



**PRELIMINARY FOUNDATION INVESTIGATION AND DESIGN REPORT
for**

**CNR OVERHEAD
HIGHWAY 58, SITE 34-111
GWP 2175-08-00
CITY OF WELLAND, ONTARIO**

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PML Ref.: 11TF023A-1
Index No.: 034FIDR
GEOCRES No.: 30L14-53
March 15, 2012



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PART A
PRELIMINARY FOUNDATION INVESTIGATION REPORT
for
CNR Overhead
Highway 58, Site 34-111
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 overhead located on Highway 58 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 structure carries the Highway 58 traffic over the CNR tracks at approximate Sta. 15+678, Highway 58 chainage, and is a three span structure of some 30 m overall length and 9 m width, accommodating two lanes of traffic. According to the Geocres data referred to below, the abutments and piers of the structure rest on spread footings founded at elevation 175.0. The foreslopes of the about 9 m high approach embankments are supported by 4.4 m high retaining walls built along the CNR alignment and founded at elevation 177.4.

Part A of the report provides preliminary subsurface information pertaining to the proposed structure and approach embankments within about 20 m of the abutments, is considered to be suitable for planning and preliminary design purposes but 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.

Information from previous foundation investigations carried out at the site (Geocres No. 30L14-24 dated March 1974 and Geocres No. 30L14-46 dated November 1998) has also been used in the preparation of this report. Copies of the relevant borehole logs are attached in Appendix A. This report supersedes all other reports for the purpose of this project.



2. SITE DESCRIPTION AND GEOLOGY

The site is located on the Highway 58 alignment at the crossing of the CNR tracks near the southern boundary of the City of Welland in the Regional Municipality of Niagara. Highway 58 is oriented roughly in the south-north direction at the structure location.

Land use in the vicinity of the site is mainly agricultural. Residences exist along both sides of Forks Road that runs parallel to the CNR tracks approximately 200 m south of the site.

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 30 m.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out on September 26 and 27, 2011 and comprised borehole H58-1 drilled at the toe of the south embankment to 31.1 m depth at the location shown on Drawing H58-1, attached. A dynamic cone penetration test was performed from the bottom of the borehole and terminated at 32.2 m upon meeting refusal on probable bedrock. One borehole planned to the north of the railway tracks has not been drilled in view of inaccessibility (no permission to enter adjacent properties or to cross the railway tracks). Two previous boreholes 1 and 2 advanced from the top of the embankments during the previous investigation (Geocres No. 30L14-24 dated March 1974) were used to supplement the information obtained from the new borehole. The previous and new subsurface data was consistent and was considered to be adequate for the preliminary recommendations in the report.

The location of and ground surface elevation at the borehole was established in the field by Peto MacCallum Ltd. using the following benchmark (BM) for vertical reference:

SP-BM 620: East end of concrete curb
approximately 180 m south of Forks Road
and 20 m east of the centreline of Highway 58



Elevation 176.592 m (geodetic).

The borehole was advanced using continuous flight hollow stem augers, powered by a truck-mounted CME-75 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 actual values due to sample disturbance and soil layering. Refer to Subsection 4.2 in this report for further details.

The groundwater conditions at the borehole location 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, a piezometer consisting of a 19 mm dia. PVC pipe slotted over the bottom 6.1 m was installed in the borehole 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 log. The water level in the piezometer was measured on October 17, 2011.

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, 4 Atterberg limits tests and 5 grain size distribution analyses were carried out on selected soil samples, with the results presented in Figures H58-PC-1 to H58-PC-3 and Figures H58-GS-1 to H58-GS-4 respectively as well as on the corresponding Record of Borehole sheet.



4. SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheet for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations, standard 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 sheet.

The borehole locations and stratigraphic profile prepared from the borehole data are shown on Drawing H58-1. 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.

Boreholes 1 and 2 advanced from the top of existing embankments during the previous investigation revealed 1.2 m of gravel fill over 8.9 and 8.6 m of very stiff to hard silty clay fill, respectively, for a total thickness of embankment fill of 10.1 and 9.8 m. It is assessed that the inclination of the embankment slopes is about 2.5H:1V (horizontal to vertical).

The subsurface stratigraphy revealed in the boreholes drilled at the site is consistent, generally comprising a surficial 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 32.2 m (elevation 145.3) in borehole H58-1 put down during the current investigation. The piezometric water level in borehole H58-1 was at 10.9 m (elevation 166.6) on October 17, 2011.

It is noted that a 1.1 m thick sand layer was found above the clayey silt till deposit in one borehole drilled about 200 m to the south of the CNR site (at Forks Road). The presence of this stratum should be checked at this site for detail design purposes.

The strata encountered in the current borehole H58-1 are summarised below.



4.1 Fill

Surficial fill was present in borehole H58-1. Composed of sand and gravel, the fill was 300 mm thick and extended to elevation 177.2.

Overlain by the sand and gravel fill at 0.3 m (elevation 177.2) was silty sand fill. This unit was 400 mm in thickness and was penetrated at 0.7 m (elevation 176.8).

The fill was compact in relative density (SPT-'N' value of 14) and had a moisture content of about 9%.

4.2 Silty Clay

Directly below the fill at 0.7 m depth (elevation 176.8) 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 125 to 188 kPa) and firm to stiff underneath (below elevation 173.8). 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 40 to 54 kPa (soil sensitivity of 2 to 3). One representative penetrometer test on a sample of the silty clay in this zone indicated a slightly lower shear strength of 38 kPa which was attributed to sampling disturbance. The deposit was 25.8 m thick and penetrated at 26.5 m (elevation 151.0).

The results of Atterberg limits testing and grain size distribution analyses conducted on 3 cohesive samples of the deposit are presented in Figures H58-PC-1, H58-PC-2 and H58-GS-1, H58-GS-2 respectively. The clay unit had a liquid limit of 54 to 55, plastic limit of 26 to 27, their plasticity index being 27 to 29. The liquid and plastic limits of the silty clay were 45 and 23 respectively, thus giving the plasticity index of 22. The moisture content of the deposit ranged from 19 to 46%.

4.3 Clayey Silt Till

Underlying the silty clay at 26.5 m (elevation 151.0) was a cohesive deposit of clayey silt till. Containing silt layers, the very stiff to hard till deposit had a total thickness of 5.7 m and extended to probable bedrock contacted at 32.2 m (elevation 145.3).



The results of Atterberg limits testing and grain size distribution analyses performed on 2 samples of the deposit are presented in Figures H58-PC-3, H58-GS-3 and H58-GS-4. The clayey silt till had a liquid limit of 16 and a plastic limit of 12, thus giving the plasticity index of 4. The moisture content of the till deposit ranged from 9 to 14%.

4.4 Bedrock

In borehole H58-1 drilled near the south abutment, bedrock was inferred by refusal to dynamic cone penetration below the clayey silt till at 32.2 m (elevation 145.3). Since the bedrock level is consistent with the findings in the boreholes advanced at Forks Road, some 200 m south of the CNR overhead site, and considering the typical relief of Dolostone bedrock, it is anticipated that the bedrock level under the north abutment will be within 1 m of the bedrock surface (elevation 145.3) inferred at the south abutment.

4.5 Groundwater

No groundwater was observed in borehole H58-1 during or upon completion of drilling. The piezometric water level measured on October 17, 2011 was at 10.9 m (elevation 166.6).

It is noteworthy that groundwater levels in the previous boreholes 1 and 2 were not established and the water level measured on December 30, 1973 in a piezometer installed by the St. Lawrence Seaway Authority some 250 m south of the site was reported to be at elevation 166.1.

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



5. MISCELLANEOUS

The field work was carried out under the supervision of Mr. A. Djirdeh, BSc, 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 Peto MacCallum Ltd.

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

Sincerely

Peto MacCallum Ltd.



