Peto MacCallum Ltd.
CONSULTING ENGINEERS

PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT

for

CNR / CPR OVERHEAD
HIGHWAY 140, SITE 34-232
GWP 2175-08-00
CITY OF WELLAND, ONTARIO

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:
5 cc: AECOM for distribution to MTO Project Manager + 1 digital copy (PDF)
2 cc: AECOM for distribution to MTO, Pavements and Foundations Section + 1 digital copy (PDF) and Drawing (AutoCAD)
2 cc: AECOM + 1 digital copy
1 cc: PML Kitchener
1 cc: PML Toronto

PML Ref.: 11TF023A-3
Index No.: 038FIDR
GEOCRES No.: 30L14-55
March 15, 2012

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PRELIMINARY FOUNDATION INVESTIGATION REPORT

for

CNR / CPR OVERHEAD
HIGHWAY 140, SITE 34-232
GWP 2175-08-00
CITY OF WELLAND, ONTARIO

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:
5 cc: AECOM for distribution to MTO Project Manager + 1 digital copy (PDF)
2 cc: AECOM for distribution to MTO, Pavements and Foundations Section + 1 digital copy (PDF) and Drawing (AutoCAD)
2 cc: AECOM + 1 digital copy
1 cc: PML Kitchener
1 cc: PML Toronto

PML Ref.: 11TF023A-3
Index No.: 038FIDR
GEOCRES No.: 30L14-55
March 15, 2012

TABLE OF CONTENTS

PART A – PRELIMINARY FOUNDATION INVESTIGATION REPORT

Peto MacCallum Ltd.
CONSULTING ENGINEERS

PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
CNR / CPR Overhead
Highway 140, Site 34-232
GWP 2175-08-00
City of Welland, Ontario

1. INTRODUCTION

This report summarises the results of a preliminary foundation investigation carried out for the proposed rehabilitation or replacement of the CNR / CPR overhead located on Highway 140 in the City of Welland, Ontario. The study was conducted for AECOM Canada Ltd. (AECOM) on behalf of the Ministry of Transportation of Ontario (MTO).

The existing overhead carries the Highway 140 traffic over the CNR / CPR tracks and Townline Tunnel Road at approximate Sta. 17+013, Highway 140 chainage. The overhead is a five span structure with a total length of 204 m and a width of 12 m, accommodating two lanes of traffic. According to the Geocres data referenced below, the approach embankments to the overhead are up to 9.5 m high. The as-built drawings received from AECOM on July 27, 2011 indicate that the abutments and piers of the structure are founded on steel pipe piles filled with concrete and embedded into bedrock. The road grades on Highway 140 and Townline Tunnel Road at the overhead location are at elevation 179.3 to 179.6 and elevation 171.8 respectively. The top of rail on the CNR / CPR tracks is at elevation 165.1.

Part A of the report provides preliminary subsurface information pertaining to the proposed structure and approach embankments near the abutments, is considered to be only suitable for planning and preliminary design purposes but should not be used for detail design. A detail foundation investigation will be required for the detail design phase of the project.

Information from previous foundation investigations carried out at the site (Geocres No. 30L14-36 dated December 1968, Geocres No. 30L14-45 dated July 1972 and Geocres No. 30L14-50 dated August 2009) has also been used in the preparation of this report. Copies of the borehole logs from the Geocres No. 30L14-36 report are attached in Appendix A. This report supersedes all other reports for the purpose of this project.

165 Cartwright Avenue, Toronto, Ontario M6A 1V5
Tel: (416) 785-5110 Fax: (416) 785-5120
E-mail: toronto@petomaccallum.com
BARRIE, HAMILTON, KITCHENER, TORONTO

2. SITE DESCRIPTION AND GEOLOGY

The site is located on the Highway 140 alignment at the crossing of the CNR / CPR tracks and Townline Tunnel Road some 500 m north of Forks Road East near the northern boundary of the City of Welland. The alignment of the overpass extends roughly in the south-north direction.

Land use in the vicinity of the site is mainly agricultural. Residences exist along Forks Road East.

The topography of the area surrounding the site is relatively flat and level. In general, the area is poorly drained. Vegetation consists of grass and small shrubs, with some large trees.

The site is located within the Haldimand Clay Plain physiographic region. The area is typically characterised by extensive deposits of lacustrine clay and silt. The bedrock at the site comprises interbedded dolostone, shale and evaporites (gypsum) of the Salina Formation deposited during Silurian geologic time and is anticipated at an approximate depth of 25 m.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out during the period of September 12 to 23, 2011 and comprised four boreholes drilled to depths of 11.3 to 25.8 m at the locations shown on Drawing H140-1, attached. A dynamic cone penetration test was performed from the bottom of borehole H140-3 and terminated at 14.0 m (elevation 157.7) due to mechanical breakdown. The remaining boreholes were terminated upon meeting refusal on probable bedrock. The drawing also shows boreholes 1 to 4 advanced during the previous investigation (Geocres No. 30L14-36 dated December 1968).

The locations of and ground surface elevations at the boreholes were established in the field by Peto MacCallum Ltd. (PML) using the following benchmark for vertical reference:

0011980u3015:	Steel protective casing marked by a white wooden post approximately 60 m north of the CNR / CPR overhead and 27 m west of the centreline of Highway 140 . Elevation 179.443 m (geodetic).

The boreholes were advanced using continuous flight hollow and solid stem augers, powered by a track-mounted CME-55 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

Representative soil samples were recovered at frequent 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. In situ vane shear and penetrometer testing was also performed to further assess the shear strength of the cohesive soils encountered. The results of the field tests and observations are reported on the Record of Borehole sheets. It is noted that penetrometer test results may be lower than the field vane values due to sample disturbance or soil layering. We refer to subsection 4.3 of this report for further details.

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, two piezometers consisting of a 19 mm dia. PVC pipe slotted over the bottom 3 m were installed in boreholes H140-1 and H140-4 to monitor groundwater conditions. The annular space around the pipe was backfilled with filter sand and a bentonite seal placed as illustrated on the corresponding borehole logs. The water level in the piezometers was measured on September 23 and October 17, 2011. Once completed, the other two boreholes were backfilled with a bentonite-cement mixture in accordance with the MTO guidelines and Ontario Regulation 903 for borehole abandonment procedures.

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 moisture content determination. In addition, 15 Atterberg limits tests and 15 grain size distribution analyses were carried out on selected soil samples, with the results presented in Figures H140-PC-1 to H140-PC-4 and Figures H140-GS-1 to H140-GS-4 respectively as well as on the corresponding Record of Borehole sheets.

4. SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations, standard and dynamic cone penetration test data, in situ vane and penetrometer undrained shear strength values and groundwater observations. The results of laboratory Atterberg limits testing, grain size distribution analyses and moisture content determinations are also shown on the Record of Borehole sheets.

The borehole locations and stratigraphic profile prepared from the borehole data are shown on Drawing H140-1. The boundaries between soil strata have been established at the borehole locations only. Between and beyond the boreholes, the boundaries are assumed and may vary.

The subsurface stratigraphy revealed in the boreholes drilled at the site is consistent, generally comprising a surficial topsoil or fill overlying a cohesive deposit of very stiff becoming firm to stiff with depth silty clay underlain by clayey silt till mantling bedrock. The bedrock surface was inferred at 24.4 to 25.8 m (elevation 152.4 to 154.3) in three out of the four boreholes put down during the current investigation. Upon completion of drilling, groundwater was at 15.4 m (elevation 162.8) in one borehole. The piezometric water levels in boreholes H140-1 and H140-4 were at 9.2 m (elevation 169.0 and 169.5) on October 17, 2011.

The strata encountered in the current boreholes H140-1 to H140-4 are summarised below.

4.1 Topsoil

Surficial topsoil was present in borehole H140-1. The silty topsoil was 200 mm thick and penetrated at elevation 178.0.

4.2 Fill

Sand and gravel fill covered by 225 mm of asphalt was present surficially in borehole H140-3. Overlain by the sand and gravel fill at 0.7 m (elevation 171.0) was silty clay fill. This unit was very stiff in consistency and had a moisture content of 17 to 23%. A penetrometer test on a sample of the silty clay fill indicated a shear strength of 225 kPa. The silty clay fill was 3.0 m in thickness and penetrated at 3.7 m (elevation 168.0).

Surficial fill was present in boreholes H140-2 and H140-4. Composed of topsoil and silty clay layers with organics and limestone inclusions, the fill was 0.5 and 0.6 m thick and penetrated at elevation 177.6 and 178.1 respectively. The fill had a moisture content ranging from 13 to 21%.

4.3 Silty Clay

Directly below the topsoil or fill at 0.2 to 3.7 m (elevation 168.0 to 178.1) in all the boreholes was a cohesive deposit of silty clay. Containing clay layers and silt seams, the silty clay was typically very stiff in the upper 3 m thick zone (penetrometer tests indicating a shear strength of 88 to 225 kPa) and firm to stiff underneath. The results of in situ vane testing carried out in the lower portion of the deposit yielded undisturbed shear strength values in a range of 36 to 100 kPa (soil sensitivity of 2 to 3). Penetrometer tests on samples of the silty clay indicated a shear strength varying between 25 and 88 kPa. In the lower zone of the deposit, the field vane test results were about 10% higher than the penetrometer results because of the presence of silt layers in the silty clay deposit.

The deposit had a thickness of 23.8 to 23.9 m at the south abutment, 18.1 m in borehole H140-4 put down at the north abutment and was penetrated at respective depths of 24.1 to 24.3 m (elevation 153.8 to 154.1) and 18.7 m (elevation 160.0).

The results of Atterberg limits testing and grain size distribution analyses conducted on 12 cohesive samples of the deposit are presented in Figures H140-PC-1 to H140-PC-3 and H140-GS-1 to H140-GS-3 respectively. The clay units had a liquid limit of 51 to 53, plastic limit of 25, their plasticity index being 26 to 28. The liquid and plastic limits of the silty clay ranged from 35 to 49 and from 17 to 24 respectively, with the plasticity index of 18 to 25. Where silt seams were present, the silty clay had a liquid limit of 31 to 34, plastic limit of 18, thus giving the plasticity index of 13 to 16. The moisture content of the deposit varied between 19 and 52%.

4.4 Clayey Silt Till

Underlying the silty clay at depths of 24.1 to 24.3 m (elevation 153.8 to 154.1) at the south abutment and 18.7 m depth (elevation 160.0) in borehole H140-4 put down at the north abutment was a cohesive deposit of clayey silt till. Very stiff to hard in consistency, this deposit had a thickness of 1.0 to 1.7 m at the south abutment, up to 5.7 m at the north abutment and extended to probable bedrock contacted at depths of 24.4 to 25.8 m (elevation 152.4 to 154.3).

The results of Atterberg limits testing and grain size distribution analyses conducted on 3 cohesive samples of the deposit are presented in Figures H140-PC-4 and H140-GS-4 respectively. The clayey silt till had a liquid limit of 16 to 18, plastic limit of 11 to 12, its plasticity index being 5 to 6. The moisture content of the deposit was in a range of 7 to 10%.

4.5 Bedrock

Dolostone bedrock was inferred by refusal to penetration below the clayey silt till at 24.4 to 25.8 m (elevation 152.4 to 154.3). Borehole H140-3 was terminated at 14.0 m (elevation 157.7) due to mechanical breakdown before reaching the bedrock.

It is noted that the bedrock was contacted / inferred at elevation 151.6 to 152.4 in boreholes 1 to 4 advanced during the previous investigation.

4.6 Groundwater

In the course of the field work, groundwater was observed in two boreholes. In the process of augering, water was detected at 14.0 m (elevation 164.2) in borehole H140-1 and at 15.2 m (elevation 163.5) in borehole H140-4. Upon completion of drilling, groundwater was at 15.4 m (elevation 162.8) in borehole H140-1. The piezometric water level in boreholes H140-1 and H140-4 was at 9.2 and 9.5 m (elevation 169.0 and 169.2) on September 23, 2011 and at a depth of 9.2 m (elevation 169.0 and 169.5) on October 17, 2011, respectively. It is consistent with the water level in the deep piezometers installed in the previous boreholes 1 and 2 (elevation 168.9 and 170.1).

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

5. <u>MISCELLANEOUS</u>

The field work was carried out under the supervision of Mr. F. Portela, Senior Technician, and direction of Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and Mr. C.M.P. Nascimento, P.Eng., Project Manager. The equipment was supplied by Elite Drilling. The testing of soil samples was carried out in the Toronto laboratory of PML.

The report was prepared by Mr. Grigory O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. Brian R. Gray, MEng, P.Eng., MTO Designated Principal Contact. Mr. C.M.P. Nascimento, P.Eng., Project Manager, carried out an independent review of the report.

Sincerely

Peto MacCallum Ltd.

