



FOUNDATION INVESTIGATION AND DESIGN REPORT

for

FOUR MILE CREEK BRIDGE REPLACEMENT

AND TEMPORARY DETOUR BRIDGE

SITE NO. 35-78

HIGHWAY 6 IMPROVEMENTS

FROM ARTHUR (WELLS STREET) TO SOUTH OF MOUNT FOREST

AGREEMENT NUMBER 3005-E-0036

GWP NO. 342-97-00

TOWNSHIP OF ARTHUR

WELLINGTON NORTH COUNTY, ONTARIO

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Geocres No.: 40P15-39
January 12, 2007

Re-issued July 24, 2007
with Revised Foundation Drawings



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Four Mile Creek Bridge

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Drawing D1 and D2: Borehole Locations and Soil Sections

FOUNDATION INVESTIGATION REPORT
for
Four Mile Creek Bridge Replacement
And Temporary Detour Bridge
Site No. 35-78
Highway 6 Improvements
From Arthur (Wells Street) to South of Mount Forest
Agreement Number 3005-E-0036
G.W.P. 342-97-00
Township of Arthur
Wellington North County, Ontario

1. INTRODUCTION

The reconstruction of the Four Mile Creek Bridge is planned under the scope of the improvement project of an approximately 21.2 km long section of Highway 6 that extends from Arthur (Wells Street) to south of Mount Forest in the Township of Arthur, Ontario. The construction of an associated temporary detour bridge is also planned as part of the construction staging for the project. This report was prepared for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

This report summarizes the results of the foundation investigation carried out at the Four Mile Creek Bridge site and at the site of the temporary detour bridge that will be located about 18 m to the west of the Four Mile Creek Bridge. Both sites are located about 0.7 km south of Arthur Road No. 9.

The Four Mile Creek Bridge is designated by the MTO Site No. 35-78 and is located at about Station 16+410 (Highway 6 chainage). The detour bridge is located at about Station 25+404 (Highway 6 Detour chainage).

This report describes the site conditions at both bridge sites and approach embankments within 20 m of the abutments.

2. SITE DESCRIPTION

The existing Four Mile Creek Bridge is a single span earth filled concrete arch structure that was built in 1932 and rehabilitated in 1989 and carries the two-lane Highway 6 over the Four Mile Creek between Arthur and Mount Forest.



The Four Mile Creek flows east to west at the bridge sites, as illustrated on the attached Borehole Location plans.

The sites of both bridges are located about 3.8 km north of Arthur. The local land use is mainly agricultural with farms and residences in relative proximity of the bridge sites. The ground cover comprises of agricultural crops. Grasses cover the ground outside of the farms and the banks of the Four Mile Creek.

The Highway 6 pavement surface dips about 9.5 m from the south to a sag located about 90 m north of the creek and rises to the north about 5.5 m. There is a 3.5 m deep earth cut on both east and west sides of the highway starting about 150 m south of the bridge. Immediately north of the bridge site the highway was constructed over a 3.0 m high earth fill section about 30 to 50 m long.

The site is located within the physiographic region known as the Dundalk Till Plain characterised by a gently undulating glacial till plain. The typical surficial soil is a shallow medium textured sandy silt which overlays cohesive glacial tills. Some of the low lying areas are swampy with poor drainage (L. J. Chapman and D. J. Putnam, the Physiography of Southern Ontario, 3rd Edition, Ontario Research Foundation, 1984).

The bedrock formations underlying the site are of Salina Formation and are largely composed of dolostone, shale, gypsum and salt. The estimated bedrock level is at about 50 m depth.

The frost penetration depth for the area of the Four Mile Creek Bridge and proposed temporary detour bridge is 1.6 m.

3. INVESTIGATION PROCEDURES

The field work was carried out during the period from May 31 to June 21, 2006 and comprised a total of twelve sampled boreholes. The six boreholes drilled at the Four Mile Creek Bridge site were designated M1 to M6 and were advanced to depths of 8.2 to 17.0 m. The six boreholes drilled at the temporary bridge site were designated D1 to D6 and advanced to depths of 5.2 to 15.5 m.



The borehole layout for the Four Mile Creek bridge was established in accordance with the requirements noted in the Request for Proposal and the layout for the detour bridge was in accordance with the subsequent MTO Change Order. PML selected the location of the boreholes. The ground surface elevations of the boreholes were determined by MRC and referred to a geodetic benchmark. All elevations in this report are expressed in meters.

The boreholes were advanced using continuous flight solid stem augers through the soil cover with 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.

Soils were identified in accordance with the MTO Soil Classification Manual procedures. The groundwater conditions in the boreholes were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, where encountered, by measuring the groundwater level in the open boreholes. All of the boreholes were backfilled with a bentonite/cement mixture in accordance with the MTO guideline for borehole abandonment.

The recovered soil samples were returned to our laboratory for detailed visual examination and classification. The laboratory testing program for the Four Mile Creek Bridge and Detour Bridge consisted of the tests listed below. The table includes the designation of the Figures for the results of the grain size analyses and Atterberg limits for each bridge.

BRIDGE	WATER CONTENT DETERMINATIONS	GRAIN SIZE ANALYSES	ATTERBERG LIMITS DETERMINATIONS
Four Mile Creek	52	16	8
		M-GS-1 to M-GS-5	M-PC-1 to M-PC-3
Detour	46	15	6
		D-GS-1 to D-GS-4	D-PC-1 and D-PC-2

The Atterberg plasticity limits were not determined on samples that were judged to be non-plastic by visual and tactile examination. The test results are shown on the attached Record of Borehole sheets.



4. SUMMARIZED SUBSURFACE CONDITIONS

4.1. General

Reference is made to the appended Record of Borehole sheets M1 to M6 and D1 to D6 for details of the subsurface conditions including soil classifications, inferred soil stratigraphy, groundwater observations, natural moisture content determinations, and results of grain size analyses and plasticity limits tests. Borehole location plans and stratigraphic profiles prepared from the borehole data are presented on Drawings M1 and M2 for the Four Mile Creek Bridge and D1 and D2 for the Detour Bridge.

The Highway 6 asphalt and gravel shoulder pavement and approach embankment fills were encountered at the Four Mile Creek Bridge location. The native soil stratigraphy revealed in the boreholes was generally consistent at both bridge locations. On the south side of the Four Mile Creek, the native soils comprised discontinuous deposits of silty clay or clayey silt, sandy silt/silty sand and discontinuous silt layers overlying or interbedded with clayey silt till which in turn overlaid sandy silt till units. North of the Creek, the soil stratigraphy comprised gravelly sand/sand and gravel deposits overlying clayey silt till and sandy silt till deposits. Scattered cobbles and boulders were found in the boreholes.

The stratigraphy for each of the bridges is summarized separately on the following paragraphs.

4.2 Four Mile Creek Bridge

4.2.1 Pavement

The 130 mm thick asphalt of the Highway 6 pavement was encountered in five of the boreholes. The sixth borehole (M1) was drilled from the gravel shoulder south of the bridge. The granular base and subbase materials were 570 to 1070 mm thick for totals of 700 to 1200 mm thick pavements. The granular material was judged to be in a compact to dense condition with one N-value of 35 blows for 300 mm penetration of the sampler. One water content determination was about 3%.



4.2.2 Fill

Typically firm mixtures of clayey silt and silty clay with variable amounts of sand, gravel, cobbles, organics, topsoil and wood chips were encountered below the pavement granular materials in all boreholes except borehole M1 where the road base granular materials overlaid directly the native soils. The fill extended to depths ranging from 3.3 to 6.1 m, elevations 441.2 to 443.0. It is inferred that the fill will extend to deeper levels behind the existing abutment walls and footings. N-values in the embankment fill typically ranged from 4 to 8 blows, with one value of 48 blows due to the presence of wood in the sample.

The particle size distribution charts of three samples of the embankment fill are shown on Figure M-GS-1 and the Plasticity chart on Figure M-PC-1. Water content determinations ranged from 18 to 30%. The Atterberg plasticity liquid limits on the three fill samples ranged from 27 to 32 and the plastic limits from 13 to 18, indicating plasticity indexes from 13 and 14.