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

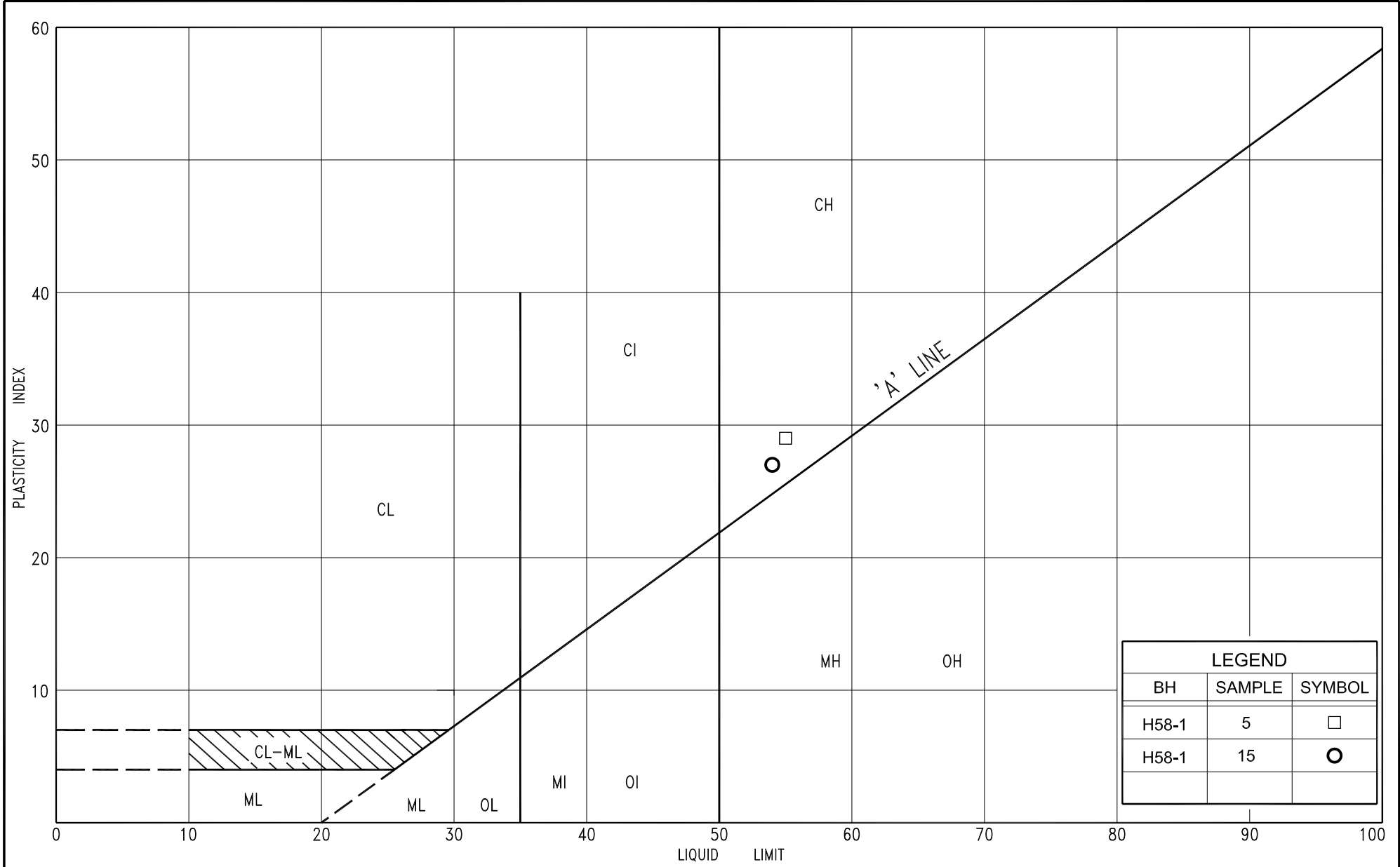


Carlos M.P. Nascimento, P.Eng.
Project Manager



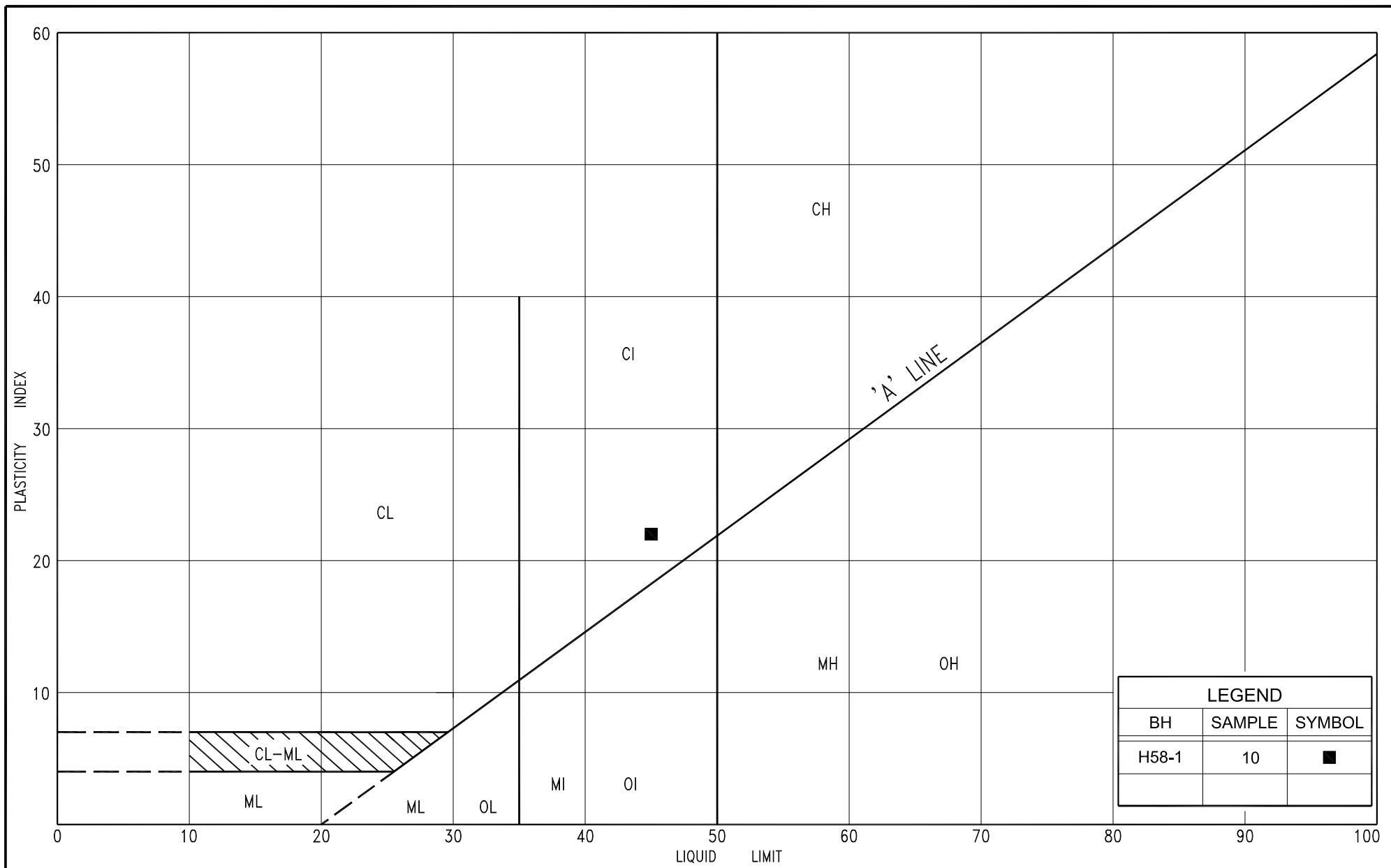
Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

GD/CN/BRG:gd-mi



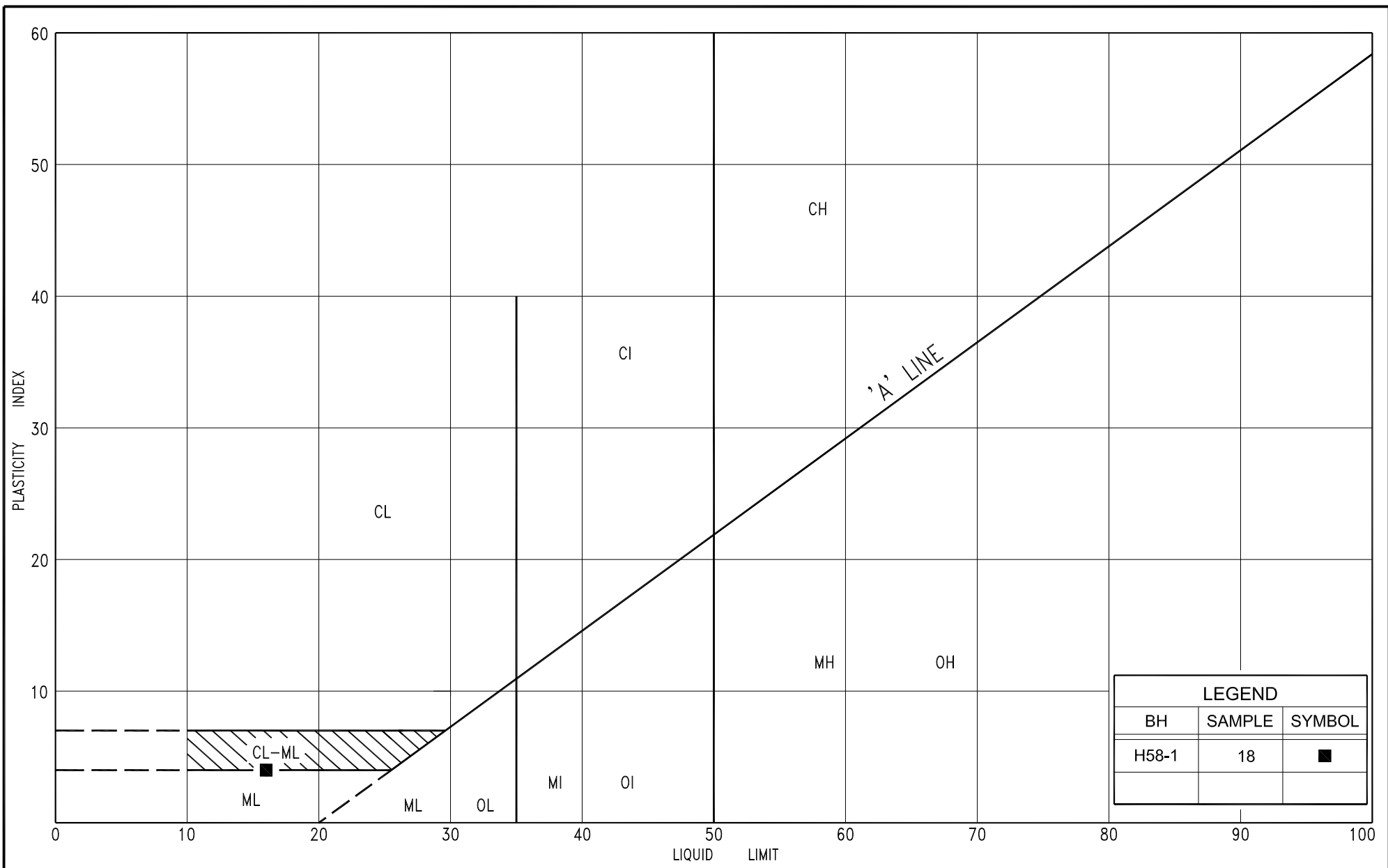
PLASTICITY CHART
CLAY, with silt, trace sand