LICENSED PROFESSIONAL ENGINEER
G. DEGIL
100056241
Mar. 15/12
PROVINCE OF ONTARIO

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

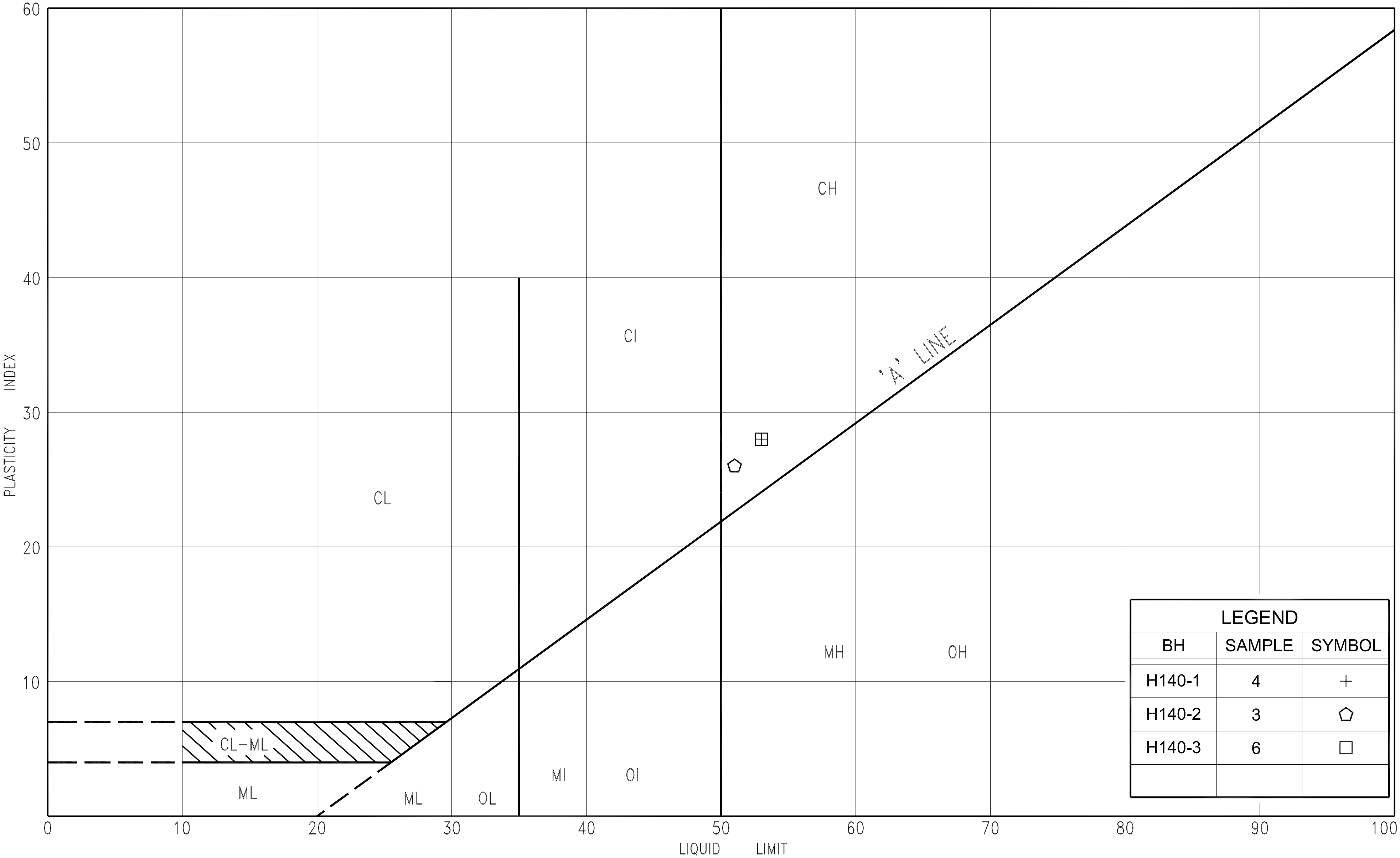
REGISTERED PROFESSIONAL ENGINEER
C. M. P. Nascimento
March 15 2012
PROVINCE OF ONTARIO

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

REGISTERED PROFESSIONAL ENGINEER
B. R. GRAY
Mar. 15/12
PROVINCE OF ONTARIO

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

GD/CN/BRG:gd-mi



LEGEND

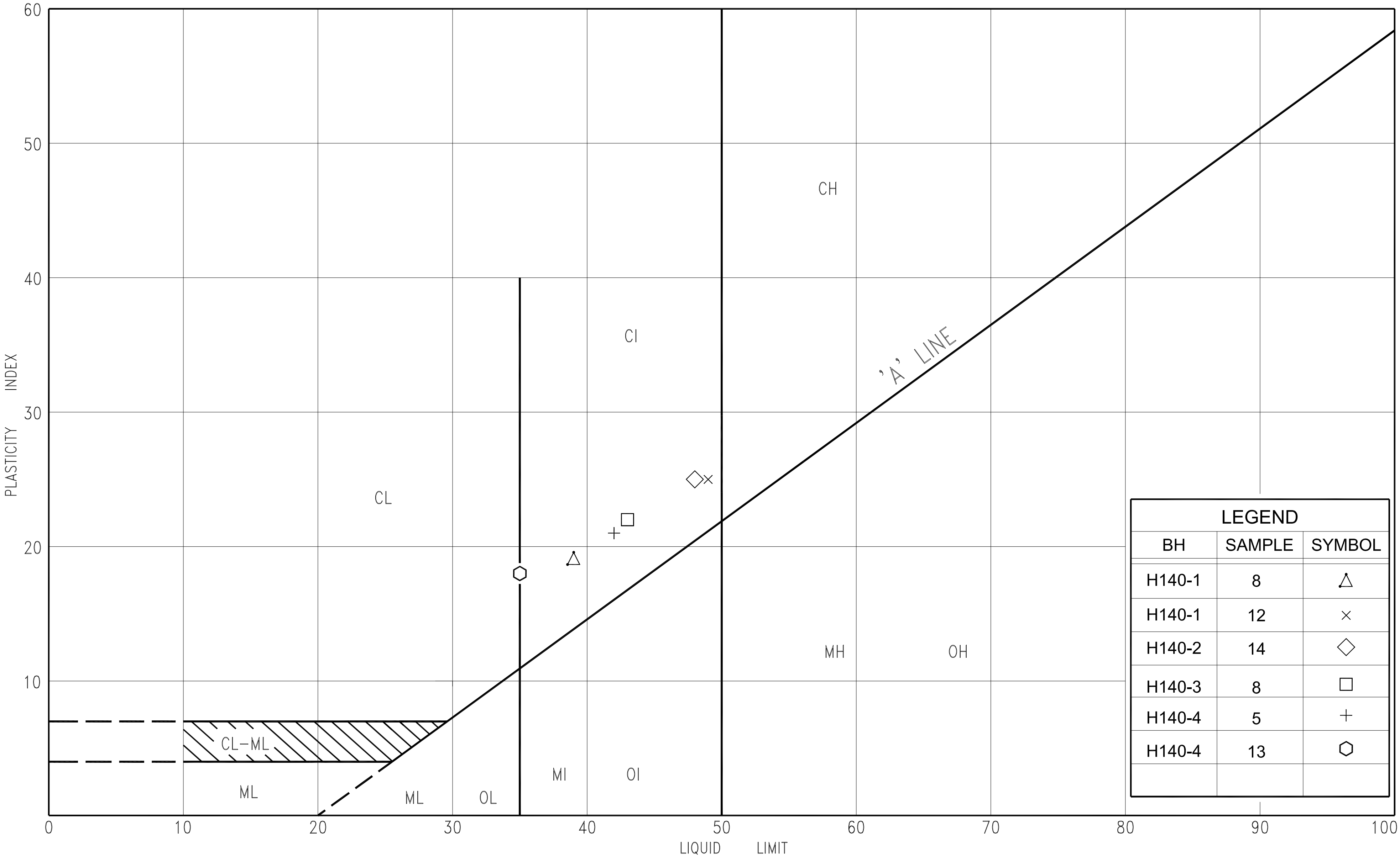
BH	SAMPLE	SYMBOL
H140-1	4	+
H140-2	3	⬠
H140-3	6	□

Ontario

PLASTICITY CHART

CLAY, with silt, trace sand

FIG No.	H140-PC-1
HWY:	140
G.W.P. No.	2175-08-00

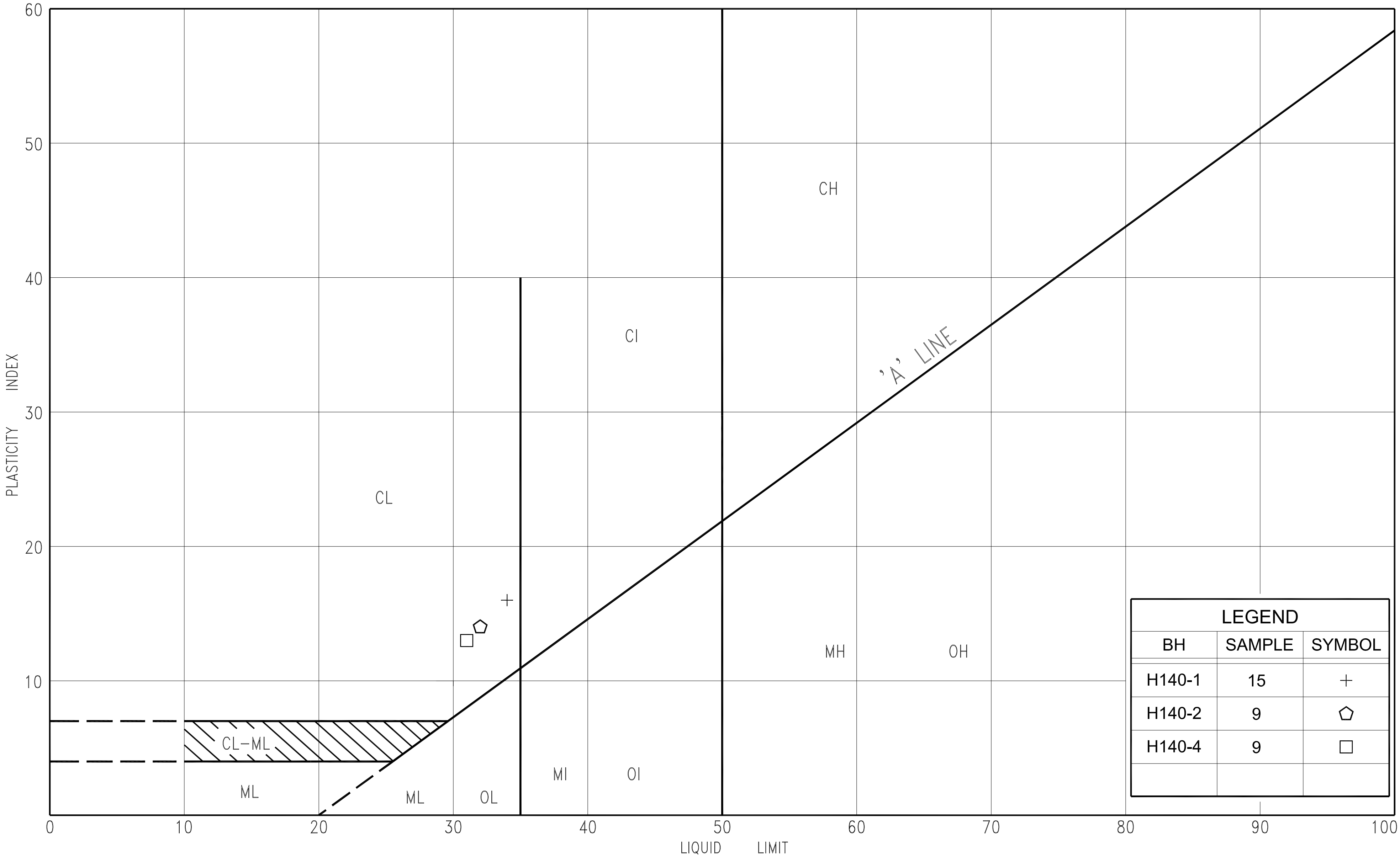


Ontario

PLASTICITY CHART

SILTY CLAY, trace sand

FIG No.	H140-PC-2
HWY:	140
G.W.P. No.	2175-08-00



LEGEND

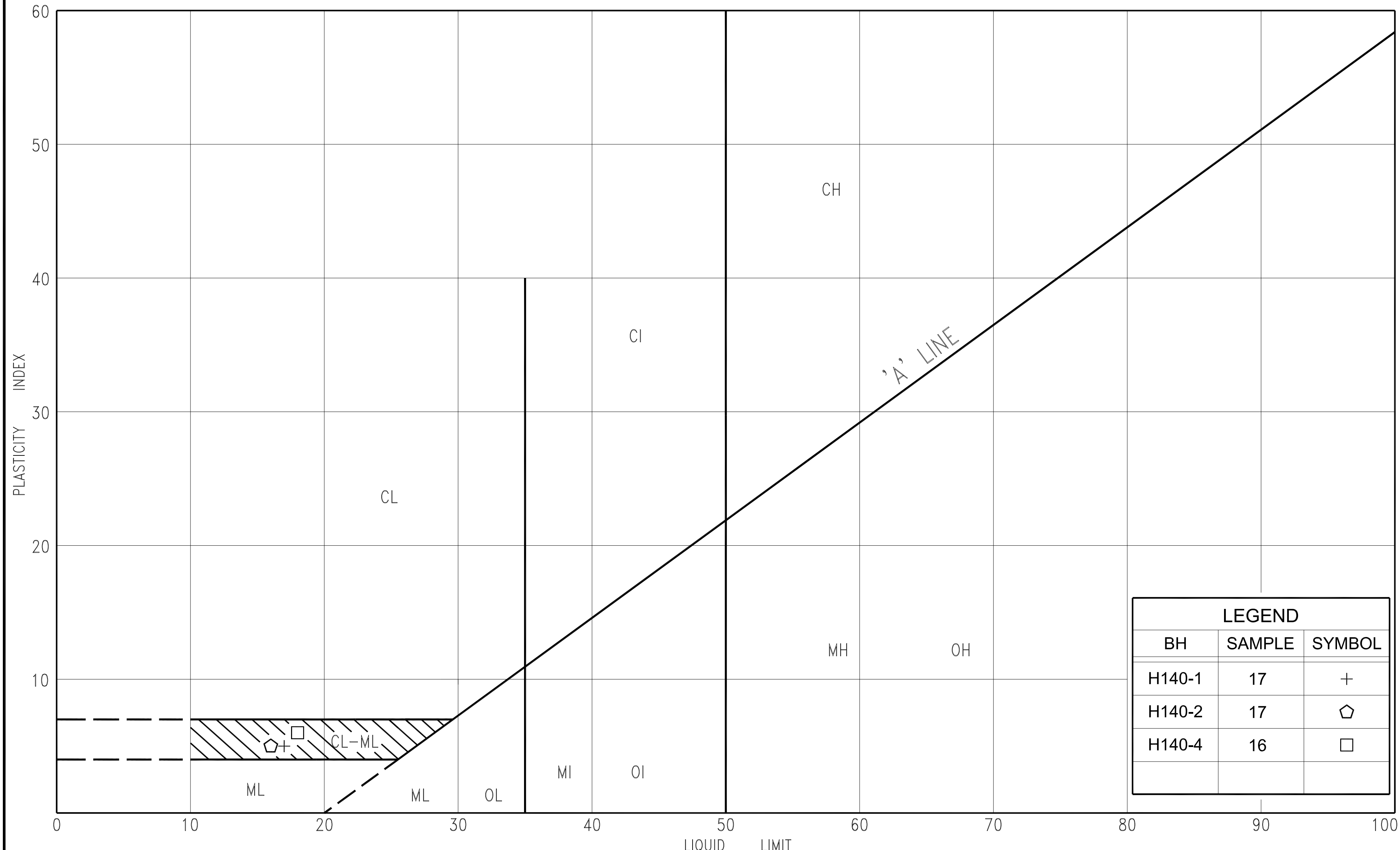
BH	SAMPLE	SYMBOL
H140-1	15	+
H140-2	9	⬠
H140-4	9	□