4.2.3 Silty Clay

A localized deposit of cohesive firm silty clay was found in borehole M3 between 4.3 and 5.3 m depths, elevations 443.0 and 442.0, respectively. A single N-value of 6 blows was obtained on the material.

4.2.4 Silt

A discontinuous layer of cohesionless compact to dense silt trace to some sand trace clay was encountered immediately below the embankment fill in borehole M2 and extended to 7.6 m depth, elevation 439.7. The unit was also found as an interbedded layer within the clayey silt till unit (described on the following subsection). The interbedded layers occurred between 8.7 and 11.1 m depths (elevations 436.2 and 438.6) in borehole M2; between 6.7 and 8.7 m depths (elevations 438.6 and 440.6) and also between 9.4 and 11.8 m depths (elevations 435.5 and 437.9) in borehole M3. The thickness of these layers varied from 1.5 to 2.4 m. N-values in the silt units ranged from 14 to 42 blows.



The grain size chart of two silt samples are shown on the attached Figure M-GS-2. Natural moisture content determinations in this cohesionless material ranged from 18 to 19%.

4.2.5 Clayey Silt Till

South of the Four Mile Creek (boreholes M1 to M3), a deposit of glacial till comprising very stiff to hard clayey silt till was encountered below the embankment/road fill in borehole M1 where it extended to the 8.2 m termination depth of the borehole, elevation 439.6. The till was also found below silt and local silty clay layers in boreholes M2 and M3 at 7.6 and 5.3 m depths, respectively (elevations 439.7 and 442.0) and extended to 8.7 and 6.7 m (elevations 438.6 and 440.6). A second layer of the clayey silt till was also found in borehole M3 between 8.7 and 9.4 m depths (elevations 437.9 and 438.6). N-values in the unit ranged from 18 to 43 blows.

North of the creek (boreholes M4 to M6), very stiff clayey silt till was found at 7.0 m in borehole M6 and extended to the 8.2 m termination depth of the borehole, elevation 437.8. The single N-value in the unit north of the creek was 25 blows.

The grain size distribution charts of two samples of the clayey silt till are shown on Figure M-GS-3, attached. The results of the Atterberg tests are shown on Figure M-PC-2. The liquid limits of the material were 25 to 30, the plastic limits 12 and 16 and the computed plasticity indexes 13 and 14. Natural moisture content determinations ranged from 8 to 19%.

4.2.6 Sand/Gravel

A cohesionless compact to very dense deposit of sand with gravel and gravel with sand trace to some silt was encountered in borehole M4 to M6 drilled on the north side of the Four Mile Creek. The soils occurred below the road embankments at depths ranging from 3.3 to 4.3 m (elevations 442.3 to 442.7) and extended to depths from 7.0 and 11.6 m (elevations 434.9 to 439.0). N-values in the deposit ranged from 12 to over 100 blows.

The envelope of the grain size distribution charts of four samples of the material is shown on Figure M-GS-4, attached. Natural moisture content determinations of 8 to 12%.



4.2.7 Sandy Silt Till

A cohesionless very dense glacial till consisting of sandy silt trace to some clay and variable gravel content was encountered below the clayey silt till and silt units at depths ranging from 10.2 to 11.8 m (elevations 434.9 to 436.5) in boreholes M2 to M5. The deposit extended to the 11.3 to 17.0 m termination depths of the boreholes (elevations 430.2 to 435.4). N-values in the deposit were in excess to 100 blows.

The envelope of the grain size distribution charts of five samples of the material is shown on Figure M-GS-5, attached. The results of the Atterberg tests are shown on Figure M-PC-3. The liquid limits of the material of three of the samples were 13 and 15, the plastic limits 10 and 11 and the computed plasticity indexes 2 to 4. Two other samples were non-plastic. Natural moisture content determinations were 6 to 8%.

4.2.8 Groundwater

The boreholes encountered groundwater during and upon completion of drilling. During the drilling, groundwater strikes were noted at depths ranging from 3.3 to 5.5 m, elevations 441.8 to 443.0. Upon completion of the drilling, the groundwater stabilized in the open boreholes at the depths and elevations summarized on the following table.

LOCATION	BOREHOLE NO.	DEPTH (m)	ELEVATION
South Approach	M1	5.5	442.3
South Abutment	M2	5.5	441.8
South Abutment	M3	4.3	443.0
North Abutment	M4	4.6	442.1
North Abutment	M5	4.3	442.2
North Approach	M6	3.5	442.5

It is inferred that the groundwater encountered reflects and is controlled by the local water level in the Four Mile Creek, which was measured at elevation 441.5 in June 2006.



The groundwater levels at this site are subjected to fluctuations due to seasonal rainfall patterns.

4.3 Detour Bridge

4.3.1 Topsoil

A layer of topsoil about 200 mm thick was encountered at the surface of boreholes D1, D2 and D6 drilled on the south bank of the Four Mile Creek. The topsoil extended to elevations 447.4 to 448.1.

4.3.2 Clayey Silt/Silty Sand/Sandy Silt

Typically very stiff/loose to compact layers of clayey silt or silty sand/sandy silt were encountered below the topsoil in borehole D2 and surficially in the boreholes D3 to D5. These layers were 0.7 to 1.2 m thick extended to 0.7 to 1.2 m depths, elevations 441.8 to 442.5 north of the creek and to elevation 446.5 south of the creek. N-values in the materials ranged from 8 to 17 blows.

Water content determinations in these deposits ranged from 14 to 22%.

4.3.3 Silt

A discontinuous layer of cohesionless compact to dense silt with varying amounts of clay and sand was interbedded within the clayey silt till unit (described on the following subsection) in boreholes D1 and D2 drilled south of the creek. In borehole D1, the interbedded layers were found between 8.4 and 9.5 m (elevations 439.3 and 438.2, respectively) and also between 10.2 and 13.2 m depths (elevations 437.5 and 434.5). In borehole D2, the silt occurred between 10.9 and 11.8 m depths (elevations 436.7 and 435.8). The thickness of these silt layers varied from 0.9 to 3.0 m. N-values in the silt deposits ranged from 28 to 43 blows.

The grain size distribution chart of one silt sample is shown on the attached Figure D-GS-1. Natural moisture content determinations in this cohesionless material ranged from 17 to 19%.



4.3.4 Clayey Silt Till

A deposit of cohesive glacial till comprising very stiff to hard clayey silt till was encountered in all boreholes. In borehole D1 and D6 the unit extended from below the topsoil layer. In borehole D2 the material was found below the clayey silt unit and in the boreholes D3, D4 and D5 the clayey silt till was found below a layer of gravel/sand described on the following subsection.

The material was interbedded by a layer(s) of silt in boreholes D1 and D2 as described previously. The unit extended to depths of 6.4 to 10.9 m (elevations 436.2 to 437.5) in boreholes D1 to D4 and to the 5.0 and 5.2 m termination depths (elevations 438.2 and 443.1) of boreholes D5 and D6. N-values in the unit ranged from 15 to 36 blows with isolated values of 13 and 14 blows.

The envelope of the grain size distribution charts of four samples of the clayey silt till is shown on Figure D-GS-2. The results of the Atterberg limits tests are shown on Figure D-PC-1. The liquid limits of the material were 22 and 23, the plastic limit was 13 and the computed plasticity indexes were 9 and 10. Natural moisture content determinations ranged from 8 to 21%.

4.3.5 Gravelly Sand/Sandy Gravel

A cohesionless compact to very dense deposit of gravelly sand to sandy gravel trace silt was encountered in boreholes D3, D4 and D5 drilled on the north side of the Four Mile Creek. These soils occurred below the surficial clayey silt/sandy silt/silty sand deposits and extended to depths ranging from 4.1 to 7.2 m (elevations 435.8 to 439.1). The material was interbedded with a layer of clayey silt till between 5.6 and 6.4 m depths (elevations 436.6 and 437.4) in borehole D4. N-values in the deposit ranged from 17 to 55 blows.

The envelope of the grain size distribution charts of six samples of the material is shown on Figure D-GS-3. Natural moisture content determinations in the materials ranged from 5 to 19%.