FIG No. H58-PC-1
HWY: 58
G.W.P. No. 2175-08-00



PLASTICITY CHART
 SILTY CLAY, trace sand, trace gravel

FIG No. H58-PC-2
 HWY: 58
 G.W.P. No. 2175-08-00

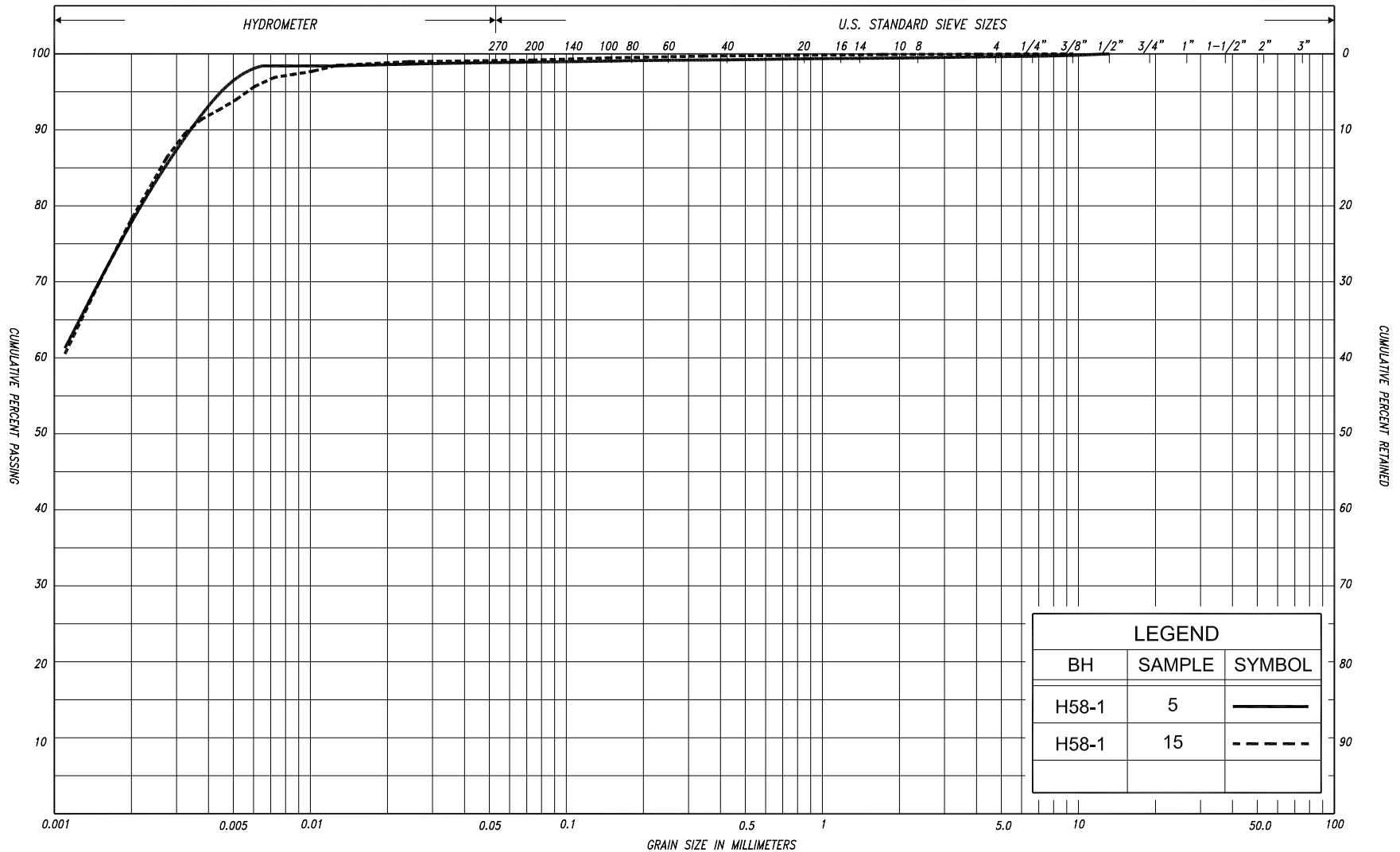


PLASTICITY CHART
 CLAYEY SILT, with sand, some gravel
 (TILL)

