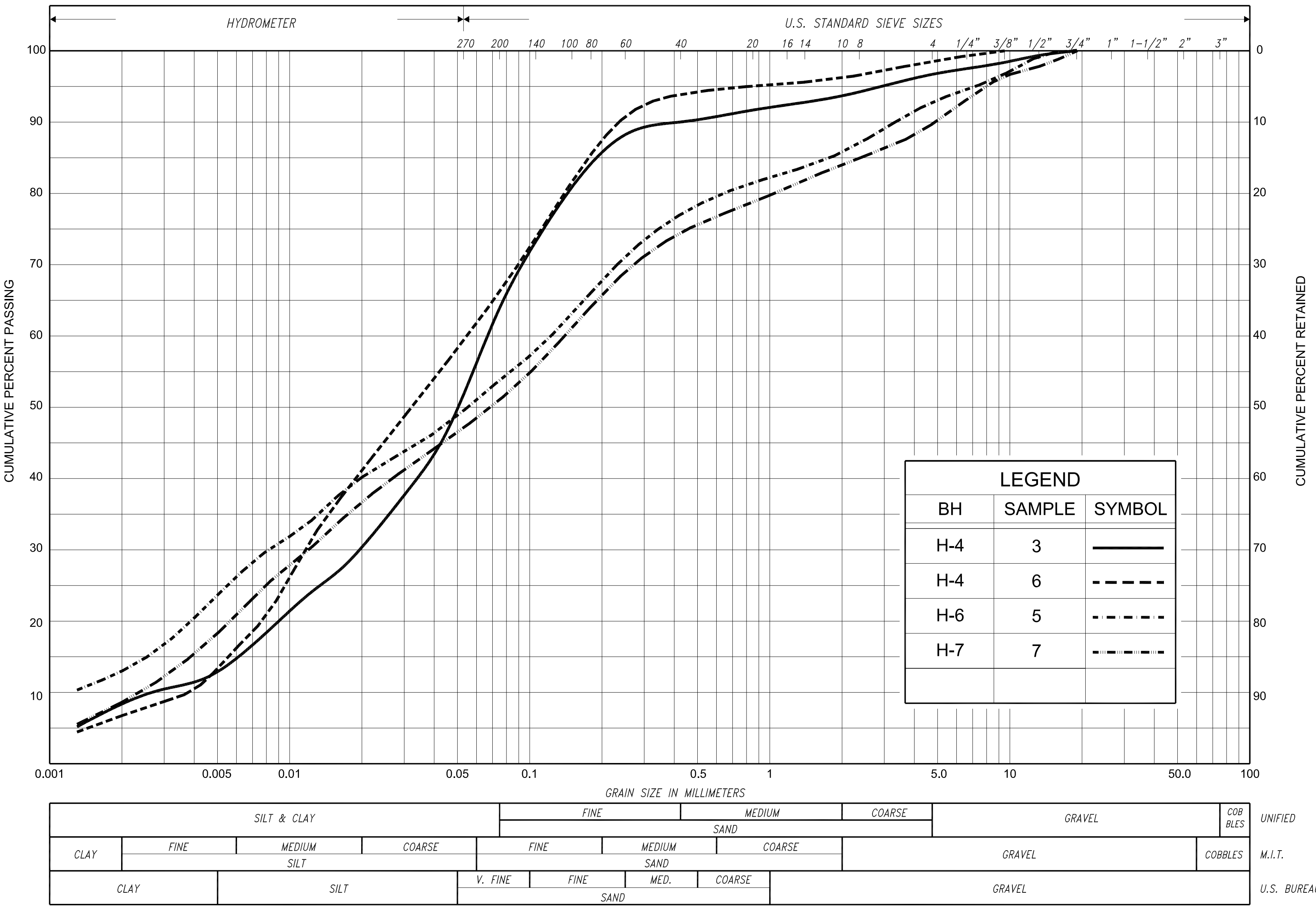


Ontario

GRAIN SIZE DISTRIBUTION

SAND AND SILT, some gravel, trace to some clay
(TILL)

FIG No.	H-GS-4
HWY:	401
P.O. No.	09-20009

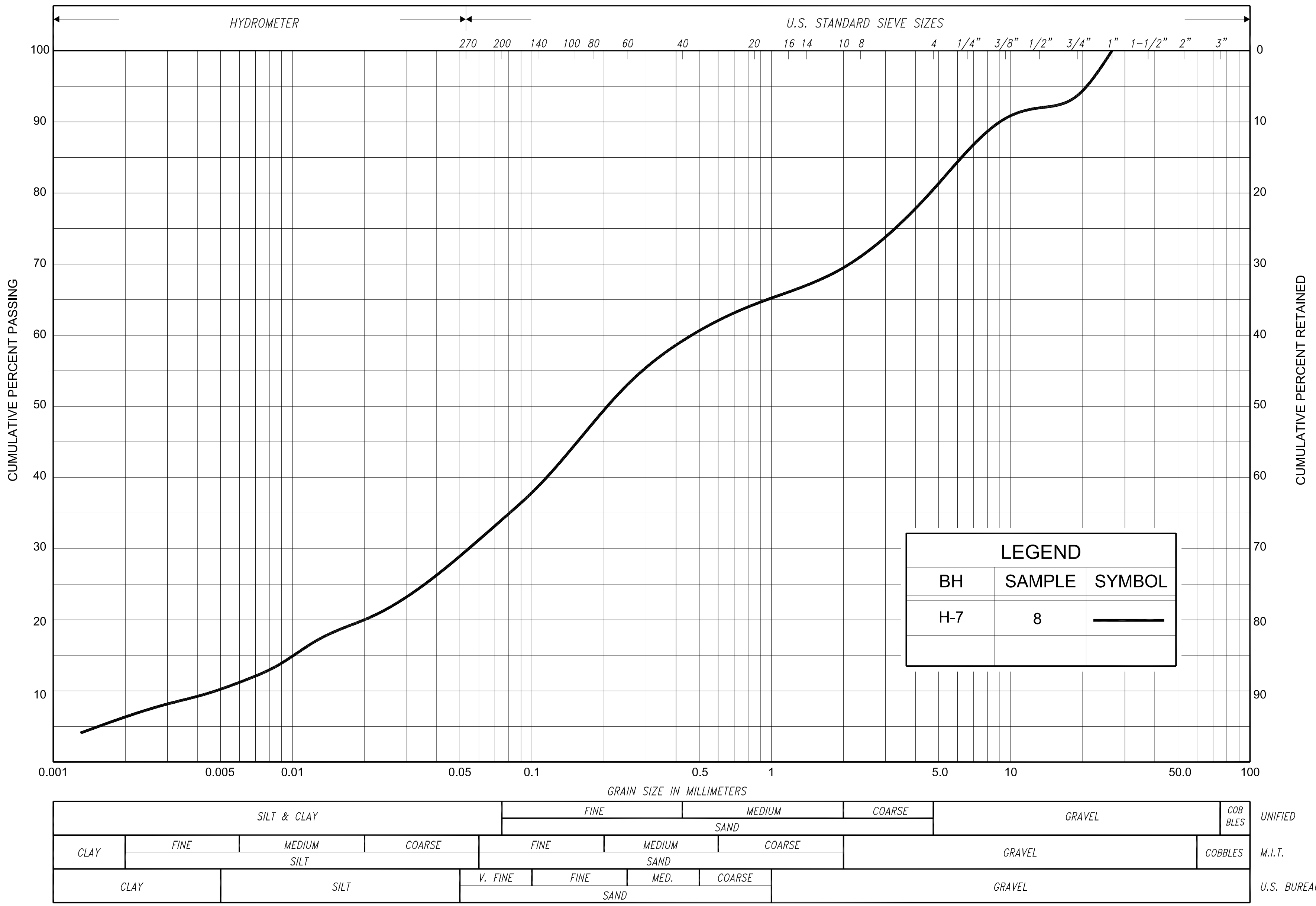


Ontario

GRAIN SIZE DISTRIBUTION

SANDY SILT, trace to some clay, trace to some gravel
(TILL)