Ontario

PLASTICITY CHART

SILTY CLAY, with silt seams, trace sand

FIG No.	H140-PC-3
HWY:	140
G.W.P. No.	2175-08-00



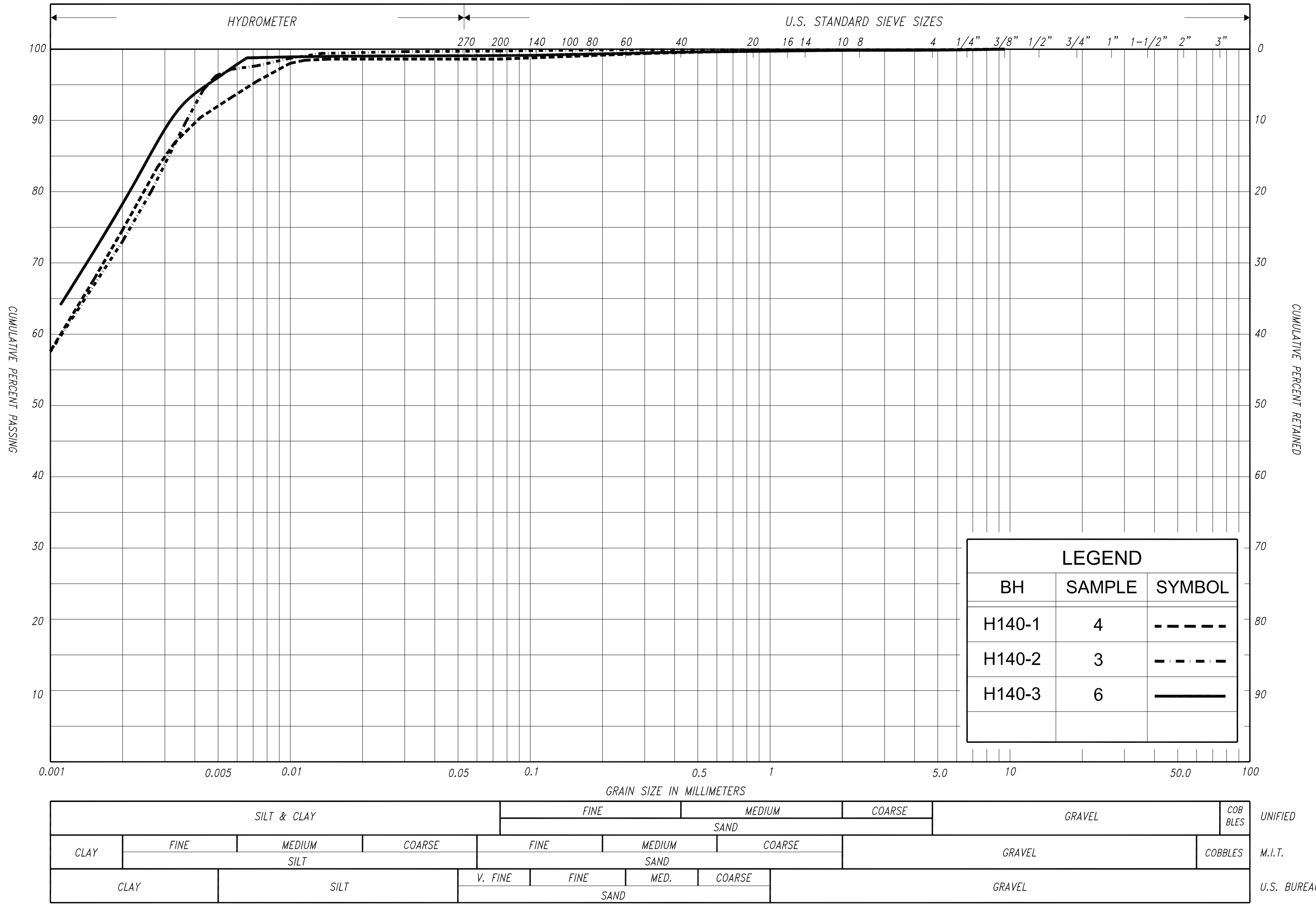
LEGEND		
BH	SAMPLE	SYMBOL
H140-1	17	+
H140-2	17	⬠
H140-4	16	□

Ontario

PLASTICITY CHART

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

FIG No.	H140-PC-4
HWY:	140
G.W.P. No.	2175-08-00

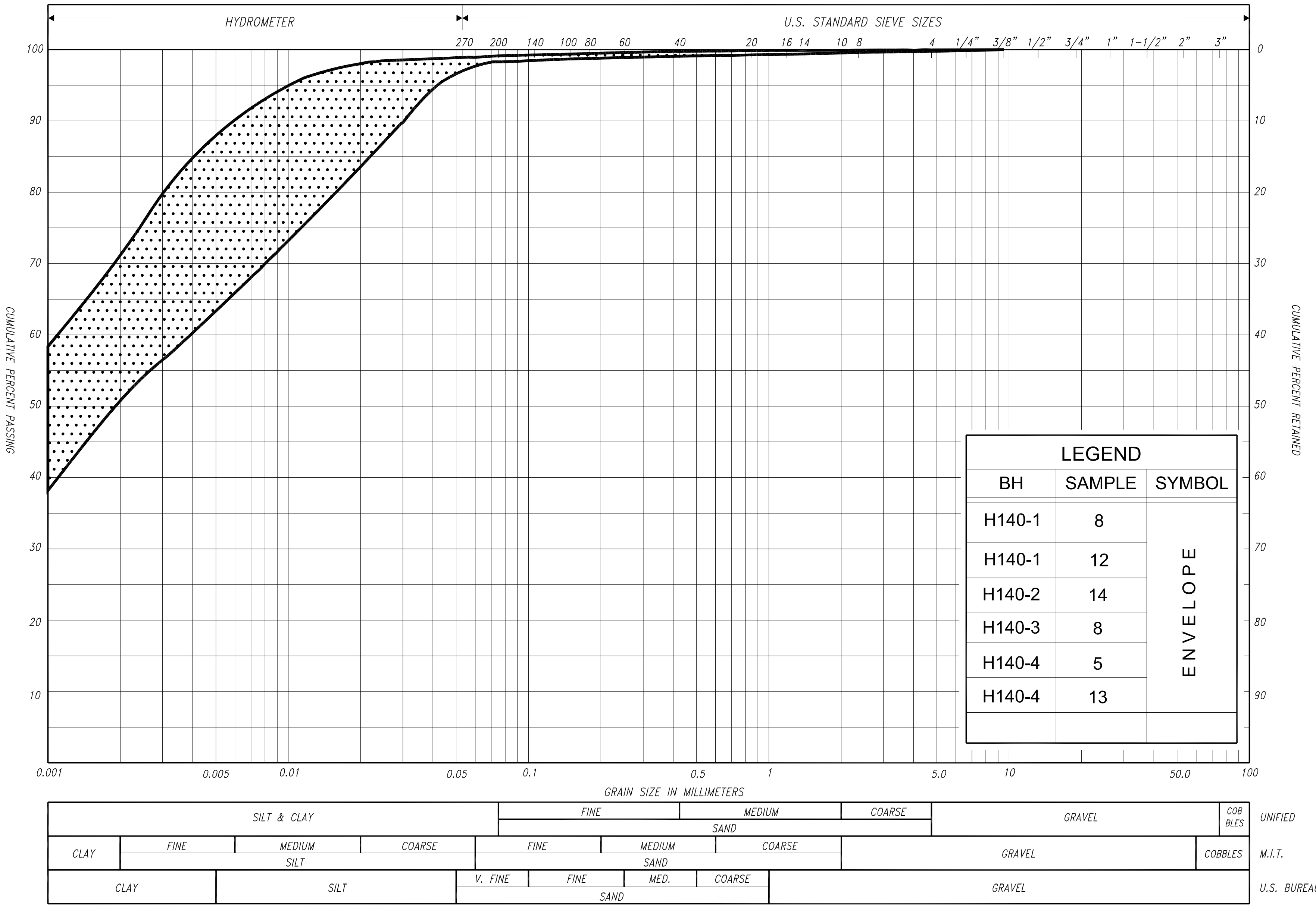


Ontario

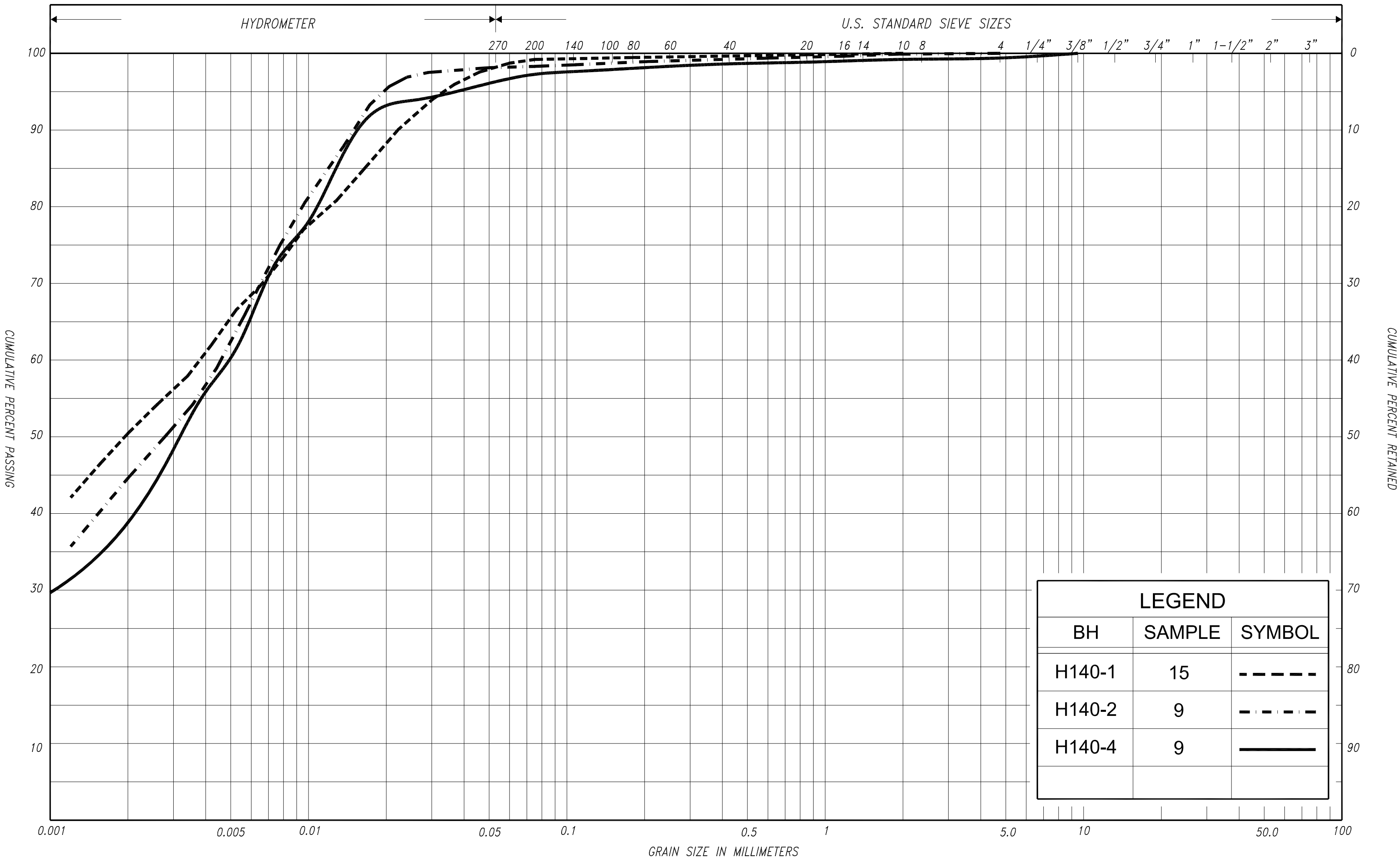
GRAIN SIZE DISTRIBUTION

CLAY, with silt, trace sand

FIG No.	H140-GS-1
HWY:	140
G.W.P. No.	2175-08-00



Ontario	**GRAIN SIZE DISTRIBUTION**	FIG No. H140-GS-2
	SILTY CLAY, trace sand	HWY: 140
		G.W.P. No. 2175-08-00