FIG No. H58-PC-3

HWY: 58

G.W.P. No. 2175-08-00



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



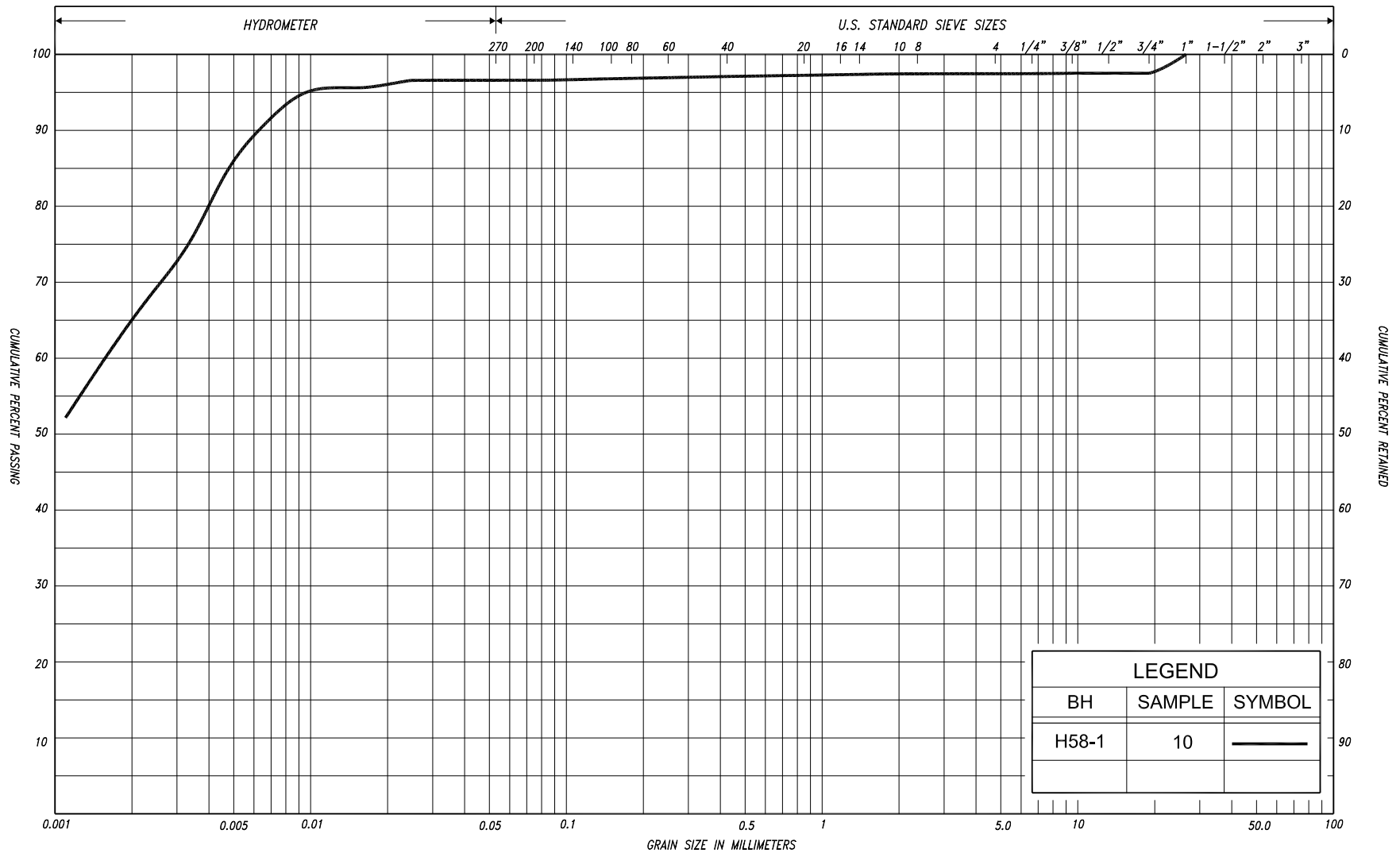
GRAIN SIZE DISTRIBUTION

CLAY, with silt, trace sand

FIG No. H58-GS-1

HWY: 58

G.W.P. No. 2175-08-00



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



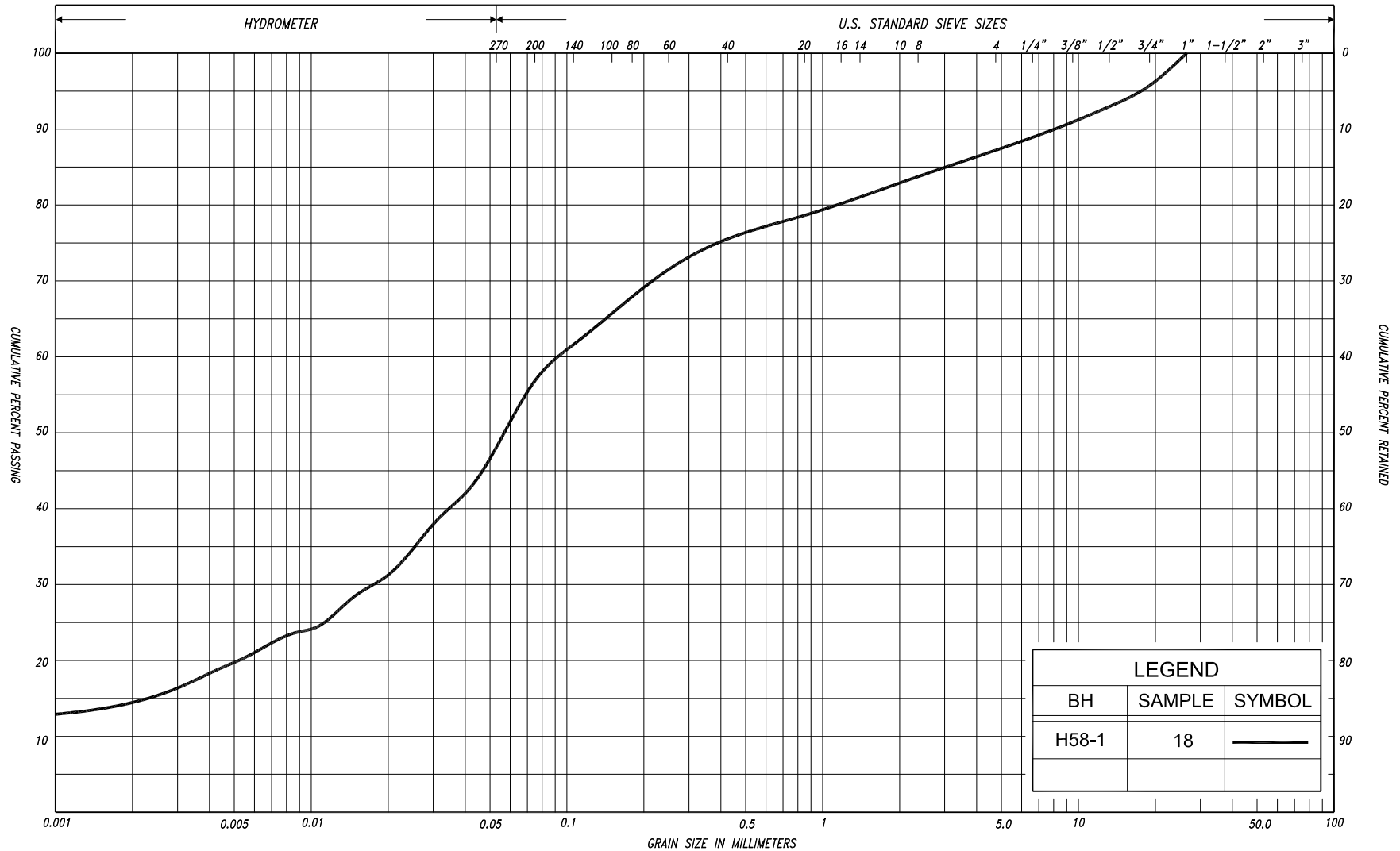
GRAIN SIZE DISTRIBUTION

SILTY CLAY, trace sand, trace gravel

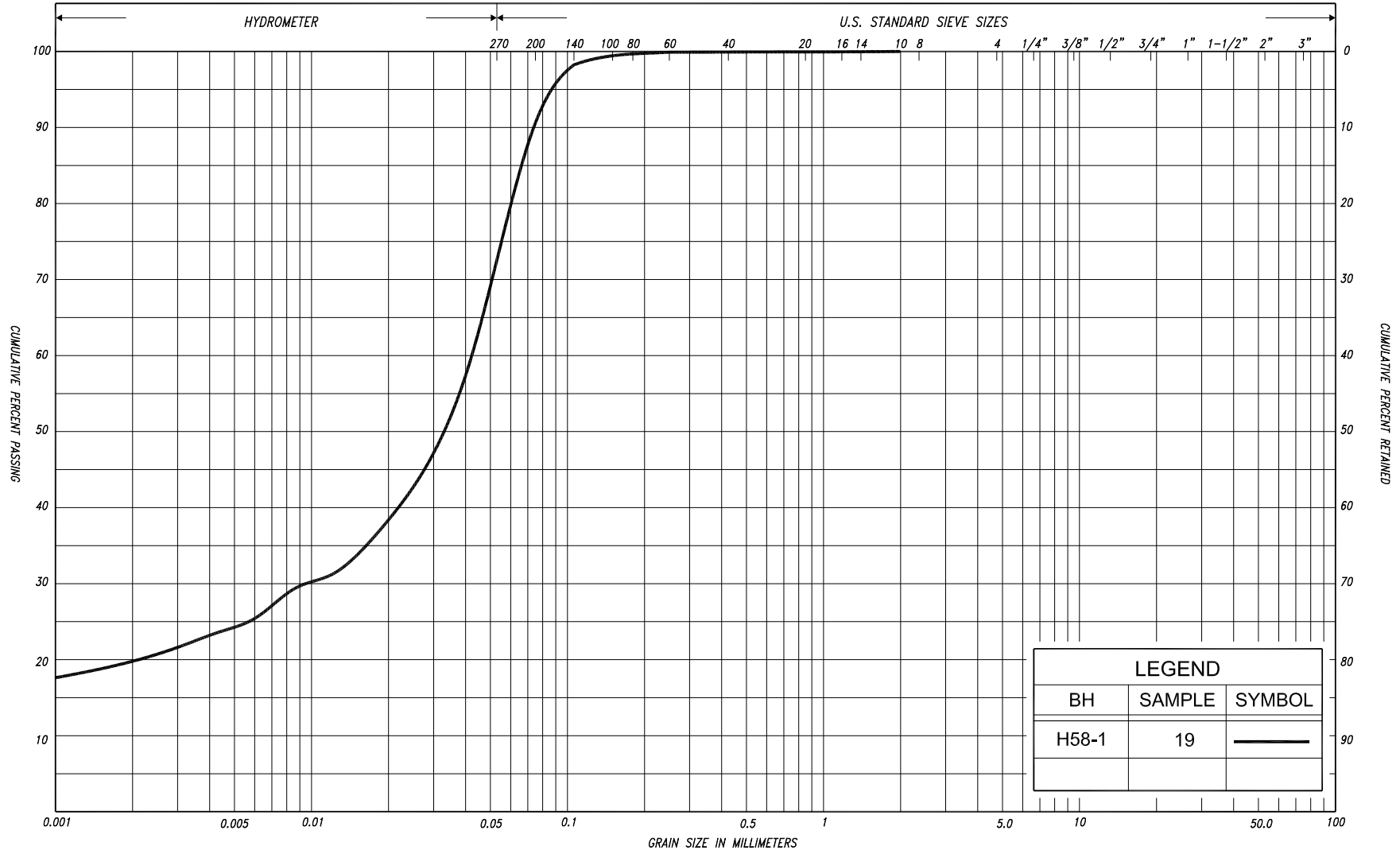
FIG No. H58-GS-2

HWY: 58

G.W.P. No. 2175-08-00



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



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

EXPLANATION OF TERMS USED IN REPORT

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

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

SOILS ARE DESCRIBED BY THEIR COMPOSITION AND CONSISTENCY OR DENSENESS.

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

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

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

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

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

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

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

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

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

R Q D (%)	0 - 25	25 - 50	50 - 75	75 - 90	90 - 100
	VERY POOR	POOR	FAIR	GOOD	EXCELLENT

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
u	1	PORE PRESSURE RATIO
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_i	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

PHYSICAL PROPERTIES OF SOIL

ρ_s	kg/m ³	DENSITY OF SOLID PARTICLES	n	1, %	POROSITY	e_{max}	1, %	VOID RATIO IN LOOSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	w	1, %	WATER CONTENT	e_{min}	1, %	VOID RATIO IN DENSEST STATE
ρ_w	kg/m ³	DENSITY OF WATER	S_r	%	DEGREE OF SATURATION	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
γ_w	kN/m ³	UNIT WEIGHT OF WATER	w_L	%	LIQUID LIMIT	D	mm	GRAIN DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_p	%	PLASTIC LIMIT	D_n	mm	n PERCENT - DIAMETER
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_s	%	SHRINKAGE LIMIT	C_u	1	UNIFORMITY COEFFICIENT
ρ_d	kg/m ³	DENSITY OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	h	m	HYDRAULIC HEAD OR POTENTIAL
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	q	m ³ /s	RATE OF DISCHARGE
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	v	m/s	DISCHARGE VELOCITY
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	DTPL		DRIER THAN PLASTIC LIMIT	i	1	HYDRAULIC GRADIENT
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	APL		ABOUT PLASTIC LIMIT	k	m/s	HYDRAULIC CONDUCTIVITY
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL	WTPL		WETTER THAN PLASTIC LIMIT	j	kN/m ³	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No H58-1

1 of 3

METRIC

G.W.P. 2175-08-00 LOCATION Coords: 4 756 573.1 N; 324 172.2 E ORIGINATED BY A.K.
DIST Central HWY 58 BOREHOLE TYPE C.F.H.S.A. and Dynamic Cone Penetration Test COMPILED BY G.D.
DATUM Geodetic DATE September 26 & 27, 2011 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE × LAB VANE						
177.5 0.0	Ground Surface Sand and gravel		1	SS	14										
176.8 0.7	Silty sand, trace clay		2	SS	10										
	Compact Brown Moist (FILL)		3	SS	8				175						
	Silty clay, trace sand clay layers		4	SS	10				125						
	Very stiff Brown Moist		5	SS	13				188					0 1 21 78	
	Firm to stiff Reddish brown		6	SS	8										
			7	SS	4										
			8	SS	2										
			9	SS	2										
				FV											
	trace gravel		10	SS	2									2 1 32 65	
	Reddish brown/ Moist to wet grey			FV											
			11	SS	3										
				FV											
			12	SS	3										
				FV											
			13	SS	3										
				FV											
162.5															

RECORD OF BOREHOLE No H58-1

2 of 3

METRIC

G.W.P. <u>2175-08-00</u>	LOCATION <u>Coords: 4 756 573.1 N; 324 172.2 E</u>	ORIGINATED BY <u>A.K.</u>
DIST <u>Central</u> HWY <u>58</u>	BOREHOLE TYPE <u>C.F.H.S.A. and Dynamic Cone Penetration Test</u>	COMPILED BY <u>G.D.</u>
DATUM <u>Geodetic</u>	DATE <u>September 26 & 27, 2011</u>	CHECKED BY <u>C.N.</u>

[illegible]