FIG No.	H-GS-5
HWY:	401
P.O. No.	09-20009



Ontario

GRAIN SIZE DISTRIBUTION

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

FIG No.	H-GS-6
HWY:	401
P.O. No.	09-20009

EXPLANATION OF TERMS USED IN REPORT

N VALUE: THE STANDARD PENETRATION TEST (SPT) N VALUE IS THE NUMBER OF BLOWS REQUIRED TO CAUSE A STANDARD 51mm O.D. SPLIT BARREL SAMPLER TO PENETRATE 0.3m INTO UNDISTURBED GROUND IN A BOREHOLE WHEN DRIVEN BY A HAMMER WITH A MASS OF 63.5kg, FALLING FREELY A DISTANCE OF 0.76m. FOR PENETRATIONS OF LESS THAN 0.3m N VALUES ARE INDICATED AS THE NUMBER OF BLOWS FOR THE PENETRATION ACHIEVED. AVERAGE N VALUE IS DENOTED THUS $\bar{N}$.

DYNAMIC CONE PENETRATION TEST: CONTINUOUS PENETRATION OF A CONICAL STEEL POINT (51mm O.D. 60° CONE ANGLE) DRIVEN BY 475 J IMPACT ENERGY ON 'A' SIZE DRILL RODS. THE RESISTANCE TO CONE PENETRATION IS MEASURED AS THE NUMBER OF BLOWS FOR EACH 0.3m ADVANCE OF THE CONICAL POINT INTO THE UNDISTURBED GROUND.

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

S S	SPLIT SPOON	T P	THINWALL PISTON
W S	WASH SAMPLE	O S	OSTERBERG SAMPLE
S T	SLOTTED TUBE SAMPLE	R C	ROCK CORE
B S	BLOCK SAMPLE	P H	T W ADVANCED HYDRAULICALLY
C S	CHUNK SAMPLE	P M	T W ADVANCED MANUALLY
T W	THINWALL OPEN	F S	FOIL SAMPLE
F V	FIELD VANE		

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_u	1	PORE PRESSURE RATIO
σ	kPa	TOTAL NORMAL STRESS
σ'	kPa	EFFECTIVE NORMAL STRESS
τ	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
ϵ	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
E	kPa	MODULUS OF LINEAR DEFORMATION
G	kPa	MODULUS OF SHEAR DEFORMATION
μ	1	COEFFICIENT OF FRICTION

MECHANICAL PROPERTIES OF SOIL

m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
τ_f	kPa	SHEAR STRENGTH
c'	kPa	EFFECTIVE COHESION INTERCEPT
ϕ'	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
c_u	kPa	APPARENT COHESION INTERCEPT
ϕ_u	-°	APPARENT ANGLE OF INTERNAL FRICTION
τ_R	kPa	RESIDUAL SHEAR STRENGTH
τ_r	kPa	REMOULDED SHEAR STRENGTH
S_t	1	SENSITIVITY $= \frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m^3	DENSITY OF SOLID PARTICLES
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES
ρ_w	kg/m^3	DENSITY OF WATER
γ_w	kN/m^3	UNIT WEIGHT OF WATER
ρ	kg/m^3	DENSITY OF SOIL
γ	kN/m^3	UNIT WEIGHT OF SOIL
ρ_d	kg/m^3	DENSITY OF DRY SOIL
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL
e	1,%	VOID RATIO

n	1,%	POROSITY
w	1,%	WATER CONTENT
S_r	%	DEGREE OF SATURATION
w_L	%	LIQUID LIMIT
w_P	%	PLASTIC LIMIT
w_S	%	SHRINKAGE LIMIT
I_P	%	PLASTICITY INDEX $= w_L - w_P$
I_L	1	LIQUIDITY INDEX $= \frac{w - w_P}{I_P}$
I_C	1	CONSISTENCY INDEX $= \frac{w_L - w}{I_P}$
DTPL		DRIER THAN PLASTIC LIMIT
APL		ABOUT PLASTIC LIMIT
WTPL		WETTER THAN PLASTIC LIMIT

e_{max}	1,%	VOID RATIO IN LOOSEST STATE
e_{min}	1,%	VOID RATIO IN DENSEST STATE
I_D	1	DENSITY INDEX $= \frac{e_{max} - e}{e_{max} - e_{min}}$
D	mm	GRAIN DIAMETER
D_n	mm	n PERCENT - DIAMETER
C_u	1	UNIFORMITY COEFFICIENT
h	m	HYDRAULIC HEAD OR POTENTIAL
q	m^3/s	RATE OF DISCHARGE
v	m/s	DISCHARGE VELOCITY
i	1	HYDRAULIC GRADIENT
k	m/s	HYDRAULIC CONDUCTIVITY
j	kN/m^3	SEEPAGE FORCE

Ontario

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Foundation Design

RECORD OF BOREHOLE No. H-1

METRIC

G.W.P. 09-20009 **LOCATION** Co-ord: 4 858 717.8 N; 349 644.4 E **ORIGINATED BY** F.P.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** April 25, 2014 **CHECKED BY** G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	WATER CONTENT (%) (approx.)
92.0	Ground Surface						
0.0	Sand and gravel (PAVEMENT FILL)						
91.7 / 0.3	Clayey silt, trace sand, topsoil inclusions. Stiff Brown Moist to wet (FILL)	1	SS	12			
		2	SS	12		91	
90.6 / 1.4	Silty clay, trace sand, trace gravel. Very stiff Brown Moist (TILL)	3	SS	26		90	
		4	SS	30			
89.0 / 3.0	Sandy silt, trace clay, trace gravel. Very dense Brown Moist (TILL)	5	SS	104/23cm		89	
		6	SS	100/15cm		88	
		7	SS	112/25cm		87	
	seams of sand				▽* ▼*	86	
		8	SS	105/20cm			
85.5 / 6.5	End of borehole						

Column headings: SOIL PROFILE; STRAT PLOT; SAMPLES; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³; REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