4.3.6 Sandy Silt Till/Silt Till

A cohesionless very dense glacial till consisting of sandy silt to silt with sand some clay and variable gravel content was encountered below the silt, clayey silt till and gravelly sand/sandy gravel deposits. The material was found at depths ranging from 6.8 to 7.2 m (elevations 435.8 to 436.2) in boreholes D3 and D4, drilled north of the creek and 11.8 and 13.2 m (elevations 434.5 to 435.8) in boreholes D1 and D2 drilled south of the creek. The deposit extended to the 8.0 to 15.5 m termination depths (elevations 432.1 to 435.0) of boreholes D1 to D4. N-values in the deposit were in excess of 100 blows.

The envelope of the grain size distribution charts of four samples of the material is shown on Figure D-GS-4. Natural moisture content determinations of 2 to 4% were obtained.

4.3.7 Groundwater

The boreholes encountered groundwater during and after completion of drilling except borehole D2 (after drilling) and borehole D6 drilled on the south bank. During the drilling, groundwater strikes were noted at depths ranging from 1.2 to 8.5 m, elevations 439.2 to 441.6. Upon completion of the drilling, the groundwater stabilized in the open boreholes except D2 and D6 at the depths and elevations summarized on the following table.

LOCATION	BOREHOLE NO.	DEPTH (m)	ELEVATION
South Approach	D6	No free water	-
South Abutment	D1	8.8	438.9
South Abutment	D2	No free water	-
North Abutment	D3	1.2	441.8
North Abutment	D4	1.2	441.8
North Approach	D5	1.8	441.4



It is inferred that the groundwater encountered reflects and is controlled by the local water level in the Four Mile Creek, which was measured at elevation 441.5 in June 2006.

The groundwater levels at this site are subjected to fluctuations due to seasonal rainfall patterns.

5. CLOSURE

The subsurface investigation was carried out under the supervision of Mr. F. Portela and direction of Mr. C. M. P. Nascimento, P. Eng., Senior Project Engineer. Geo-Environmental Drilling Inc. supplied the drilling equipment. This report was prepared by Mr. C. M.P. Nascimento, P. Eng. and reviewed by Mr. Brian R. Gray, M. Eng., P. Eng, MTO Designated Contact and Project Manager.

Sincerely,

Peto MacCallum Ltd.

A handwritten signature in black ink, appearing to read "C. Nascimento", is written over the name and title of Carlos M.P. Nascimento.

Carlos M.P. Nascimento, P.Eng.
Senior Project Engineer



A handwritten signature in blue ink, appearing to read "Brian R. Gray", is written over the name and title of Brian R. Gray.

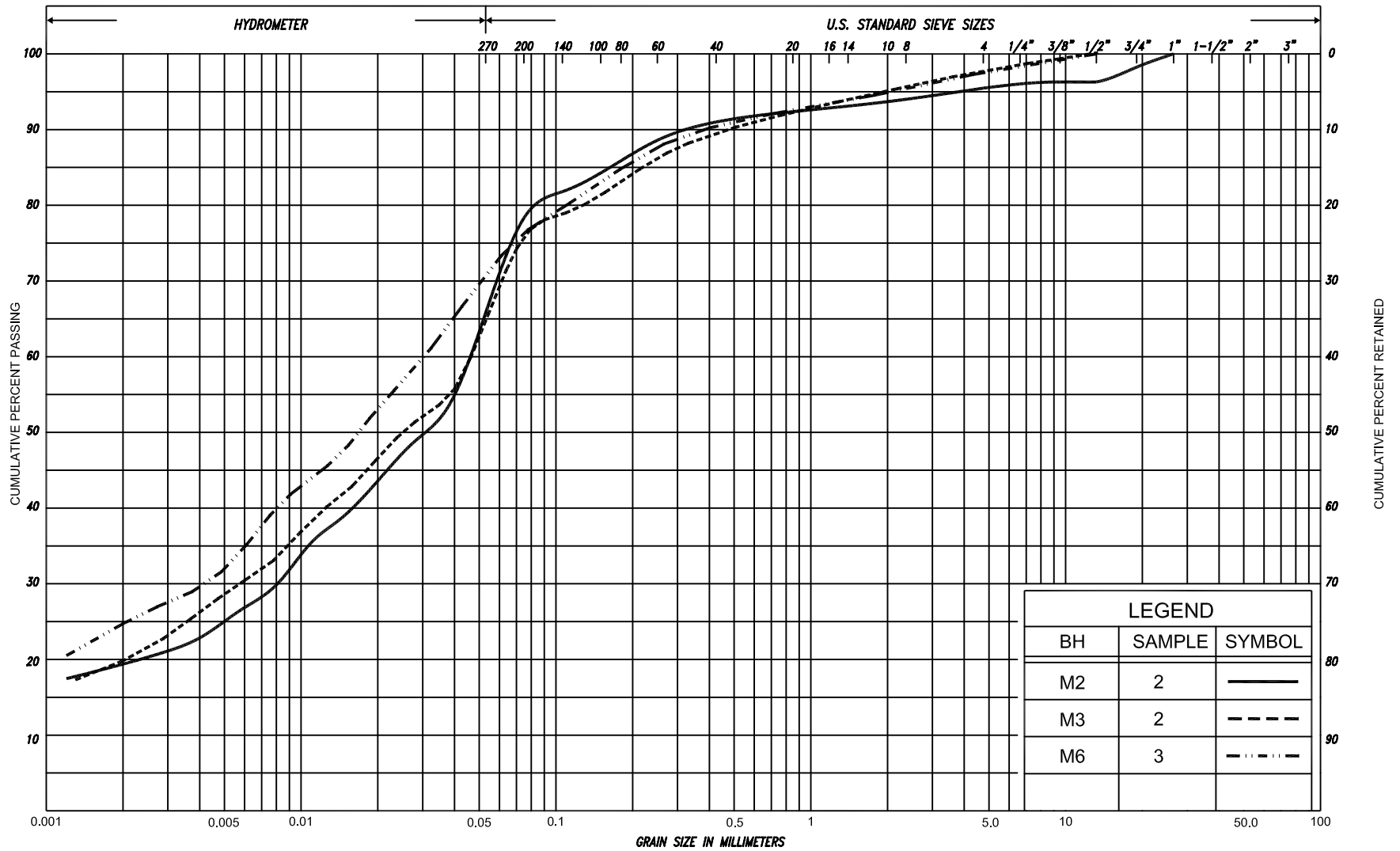
Brian R. Gray, M. Eng., P. Eng,
MTO Designated Contact and Project Manager