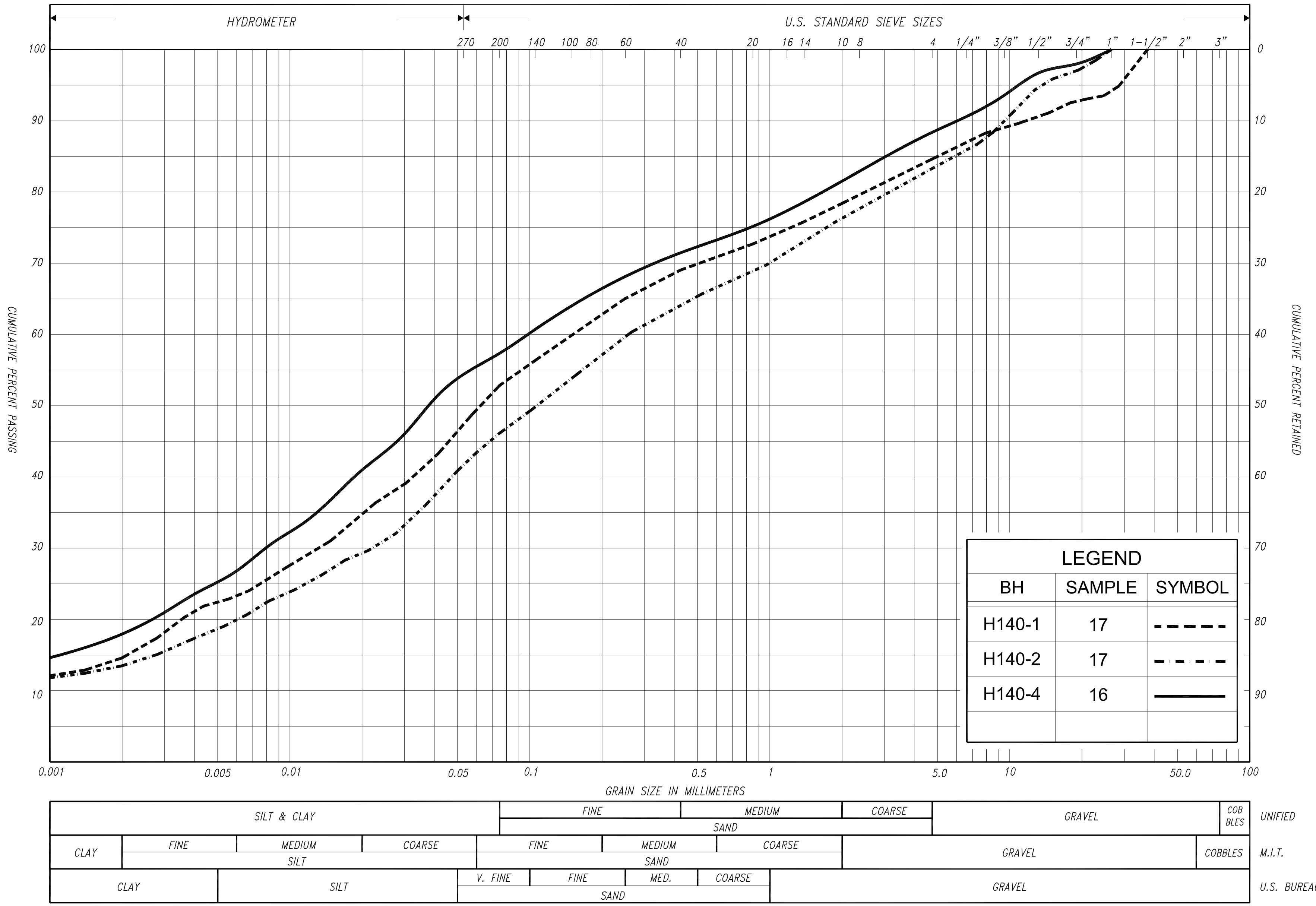
SILT & CLAY			FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL	COBBLES			UNIFIED
CLAY	FINE SILT	MEDIUM SILT	COARSE SILT	FINE SAND	MEDIUM SAND	COARSE SAND	GRAVEL	COBBLES		M.I.T.
CLAY	SILT	V. FINE SAND	FINE SAND	MED. SAND	COARSE SAND	GRAVEL				U.S. BUREAU

Ontario

GRAIN SIZE DISTRIBUTION

SILTY CLAY, with silt seams, trace sand

FIG No.	H140-GS-3
HWY:	140
G.W.P. No.	2175-08-00



Ontario

GRAIN SIZE DISTRIBUTION

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

FIG No.	H140-GS-4
HWY:	140
G.W.P. No.	2175-08-00

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

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

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

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

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No H140-1

METRIC

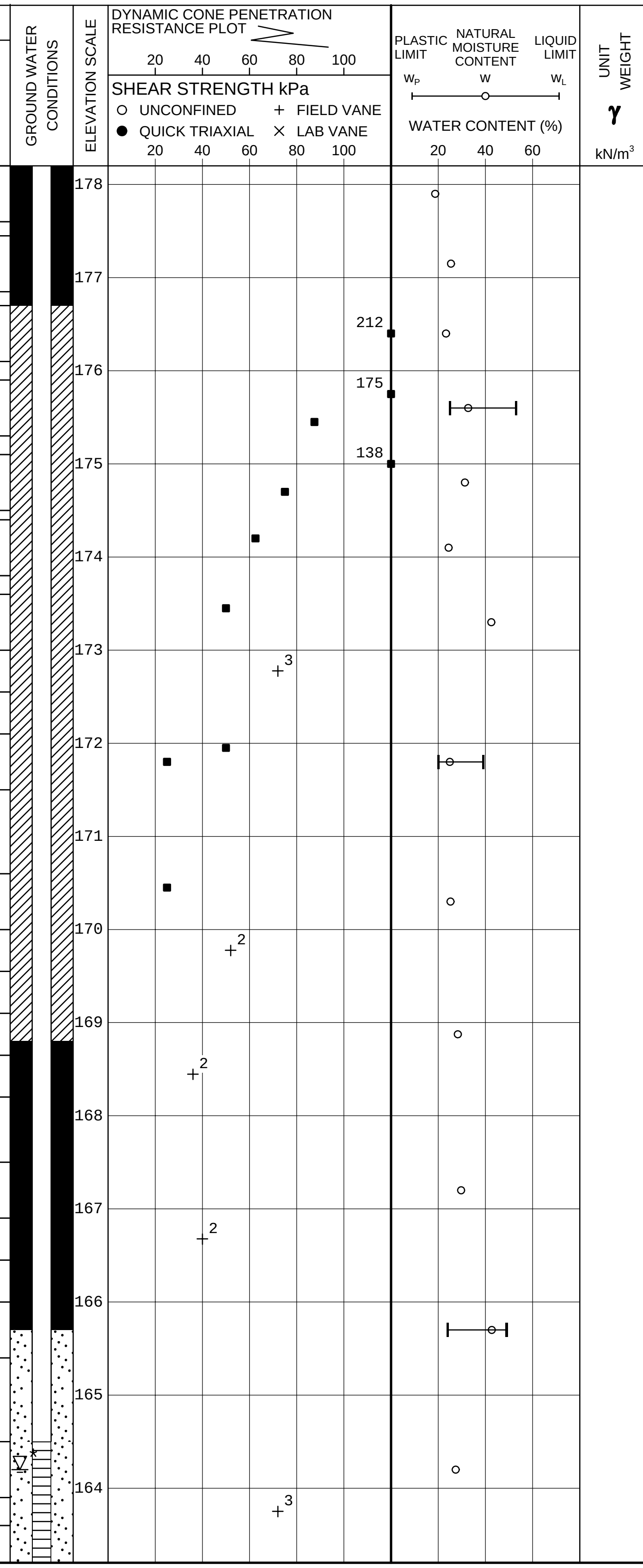
G.W.P. 2175-08-00	LOCATION Coords: 4 757 691.1 N; 328 641.0 E		ORIGINATED BY F.P.
DIST Central HWY 140	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY G.D.
DATUM Geodetic	DATE September 12 & 13, 2011		CHECKED BY C.N.

SOIL PROFILE

ELEV DEPTH	DESCRIPTION
178.2	Ground Surface
0.0	Topsoil
178.0 0.2	Silty clay, trace sand organic inclusions silt partings to 2.1 m Very stiff to stiff / Mottled red/brown / Moist
	clay layers
	Firm to stiff / Reddish brown
	varved Reddish brown/ grey / Moist to wet
	thin silt seams
163.2	Cont'd

SAMPLES

NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
1	SS	9	
2	SS	15	
3	SS	17	
4	SS	9	0 1 24 75
5	SS	6	
6	SS	5	
7	SS	4	
	FV		
8	SS	4	0 2 40 58
9	SS	3	
	FV		
10	TW	-	
	FV		
11	TW	-	
	FV		
12	SS	3	0 1 30 69
13	SS	1	
	FV		



+7, ×5 : Numbers refer to Sensitivity

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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No H140-1

METRIC

G.W.P. 2175-08-00 LOCATION Coords: 4 757 691.1 N; 328 641.0 E ORIGINATED BY F.P.

DIST Central HWY 140 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.

DATUM Geodetic DATE September 12 & 13, 2011 CHECKED BY C.N.

ELEV / DEPTH	SOIL PROFILE DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH (kPa)	WATER CONTENT	GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
163.2 / 15.0	Silty clay, trace sand silt seams. Firm to stiff, Reddish brown, Moist to wet *(Continued)*	14	TW	-			
			FV		+2		
		15	SS	3		Wp–w–WL	0 1 49 50
		16	SS	3		w	
			FV		+2		
154.1 / 24.1	Clayey silt, sandy some gravel. Very stiff to hard, Reddish brown, Moist (TILL)	17	SS	31		w Wp–WL	15 32 38 15
		18	SS	50/3cm			
152.4 / 25.8	End of borehole						

Elevation scale: 163, 162, 161, 160, 159, 158, 157, 156, 155, 154, 153

Refusal on probable bedrock

* 2011 09 12/13

▽ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

Water Level Readings

Date	Depth (m)	Elev.
Sep.23/2011	9.2	169.0
Oct.17/2011	9.2	169.0

Piezometer Legend:

- Bentonite seal
- Portland / grout
- Filter sand
- 19mm dia. PVC screen
- Bentonite bottom

+7, ×5 : Numbers refer to Sensitivity

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

RECORD OF BOREHOLE No H140-2

METRIC

G.W.P. 2175-08-00 LOCATION Coords: 4 757 691.1 N; 328 660.0 E ORIGINATED BY F.P.

DIST Central HWY 140 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.

DATUM Geodetic DATE September 14 & 15, 2011 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	NUMBER	TYPE	"N" VALUES	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	GR SA SI CL
178.1	Ground Surface				178			
0.0	Topsoil							
177.6 / 0.5	Silty clay, with sand organic inclusions Stiff Brown Moist (FILL)	1	SS	11				
	Silty clay, trace sand clay layers. Very stiff to stiff, Mottled brown/grey, Moist	2	SS	15	177			
		3	SS	18		225		0 0 27 73
		4	SS	13	176	225		
		5	SS	11	175	175		
		6	SS	8	174	163		
	Reddish brown	7	SS	5	173			
			FV			+2		
		8	SS	5	172			
	silt seams. Firm to stiff	9	SS	3	171 / 170			0 2 53 45
			FV			+2		
		10	TW	-	169			
			FV		168	+2		
		11	TW	-	167			
			FV			+3		
	varved. Reddish brown/grey, Moist to wet	12	SS	2	166			
	silt seams	13	TW	-	165 / 164			
			FV			+3		
163.1	Cont'd							

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No H140-2

METRIC

G.W.P. 2175-08-00 LOCATION Coords: 4 757 691.1 N; 328 660.0 E ORIGINATED BY F.P.

DIST Central HWY 140 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.

DATUM Geodetic DATE September 14 & 15, 2011 CHECKED BY C.N.

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	GROUND WATER CONDITIONS	ELEVATION SCALE	SHEAR STRENGTH kPa	WATER CONTENT (%)	UNIT WEIGHT γ kN/m³	GR	SA	SI	CL
163.1 / 15.0	Silty clay, trace sand. Firm to stiff, Reddish brown/grey, Moist to wet *(Continued)*	14	SS	2		163				0	1	28	71
		15	SS	4		160							
			FV			159	+ 2						
		16	SS	2		157							
153.8 / 24.3	Clayey silt, sandy some gravel. Hard, Reddish brown, Moist (TILL)	17	SS	42		154				17	36	33	14
152.8 / 25.3	End of borehole. Refusal on probable bedrock												

* Borehole dry

+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; REMARKS & GRAIN SIZE DISTRIBUTION (%)

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No H140-3

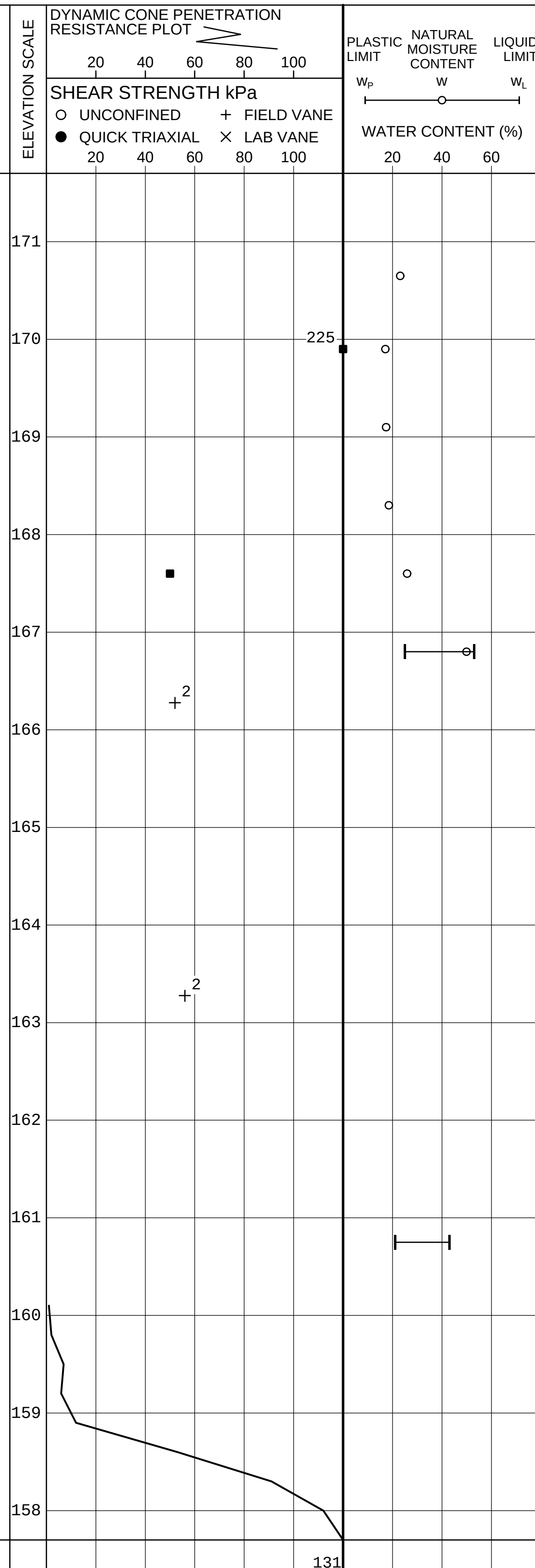
METRIC

G.W.P. 2175-08-00 LOCATION Coords: 4 757 872.2 N; 328 665.1 E ORIGINATED BY F.P.

DIST Central HWY 140 BOREHOLE TYPE C.F.S.S.A. and Dynamic Cone Penetration Test COMPILED BY G.D.

DATUM Geodetic DATE September 23, 2011 CHECKED BY C.N.