* 2014 04 25

▽ Water level observed during drilling

▼ Water level measured after drilling

+7, ×5 : Numbers refer to Sensitivity

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

Ontario

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RECORD OF BOREHOLE No. H-2

METRIC

P.O. # 09-20009 **LOCATION** Co-ord: 4 858 703.1 N; 349 650.7 E **ORIGINATED BY** F.P.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** April 25, 2014 **CHECKED BY** G.D.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
92.6						
0.0	125mm asphalt over sand and gravel					
92.2 / 0.4	Loose Brown Moist (PAVEMENT FILL)	1	SS	8	92	
	Silty clay, trace sand, trace gravel, topsoil inclusions; Firm Brown Moist (FILL)	2	SS	7		
91.0 / 1.6	Silty clay, trace sand, trace gravel; Firm Brown Moist				91	
90.7 / 1.9	Clayey silt, sandy, trace gravel; Stiff to hard Brown Moist; cobbles (TILL)	3	SS	6		
		4	SS	39	90	9 34 33 24
89.5 / 3.1	Silty sand, some clay, some gravel; Very dense Brown Moist (TILL)	5	SS	54	89	16 43 30 11
88.6 / 4.0	Sand and silt, some gravel, trace clay; Very dense Grey Moist (TILL)	6	SS	100/13cm		
		7	SS	101/25cm	88	15 39 38 8
					87	
		8	SS	100/13cm	86	
		9	SS	100/8cm	85	
84.8 / 7.8	End of borehole. Refusal on probable boulder					

* Borehole dry

Column headings: SOIL PROFILE; SAMPLES; STRAT PLOT; GROUND WATER CONDITIONS *; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L, WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m³.

$+^7$, $\times^5$: Numbers refer to Sensitivity   20 / 15 –○– 5 / 10 (%) STRAIN AT FAILURE

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No. H-3

METRIC

G.W.P. 09-20009 **LOCATION** Co-ord: 4 858 698.6 N; 349 669.7 E **ORIGINATED BY** S.A.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** May 07, 2014 **CHECKED BY** G.D.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	WATER CONTENT (%)	REMARKS GR SA SI CL
91.5 / 0.0	Clayey silt, rootlets topsoil inclusions layers of sand. Stiff, Dark brown, Moist (FILL)	1	SS	9		
		2	SS	10		
90.1 / 1.4	Clayey silt with sand, trace gravel. Very stiff, Brown/grey, Moist (TILL)	3	SS	19		4 30 39 27
89.3 / 2.2	Sand and silt some clay, trace gravel cobbles. Compact to very dense, Brown, Moist (TILL)	4	SS	23		
		5	SS	50/10cm		
87.0 / 4.5	some gravel, Grey	6	SS	78/28cm		14 37 35 14
		7	SS	50/10cm		
84.5 / 7.0	trace clay, Wet	8	SS	50/8cm		
		9	SS	70/13cm		
82.2 / 9.3	End of borehole					

Elevation scale: 91, 90, 89, 88, 87, 86, 85, 84, 83

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

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

UNIT WEIGHT γ kN/m³

* 2014 05 07

▽ Water level observed during drilling

▼ Water level measured after drilling

NOTE: Piezometer was installed just south of borehole at location with the same ground surface elevation.

Piezometer Readings:

Date	Depth (m)	Elev.
06/18/2014	3.1	88.4
07/09/2014	3.4	88.1

Piezometer Legend:

- Bentonite seal
- Native
- Filter sand
- Screen

+7, ×5 : Numbers refer to Sensitivity

20, 15, 5, 10 (%) STRAIN AT FAILURE

Ontario

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RECORD OF BOREHOLE No. H-4

METRIC

P.O. # 09-20009 **LOCATION** Co-ord: 4 858 655.4 N; 349 660.0 E **ORIGINATED BY** F.P.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** April 28 & 29, 2014 **CHECKED BY** G.D.

SOIL PROFILE / SAMPLES / GROUND WATER CONDITIONS / ELEVATION SCALE / DYNAMIC CONE PENETRATION RESISTANCE PLOT (20 40 60 80 100) / SHEAR STRENGTH kPa (○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; 20 40 60 80 100) / PLASTIC LIMIT w_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT w_L / WATER CONTENT (%) (20 40 60) / UNIT WEIGHT γ kN/m³ / REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GR	SA	SI	CL
87.6 / 0.0	225mm asphalt over sand and gravel. Compact Brown Moist (PAVEMENT FILL)	1	SS	14				
86.9 / 0.7	Silty sand, some gravel, trace clay. Compact to very dense Grey Moist (TILL)	2	SS	101	13	46	34	7
86.2 / 1.4		3	SS	115	3	33	56	8
85.5 / 2.1	Sandy silt, trace clay, trace gravel. Very dense Grey Moist	4	SS	71/15cm				
84.9 / 2.7	shale fragments. Wet (TILL)	5	SS	59				
		6	SS	91	2	32	59	7
		7	SS	107/23cm				
		8	SS	73				
		9	SS	100/13cm				
		10	SS	103				
75.7 / 11.9	Shale bedrock	11	SS	100/10cm				
75.2 / 12.4	End of borehole							

Elevation scale: 87, 86, 85, 84, 83, 82, 81, 80, 79, 78, 77, 76

* 2014 04 28/29

▽ Water level observed during drilling

▼ Water level measured after drilling

+7, ×5 : Numbers refer to Sensitivity

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

Ontario

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Foundation Design

RECORD OF BOREHOLE No. H-5

METRIC

G.W.P. 09-20009 **LOCATION** Co-ord: 4 858 654.2 N; 349 682.4 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** May 02, 2014 **CHECKED BY** G.D.

ELEV DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
87.6	Ground Surface									
0.0	200mm asphalt over sand and gravel									
87.0 / 0.6	Compact Brown Moist to dense (PAVEMENT FILL)	1	SS	11		87		○		
		2	SS	89/23cm				○		
86.4 / 1.2	Sand, trace gravel (FILL)									
	Silty sand some gravel, trace clay	3	SS	78		86		○		11 46 39 4
	Very dense Grey Moist to wet				∇*					
	(TILL)	4	SS	103/23cm		85		○		12 45 37 6
		5	SS	103/18cm		84		○		
		6	SS	102/25cm		83		○		
						82				
						81				
						80				
						79				
						78				
						77				
						76				
						75				
74.4 / 13.2	Shale bedrock with interbedded limestone	7	RC	REC 66%		74				RQD 44%
	Unweathered									
	Soft to medium strength									
	Poor to good quality	8	RC	REC 93%		73	UCS=40.1 MPa	○	22.9	RQD 88%
	Cont'd									

Legend: PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE; scales 20 40 60 80 100.

+7, ×5: Numbers refer to Sensitivity   20 / 15–○–5 / 10 (%) STRAIN AT FAILURE

Ontario

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Foundation Design

RECORD OF BOREHOLE No. H-5

METRIC

G.W.P. 09-20009 **LOCATION** Co-ord: 4 858 654.2 N; 349 682.4 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** May 02, 2014 **CHECKED BY** G.D.

ELEV / DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT / SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
72.6	Shale bedrock with interbedded limestone Unweathered Soft to medium strength Fair to good quality (Cont'd.)		9	RC	REC 90%		72	UCS=62.8 MPa		24.2	RQD 74%
			10	RC	REC 99%		71 70				RQD 86%
69.4 18.2	End of borehole										

* 2014 05 02

▽ Water level observed during drilling

* Borehole charged with drilling water

UCS denotes Unconfined Compressive Strength

+7, ×5: Numbers refer to Sensitivity

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

Legend: DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT W, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60

Ontario

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RECORD OF BOREHOLE No. H-6

METRIC

G.W.P. 09-20009 **LOCATION** Co-ord: 4 858 614.9 N; 349 666.5 E **ORIGINATED BY** S.A.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** April 28, 2014 **CHECKED BY** G.D.