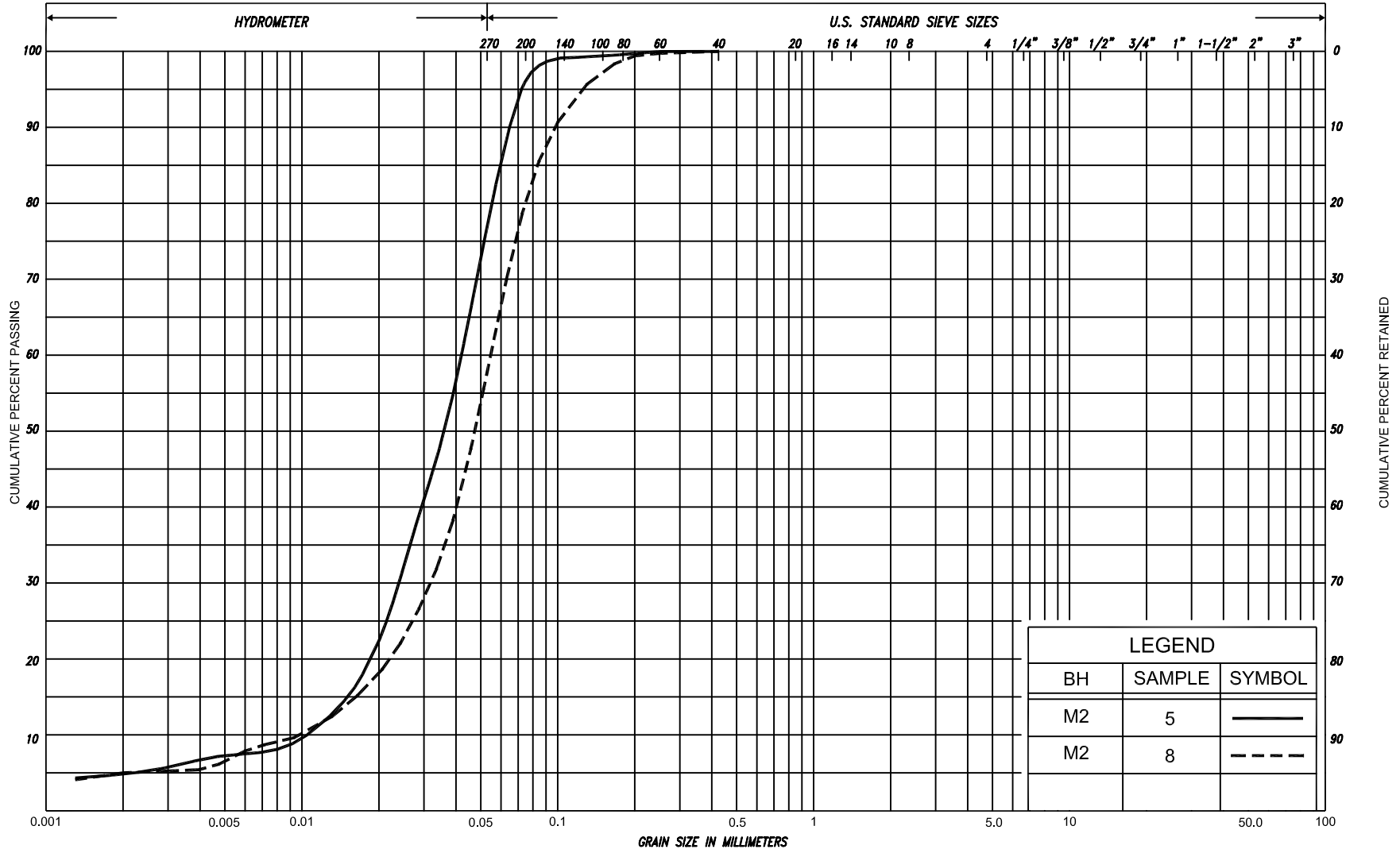
Four Mile Creek Bridge Replacement, Site No. 35-78
Improvement of Highway 6, Arthur to Mount Forest
GWP No.: 342-97-00, Index No.: 151FIR
PML Ref.: 05KF104A, January 12, 2007