ELEV / DEPTH	SOIL PROFILE – DESCRIPTION	SAMPLE NUMBER	TYPE	"N" VALUES	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
171.7	Ground Surface				
0.0	225 mm thick asphalt over sand and gravel				
	Silty clay, with sand organic inclusions	1	SS	10	
	Silty clay trace sand, trace gravel Very stiff Reddish brown Moist (FILL)	2	SS	13	
		3	SS	19	
	Grey	4	SS	27	
168.0 / 3.7	Silty clay, trace sand clay layers Firm to stiff Reddish brown/ grey Moist to wet	5	SS	6	
		6	SS	4	0 1 21 78
			FV		
		7	SS	3	
			FV		
		8	SS	3	0 2 34 64
160.4 / 11.3	End of borehole Dynamic cone penetration test started Probable silty clay/ clayey silt (Till)				
157.7 / 14.0	End of dynamic cone penetration test				



GROUND WATER CONDITIONS; UNIT WEIGHT γ kN/m³

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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No H140-4

METRIC

G.W.P. 2175-08-00	LOCATION Coords: 4 757 919.1 N; 328 691.6 E		ORIGINATED BY F.P.
DIST Central	HWY 140	BOREHOLE TYPE Continuous Flight Hollow Stem Augers	COMPILED BY G.D.
DATUM Geodetic	DATE September 15 & 16, 2011		CHECKED BY C.N.

Column headings: SOIL PROFILE (ELEV / DEPTH, DESCRIPTION, STRAT PLOT) | SAMPLES (NUMBER, TYPE, "N" VALUES) | 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)

Soil Profile

ELEV / DEPTH	DESCRIPTION
178.7	Ground Surface
0.0	Topsoil, limestone inclusions (FILL) Moist
178.1 / 0.6	Silty clay, trace sand, organics to 3.0m. Very stiff, Mottled grey/brown, Moist
	Firm to stiff, Reddish brown/grey
	silt seams
	varved, Moist to wet
163.7	Cont'd

Samples

NUMBER	TYPE	"N" VALUES	SHEAR STRENGTH (kPa)	GRAIN SIZE GR SA SI CL
1	SS	18		
2	SS	13		
3	SS	14	188	
4	SS	15	225	
5	SS	13	138	0 2 46 52
6	SS	7		
7	SS	8		
8	SS	8		
9	SS	5		1 2 58 39
	FV		+2	
10	TW	-		
	FV		+2	
11	SS	6		
12	SS	10		
	FV		+3	
13	SS	2		0 1 48 51
	FV		+3	

Elevation scale: 178, 177, 176, 175, 174, 173, 172, 171, 170, 169, 168, 167, 166, 165, 164

$+^{7}$, $\times^{5}$: Numbers refer to Sensitivity

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

Ontario

Peto MacCallum Ltd. CONSULTING ENGINEERS

Foundation Design

RECORD OF BOREHOLE No H140-4

METRIC

G.W.P. 2175-08-00	LOCATION Coords: 4 757 919.1 N; 328 691.6 E		ORIGINATED BY F.P.
DIST Central HWY 140	BOREHOLE TYPE Continuous Flight Hollow Stem Augers		COMPILED BY G.D.
DATUM Geodetic	DATE September 15 & 16, 2011		CHECKED BY C.N.

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	Shear strength / Water content	GR SA SI CL
163.7						
15.0	Silty clay, trace sand silt seams. Firm to stiff, Reddish brown/ grey, Moist to wet *(Contined)*	14	TW	-	Water content ○ approx. 38%	
			FV		Field vane + approx. 55 kPa, sensitivity 3	
	trace gravel					
		15	SS	7	Water content ○ approx. 25%	
160.0 / 18.7	Clayey silt with sand, some gravel. Very stiff to hard, Reddish brown, Moist (TILL)					
		16	SS	97	Water content ○ approx. 22%, plastic–liquid limit range shown	12 30 40 18
154.3 / 24.4	End of borehole. Refusal on probable bedrock	17	SS	30/3cm		

Elevation scale: 163, 162, 161, 160, 159, 158, 157, 156, 155

* 2011 09 16

∇ Water level observed during drilling

■ Penetrometer test

Water Level Readings

Date	Depth (m)	Elev.
Sep.23/2011	9.5	169.2
Oct.17/2011	9.2	169.5

Piezometer Legend:

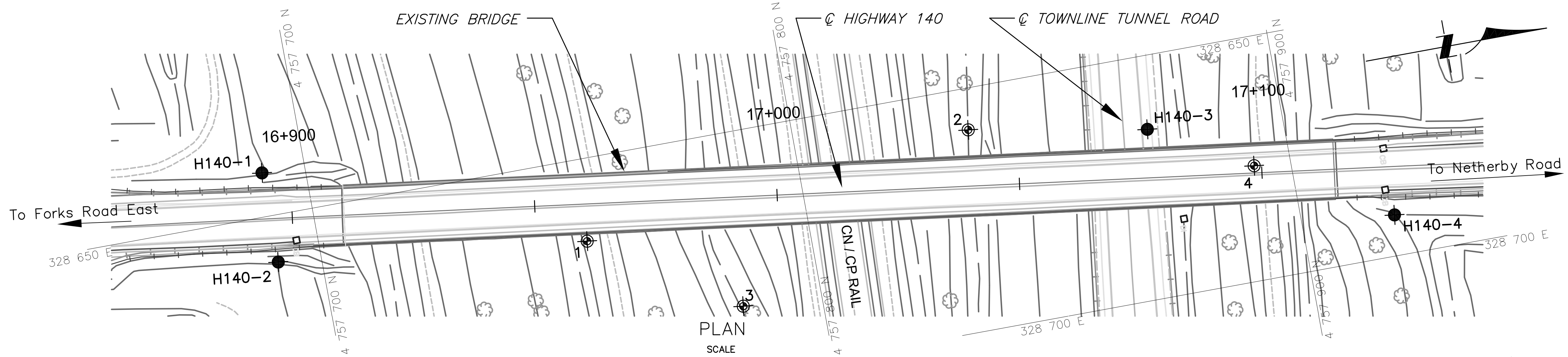
- Bentonite seal
- Portland / grout
- Filter sand
- 19mm dia. PVC screen
- Bentonite bottom

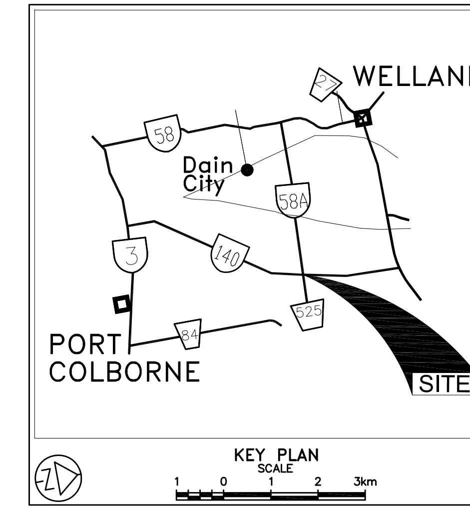
$+^7$, $\times^5$: Numbers refer to Sensitivity

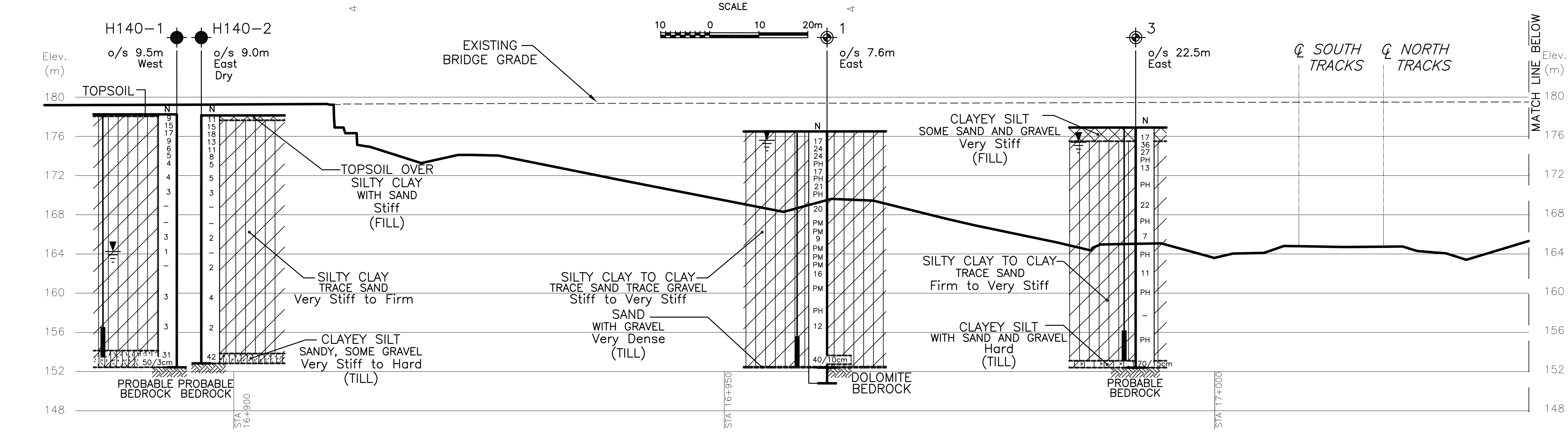
20, 15, 5, 10 (%) STRAIN AT FAILURE

CONT No	
GWP No 2175-08-00	
CNR/CPR OVERHEAD HIGHWAY 140 BOREHOLE LOCATIONS AND SOIL STRATA	SHEET

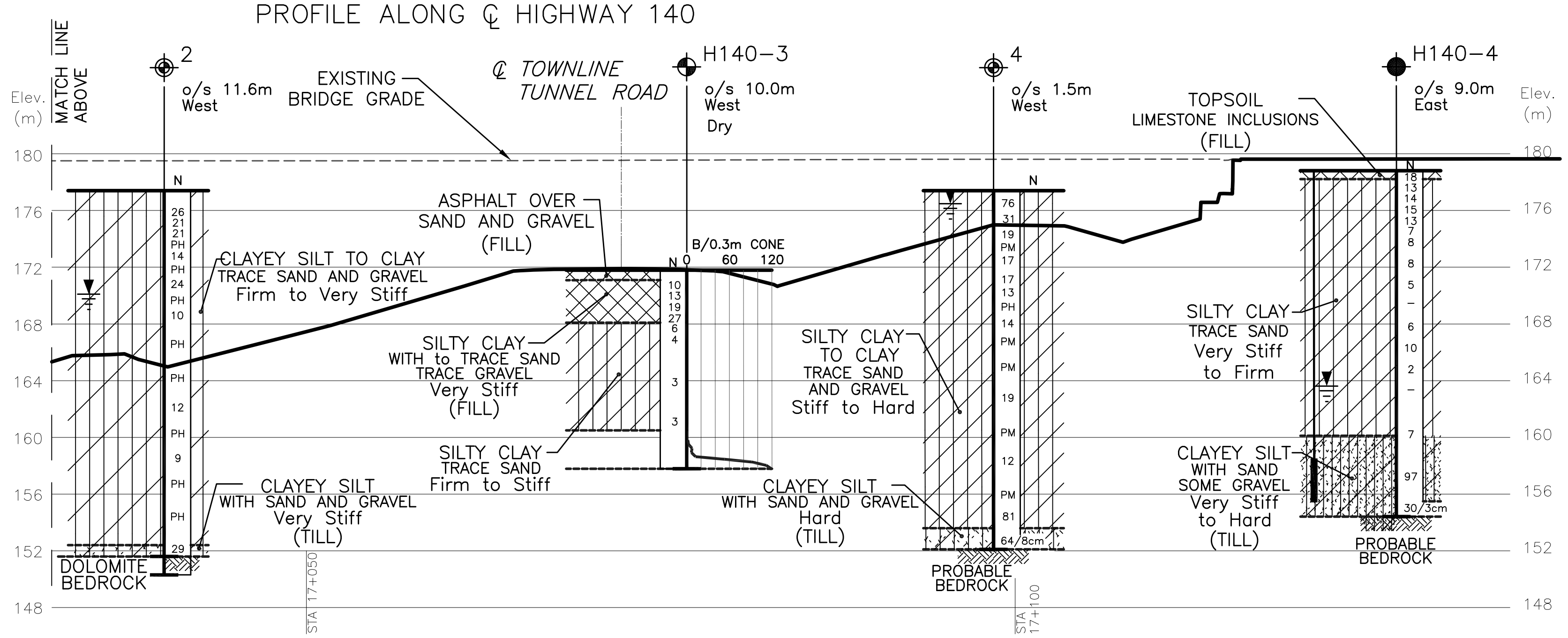
Peto MacCallum Ltd.
CONSULTING ENGINEERS





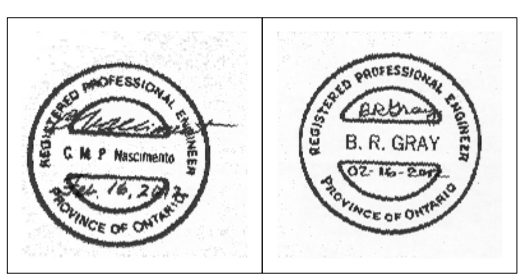


PROFILE ALONG ℄ HIGHWAY 140



PROFILE ALONG ℄ HIGHWAY 140 (Continued)

SCALE



NOTES:

1. BOREHOLES 1 TO 4 HAVE BEEN REPRODUCED FROM THE GEOCRES REPORT NO 30L14-36 DATED DECEMBER 1968. REFER TO THE LOGS OF BOREHOLES 1 TO 4 ATTACHED TO THE REPORT.
2. THE INFERRED STRATIGRAPHY REFERRED TO IN THIS REPORT IS BASED ON DATA FROM THESE BOREHOLES, SUPPLEMENTED BY GEOLOGICAL EVIDENCE. THE ACTUAL STRATIGRAPHY AT OTHER POINTS BETWEEN THE BORINGS MAY VARY FROM THAT SHOWN.
3. THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT AND THE RECORD OF BOREHOLE LOGS.
4. THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
5. DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.

LEGEND

- Borehole — Current Investigation
- Borehole and Cone — Current Investigation
- Borehole from previous investigation
- N — Blows/0.3m (Std. Pen Test, 475 J/blow)
- CONE — Blows/0.3m (60° Cone, 475 J/blow)
- WL at time of investigation September and October 2011
- Head ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES NORTHINGS	EASTINGS
H140-1	178.2	4 757 691.1	328 641.0
H140-2	178.1	4 757 691.1	328 660.0
H140-3	171.7	4 757 872.2	328 665.1
H140-4	178.7	4 757 919.1	328 691.6
1	176.5	4 757 754.5	328 666.9
2	177.4	4 757 835.9	328 658.5
3	176.9	4 757 783.7	328 685.9
4	177.4	4 757 892.5	328 676.4

– NOTE –

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

REVISIONS		
DATE	BY	DESCRIPTION

Geocres No. 30L14-55

HWY No 140			DIST Central
SUBM'D GD	CHECKED GD	DATE FEB. 16, 2012	SITE 34-232
DRAWN NA	CHECKED CN	APPROVED BRG	DWG H140-1

REF No. AECOM DRAWING: Hwy 140, CNR & CPR Overhead.dwg;

APPENDIX A

RECORD OF BOREHOLE SHEETS FROM PREVIOUS INVESTIGATION
(GEOCRES NO. 30L14-36)

FORM ...