SOIL PROFILE

ELEV / DEPTH	DESCRIPTION
90.4	
0.0	Clayey silt, rootlets topsoil inclusions. Soft to firm, Dark brown, Moist (FILL)
89.6 / 0.8	Silty clay, trace gravel. Stiff, Grey, Moist (TILL)
88.6 / 1.8	Sandy silt, some clay trace gravel, cobbles. Compact to very dense, Brown/grey, Moist to wet (TILL)
81.9 / 8.5	Silty sand trace clay, trace gravel. Very dense, Grey, Moist to wet (TILL)
79.6 / 10.8	End of borehole

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	1	
2	SS	13	
3	SS	11	
4	SS	50/13cm	
5	SS	80	7 39 41 13
6	SS	50/13cm	
7	SS	50/13cm	
8	SS	50/8cm	
9	SS	70	8 45 38 9
10	SS	50/13cm	

GROUND WATER CONDITIONS

ELEVATION SCALE: 90, 89, 88, 87, 86, 85, 84, 83, 82, 81, 80

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

PLASTIC LIMIT W_P — NATURAL MOISTURE CONTENT W — LIQUID LIMIT W_L

WATER CONTENT (%): 20 40 60

UNIT WEIGHT γ kN/m³

* 2014 05 07

▽ Water level observed during drilling

▼ Water level measured after drilling

Cont'd

+7, ×5 : Numbers refer to Sensitivity

15—○—5 (20 above, 10 below) (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No. H-6

METRIC

G.W.P. 09-20009 **LOCATION** Co-ord: 4 858 614.9 N; 349 666.5 E **ORIGINATED BY** S.A.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** April 28, 2014 **CHECKED BY** G.D.

SOIL PROFILE			SAMPLES					DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE 20 40 60 80 100	PLASTIC LIMIT W_P NATURAL MOISTURE CONTENT W LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60	UNIT WEIGHT γ kN/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE				
75.4											

Piezometer Readings:

Date	Depth (m)	Elev.
06/18/2014	3.8	86.6
07/09/2014	4.6	85.8

Piezometer Legend:

- Bentonite seal
- Native
- Filter sand
- Screen

NOTE: Piezometer was installed 1m south of borehole at location with ground surface elevation about 0.3m higher than that of the borehole. The difference in elevation has been taken into account in determination of the piezometric water level.

+7 , ×5 : Numbers refer to Sensitivity

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

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RECORD OF BOREHOLE No. H-7

METRIC

P.O. # 09-20009 **LOCATION** Co-ord: 4 858 618.4 N; 349 683.0 E **ORIGINATED BY** F.P.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** April 25, 2014 **CHECKED BY** G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	WATER CONTENT (%) natural w (%)	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
93.2							
0.0	200mm asphalt over sand and gravel				93		
92.7 / 0.5	Compact to loose, Brown, Moist	1	SS	13		~2	
92.3 / 0.9	Clayey silt, trace sand organic inclusions; Firm to stiff, Dark brown, Moist	2	SS	7	92	~11	
	Silty clay, rootlets organic inclusions	3	SS	9		~15	
91.0 / 2.2	with sand, trace gravel (FILL)	4	SS	11	91	~5 (Wp ~12, WL ~22)	3 24 40 33
		5	SS	14	90	~13	
89.5 / 3.7	Sandy silt trace to some gravel trace clay; Compact to dense, Brown/grey, Moist (TILL)	6	SS	17	89	~5	
		7	SS	38	88	~5	11 38 43 8
87.4 / 5.8	Sand, with silt some gravel, trace clay; Very dense, Brown, Moist to wet; Grey (TILL)	8	SS	108/23cm	87	~8	19 47 27 7
					86		
		9	SS	100/10cm	85	~4	
		10	SS	103/20cm	84	~5	
83.7 / 9.5	End of borehole						

* Borehole dry

STRAT PLOT; GROUND WATER CONDITIONS *; DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100; SHEAR STRENGTH kPa: ○ UNCONFINED, + FIELD VANE, ● QUICK TRIAXIAL, × LAB VANE, 20 40 60 80 100; PLASTIC LIMIT W_P, NATURAL MOISTURE CONTENT w, LIQUID LIMIT W_L; WATER CONTENT (%) 20 40 60; UNIT WEIGHT γ kN/m^3

$+^7$, $\times^5$: Numbers refer to Sensitivity; 20 / 15—○—5 / 10 (%) STRAIN AT FAILURE

Ontario

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RECORD OF BOREHOLE No. H-8

METRIC

P.O. # 09-20009 **LOCATION** Co-ord: 4 858 601.8 N; 349 666.3 E **ORIGINATED BY** S.A.

DIST Durham **HWY** 401 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** A.D.

DATUM Geodetic **DATE** April 28, 2014 **CHECKED BY** G.D.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
92.3							
0.0 / 91.9 0.4 / 91.5 0.8	Clayey silt, rootlets topsoil inclusions / Firm to stiff, Dark brown, Moist / silty sand seams organic inclusions / Silty clay, some sand trace gravel, rootlets organic inclusions (FILL)	1	SS	7		92	
		2	SS	5		91	
		3	SS	9			1 18 45 36
		4	SS	9		90	
89.3 / 3.0	Clayey silt trace sand, trace gravel cobbles / Very stiff to hard, Brown/grey, Moist (TILL)	5	SS	21		89	
						88	
87.6 / 4.7	Sandy silt trace clay, trace gravel / Very dense, Grey, Wet (TILL)	6	SS	93	▼*	87	
					▽*		
		7	SS	60/13cm		86	
						85	
		8	SS	50/10cm		84	
		9	SS	50/8cm			
83.1 / 9.2	End of borehole						

* 2014 04 28

▽ Water level observed during drilling

▼ Water level measured after drilling

DYNAMIC CONE PENETRATION RESISTANCE PLOT: 20 40 60 80 100

SHEAR STRENGTH kPa: ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE; 20 40 60 80 100