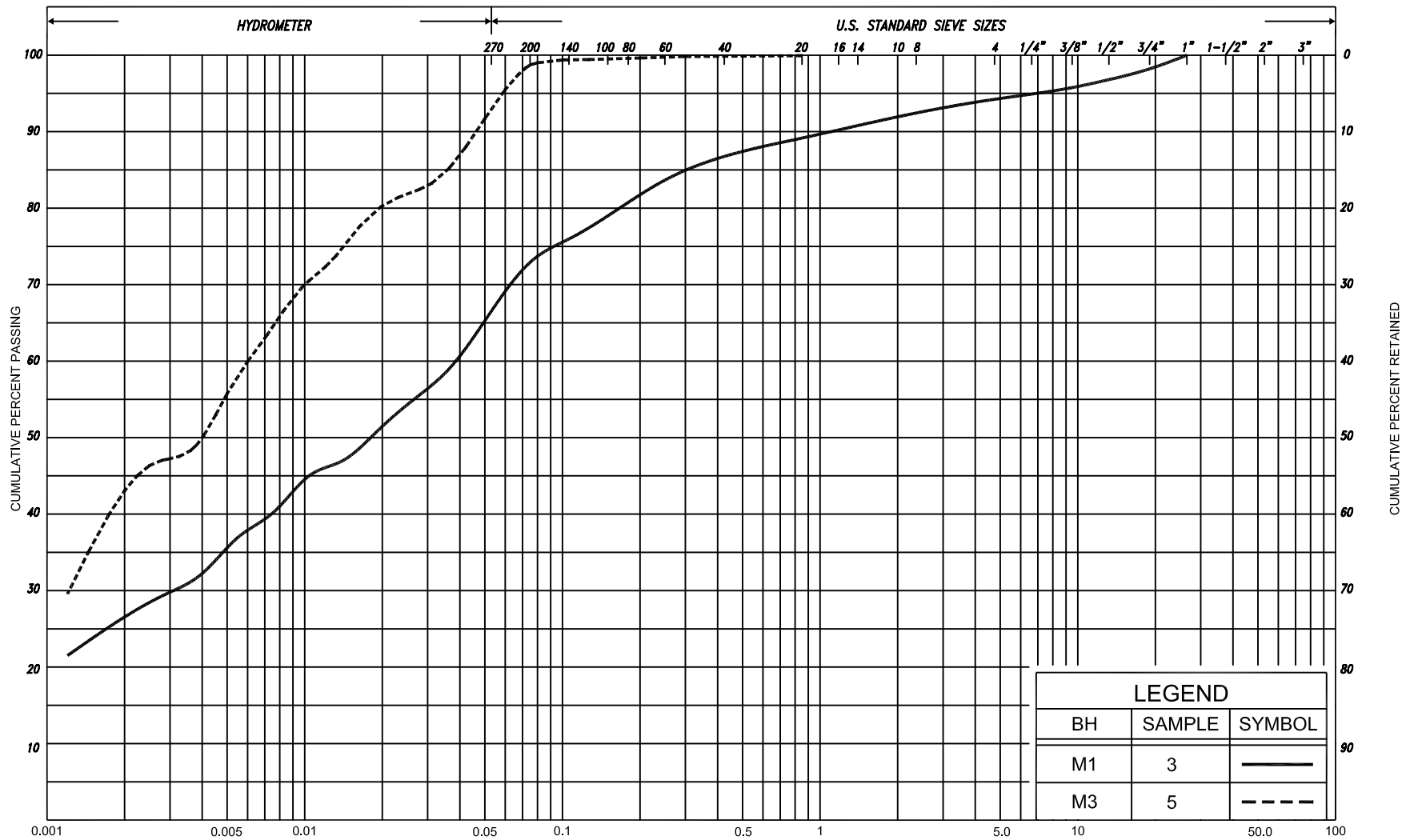
FOUR MILE CREEK BRIDGE



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									



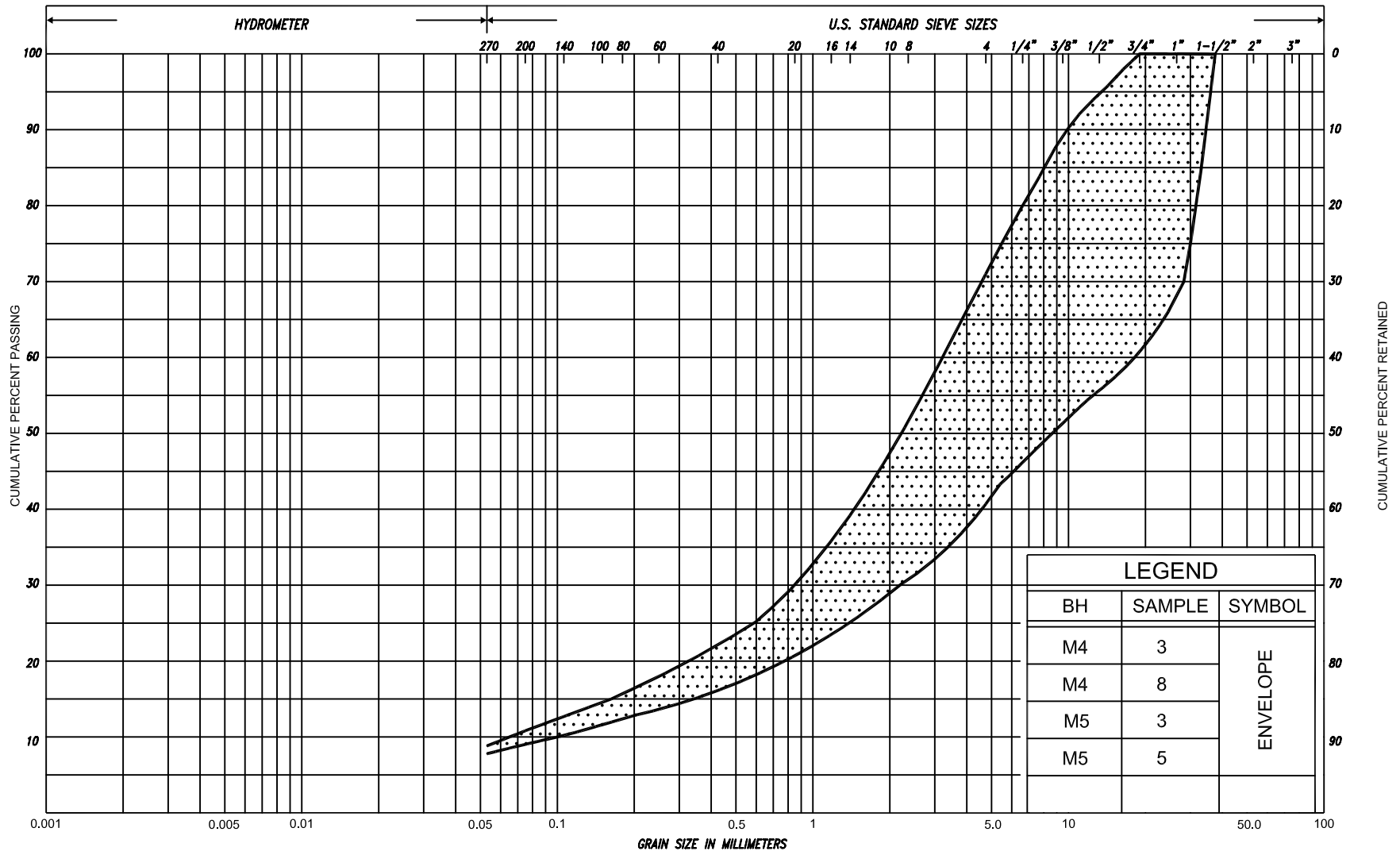
SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL		COB BLES	UNIFIED
CLAY	SAND			FINE		MEDIUM		COARSE	GRAVEL		COBBLES	
	SILT			V. FINE		FINE		MED.	COARSE	GRAVEL		U.S. BUREAU



LEGEND		
BH	SAMPLE	SYMBOL
M1	3	—
M3	5	- - -

SILT & CLAY				GRAIN SIZE IN MILLIMETERS				COBBLES		UNIFIED	
				FINE	MEDIUM		COARSE	GRAVEL			
CLAY	FINE	MEDIUM	COARSE	SAND			GRAVEL			COBBLES	
				FINE	MEDIUM	COARSE					
				V. FINE	FINE	MED.	COARSE	GRAVEL			U.S. BUREAU
				SAND							

GRAIN SIZE DISTRIBUTION CLAYEY SILT trace to with sand, trace gravel (TILL)



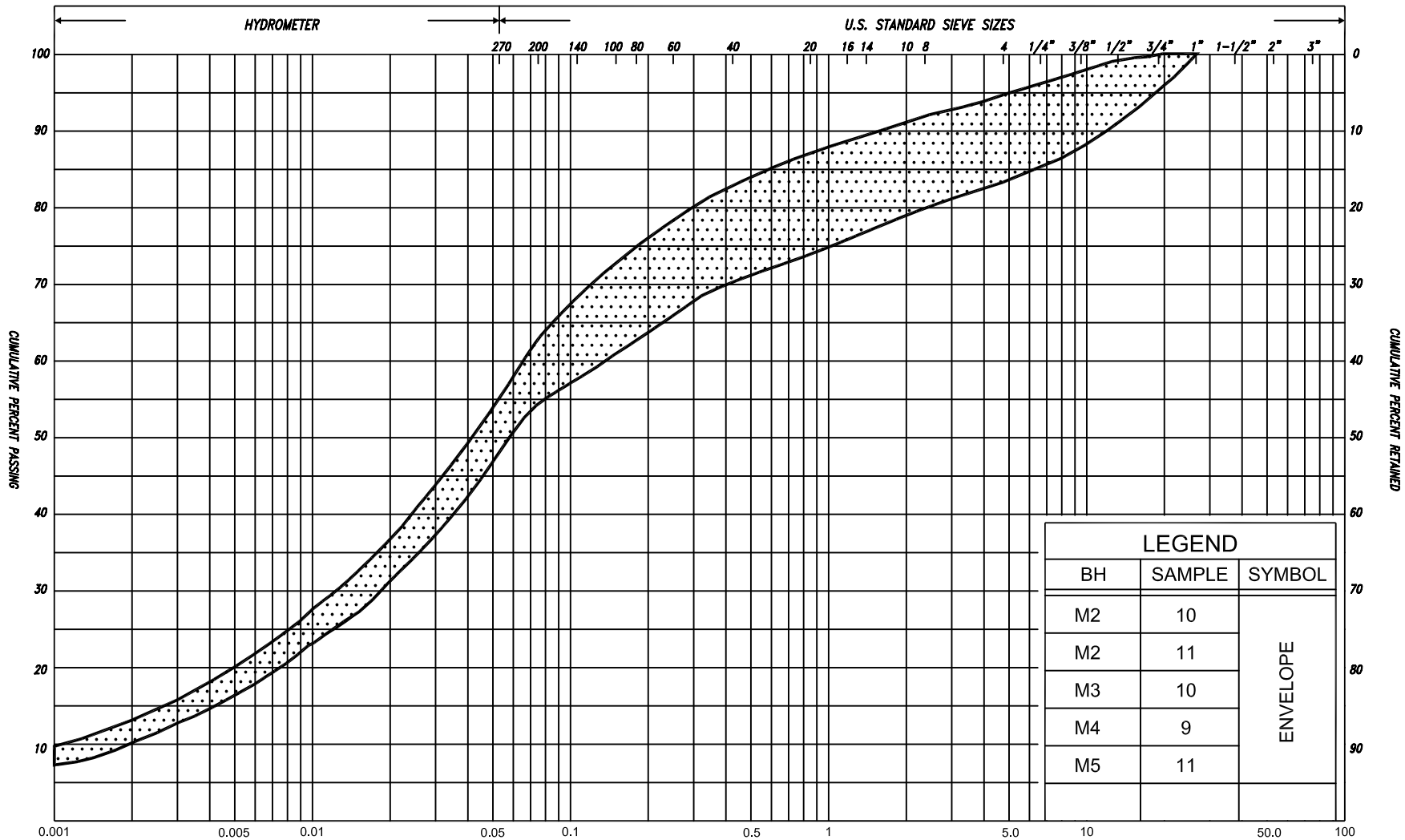
SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED	
CLAY	FINE		MEDIUM		COARSE	SAND										
	SILT				FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.	
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL						U.S. BUREAU