RECORD OF BOREHOLE No H58-1

3 of 3

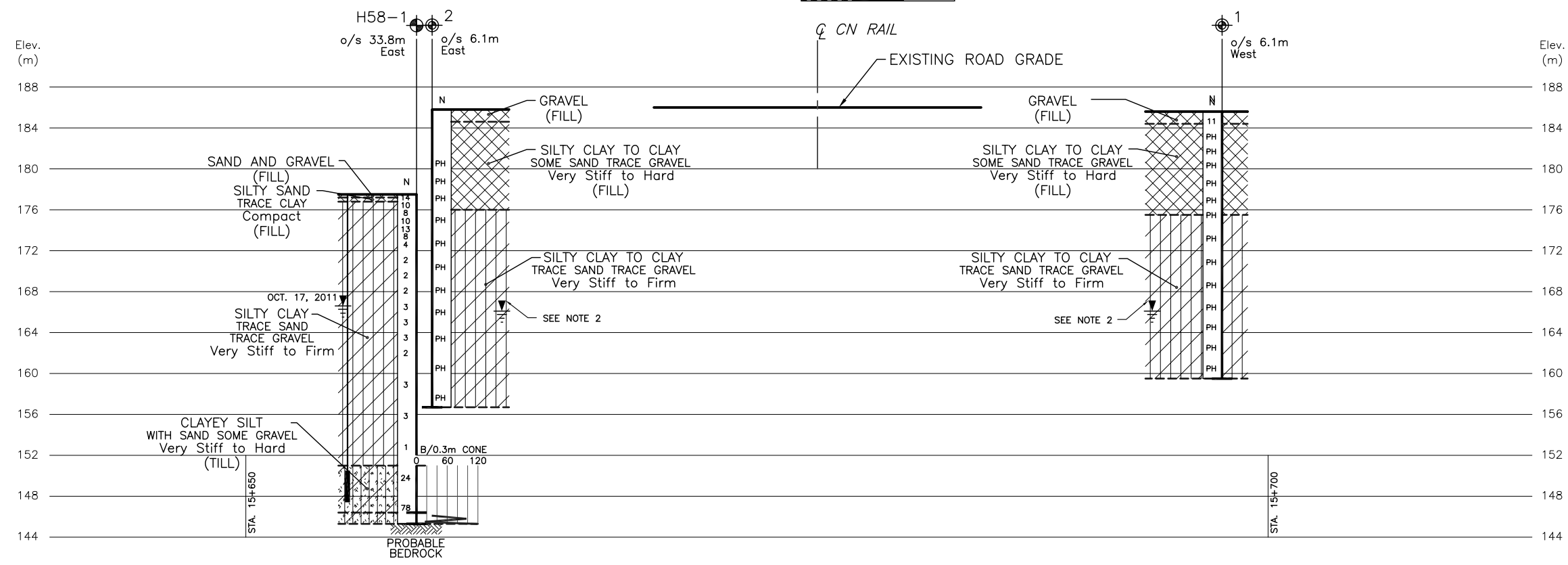
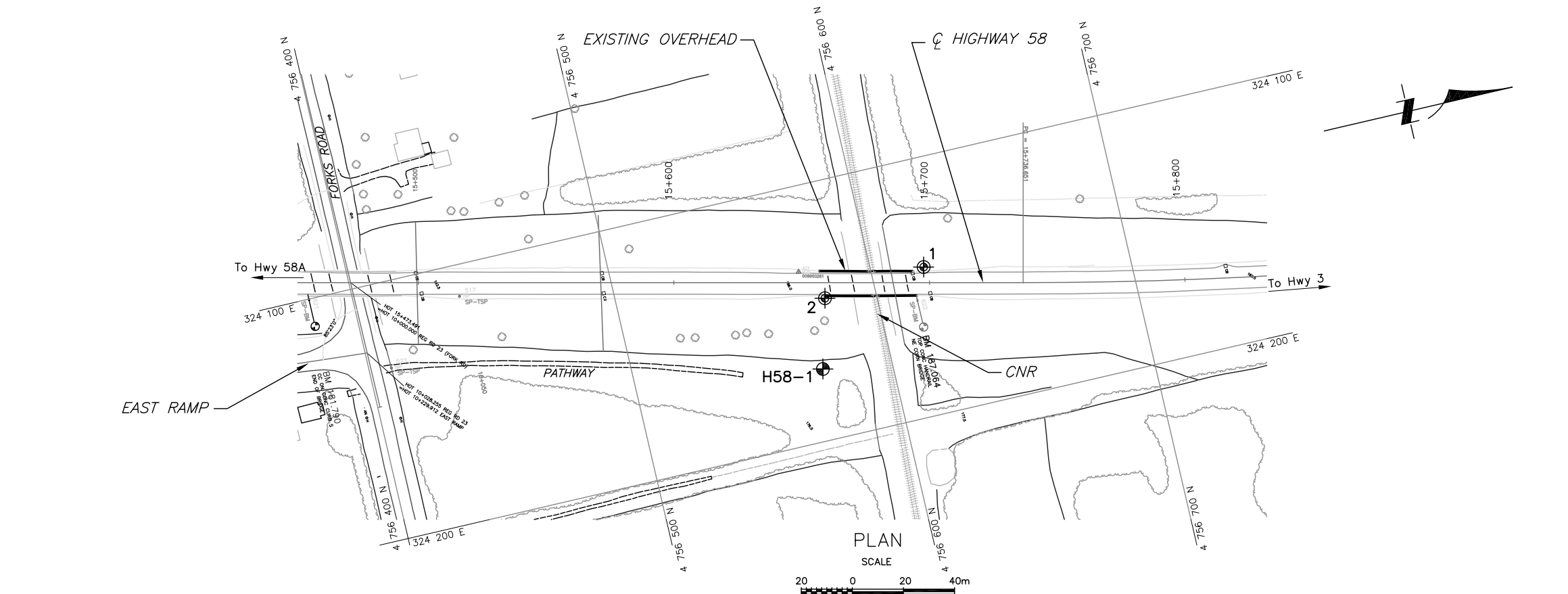
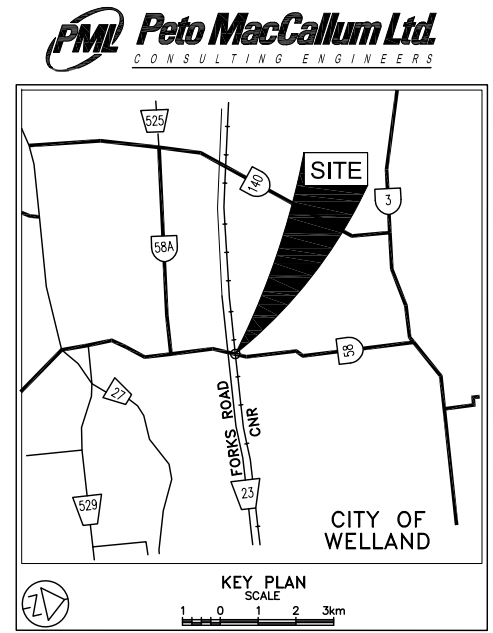
METRIC

G.W.P. 2175-08-00	LOCATION	Coords: 4 756 573.1 N; 324 172.2 E	ORIGINATED BY A.K.
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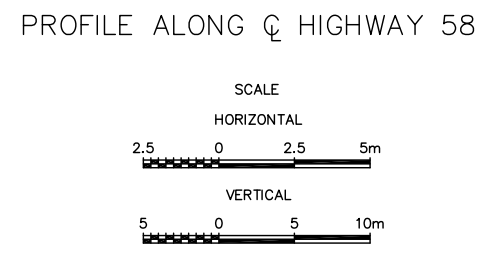
DIST Central HWY 58 BOREHOLE TYPE C.F.H.S.A. and Dynamic Cone Penetration Test COMPILED BY G.D.

DATUM Geodetic DATE September 26 & 27, 2011 CHECKED BY C.N.

[illegible]



- NOTES:
- BOREHOLES 1 AND 2 HAVE BEEN REPRODUCED FROM THE GEOCRIS REPORT No. 30L14-24 DATED MARCH 1974. FOR DYNAMIC CONE PENETRATION TEST DATA, REFER TO THE LOGS OF BOREHOLES 1 AND 2 ATTACHED TO THE REPORT.
 - GROUNDWATER LEVELS IN BOREHOLES 1 AND 2 WERE NOT ESTABLISHED IN THE COURSE OF THE FIELD WORK. THE WATER LEVEL SHOWN WAS MEASURED ON DECEMBER 30, 1973 IN A PIEZOMETER INSTALLED SOME 250 m SOUTH OF THE SITE BY THE ST. LAWRENCE SEAWAY AUTHORITY.
 - THIS DRAWING SHOULD BE READ IN CONJUNCTION WITH THE TEXT AND RECORD OF BOREHOLE LOGS.
 - THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.
 - DIMENSIONS ARE IN METRES AND/OR MILLIMETRES UNLESS OTHERWISE SHOWN. STATIONS ARE IN KILOMETRES AND METRES.



REF No. AECOM DRAWING: b-190-58-25202.dwg;

LEGEND			
	Borehole and Cone for present 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. Also see Note 2		
	Head		
	ARTESIAN WATER		
	Encountered		
	PIEZOMETER		

BH No	ELEVATION	CO-ORDINATES	
		NORTHINGS	EASTINGS
H58-1	177.5	4 756 573.1	324 172.2
1	185.6	4 756 620.5	324 142.3
2	185.8	4 756 580.1	324 145.4

- 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

Geocris No. 30L14-53

HWY No 58	CHECKED GD	DATE FEB. 16, 2012	DIST Central
SUBM'D NA	CHECKED CN	APPROVED BRG	SITE 34-111
DRAWN NA	CHECKED CN	APPROVED BRG	DWG H58-1



APPENDIX A

Record of Borehole Sheets from Previous Investigation
(Geocres No. 30L14-24)

DESIGN SERVICES BRANCH

FOUNDATIONS OFFICE

RECORD OF BOREHOLE NO 1

JOB 73-11098

LOCATION Hwy. 88, Sta. 167 + 26, c/s: 20' LT &

ORIGINATED BY L.M.

WP 78-73-02

BORING DATE December 13, 13, 1973

COMPILED BY J.M.