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 1

FOUNDATION SECTION

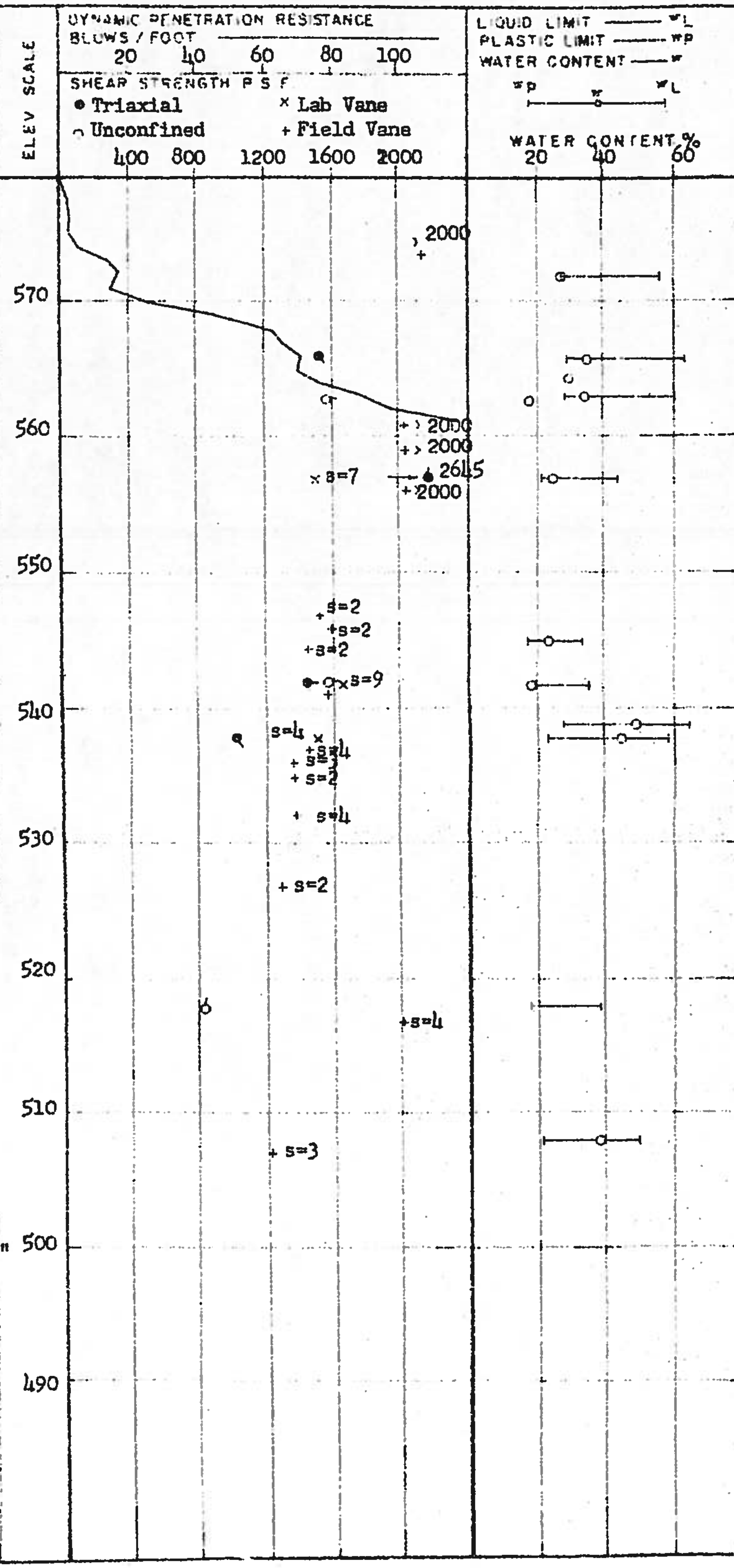
JOB 68-F-73	LOCATION Sta. 216+50 ℄ East Side Hwy. o/s 25' Rt.	ORIGINATED BY WH
W.P. 60-68-03	BORING DATE Oct. 17 - Nov. 1, 1968	COMPILED BY WH
DATUM Geodetic	BOREHOLE TYPE Cont. Flight auger & diamond drill	CHECKED BY

SOIL PROFILE

ELEV. DEPTH	DESCRIPTION
579.0	Ground Level
0.0	Silty clay to clay with trace of sand & gravel with occ. silt layers up to 3" thick throughout. Pocket of silty sand with gravel 3" thick at elev. 539
546.0 / 33.0	Clayey silt zone
540.0 / 39.0	Brown to reddish brown. Stiff to very stiff.
500.5 / 78.5	(Glacial Till) Very dense Sand with gravel & fragments of bedrock.
79.3	Dolomite bedrock with gypsum lenses. Occ. layers of calcareous shale. Sound.
494.6 / 84.4	End of Borehole

SAMPLES

NUMBER	TYPE	BLOWS / FOOT
1A	SS	17
2	SS	24
3	SS	24
4	TW	PH
5	SS	17
6	3" TW	PH
7	SS	21
8	TW	PH
9	SS	20
10	TW	PM
10A	TW	PM
11	SS	9
11A	TW	PH
12A	TW	PM
12	TW	PM
13	SS	16
14	TW	PM
15	TW	PH
16	SS	12
17	SS	40/4"
18	BXL RC	80% Rec.



BULK DENSITY γ P.C.F.
116
114
127.5
127
111
121

REMARKS

Gr.Sa.Si.Cl

576.

0 1 29 70

0 1 33 66

Piez.#2 Tip.elev.564

554.

0 1 22 77

Piez.#1 Tip Elev.49

DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO. 2

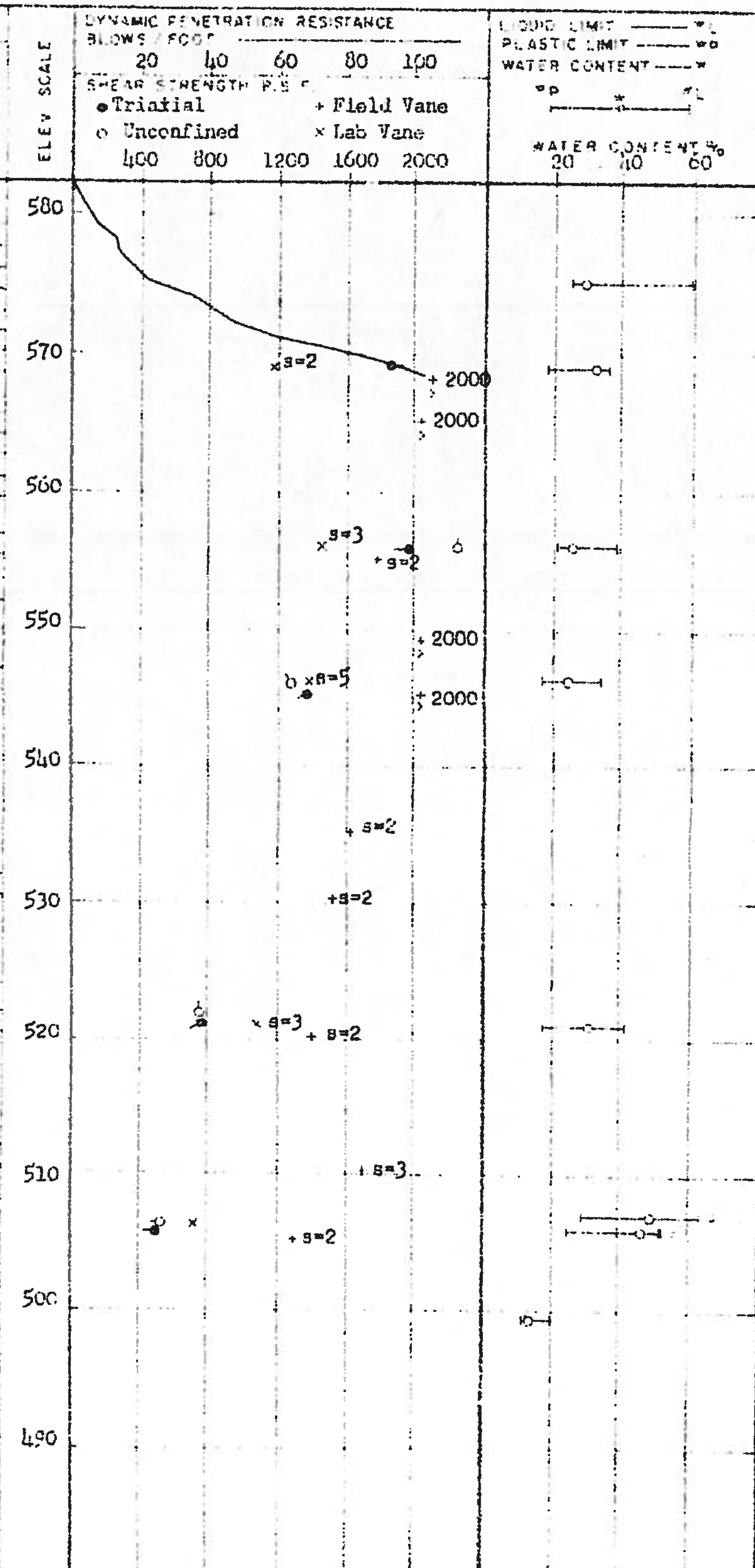
FOUNDATION SECTION

JOB 68-F-73 LOCATION Sta. 219+10 Ø East Side Hwy. o/s 38' Lt. ORIGINATED BY WH

W.P. 60-68-03 BORING DATE Oct. 23-29, 1968 COMPILED BY WH

DATUM Geodetic BOREHOLE TYPE Cont. flight auger & diamond drill CHECKED BY

ELEV. DEPTH	DESCRIPTION	NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY P.C.F.	REMARKS Gr.Sa.Si.Cl
582.0	Ground Level					
0.0	Clayey silt to clay with trace of sand & gravel Occ. very thin grey silt seams containing clear gypsum crystals above elev. 559. Brown to reddish brown Firm to very stiff	1	SS	26		
		2	SS	21		0 1 (99)
		3	SS	21		
		4	TW	PH	116	
		5	SS	14		
		6	TW	PH		
		7	SS	24		
		8	TW	PH	128	558
		9A	SS	10		
		10	TW	PH	125	
		11	TW	PH		
		12	SS	12		
		13	TW	PH	121	
		14	SS	2		
510.0 72.0	Occ. thin sand seams up to ½" thick	15	TW	PH	112 108	
500.0 82.0	Glac. Till-clayey silt with sand, gravel and gypsum. Very stiff.	16	TW	PH		Piez. #3 Tip Elev. 500
497.5 84.5	Dolomite bedrock with a bed of gypsum 2' thick Sound. Grey.	17	SS	29		12 34 39 15
493.0		18	BXL RC	76% Rec		
89.0	End of Borehole					



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

DEPARTMENT OF HIGHWAYS ONTARIO
MATERIALS & TESTING DIVISION

RECORD OF BOREHOLE NO 3

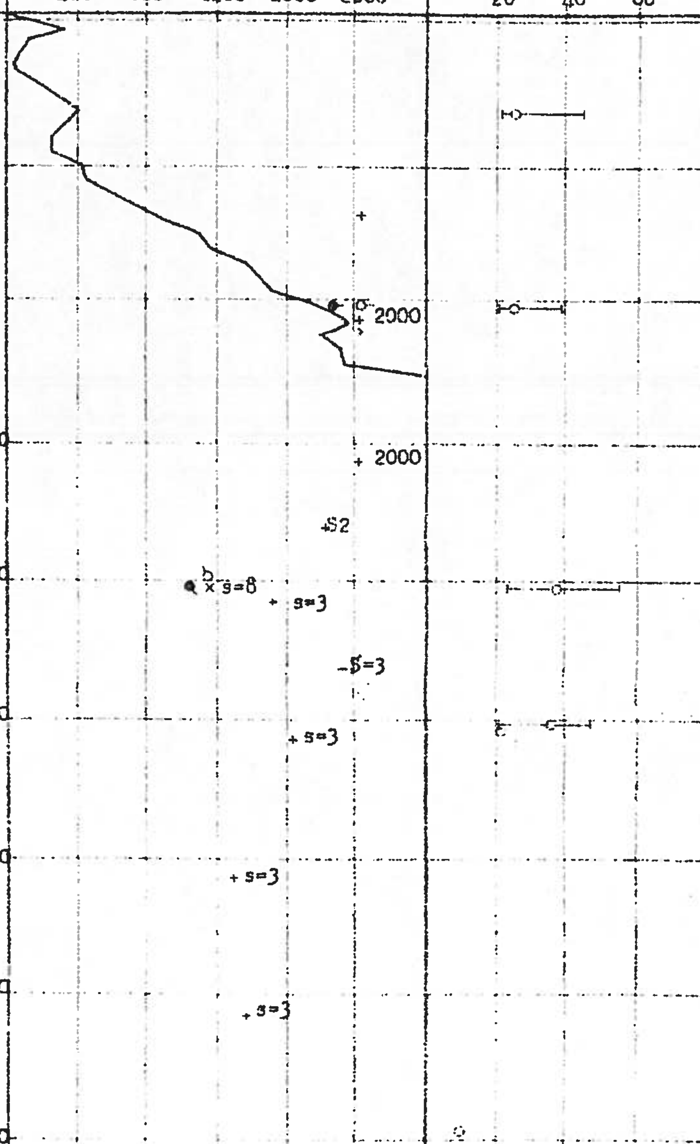
FOUNDATION SECTION

JOB 68-F-73
W P 69-68-03
DATUM Geodetic
LOCATION Sta. 217+53 ℄ East Side Hwy. o/s 73' Rt.
BORING DATE Oct. 28-29, 1968
BOREHOLE TYPE Cont. Flight Auger
ORIGINATED BY WH
COMPILED BY WH
CHECKED BY