GRAIN SIZE DISTRIBUTION SAND with gravel to GRAVEL with sand trace to some silt

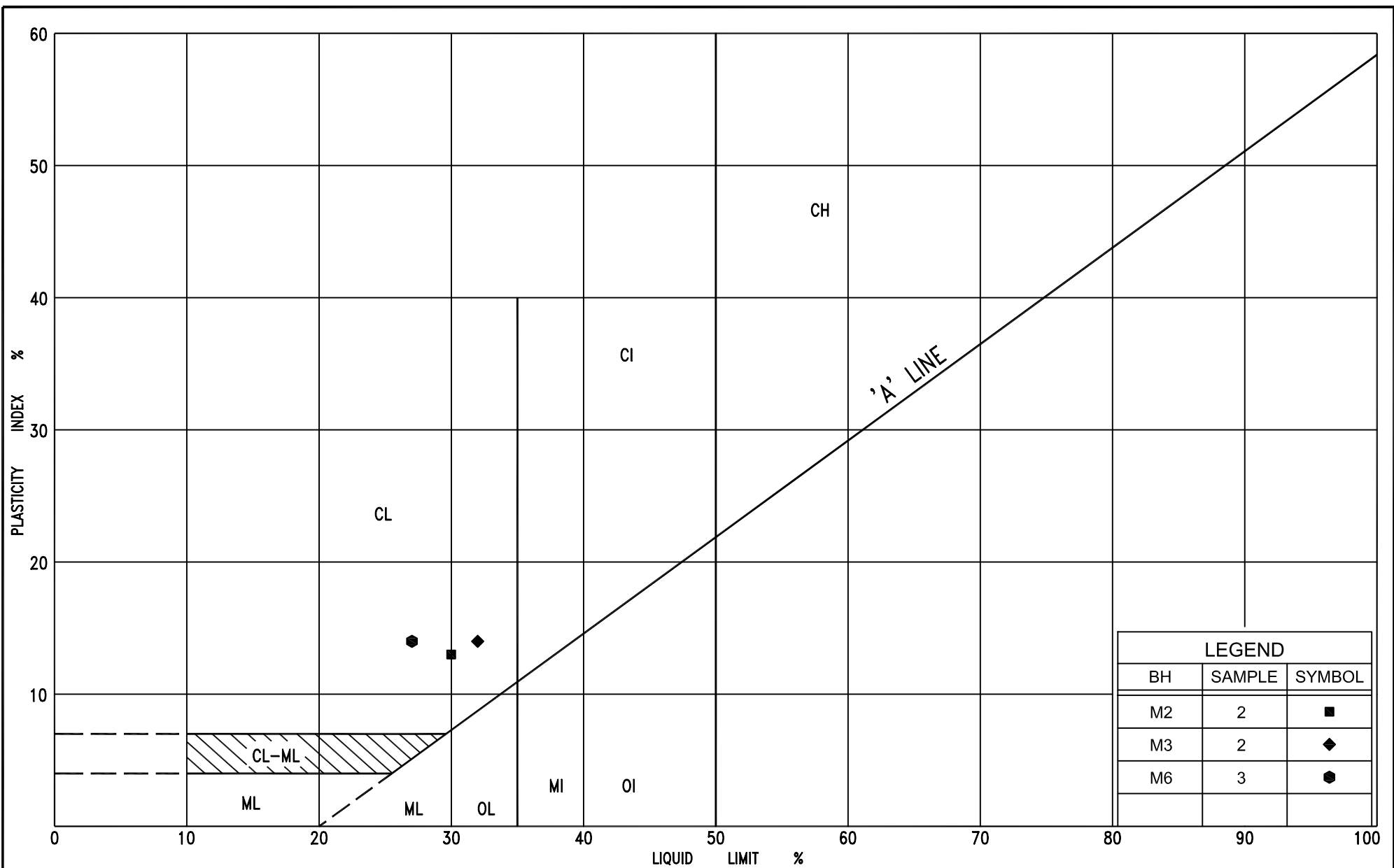
FIG No. M-GS - 4

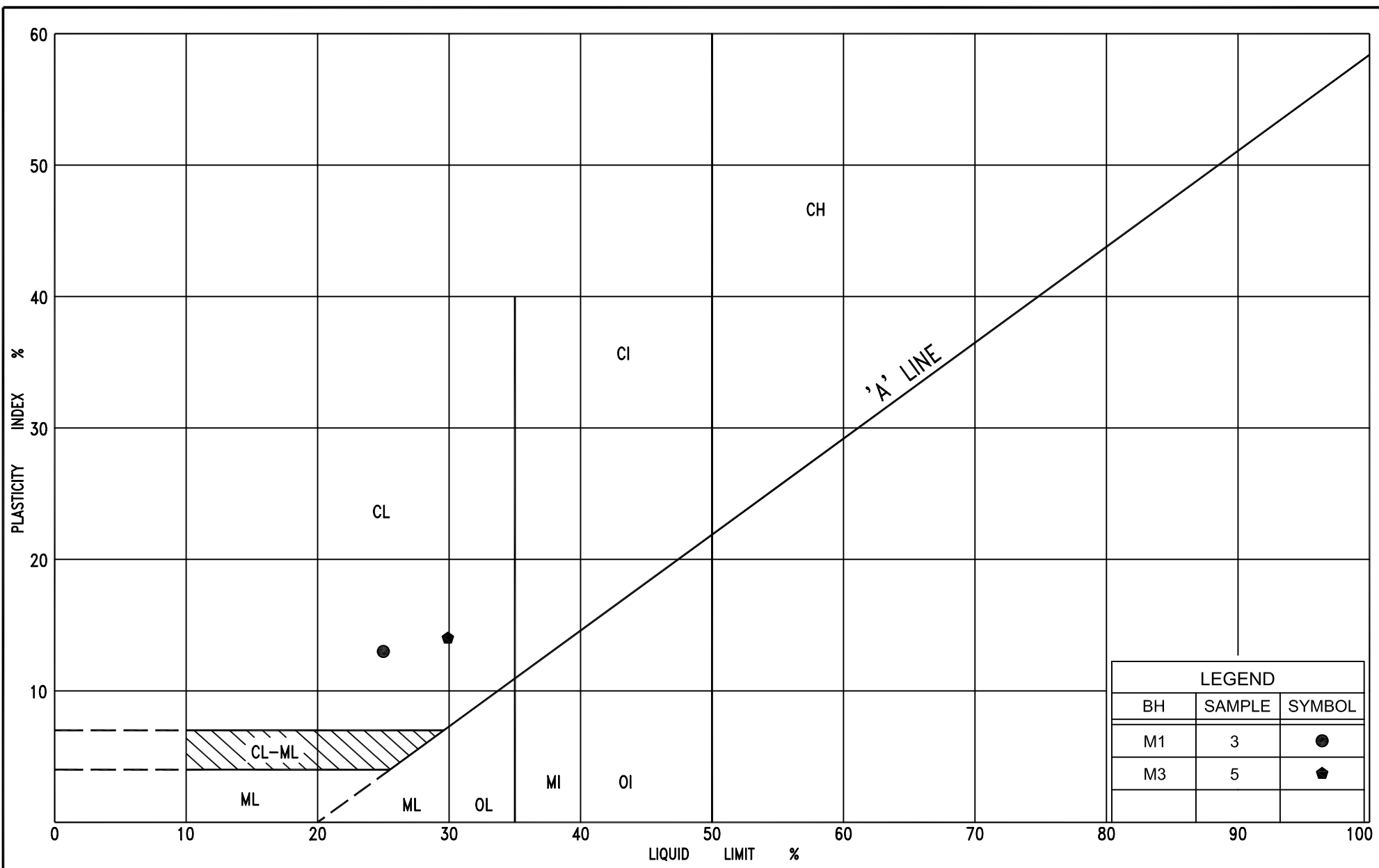
PROJECT: FOUR MILE CREEK BRIDGE REPLACEMENT