Datum: Geodetic

BOREHOLE TYPE Hollow Stem Jumbo Borehole Mounted

CHECKED BY

SOIL PROFILE			SAMPLES		ELEV. SCALE (ft. / m)	DYNAMIC PENETRATION RESISTANCE BLOWS / FOOT (15.2 m)		LIQUID LIMIT w_L PLASTIC LIMIT w_p WATER CONTENT w		BULK DENSITY γ (T/m^3)	REMARKS
ELEV. DEPTH ft.	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		20	40	50	70		
185.6 0.0	Ground Level										
184.4 1.2	Gravel FILL		1	SS	11						
	Silty Clay to clay some sand and traces of gravel Brown to red-brown Very Stiff to Hard.		2	TH	TH	800 (197.9)					
			3	TH	TH						
			4	TH	TH	590 (179.8)					
			5	TH	TH						
			6	TH	TH	580 (176.8)					
175.6 10.1	(Zone 1)		7	TH	TH						
	Silty clay to cl. numerous silt seams/pockets, occ. gravel, pockets sandy Grey-brown to mottled layered red, brown, grey Firm to Stiff		8	TH	TH	570 (173.7)					
			9	TH	TH						
171.3 (14.3)	(Zone 2)		10	TH	TH	560 (170.7)					
	Silty clay, occ. silt seams/pockets Grey Firm.		11	TH	TH						
168.3 17.4	(Zone 3)					550 (167.6)					
	Silty clay to clay occasional silt pockets, occ. gravel Red-brown. Firm to Stiff mottled red, brown, grey		12	TH	TH						
						540 (164.6)					
			13	TH	TH						
						530 (161.9)					
159.6 26.1	End of Borehole		14	TH	TH						
						520 (159.5)					

50
(kN/m^2) 100

ORIGINATED BY J.M.

COMPILED BY JEH

CHECKED BY _____

20
10 5 % STRAIN AT FAILURE
30



PRELIMINARY FOUNDATION DESIGN REPORT

for

**CNR OVERHEAD
HIGHWAY 58, SITE 34-111
GWP 2175-08-00
CITY OF WELLAND, ONTARIO**

PETO MacCALLUM LTD.
165 CARTWRIGHT AVENUE
TORONTO, ONTARIO
M6A 1V5
Phone: (416) 785-5110
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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-1
Index No.: 034FIDR
GEOCRES No.: 30L14-53
March 15, 2012



PART B – PRELIMINARY FOUNDATION DESIGN REPORT

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Table 1 – List of Standard Specifications Referenced in Report

Figure 1 – Abutment on Compacted Fill Showing Granular A Core

Appendix FDR-A – Embankment Settlement Criteria for Design

PART B
PRELIMINARY FOUNDATION DESIGN REPORT

for
CNR Overhead
Highway 58, Site 34-111
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 overhead located on Highway 58 in the City of Welland, Ontario.

The existing overhead is a three span structure of some 30 m overall length and 9 m width, accommodating two lanes of traffic (Drawing 'Hwy 58 – CNR Overhead' prepared by AECOM in June 2011). The abutments and piers of the structure rest on spread footings founded at elevation 175.0. The foreslopes of the about 9 m high approach embankments are supported by 4.4 m high retaining walls built along the CNR alignment and founded at elevation 177.4. No details of the proposed rehabilitation / replacement of the structure were available at the time of preparation of this report. However, it is planned to raise the road grade by about 0.6 m at the south abutment and 0.3 m at the north abutment.

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 previous foundation investigations at the site (Geocres No. 30L14-24 dated March 1974 and Geocres No. 30L14-46 dated November 1998). 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 soil resistance to carry the rehabilitated structure loading to be confirmed by the Design-Build contractor, reuse of the existing spread footings founded in the upper zone of the cohesive deposit near elevation 175 may be considered by the Design-Build contractor, subject to a structural analysis including verification of adequate foundation resistance by the Design-Build contractor.

In case the structure is to be replaced, steel H-piles driven to refusal on bedrock or, alternatively, footings placed on either the native soils or a pad of engineered fill may be used at the site.

Drilled cast-in-place concrete caissons bearing on the bedrock to support the structure are considered to be practical at this site only if the difficulties caused by the dewatering of the high groundwater table (about 21 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.

A summary of advantages, disadvantages and recommended foundations is given below:

FOUNDATION TYPE	ADVANTAGES	DISADVANTAGES	RISKS / CONSEQUENCES
Spread footings on native soil	Existing foundation system	Lower bearing resistance than for driven piles	Limited support for increase in loading
Spread footings on engineered fill pad at new locations	Ease of construction	Lower bearing resistance than for driven piles Need to provide erosion protection	Significant long-term consolidation settlements expected
Driven piles	High capacity	High cost relative to footings	Difficulty of installation due to potential presence of boulders
Caissons	High capacity	High cost relative to footings Potential for difficulties to be encountered during drilling through the hard glacial till that overlies bedrock	Augering difficulties could result in construction delays and cost overrun



All footings or 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 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 Spread Footings

6.2.2.1 Spread Footings on Native Soil

As indicated previously, supporting the abutments and piers of the overhead structure on conventional spread footings founded on native soil is considered to be feasible.

Spread footings (new or existing) should be founded on the typically very stiff silty clay near elevation 175. The recommended preliminary bearing resistance at ultimate and serviceability limit states (ULS and SLS) for minimum 2.0 m wide footings constructed at a new location (new footings) or for the existing footings bearing on the native soils is as follows:

Factored Geotechnical Resistance at ULS, kPa	500 (new footings) 550 (existing footings)
Geotechnical Resistance at SLS, kPa	200 (new footings) 220 (existing footings)

The resistance at SLS normally allows for 25 mm compression of the founding medium. Differential settlement is expected to be less than 75% of this value.

Settlements resulting from the planned road grade raise are expected to be up to 30 mm. Refer to section 6.4 of the report for further details.



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 developed between the underside of the footing and the clayey soil. An unfactored friction factor of 0.4 is recommended for footings placed on clay.

Construction of the spread footings on native soil should be performed and monitored in accordance with OPSS 902 and SP 902S01 to verify the competency of the founding surface.

6.2.2.2 Spread Footings on Structural Fill

Construction of new abutment footings on structural fill placed in the approach embankments could also be employed to support the foundation loads. The structural fill should comprise Ontario Provincial Standards Specifications (OPSS) Granular A material placed in maximum 200 mm thick lifts, compacted to 100% of the ASTM D-698 (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 a minimum 2.5 m thick structural fill is as follows:

Factored Geotechnical Resistance at ULS, kPa	900
Geotechnical Resistance at SLS, kPa	350

The resistance at SLS normally allows for 25 mm 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 developed between the underside of the footing and the structural fill. An unfactored friction factor of 0.7 is recommended for footings placed on granular fill.

6.2.3 Piles

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 an approximate depth of 32.2 m below existing grade (elevation 145.3). For preliminary design purposes, the founding levels for driven piles at the proposed pier locations are likely to be similar to the founding levels at the abutments in view of the relatively flat characteristics of the dolostone bedrock surface anticipated at this site. These levels should be confirmed during detail design.

The recommended factored axial resistance at ULS for the HP 310x110 pile section 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 structural computation purposes, the geotechnical resistance at SLS may be taken the same as the factored geotechnical resistance at ULS of 2000 kN.

The piles will have to be driven through native soils containing 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 embankment will not be widened. However, it is proposed to raise the road grade by about 0.6 m at the south abutment and 0.3 m at the north abutment. 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 long-term settlement of the approach embankment, negative skin friction will not need to be considered.

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 ϕ = angle of internal friction of retained soil (35° for Granular A or B Type II)

δ = angle of friction between soil and wall (23.5° 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, ϕ (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).

For preliminary design of the retaining walls at the piers, the earth pressure coefficients of 0.5 (active), 0.7 (at rest) and 2.77 (passive) should be employed for the clay fill to take account of sloping backfill.

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 embankments on the 20 m long north and south approaches to the overpass. 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 to be expected if the road grade remains unchanged. Subsurface investigations should be carried out at these locations for detail design to assess the condition of the existing fill within the approaches.

For the current preliminary design concept, it is proposed to raise the road grade by about 0.6 m at the south abutment and 0.3 m at the north abutment. Based on the settlement analysis previously carried out for the site (Geocres No. 30L14-24) the total consolidation settlements resulting from the increase in loading are assessed to be up to 30 mm at the south abutment, with the period for 90% completion of consolidation of 20 to 25 years. These settlements may be eliminated if mitigation measures such as use of lightweight fill or EPS are undertaken to balance the increase in weight. It is considered that preloading or surcharging the embankment should be avoided because these alternatives would extend the period of construction and possibly induce settlements of the existing spread footing foundations. In addition, the surcharge option would require that the embankment be temporarily widened to avoid instability issues with the embankment slope. These mitigation measures should be further considered during detail design.

Any new approach embankments or widenings, if required 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 2.5H:1V for earth fill. It is noted that the slopes of the existing approach embankments are assessed to be about 2.5H:1V and this inclination should be confirmed during detail design.

It is considered feasible to backfill a possible new abutment structure 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 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 the approach fill surface near the abutments should be essentially complete within 3 to 4 months after placement of fill within the existing embankment footprint.

As indicated above, a settlement analysis was carried out during the previous investigation at the site (Geocres No. 30L14-24) and showed that total settlements due to imposed loading (excluding drawdown settlements resulting from the lowering of groundwater levels for the relocation of the Welland Canal) were expected to be 100 mm at the pier locations and 300 mm at the abutment locations, with about 90% of these settlements occurring for a period of 20 to 25 years after construction. It is considered that settlements of similar magnitude should be anticipated in case of the new abutments of the possible replacement structure being relocated closer to the CNR alignment.

It is also noted that the section of embankment between the overhead and the Forkes Road overpass located approximately 200 m to the south will need up to be raised with up to 1.2 m of fill to correct its vertical alignment. The added fill will induce embankment settlements that are estimated to be up to 60 mm. The Design-Build contractor should carry out a detailed analysis for detail design of the embankment grade raise. The design should include all mitigation measures required to ensure that the embankment grades will satisfy the MTO Embankment Settlement Criteria for Design (Final Draft dated March 2, 2010) including the transitions at the bridge abutments. A copy of this document is attached in Appendix FDR-A. Settlement monitoring details are recommended in Subsection 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

No groundwater was observed in borehole H58-1 during or upon completion of drilling. The piezometric water level measured on October 17, 2011 was at 10.9 m depth (elevation 166.6). 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 the excavation should be readily handled by conventional sump pumping techniques.

Groundwater conditions should be further assessed during detail design.