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

UNIT WEIGHT γ kN/m³

+7, ×5 : Numbers refer to Sensitivity   20 / 15 —○— 5 / 10 (%) STRAIN AT FAILURE

CONT No
P.O. No 09-20009
GWP No

SHEET

HENRY STREET UNDERPASS
HIGHWAY 401
GEOGRAPHIC TOWNSHIP OF WHITBY
BOREHOLE LOCATIONS AND SOIL STRATA

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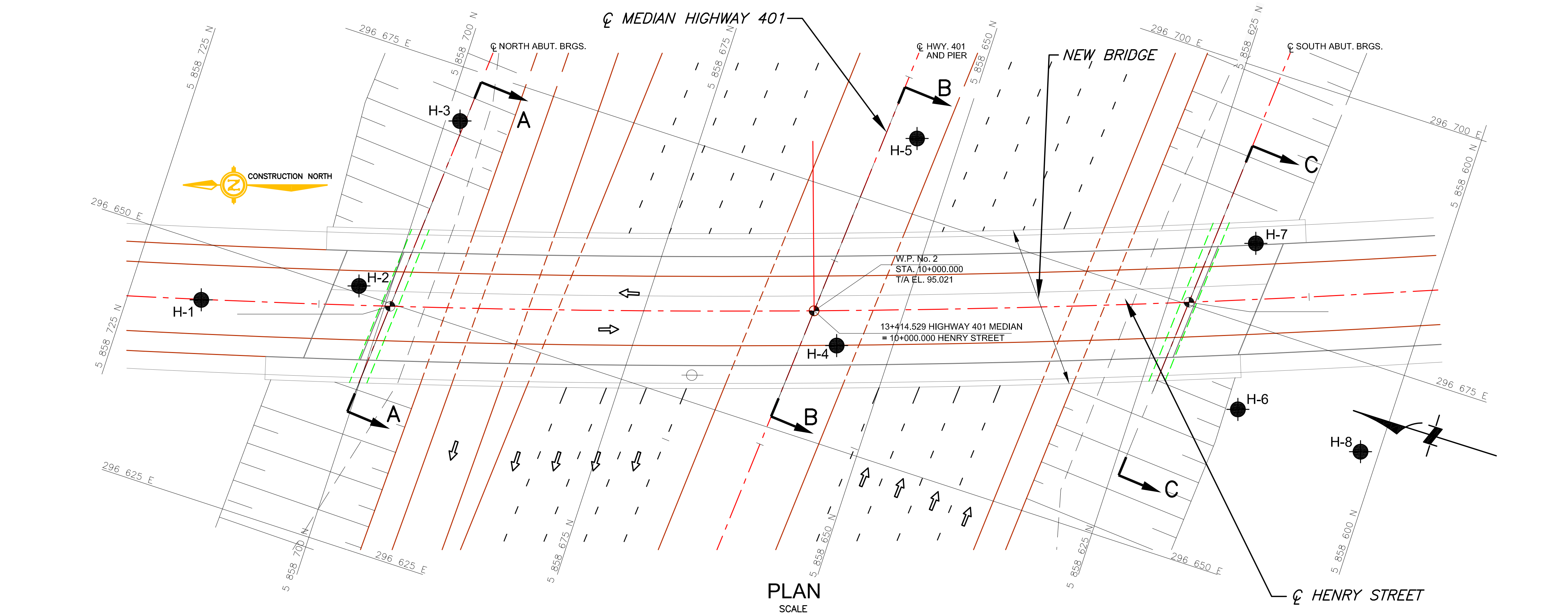
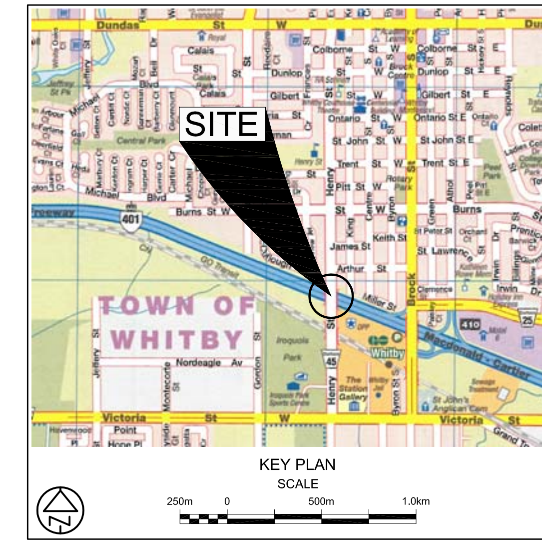


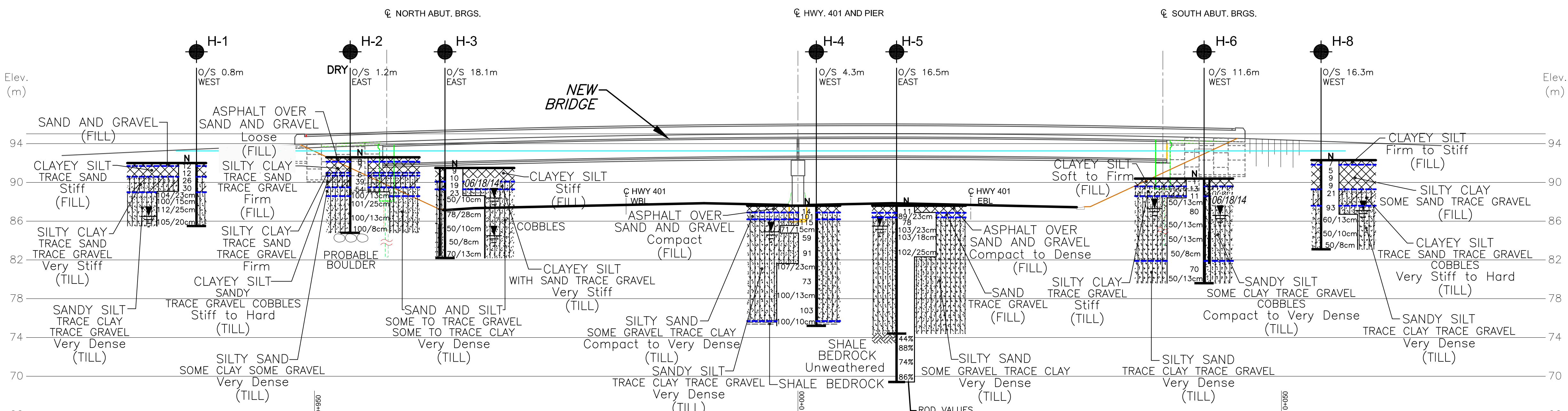
PLAN

SCALE



KEY PLAN

SCALE



PROFILE ALONG ℄ HENRY STREET

SCALE

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 & May 2013
- WH Penetration due to weight of hammer
- Head ARTESIAN WATER Encountered
- PIEZOMETER

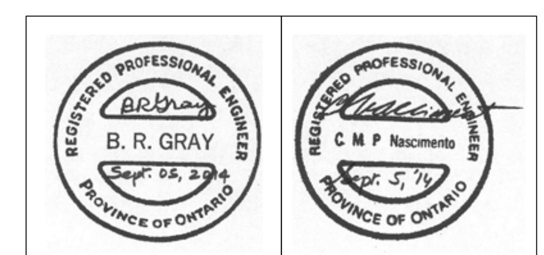
BH No	ELEVATION	NORTHINGS	EASTINGS
H-1	92.0	4 858 717.8	349 644.4
H-2	92.6	4 858 703.1	349 650.7
H-3	91.5	4 858 698.6	349 669.7
H-4	87.6	4 858 655.4	349 660.0
H-5	87.6	4 858 654.2	349 682.4
H-6	90.4	4 858 614.9	349 666.5
H-7	93.2	4 858 618.4	349 683.0
H-8	92.3	4 858 601.8	349 666.3

– NOTE –

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

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. REFER TO DRAWING H-2 FOR SECTIONS A-A, B-B AND C-C.
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.