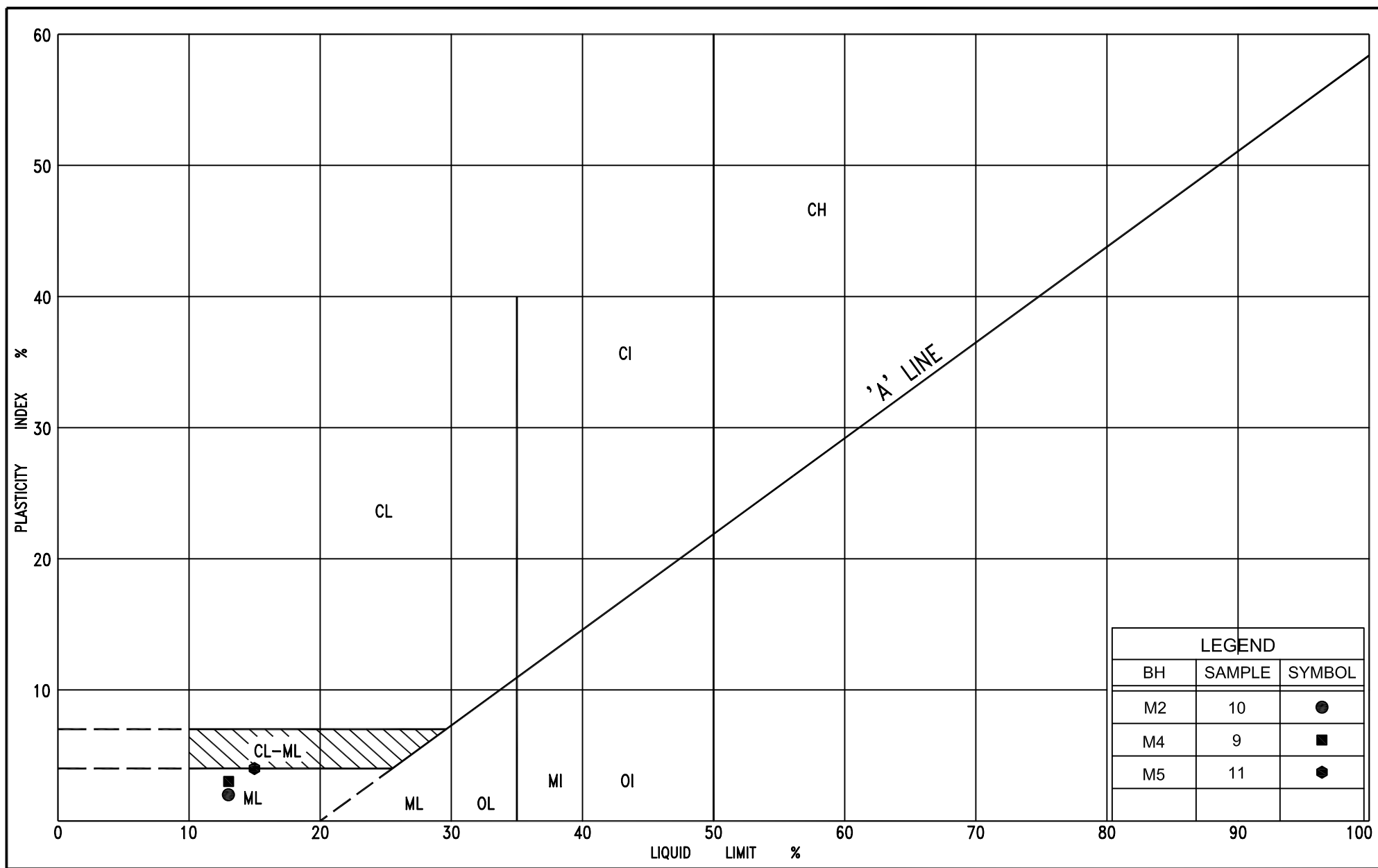
G.W.P. No. 342-97-00



SILT & CLAY				FINE		MEDIUM		COARSE		GRAVEL				COBBLES	UNIFIED	
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL		COBBLES	M.I.T.
	SILT						V. FINE		FINE		MED.		COARSE		GRAVEL	







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.

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_t	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 M1

1 of 1

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 730 N; 217 136 E
Sta. 16+373.4, o/s 5.3m Lt. ORIGINATED BY F.P.
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.
DATUM Geodetic DATE May 31, 2006 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					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							PLASTIC LIMIT W _P	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L
447.8	Ground surface																
0.0	Sand and gravel		1	SS	35												
447.1	Dense Brown Damp (PAVEMENT FILL)																
0.7	Clayey silt some sand, trace gravel		2	SS	18		447										
	Very Brown Moist stiff																
			3	SS	19		446	225							6 21 46 27		
	Grey (TILL)		4	SS	24		445	175									
			5	SS	22		444	138									
							443	213									
			6	SS	24												
							442										
							441										
							440										
439.6	Hard (TILL)		8	SS	38												
8.2	End of borehole																
				</													

RECORD OF BOREHOLE No M2

1 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 747 N; 217 125 E
Sta. 16+393.4, o/s 5.8m Lt. ORIGINATED BY F.P.
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.
DATUM Geodetic DATE June 05, 2006 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 (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									WATER CONTENT (%)			GR
447.3	Ground surface					↓* ↓*														
0.0	Asphalt (130mm)																			
0.1	Sand and gravel																			
446.4	Compact Brown Damp (PAVEMENT FILL)																			
0.9	Clayey silt some sand, trace gravel topsoil and gravel inclusions																			
	Firm Brown Moist to wet		1	SS	7															
			2	SS	4															
	organic inclusions																			
	Grey Wet																			
	(FILL)		3	SS	5															
	wood chips																			
			4	SS	48															
441.2																				
6.1	Silt trace sand, trace clay		5	SS	26															
	Compact Grey Wet to dense																			
			6	SS	33															
439.7																				
7.6	Clayey silt, trace sand silt lenses		7	SS	36															
	Hard Grey Moist (TILL)																			
438.6																				
8.7	Silt some sand, trace clay																			
	Dense Grey Wet to compact		8	SS	42															
			9	SS	14															
436.2																				
11.1	Sandy silt some clay, trace gravel																			
	Compact Grey Moist																			
	Very dense Brown (TILL)		10	SS	60/ 15cm															
	trace to some clay some gravel																			
	Damp		11	SS	50/ 15cm															

RECORD OF BOREHOLE No M2

2 of 2

METRIC

G.W.P. 342-97-00 **LOCATION** Co-ords: 4 858 747 N; 217 125 E
 Sta. 16+393.4, o/s 5.8m Lt. **ORIGINATED BY** F.P.
DIST Owen Sound HWY 6 **BOREHOLE TYPE** Continuous Flight Solid Stem Augers **COMPILED BY** G.D.
DATUM Geodetic **DATE** June 05, 2006 **CHECKED BY** C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			SHEAR STRENGTH kPa					W _p	W	W _L		
432.3			12	SS	125/ 20cm	432										
	Grey					431										
430.3			13	SS	135/ 8cm											
17.0	End of borehole															
	* 2006 06 05  Water level observed during drilling  Water level measured after drilling															

RECORD OF BOREHOLE No M3

1 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 754 N; 217 135 E
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers ORIGINATED BY F.P.
DATUM Geodetic DATE June 07, 2006 COMPILED BY G.D.
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 (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
447.3	Ground surface													
0.0	Asphalt (130 mm)													
0.1	Sand and gravel													
446.6	Compact Brown Damp (PAVEMENT FILL)													
0.7	Clayey silt some sand, trace gravel organic inclusions													
	Firm Dark Moist brown to wet (FILL)		1	SS	5									
			2	SS	6									
443.0	Silty clay, trace gravel													
4.3	Firm Grey Moist to stiff		3	SS	6									
442.0	Clayey silt, trace sand silt lenses		4	SS	43									
5.3	Hard Grey Moist (TILL)		5	SS	30									
440.6	Silt, trace sand		6	SS	31									
6.7	Compact Brown Wet to dense		7	SS	28									
438.6	Clayey silt, trace sand													
8.7	Very Brown Moist stiff (TILL)		8	SS	31									
437.9	Silt with sand, trace clay													
9.4	Dense Brown Moist													
	Compact Grey		9	SS	14									
435.5	Silt, with sand some clay, trace gravel		10	SS	83/ 15cm									
11.8	Very Grey Moist dense to damp (TILL)		11	SS	75/ 15cm									




Cont'd

RECORD OF BOREHOLE No M3

2 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 754 N; 217 135 E ORIGINATED BY F.P.
 DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.
 DATUM Geodetic DATE June 07, 2006 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			20	40	60	80	100					
432.3																	
431.8			12	SS	86/ 15cm		432										
15.5	End of borehole																
	* 2006 06 07  Water level observed during drilling  Water level measured after drilling  Penetrometer test																

RECORD OF BOREHOLE No M4

1 of 1

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 770 N; 217 113 E
Sta. 16+419.8, o/s 5.8m Lt. ORIGINATED BY F.P.
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.
DATUM Geodetic DATE June 08 and 2006 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	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100					
446.7	Ground surface																
0.0	Asphalt (130 mm)																
0.1	Sand and gravel																
	Compact Brown Damp																
445.7	(PAVEMENT FILL)																
1.0	Clayey silt with sand, trace gravel organic inclusions Firm Brown Moist		1	SS	8												
	Organics																
	Dark brown Wet (FILL)		2	SS	4												
442.4	Gravel with sand, some silt																
4.3	Compact Grey Moist to dense		3	SS	19								o				59 30 (11)
441.0	Sand, medium to coarse		4	SS	36												
5.7	Very Grey Wet dense																
440.4	Gravel, with to trace sand cobbles		5	SS	51								o				
	Dense Grey Wet to very dense		6	SS	50/ 15cm												
			7	SS	42								o				
438.0	Sand, with gravel trace to some silt																
8.7	Very Grey Wet dense		8	SS	94								o				29 61 (10)
436.5	Sandy silt some clay, trace gravel																
10.2	Very Grey Moist dense																
435.4	(TILL)		9	SS	113								oH				7 31 50 12
11.3	End of borehole																
	* 2006 06 07/08																
	Water level observed during drilling																
	Water level measured after drilling																