ELEV DEPTH	SOIL PROFILE DESCRIPTION	SAMPLES NUMBER	TYPE	BLOWS/FOOT	ELEV SCALE	BULK DENSITY P.C.F.	REMARKS Gr.Sa.Si.Cl.
580.5	Ground Level				580		
0.0	Roadway Fill Clayey silt with some sand & gravel. V.Stiff. dark grey.						
576.0 4.5	Silty clay to clay with trace of sand occ. very thin grey silt seams containing clear gypsum crystals above elev. 554.	1	SS	17			▽ 575.5
		2	SS	36			
		3	SS	27	570		
		4	TW	PH			
		5	SS	13			
		6	TW	PH	560	127	0 1 54 45
		7	SS	22			
		8	3" TW	PH	550		
		9	SS	7			
		10	3"TW	PH	540	125 111	
535.0 45.5	Occasional silt layers up to 3" thick.	11	SS	11			
		12	3"TW	PH	530		0 0 90 10 (silt seam)
525.0 55.5	Brown to reddich brown	13	SS	-	520		
	Firm to very stiff	14	TW	PH	510		
502.5 78.0	Glacial till - clayey silt with sand & grav. Hard. Brown.	15	SS	70/6"	500		
500.0 80.5	End of borehole Probable Bedrock						

DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT 20 40 60 80 100
SHEAR STRENGTH P.S.F. ● Triaxial ○ Unconfined + Field Vane × Lab Vane 400 800 1200 1600 2000

LIQUID LIMIT w_L PLASTIC LIMIT w_P WATER CONTENT w
WATER CONTENT % 20 40 60



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

HWY ... [illegible]

OFFICE REPORT ON SOIL EXPLORATION

DEPARTMENT OF HIGHWAYS - ONTARIO

MATERIALS & TESTING DIVISION

FOUNDATION SECTION

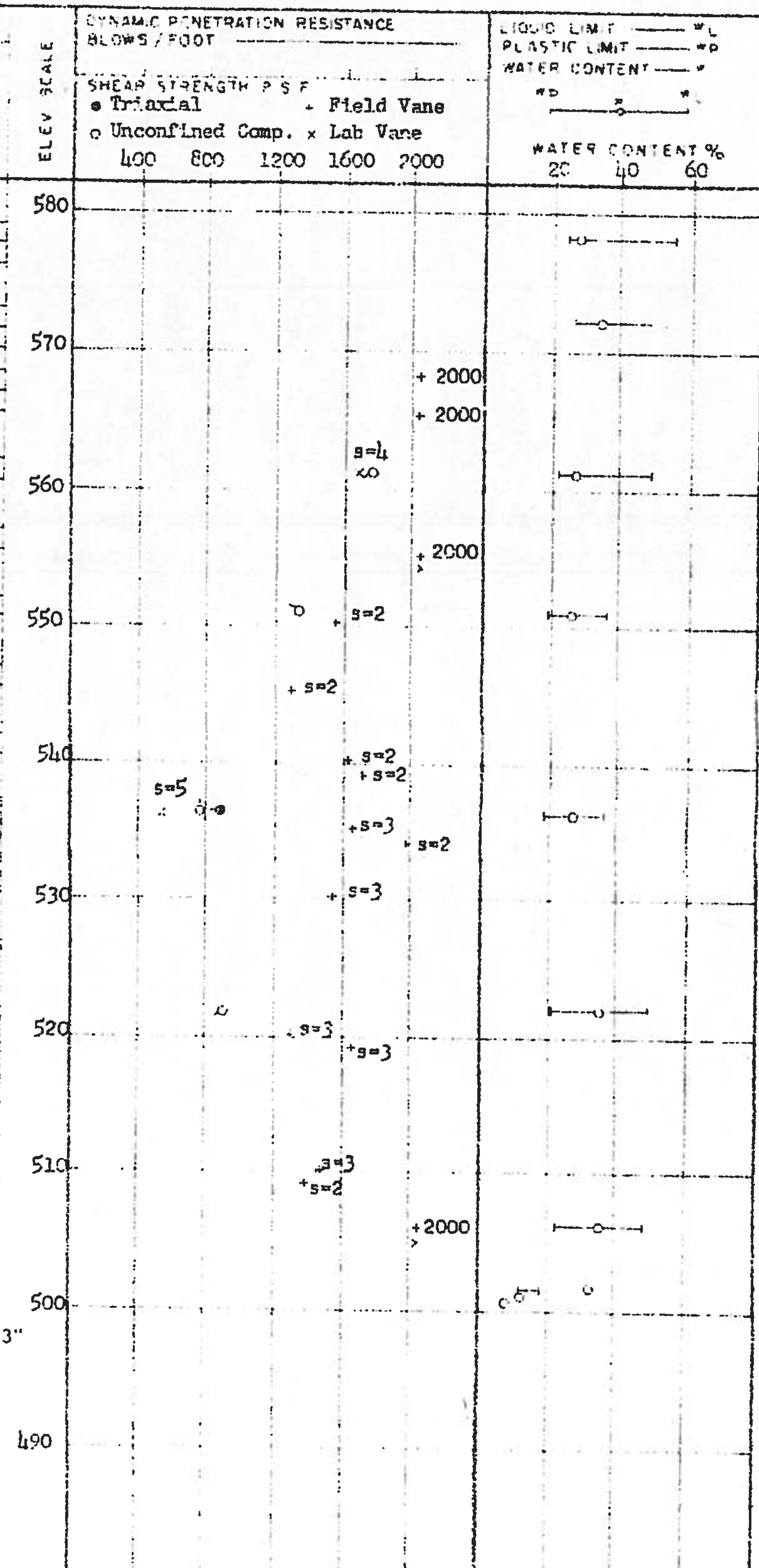
RECORD OF BOREHOLE NO 4

JOB 68-F-73 LOCATION Sta. 221+00 ℄ East Side Hwy. o/s 5' Lt. ORIGINATED BY WH

W.P. 60-68-03 BORING DATE Oct. 30-31, 1968 COMPILED BY WH

DATUM Geodetic BOREHOLE TYPE Diamond Drill - NX Casing CHECKED BY

ELEV / DEPTH	DESCRIPTION	SAMPLE NUMBER	TYPE	BLOWS / FOOT	BULK DENSITY γ P.C.F.	REMARKS Gr.Sa.Si.Cl
582.0	Ground Level					
0.0	Silty clay to clay with trace of sand & gravel	1	SS	75		▼579
		2	SS	[illegible]		0 1 30 69
		3	SS	19		
	occ. very thin grey silt seams containing clear gypsum crystals above elev. 560	4	TW	PM		
		5	SS	17		
		6	SS	17	124	
	Brown to reddish brown	6A	SS	13		
	Stiff to hard	7	TW	PM	127	
		8	SS	14		
		9	TW	PM		
534.0		10	TW	PM	123	0 2 61 37
48.0	Occ. layers of silt up to 3" thick	11	SS	12		
520.0		12	TW	PM	117	
62.0		13	SS	12		
504.0		14	TW	PM		
78.0	Glacial till-clayey silt with sand & grav. Hard. Brown to grey.	15	SS	81		
497.3		16	SS	[illegible]/3"		
84.7	End of Borehole Probable Bedrock					



DEFECTS IN NEGATIVE DUE TO CONDITION OF ORIGINAL DOCUMENT

PRELIMINARY FOUNDATION DESIGN REPORT

for

CNR / CPR OVERHEAD
HIGHWAY 140, SITE 34-232
GWP 2175-08-00
CITY OF WELLAND, ONTARIO

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
Fax: (416) 785-5120
Email: toronto@petomaccallum.com

Distribution:
5 cc: AECOM for distribution to MTO Project Manager + 1 digital copy (PDF)
2 cc: AECOM for distribution to MTO, Pavements and Foundations Section + 1 digital copy (PDF) and Drawing (AutoCAD)
2 cc: AECOM + 1 digital copy
1 cc: PML Kitchener
1 cc: PML Toronto

PML Ref.: 11TF023A-3
Index No.: 038FIDR
GEOCRES No.: 30L14-55
March 15, 2012

PART B – PRELIMINARY FOUNDATION DESIGN REPORT

PART B
PRELIMINARY FOUNDATION DESIGN REPORT
for
CNR / CPR Overhead
Highway 140, Site 34-232
GWP 2175-08-00
City of Welland, Ontario

---

6. ENGINEERING RECOMMENDATIONS

6.1 General

Part B of the report provides preliminary foundation engineering comments and recommendations regarding the design and construction of the foundations, abutments and approach embankments for the proposed rehabilitation or replacement of the CNR / CPR overhead located on Highway 140 in the City of Welland, Ontario.

The existing overhead is a five span structure with a total length of 204 m and a width of 12 m, accommodating two lanes of traffic (Drawing 'Hwy 140 – CNR / CPR Overhead' prepared by AECOM in June 2011). The approach embankments to the overhead are up to 9.5 m high. The abutments and piers are founded on steel pipe piles embedded into bedrock.

The road grades on Highway 140 and Townline Tunnel Road at the overhead location are at elevation 179.3 to 179.6 and elevation 171.8 respectively. The top of rail on the CNR / CPR tracks is at elevation 165.1. No details of the final rehabilitation / replacement plans of the structure were available at the time of preparation of this report. However, it is anticipated that the existing vertical and horizontal alignments of Highway 140 in the vicinity of the bridge will remain unchanged according to a preliminary plan and profile drawing provided by AECOM on February 21, 2012.

The recommendations in this report are preliminary and based on PML's interpretation of the factual information obtained from a limited number of boreholes and outlined in Part A of the report as well as from the previous foundation investigations at the site (Geocres No. 30L14-36 dated December 1968, Geocres No. 30L-45 dated July 1972 and Geocres No. 30L14-50 dated

August 2009). The recommendations are only provided for planning purposes and are not to be relied upon for detail design. The Design-Build contractor shall supplement the information as needed to meet the requirements for detail design and is solely responsible for selecting appropriate foundation alternatives.

All elevations in this report are expressed in metres. A list of the standard specifications referenced in the report is provided in Table 1, attached.

6.2 Foundations

6.2.1 General

Based on the preliminary data and sufficient pile resistance available, reuse of the existing piles embedded into bedrock is considered to be feasible, subject to a structural analysis including verification of adequate foundation resistance. In case the overhead is to be replaced, steel H-piles driven to refusal on bedrock may be used.

Since large spread footings would be required for foundations on the relatively weak cohesive soils that are present at the site, it is considered that these spread footings would be inducing long-term foundation settlements. In view of the settlement sensitive foundation for the multispan bridge, it is not recommended to support a replacement structure on spread footings placed on either the native soils or a pad of engineered fill.

Drilled cast-in-place concrete caissons bearing on the bedrock to support a new structure are considered to be practical at this site only if the difficulties caused by the dewatering of the high groundwater table (about 16 m above the caisson base levels) are taken into account. In addition, the Design-Build contractor should consider the potential presence of cobbles and boulders in the glacial till above the bedrock which may cause delays during the caisson installation.

The seismic site coefficient for the stratigraphic conditions at this site is 1.0 (soil profile Type I, Canadian Highway Bridge Design Code (CHBDC) 2006 Edition, clause 4.4.6).

Further preliminary comments and recommendations for design of the foundations are provided in the following sections.

6.2.2 Piles

As indicated in the previous section, supporting the abutments and piers of the overhead on the existing piles is considered to be feasible. The as-built drawings indicate that the abutments and piers are respectively founded on 0.61 and 0.76 m diameter steel pipe piles filled with concrete and embedded into bedrock. Having initial lengths of 0.61 and 0.76 m, the rock sockets are step tapered for another 1.5 and 1.8 m to about 0.3 m diameter for the abutment and pier piles, respectively. Refer to the as-built drawings in Appendix FDR-A for more details.

The factored axial resistance at ULS of the existing piles driven into bedrock is assessed to be about 1600 kN for the 0.61 m diameter abutment piles and 2500 kN for the 0.76 m diameter pier piles. The geotechnical resistance at SLS should be considered the same as the factored axial resistance at ULS for structural computation purposes because the bedrock founding medium is unyielding.

From a foundation engineering perspective, use of end-bearing piles driven to bedrock is considered to be the preferred means of supporting the foundation loads of a replacement structure supported on new foundations. Further, construction of integral abutments supported on steel H-piles is considered to be feasible (refer to MTO Report SO-96-01 for further details).

The H-piles should be driven to refusal on bedrock anticipated at depths of 25.3 to 25.8 m (elevation 152.4 to 152.8) at the south abutment and 24.4 m depth (elevation 154.3) at the north abutment. It is noteworthy that the bedrock was contacted / inferred at elevation 151.6 to 152.4 in boreholes 1 to 4 drilled during the previous investigation. For preliminary design purposes, the founding levels for driven piles at the pier locations could be linearly interpolated between the founding levels at the abutments. These levels should be confirmed during detail design.

The recommended factored axial resistance at ULS for the HP 310x110 pile section founded on the underlying bedrock is 2000 kN. Considering the bedrock to be unyielding the geotechnical resistance at SLS is not expected to be governed by settlement criteria. For structural computation purposes, the geotechnical resistance at SLS may be the same value as the factored geotechnical resistance at ULS of 2000 kN.

The piles will have to be driven through compressible clayey soils 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 and, consequently, application of negative skin friction to the pile axial resistance at ULS could be ignored if the road grade will be maintained and the embankments will not be widened. However, it is proposed to raise the road grade by some 70 mm to accommodate a thicker bridge deck, with a possible further raise to increase the longitudinal grade of the bridge to the standard 0.3%. For preliminary design purposes, the capacity of new HP 310x110 piles at the abutments should allow for a negative skin friction load of 200 kN. This value has been estimated for preliminary design and should be checked during detail design based on the actual construction by the Design-Build contractor. If mitigation measures such as use of lightweight fill or EPS are undertaken to preclude additional settlement of the embankment, negative skin friction will not need to be considered.

Pile caps should be provided with at least 1.2 m of earth cover or equivalent thermal insulation as protection against frost action. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

The piles should be installed and monitored in accordance with the requirements of MTO OPSS 903.

6.3 Lateral Earth Pressure

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

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

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

The seismic site coefficient for the conditions at this site was provided in Section 6.2.1.