Reference AECOM Drawing: 60154317-HENRY-GA dated Sept. 2014

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 30M15-199

HWY No	401			DIST	Central
SUBM'D	NA	CHECKED	GD	DATE SEPT. 05, 2014	SITE 22-152
DRAWN	NA	CHECKED	BRG	APPROVED CN	DWG H-1

CONT No
P.O. No 09-20009
GWP No

HENRY STREET UNDERPASS
HIGHWAY 401
GEOGRAPHIC TOWNSHIP OF WHITBY
SOIL STRATA

SHEET

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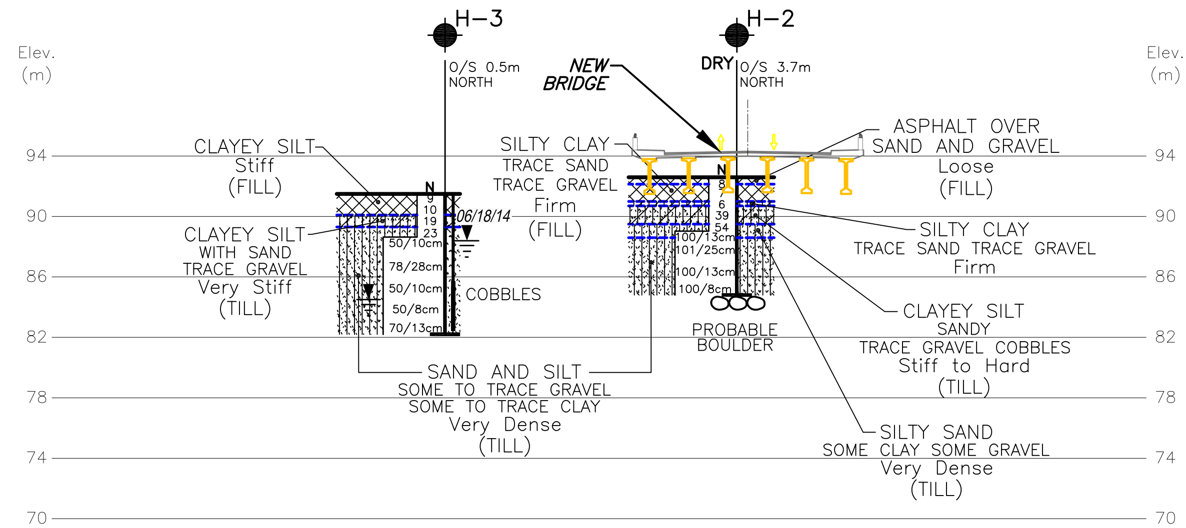
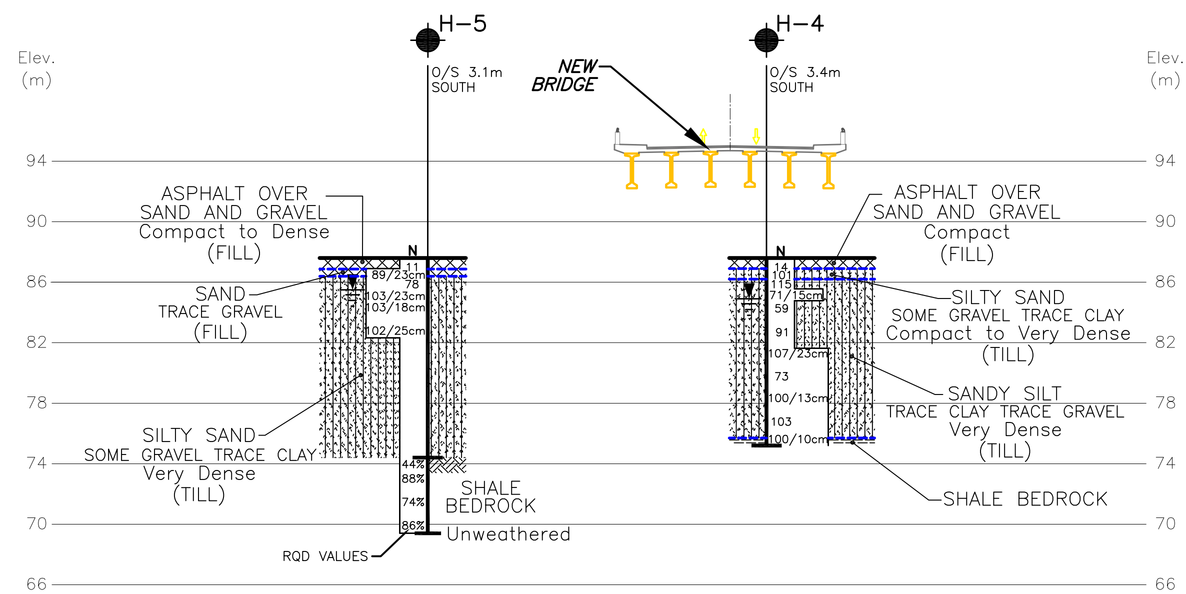
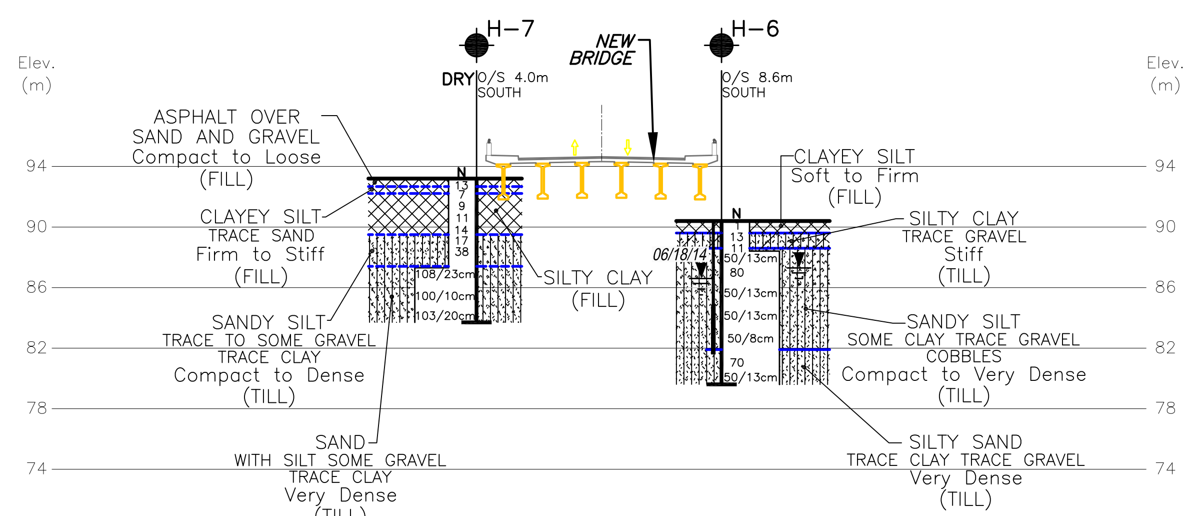