6.5.3 Settlement Monitoring

It is recommended that the tenders require that the Design-Build contractor carries out a monitoring program of the magnitude of induced settlements of the structure and of the embankment between the CNR overhead and the Forks Road overpass to the south. The monitoring program should contain the reporting protocol and the review and alarm displacement levels for the project and be specific to the proposed final design proposed by the Design-Build contractor. At least 60 days before the construction starts, 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.
4. Consider mitigation measures to balance the planned increase in loading from the road grade raise within the approach embankments.



8. CLOSURE

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., Senior Foundation Engineer, and reviewed by Mr. C.M.P. Nascimento, P.Eng., Project Manager. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Principal Contact, 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



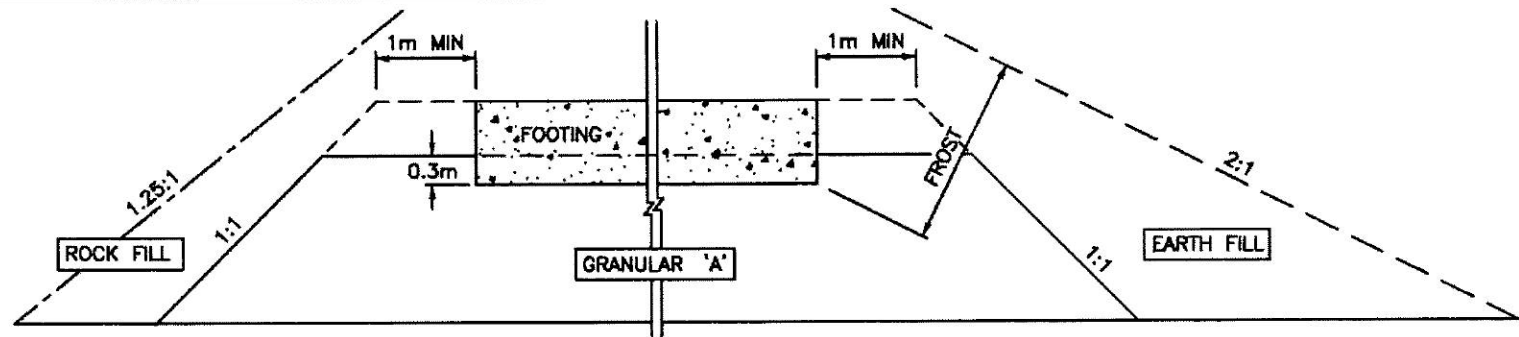
Brian R. Gray, MEng, P.Eng.
MTO Designated Principal Contact

GD/CN/BRG:gd-mi



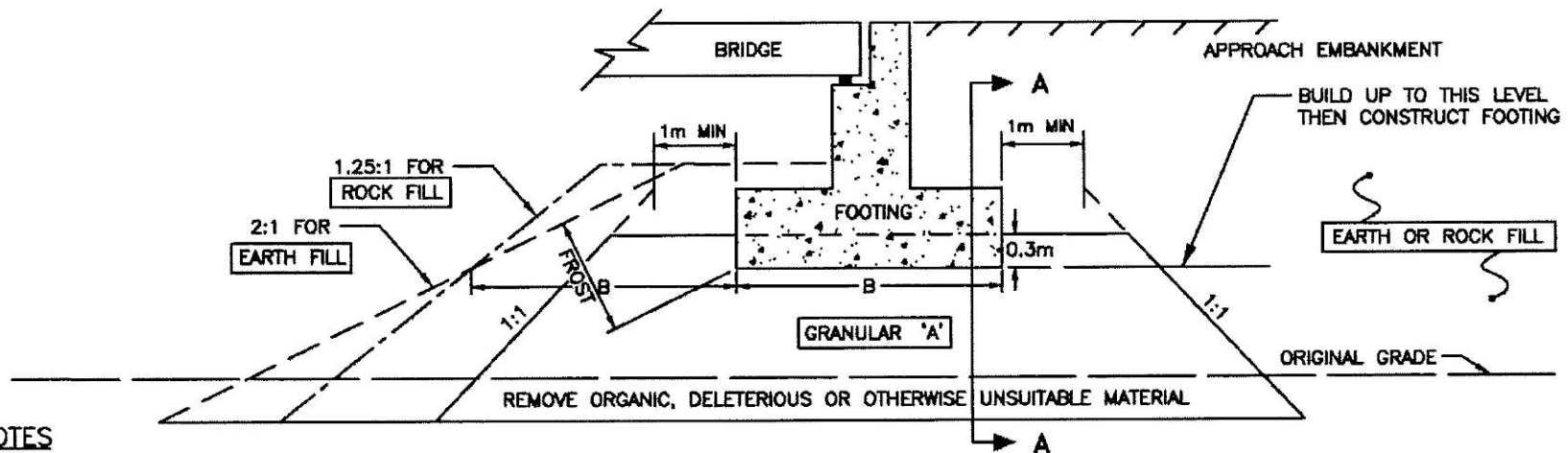
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE
OPSS 501	Construction Specification for Compacting
OPSS 902	Excavation and Backfilling of Structures
OPSS 903	Construction Specification For Deep Foundations
SP 206S03	Construction Specification for Grading
SP 405F03	Construction Specification for Pipe Subdrains
SP 902S01	Excavation and Backfilling of Structures
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



CROSS SECTION A-A

NOT TO SCALE



LONGITUDINAL SECTION

NOT TO SCALE

NOTES

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

FIGURE 1: ABUTMENT ON COMPACTED FILL SHOWING GRANULAR A CORE



APPENDIX FDR-A

Embankment Settlement Criteria for Design

EMBANKMENT SETTLEMENT CRITERIA FOR DESIGN

Final Draft – March 2 2010

SUBJECT: EMBANKMENT SETTLEMENT CRITERIA FOR DESIGN

PURPOSE: To provide direction and generic criteria for post construction settlements for new embankments, transitions, and for embankment widenings. The criteria are intended to provide guidance in developing targets for embankment performance during the design phase. The criteria should be reviewed and designs customized on a project specific basis.

BACKGROUND: An embankment refers to the materials placed within the side slopes, below subgrade, and above the original ground, excavated base or theoretical bottom as identified during design.

Embankment settlements are attributable to movements and changes that occur within the underlying native soils due to embankment loading. Settlements can also occur within the embankment.

The main types of settlement of native soils include:

1. Elastic compression
2. Primary consolidation
3. Secondary consolidation (creep)

The total settlement is the sum of these settlements.

The magnitude of these settlements is a function of the applied loading, the thickness of the compressible layer and the physical, mechanical and compressibility properties of the soil.

Elastic compression is distinguished from primary consolidation and secondary consolidation in that the settlements are immediate in nature occurring during the construction period or shortly thereafter. Primary and secondary consolidation settlements are time dependent.

Settlements within the embankment can occur due to its own weight. Care must be taken to properly compact embankments to reduce settlements associated with fill density changes. Settlements within granular fills generally occur during construction. Settlements within cohesive fills are time dependent and can occur following construction.

Settlements in rock fill can occur due to weathering, particle breakage and particle reorientation. These settlements are time dependent.

Differential settlement between any two points is the primary concern when

assessing embankment settlement performance. When differential settlements exceed limits as governed by the flexibility of the pavement structure, asphalt cracking and distortions will occur. Differential settlement limits are required to ensure adequate transitions and to avoid unacceptable surface distortions.

Generic performance criteria for highway embankments, including asphalt pavement and slopes, are necessary to ensure safety, rideability and to optimize initial construction and post construction maintenance cost. Performance requirements will also improve provincial design consistency and provide general guidance for foundation engineers. Due engineering diligence is required to define the embankment performance criteria during detailed design and to customize the selection on a project specific basis. The requirements for major highways are more stringent than secondary highways or roadways.

For the purpose of establishing settlement design criteria, the following Pavement Design Life has been assumed:

- 20 years following the construction of the pavement structure for King's highways,
- 15 years following the construction of the pavement structure for secondary highways, and
- 15 years following the construction of embankments beneath surface treated and gravel surfaces.

This document does not apply to structures. For structures, refer to the applicable design requirements of the Canadian Highway Bridge Design Code (CAN/CSA-S6-00).

POLICY:

Section 1: Settlement Design Criteria

Gravel shoulders of paved or surface treated roads can be readily restored to design crossfall by grading/addition of gravel. For paved and surface treated roads, the settlement criteria apply only to the paved or surface treated portion of the road. Designs shall account for the loss of shoulder width due to settlement and subsequent restoration of the road to design profile, by including an overbuilding of the embankment where appropriate.

1.1 New Embankments

Maximum permissible post-construction settlements for new embankments are provided in Table 1.1. Refer to Figure 1.

Total settlements are defined relative to both the longitudinal profile of the traffic lanes of the roadway, and transversely across the top of the roadway surface.

**Table 1.1: Post-Construction Settlement Criteria
For New Embankments**

	Maximum Limits During Pavement Design Life	
	Total Settlement (mm)	Differential Settlement Rate
Non-Compressible Soils	50	200:1
Freeways on Compressible Soils	100	200:1
Non-Freeways On Compressible Soils	200	100:1
Surface Treated and Gravel on Compressible Soils	300	50:1

The values in Table 1.1 are recommended maximum permissible values for settlements. Each embankment shall be designed to satisfy these maximum values. Alternative foundation designs shall be developed and compared based on the advantages, disadvantages, costs, and risk/consequences. Alternatives that exceed the maximum settlement values should be considered when the initial construction costs of the alternatives are high, compared to the cost of surface repairs to correct the longitudinal and transverse profile of the surface. Typically, the embankment design selected shall be the preferred alternative that satisfies the maximum settlement values and is the most cost effective, considering both initial construction and anticipated maintenance costs.

1.2

Transition/Tapers

A smooth transition between elements such as a bridge abutment, existing embankment or other structure constructed on non-compressible soils and the new embankment shall be taken into consideration during design. The transition point is the point where an element that will have post-construction settlement intersects an element that will not have post-construction settlement. Refer to Figure 2.