Free-draining granular material should be used as backfill behind the wall. The following parameters are recommended for preliminary design (including those for the clay fill retained by the existing abutments):

PARAMETERS	GRANULAR A or GRANULAR B TYPE II	CLAY FILL
Internal Friction Angle, Ø (degrees)	35	28
Unit weight, γ (kN/m^3)	22.8	19.0
Coefficient of Active Earth Pressure, K_a	0.27	0.36
Coefficient of Earth Pressure At Rest, K_o	0.43	0.53
Coefficient of Passive Earth Pressure, K_p	3.69	2.77

Refer to MTO Report SO-96-11 for procedures to determine the earth pressure coefficient to be employed in design of integral abutments. 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 magnitude of the passive resistance is dependent on the actual lateral movement of the structure towards the retained soil. Refer to Figure C6.16 of the CHBDC for this computation. 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 subdrain system (SP 405F03) and/or weep holes (OPSD 3190.100) should be installed to minimise the build-up of hydrostatic pressure behind the wall.

6.4 Approach Embankments

The scope of work for this preliminary study did not require that boreholes be put down for the approach embankments to the overhead. It is inferred that both embankments are founded on the very stiff to firm silty clay. Since the approach embankment areas have been preloaded by the existing fill for a considerable period of time, consolidation of the clayey deposit is likely to have taken place and no settlement is expected because the road grade will remain unchanged as currently planned. However, if it is planned to raise the road grade on the existing alignment, subsurface investigations should be carried out at these locations for detail design and to assess the condition of the existing fill within the approaches and estimate the magnitude of possible settlements.

Any new possible approach embankments or widenings should be designed and constructed in accordance with OPSD-200.010, 202.010, 208.010 and SP 206S03. Based on the historical data, the side slopes of the approach embankments will be stable where inclined no steeper than 3H:1V for earth fill. It is noted that the previous failures of the slopes of the existing approach embankments required remedial measures and slope flattening to 3H:1V.

It is considered feasible to backfill a possible new abutment structure constructed between the existing abutment and piers using granular materials. The magnitude of the "consolidation" of the granular fill depends on the workmanship employed by the contractor and, if placed in 200 mm thick lifts compacted to 100% of the standard Proctor maximum dry density in accordance with the requirements of SP 206S03 and OPSS 501 (Method A), should be in the order of 20 to 25 mm. These estimated total settlements of new approach fill surface near the abutments should be essentially complete within 3 to 4 months after placement of the fill.

The settlement analysis carried out for the previous investigation at the site (Geocres No. 30L14-36) showed that maximum total settlements due to a possible imposed embankment loading were expected to be 250 mm at possible new abutment locations. Settlements of similar magnitude should be anticipated if the abutments of the possible replacement structure are relocated closer to the CNR / CPR alignment and the existing spans at the abutments are backfilled. The period for 90% completion of consolidation in this scenario is preliminarily estimated to be 20 to 25 years. These values and timelines should be determined during detail design.

The final design that will be prepared and constructed by the Design-Build contractor may incorporate changes to the structures spans and embankment grades and slopes. It is recommended that the contract incorporates a requirement for the Design-Build contractor to carry out a displacement monitoring program as outlined in Section 6.5.3 of this report.

6.5 Construction Considerations

6.5.1 Excavation

The fill and native silty clay encountered in the boreholes are classified as Type 3 soils according to the Occupational Health and Safety Act (OHSA) criteria. Temporary cut slopes 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.

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

6.5.2 Groundwater Control

Groundwater was at 15.4 m depth (elevation 162.8) in borehole H140-1 upon completion of drilling on September 13, 2011. The piezometric water levels in boreholes H140-1 and H140-4 were at a depth of 9.2 m (elevation 169.0 and 169.5) on October 17, 2011. The proposed works should not be affected by the groundwater at the site.

Taking into account the low permeability characteristics of the clayey soils at the site, it is considered that seepage from soil and surface water run-off that enters possible excavations for new pile caps should be readily handled by conventional sump pumping techniques.

Groundwater conditions should be further assessed during detail design.

6.5.3 Monitoring Program

It is recommended that the tenders require that the Design-Build contractor prepares and submits a monitoring program of potential movements of the existing substructure, side slopes and/or ground settlement consistent with the proposed design for the project. At least 60 days before the start of the construction, the monitoring program shall be submitted for review and approval of the MTO Pavements and Foundations Section and the Structural and Geotechnical consultants to allow for the installation of the instrumentation and to record baseline readings.

7. ADDITIONAL STUDIES

The recommendations in this report are considered to be suitable for planning and preliminary design purposes only and should not be used for detail design. A detail foundation investigation will be required at the structure location during the detail design phase of the project.

The following items should be considered for the detail design studies.

1. Carry out the complete scope of detailed field investigations at the structure site. Incorporate in detail design the appropriate data from all boreholes drilled.

2. Conduct additional investigation with appropriate laboratory testing within the footprint of the approaches for settlement and stability analyses in case abandoned bridge spans will be infilled.

3. Evaluate the need to cut temporary slopes flatter than 45° to the horizontal for slope stability purposes.

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., MTO Designated Principal Contact. Mr. C.M.P. Nascimento, P.Eng., Project Manager, conducted an independent review of the report.

Sincerely

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

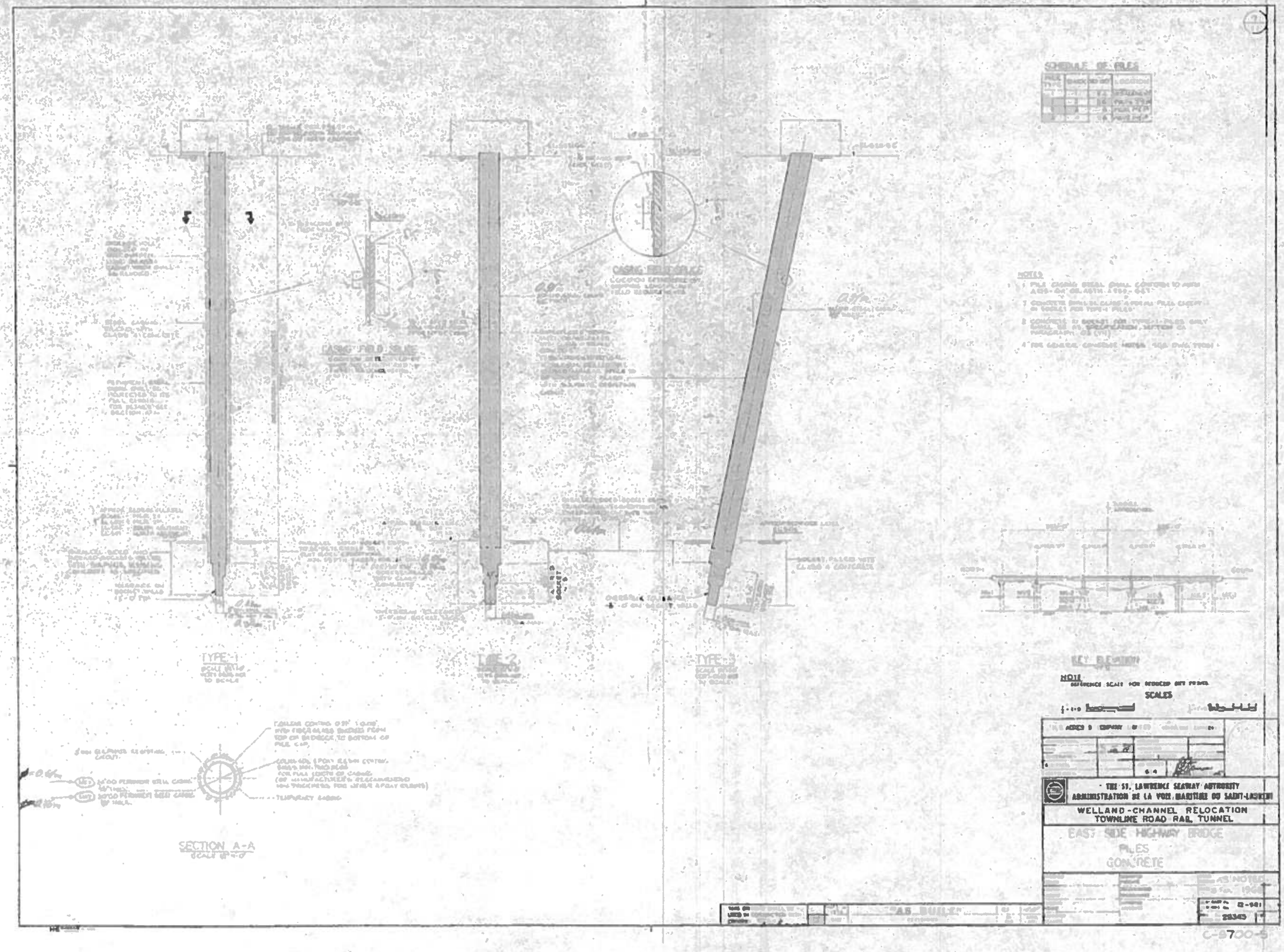
TABLE 1

LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

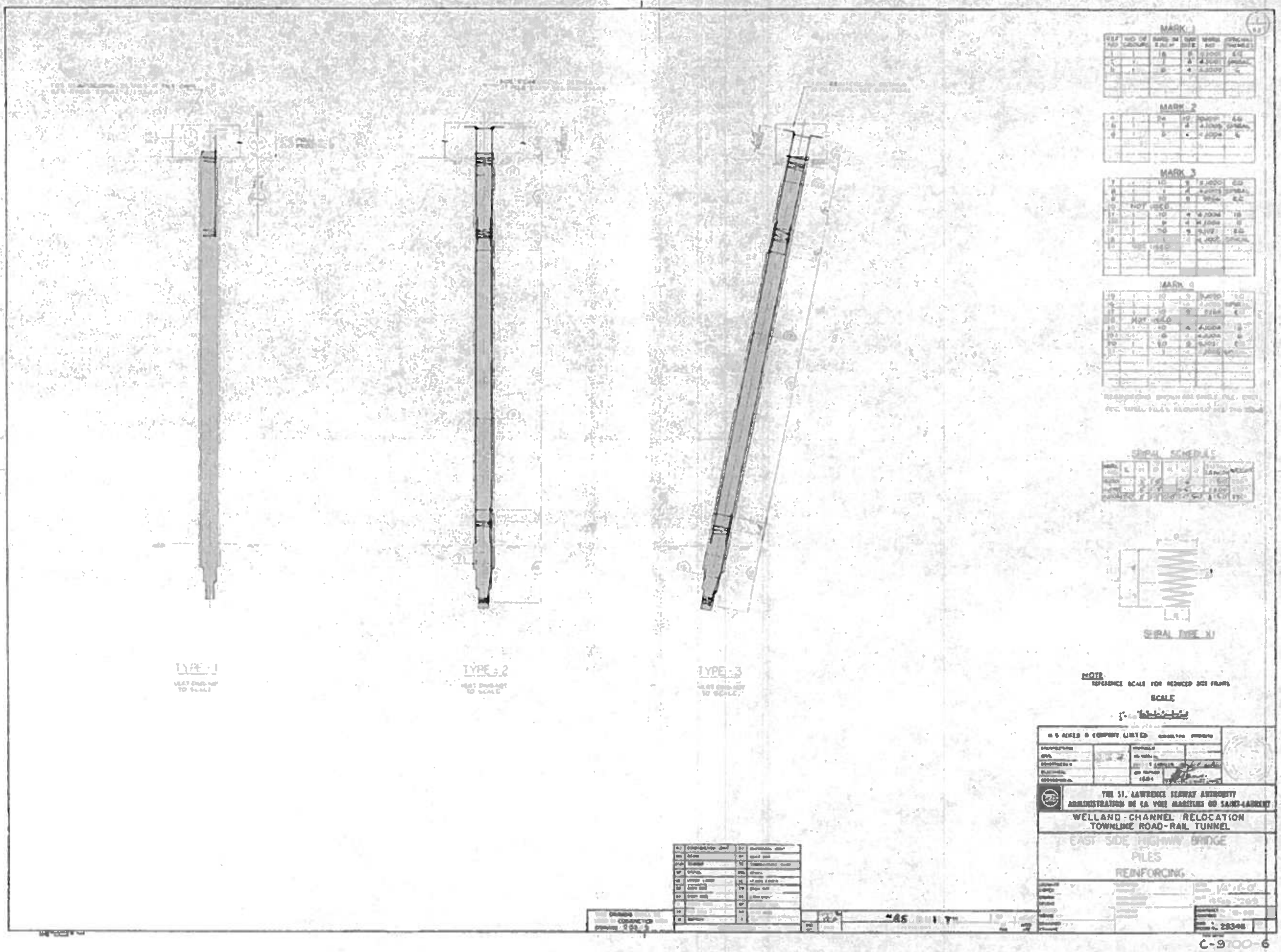
DOCUMENT	TITLE
OPSS 501	Construction Specification for Compacting
OPSS 903	Construction Specification For Deep Foundations
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
OPSD 200.010	Earth/Shale Grading – Undivided Rural
OPSD 202.010	Slope Flattening Using Excess Material on Earth or Rock Embankment
OPSD 208.010	Benching of Earth Slopes
OPSD 3190.100	Retaining Wall and Abutment Wall Drain Detail

APPENDIX FDR-A

As-Built Pile Foundation Drawings



SCHEDULE OF PILES
NOTES
TYPE-1
TYPE-2
TYPE-3
SECTION A-A
KEY ELEVATION
NOTE
SCALES
THE ST. LAWRENCE SEAWAY AUTHORITY
ADMINISTRATION DE LA VOIE MARITIME DU SAINT-LAURENT
WELLAND-CHANNEL RELOCATION
TOWNLINE ROAD RAIL TUNNEL
EAST SIDE HIGHWAY BRIDGE
PILES
CONCRETE
AS NOTED
1968
"AS BUILT"
28343
C-9700-5


MARK 1
MARK 2
MARK 3
MARK 4
SPIRAL SCHEDULE
SPIRAL TYPE XI
TYPE 1
TYPE 2
TYPE 3
NOTE
SCALE
THE ST. LAWRENCE SEAWAY AUTHORITY
ADMINISTRATION DE LA VOIE MARITIME DU SAINT-LAURENT
WELLAND-CHANNEL RELOCATION
TOWNLINE ROAD-RAIL TUNNEL
EAST SIDE HIGHWAY BRIDGE
PILES
REINFORCING
"AS BUILT"
C-9700-6