SECTION A-A



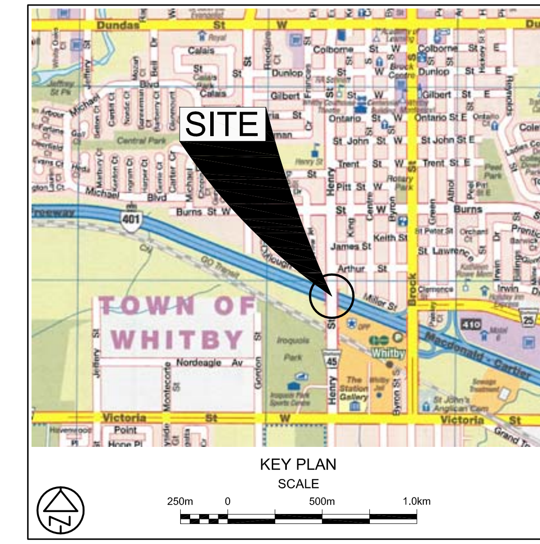
SECTION B-B



SECTION C-C

SCALE

5 0 5 10m



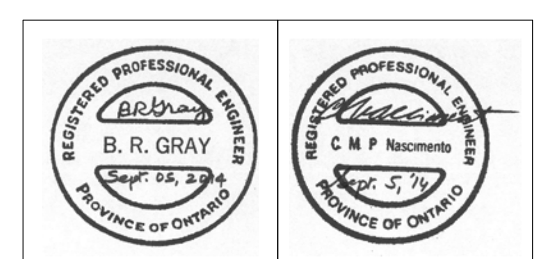
KEY PLAN
SCALE
250m 0 500m 1.0km

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 & May 2013
WH	Penetration due to weight of hammer
	Head ARTESIAN WATER Encountered
	PIEZOMETER

BH No	ELEVATION	NORTHINGS	EASTINGS
FOR DETAILS, REFER TO DRAWING H-1			

– 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-HENRY-GA.dwg dated Sept. 2014

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No. 30M15-199

HWY No 401			DIST Central
SUBM'D NA	CHECKED GD	DATE SEPT. 05, 2014	SITE 22-152
DRAWN NA	CHECKED BRG	APPROVED CN	DWG H-2

NOTES:

1. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT OF REPORT AND RECORD OF BOREHOLE LOGS.
2. REFER TO DRAWING H-1 FOR BOREHOLE LOCATION PLAN AND CENTRE LINE PROFILE.
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.

APPENDIX A

Site Photographs

Photograph 1: Facing north from the south approach to the Henry Street underpass. (April 7, 2014)

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

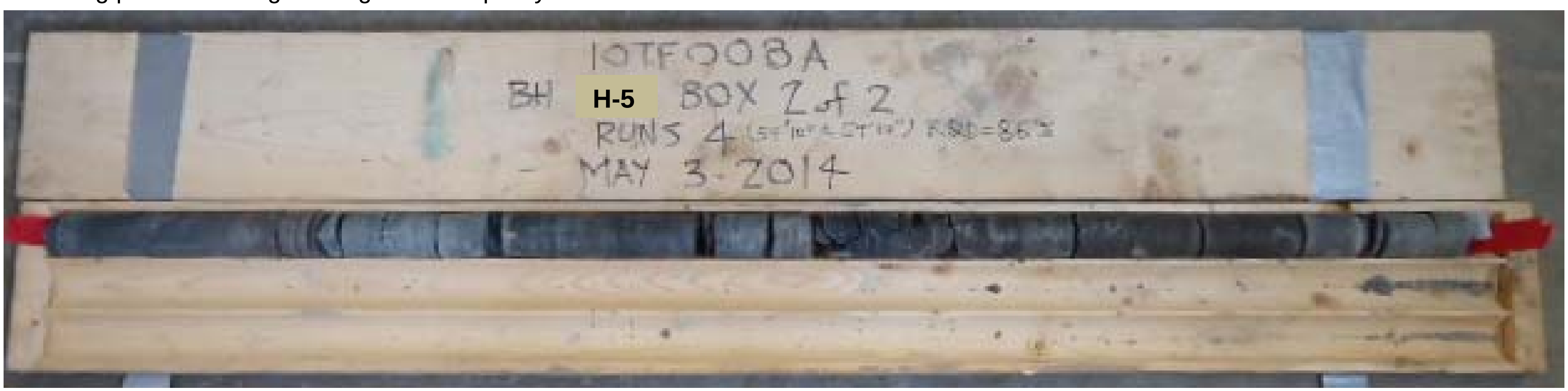
Photograph 3: Facing west at the north abutment. (April 7, 2014)

Photograph 4: Facing north at the north abutment. (April 7, 2014)

APPENDIX B

Rock Core Photographs

Photograph 1: Cores retrieved from borehole H-5. Rock cores 7 to 9 from 13.2 to 16.7 m depth. RQD values ranged from 44 to 88%, indicating poor becoming fair to good rock quality.



Photograph 2: Cores retrieved from borehole H-5. Rock core 10 from 16.7 to 18.2 m depth. RQD value was 86%, indicating good rock quality.

Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT

for

HIGHWAY 401 / HENRY STREET UNDERPASS
SITE 22-152, W.O. 09-20009
WHITBY, ONTARIO

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PML Ref.: 10TF008A-H
Index No.: 015FDR
GEOCRES No.: 30M15-199
September 5, 2014

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Peto MacCallum Ltd.
CONSULTING ENGINEERS

FOUNDATION DESIGN REPORT
for
Highway 401 / Henry Street Underpass
Site 22-152, 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 Henry Street traffic over Highway 401 in Whitby, Ontario. The project is administered as a Design Build. 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+415, Highway 401 chainage, in Whitby. The underpass is proposed to be a 2-span structure with a total length of 80.3 m and width of 16.6 m (ref. General Arrangement Drawing 1 'Highway 401 / Henry Street Underpass' prepared by AECOM in September 2014).

The road grade on Henry Street at the underpass location is planned to be at elevation 94.3 at the north abutment, elevation 95.0 at the pier and elevation 94.6 at the south abutment. The approach embankments to the structure are envisaged to be 5 to 6 m high at the north abutment and 6 to 8 m high at the south abutment (interpolated from ground surface elevations and the road grade on Henry 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 87.5.

The replacement of the underpass will require staged construction and use of roadway protection systems to maintain traffic over Highway 401. Refer to section 6 of this report for a detailed discussion.

In summary, the subsurface stratigraphy revealed in the boreholes generally comprised surficial sand and gravel and/or clayey silt / silty clay fill overlying clayey silt till or silty clay till underlain by cohesionless till mantling shale bedrock. Cobbles were encountered in 4 boreholes. Boulders are also likely to be encountered in the glacial till. At the pier location, the bedrock surface was contacted at 11.9 and 13.2 m (elevation 75.7 and 74.4). The piezometric water level was at 3.1 and 3.8 m (elevation 88.4 and 86.6) on June 18, 2014 and at 3.4 and 4.6 m (elevation 88.1 and 85.8) on July 9, 2014.

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BARRIE, HAMILTON, KITCHENER, TORONTO

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

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 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.