In order to control the differential settlements between different ground treatment areas such as areas of compressible soils adjacent to areas of non compressible soils and also to afford acceptable transitions in areas of backfill and approach embankments to structures in areas of compressible

soils consideration has to be given to transitions in these areas. Accordingly, embankments adjacent to bridge/culvert structures or to different ground treatment areas shall be designed with appropriate transitions and tapers in the longitudinal direction.

The total and differential post-construction settlement limits are shown in Table 1.2. The designer shall consider these limits and the varying profile grade to design an appropriate transition.

The settlement at structure/embankment interface shall not create any abrupt step deeper than 5 mm

Table 1.2: Post-Construction Settlement Criteria for Transitions

Distance From Transition Point	Maximum Limits During Pavement Design Life			
	0-20 m	20-50 m	50-75 m	≥ 75 m
Freeways	25	50	75	100
Non-Freeways	25	50	100	200
Surface Treated and Gravel	25	75	150	300

1.3

Embankment Widening

Post-construction settlement of the widened embankment shall comply with the limits in Table 1.3. Refer to Figure 3.

Table 1.3: Post- Construction Settlement Criteria for Embankment Widening

	Maximum Limits During Pavement Design Life	
	Total Settlement (mm)	Differential Settlement Rate
Freeways	50	200:1
Non-Freeways	75	100:1
Surface Treated and Gravel	100	50:1

The settlement across the widened embankment shall transition uniformly from the widening point (existing highway embankment rounding) to the new embankment rounding such that surface drainage is not impeded.

Section 2: Other Considerations

The design, construction and maintenance of embankments must ensure that post-construction deformations do not at any time:

- impair or compromise pavement support; or
- cause pavement to exceed, or fail to satisfy the pavement performance requirements

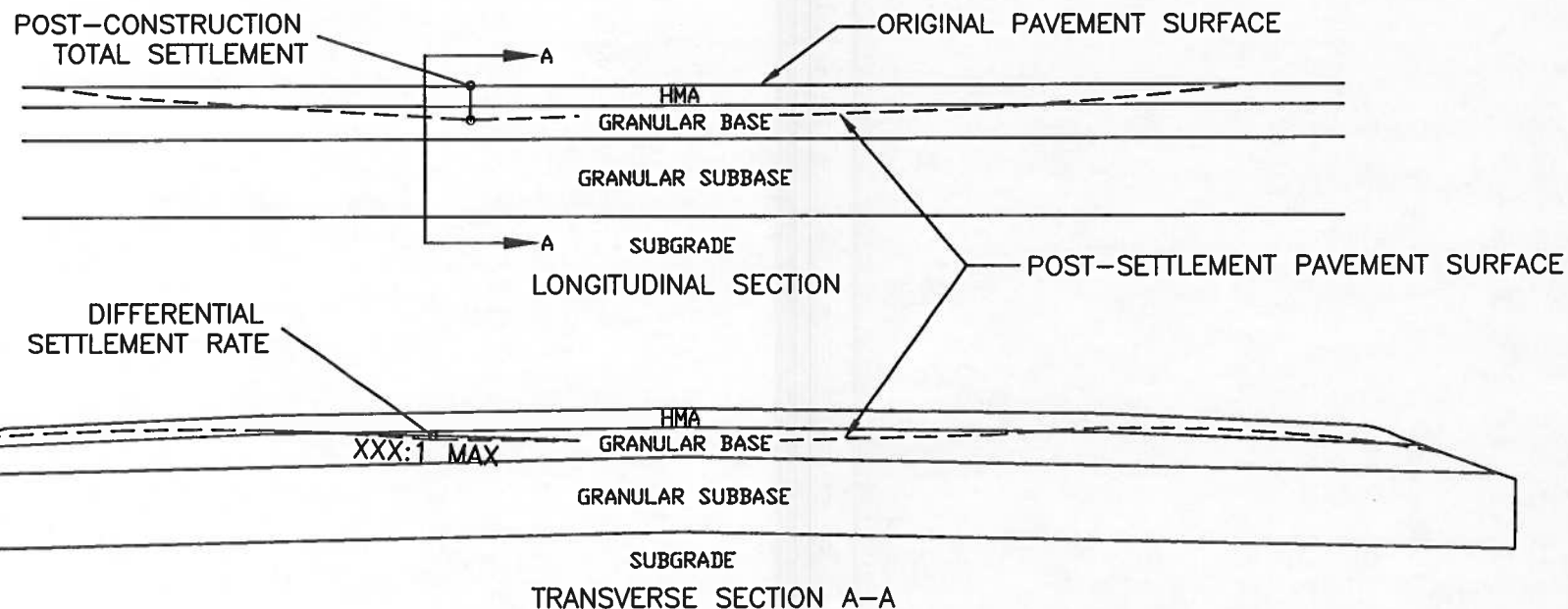
Where rigid (concrete) pavement is designed or possible by contract alternative bidding, the embankment settlement shall not result in joint faulting greater than slight (Reference Ministry of Transportation Manual for Condition Rating of Rigid Pavements, SP-026 (1995), and OPSD 551.010).

Any movement must not cause the cross-section profile to deform to an extent that would compromise surface runoff and subsurface drainage.

Embankment settlements and lateral movements of the subsoils must not adversely impact on existing structures, earthworks or services in a manner that compromises the serviceability and/or structural integrity of the existing structures, new structures, earthworks or services.

Refer to the project specific Foundations Engineering terms of reference for details regarding embankment settlement analysis and parameters for design.

In general, instrumentation to monitor settlement is only required where the foundation conditions are high complexity and/or for construction safety purposes. Typically, monitoring of post-construction settlement is by observation. Where a problem is suspected, a ground survey and comparison with as-constructed elevations may be carried out.



NOTES:

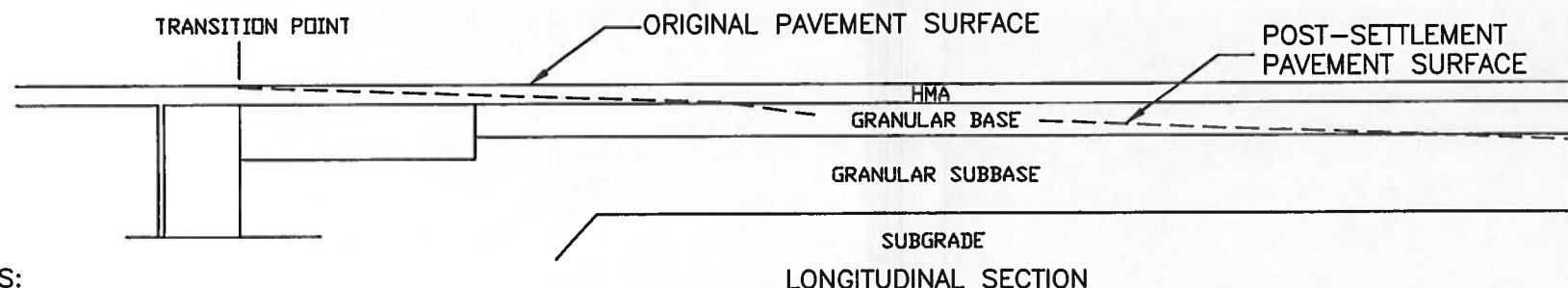
1. SETTLEMENT DESIGN SHALL MEET THE TABULATED LIMITS.
2. FOR PAVED AND SURFACE TREATED ROADS, LIMITS SHALL APPLY ONLY TO THE PAVED OR SURFACE TREATED PORTION.
3. POST-CONSTRUCTION SETTLEMENT PERIODS ARE:
 - 20 YEARS FOR KING'S HIGHWAYS AND FREEWAYS
 - 15 YEARS FOR SECONDARY HIGHWAYS (500 SERIES AND HIGHER NUMBERED HIGHWAYS)
 - 15 YEARS FOR SURFACE TREATED AND GRAVEL HIGHWAYS

	SETTLEMENT LIMITS	
	TOTAL (mm)	DIFFERENTIAL
EMBANKMENT ON NON-COMPRESSIBLE SOILS	100 ⁵⁰	200:1
FREEWAYS ON COMPRESSIBLE SOILS	100	200:1
NON-FREEWAYS ON COMPRESSIBLE SOILS	200	100:1
SURFACE TREATED AND GRAVEL ON COMPRESSIBLE SOILS	300	50:1

NOT TO SCALE

SETTLEMENT DESIGN CRITERIA	MARCH 2010	
NEW EMBANKMENTS	-----	

FIGURE 1		



NOTES:

1. POST-CONSTRUCTION SETTLEMENT IN TRANSITION ZONE SHALL NOT EXCEED THE TABULATED LIMITS.
2. CRITERIA FOR TOTAL AND DIFFERENTIAL SETTLEMENT SHALL NOT EXCEED LIMITS IN FIGURE 1.
3. THE TRANSITION POINT IS THE POINT BETWEEN TWO DIFFERENT GROUND TREATMENT ZONES, INCLUDING STRUCTURE ZONE.
4. FOR PAVED AND SURFACE TREATED ROADS, LIMITS SHALL APPLY ONLY TO THE PAVED OR SURFACE TREATED PORTION.
5. POST-CONSTRUCTION SETTLEMENT PERIODS ARE:
 - 20 YEARS FOR KING'S HIGHWAYS AND FREEWAYS
 - 15 YEARS FOR SECONDARY HIGHWAYS (500 SERIES AND HIGHER NUMBERED HIGHWAYS)
 - 15 YEARS FOR SURFACE TREATED AND GRAVEL HIGHWAYS

DISTANCE FROM TRANSITION POINT	SETTLEMENT LIMITS (mm)			
	0-20 m	20-50 m	50-75 m	> 75 m
FREEWAYS	25	50	75	100
NON-FREEWAYS	25	50	100	200
SURFACE TREATED AND GRAVEL	25	75	150	300

NOT TO SCALE

POST-CONSTRUCTION SETTLEMENT LIMITS

LONGITUDINAL TRANSITIONS

MARCH 2010

FIGURE 2