2. FOUNDATIONS

2.1 General

The design road grade at the underpass location is at elevation 94.0 to 95.0, about 7 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:

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES
Spread footings on native soil	• Ease of construction	• Lower bearing resistance than for driven piles or caissons	• Limited support for increase in loading
Spread footings on engineered fill pad	• Ease of construction	• Lower bearing resistance than for driven piles or caissons • Need to provide erosion protection	• Consolidation settlements expected

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES
Driven piles	• High bearing resistance	• High cost relative to footings • Vibration induced during driving • Pilot holes required due to presence of cobbles and boulders	• Pile driving next to existing bridge may damage foundations or superstructure
Caissons	• Higher bearing resistance than for other options • May address space limitations at the median for pier construction	• High cost relative to footings • Potential for difficulties to be encountered during drilling through the hard glacial till	• Augering difficulties could result in construction delays and cost overrun

Spread footings bearing on the hard / very dense till in combination with engineered fill pads as outlined in this report are considered to be the preferred means of supporting the foundation loads at the abutments and pier from a foundation engineering perspective. However, use of steel H-piles driven to 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.

The seismic 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 underpass site is located in Seismic Performance Zone 1.

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.

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 Table 1.

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 at the actual foundation locations, the founding levels for 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) is given below:

Factored Geotechnical Bearing Resistance at ULS	**Geotechnical Reaction at SLS**	**North Abutment**		**Pier**	**South Abutment**	
		Borehole H-2	**Borehole H-3**	**Boreholes H-4 and H-5**	**Borehole H-6**	**Borehole H-7**
550	350	Elev. 90.0	Elev. 88.5	–	Elev. 88.5*	Elev. 88.5
1000	650	Elev. 89.0	Elev. 88.0	Elev. 86.5	Elev. 88.0	Elev. 87.0

(*) Replace with engineered fill (refer to section 2.2.1)

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 6.7.4 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 till. 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.

The structural fill should comprise OPSS Granular A material placed 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 3 m thick structural fill is as follows:

Factored Bearing Resistance at ULS	=	900 kPa
Bearing Resistance at SLS	=	350 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 6.7.4 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 is a suitable method of supporting the abutment and pier foundations of the underpass. In view of the denseness of the native soils and presence of cobbles and boulders, the installation of these piles will likely require the pre-augering of pilot holes to approximate elevation 74.5 to 75.5. 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 at the location of the pier was contacted at 11.9 and 13.2 m (elevation 75.7 and 74.4). In borehole H-5, the bedrock comprised an unweathered soft to medium strength shale with interbedded limestone and was classified as poor becoming fair to good quality (RQD of 44 to 88%) with a core recovery of 66 to 99%. Poor quality rock was only identified in the upper 0.4 m core sample. It is considered that the rock is capable of adequately supporting the foundation loads.

The H-piles should be driven to refusal on bedrock anticipated at 11.9 to 13.2 m below existing grade (elevation 74.4 to 75.7). The founding levels for driven piles at the proposed foundation locations should be confirmed during detail design. It is worth noting that 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 preliminary design purposes, however, the resistance at SLS may be taken equal to the factored axial resistance at ULS.

For steel HP 310 x 110 piles driven into the very dense glacial till below elevation 83.0 at the abutment locations, 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 both abutments should be confirmed during detail design.

The piles will have to be driven through the native soils containing relatively compressible clayey silt till / silty clay till at the abutments. Taking into account that the abutment locations have been preloaded by the 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, up to 15 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 detail 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 make pile driving operations slow and arduous. An NSSP prepared to alert the Contractor to the presence of cobbles and boulders at the site is attached to this report. It may be necessary to provide pre-auger holes if the piles are designed to be driven to bedrock.

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 should be driven to the hard / very dense glacial till or bedrock. 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. In case of the H-piles driven to bedrock, the installation of these piles is likely to require the pre-augering of pilot holes.

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. 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.

The factored axial resistance for caissons founded in the hard / dense to 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 / 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

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.

The factored axial resistance for caissons socketed at least 1 m into sound shale is given below:

Caisson Diameter, m	**Factored Axial Resistance, kN, for Socket / Diameter Ratio (Caisson Base Elevation)**		
	1	**2**	**3**
0.9	1500 (el. 73.0)	1900 (el. 72.2)	2250 (el. 71.3)
1.2	2550 (el. 72.8)	3150 (el. 71.6)	3750 (el. 70.4)
1.5	3875 (el. 72.5)	4800 (el. 71.0)	5725 (el. 69.5)
1.8	5450 (el. 72.2)	6800 (el. 70.4)	8150 (el. 68.6)

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 possible 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 Table 3.

The modulus of horizontal subgrade reaction k_s, 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 MN/m^3 for dense to very dense cohesionless till
= 25 MN/m^3 for shale bedrock
z = depth, m
d = pile diameter, m

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:

$$p = K(\gamma h + q) + Cp + Cs$$

where K = coefficient of lateral earth pressure (dimensionless)
γ = unit weight of free-draining granular material, kN/m^3
h = depth below final grade, m
q = surcharge load, kPa, if present
Cp = compaction pressure, kPa (refer to clause 6.9.3 of CHBDC)
Cs = earth pressure induced by seismic events, kPa (refer to clause 4.6.4 of CHBDC)
where Ø = angle of internal friction of retained soil (35º for Granular B Type II)
δ = angle of friction between the soil and wall (23.5º 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.

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^3	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 and lead to a frost-free outlet.

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 6.9.3 of the CHBDC. Refer to OPSS 501 for additional information in this regard.

4. APPROACH EMBANKMENTS

The height of fill embankment widenings will be 5 to 6 m high at the north approach and 6 to 8 m high at the south approach. 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 5 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.

5. 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 1.5 to 3.0 m at both abutments and the pier (about 5 m at the location of borehole H-7).

The typically firm to stiff / 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 measured in boreholes H-3 and H-6 was at 3.1 and 3.8 m (elevation 88.4 and 86.6) on June 18, 2014 and at 3.4 and 4.6 m (elevation 88.1 and 85.8) on July 9, 2014, respectively. 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.

6. CONSTRUCTION STAGING AND ROADWAY PROTECTION

The current plans call for the centreline of the proposed two-lane 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 a 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 road protection scheme.

6. 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., 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.

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

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

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

GD/CN/BRG:gd-mi

TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
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-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

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
≥ 4D	1.00
3D	0.83
2D	0.67
1D	0.50

Notes: 1) D is pile diameter

2) Spacing is considered the distance from centre to centre

APPENDIX A

Non-Standard Special Provisions (NSSP)

NON-STANDARD SPECIAL PROVISIONS (NSSP)

NSSP – Potential for Cobbles and Boulders During Pile Driving

The Contractor shall be advised that cobbles and boulders were identified within the sandy silt / silty sand deposit at the site and that although no cobbles and boulders were encountered in the glacial till deposits the possibility of cobbles or boulders within the glacial till deposits should be considered. The contractor shall ensure that more comprehensive engineering supervision is required than is called for in OPSS 903 and that heavy pile driving with local pre-augering may be required.

If during pile driving there is evidence that a pile meets refusal on a cobble or boulder the contractor shall inform the Contract Administrator. The contractor shall be advised that piles meeting refusal on a cobble or boulder may need to be relocated, have their capacity reduced and / or require additional piles to be installed.

NSSP – Temporary Roadway Protection

The possibility of the existing granular fill migrating through the temporary roadway protection and difficulties associated with the presence of cobbles and boulders within the sandy silt, silty sand and possibility of cobbles and boulders within the glacial till soils encountered at the site should be considered by the contractor during the installation of the temporary protection systems.