RECORD OF BOREHOLE No M5

1 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 780 N; 217 121 E
Sta. 16+424.3, o/s 5.8m Rt. ORIGINATED BY F.P.
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.
DATUM Geodetic DATE June 06, 2006 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
446.5	Ground surface													
0.0	Asphalt (130 mm)													
0.1	Sand and gravel													
	Compact Brown Moist (PAVEMENT FILL)													
445.3	Clayey silt with sand, trace gravel organic inclusions		1	SS	6									
1.2	Firm Dark Moist brown (FILL)													
	Wet													
			2	SS	4									
442.3	Gravel, with sand trace to some silt cobbles													
4.2	Compact Brown Wet		3	SS	19									56 34 (10)
			4	SS	29									
440.6	Sand with gravel, trace silt													
5.9	Dense Brown Wet		5	SS	33									32 59 (9)
439.8	Sandy gravel, cobbles													
6.7	Very Grey Wet dense		6	SS	50/ 15cm									
			7	SS	78									
437.9	Sand, coarse trace gravel													
8.6	Dense Grey Wet		8	SS	42									
436.3	Gravelly sand, cobbles													
10.2	Very Grey Wet dense		9	SS	15/8cm									
434.9	Sandy silt some clay, some gravel													
11.6	Very Grey Moist dense (TILL)		10	SS	110/ 15cm									
			11	SS	105/ 15cm									17 28 43 12

Cont'd

RECORD OF BOREHOLE No M5

2 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 780 N; 217 121 E
Sta. 16+424.3, o/s 5.8m Rt. ORIGINATED BY F.P.
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.
DATUM Geodetic DATE June 06, 2006 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES		20	40	60	80	100	W _p	W	W _L		
431.5			12	SS	82/ 15cm	431						0				
430.2																
16.3	End of borehole															
	* 2006 06 06															
	▽ Water level observed during drilling															
	▼ Water level measured after drilling															

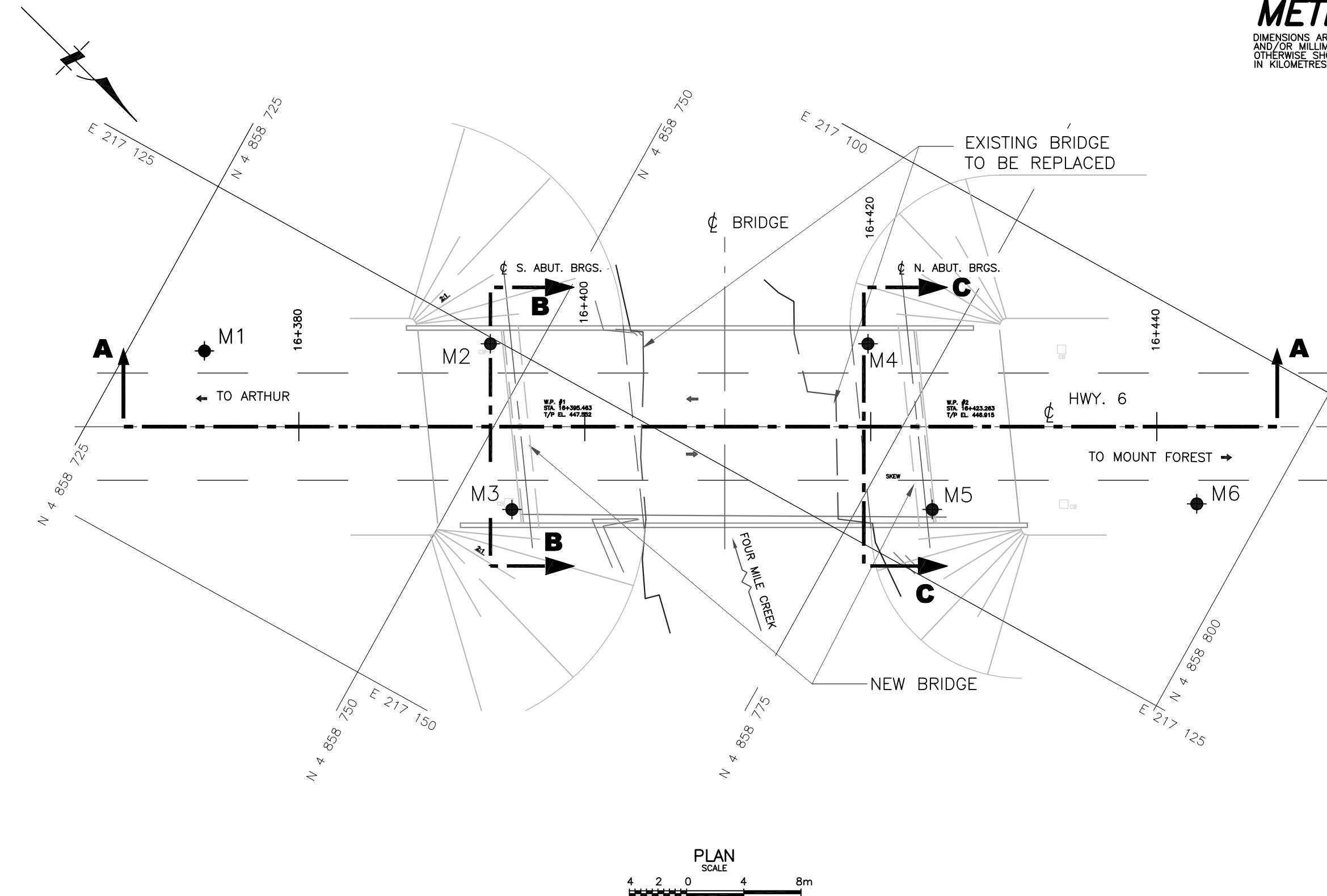
RECORD OF BOREHOLE No M6

1 of 1

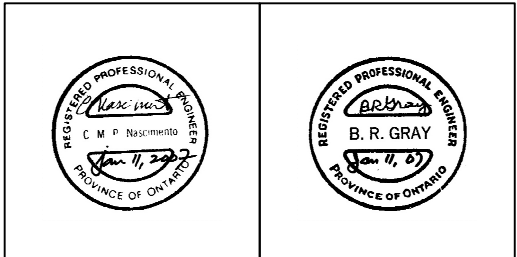
METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 796 N; 217 111 E
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers ORIGINATED BY F.P.
DATUM Geodetic DATE June 01, 2006 COMPILED BY G.D.
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	REMARKS & GRAIN SIZE DISTRIBUTION (%)			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa										WATER CONTENT (%)		
								○ UNCONFINED		+ FIELD VANE								● QUICK TRIAXIAL		
446.0	Ground surface						20	40	60	80	100									
0.0	Asphalt (130 mm)																			
0.1	Sand and gravel clayey silt inclusions Brown		1	SS	8															
445.2	(PAVEMENT FILL)																			
0.8	Clayey silt with sand, trace gravel		2	SS	16															
	very stiff Brown Moist to stiff /grey																			
	(FILL)		3	SS	12															
	Firm		4	SS	6															
442.7	organic inclusions																			
3.3	Sand, trace gravel		5	SS	12															
	Compact Brown Moist to wet																			
441.9																				
4.1	Sandy gravel to Gravelly sand, trace silt																			
	Compact Grey Wet		6	SS	28															
	Dense																			
			7	SS	45															
439.0	Clayey silt trace sand, trace gravel																			
7.0	Very stiff Grey Moist (TILL)																			
437.8			8	SS	25															
8.2	End of borehole																			
								</												



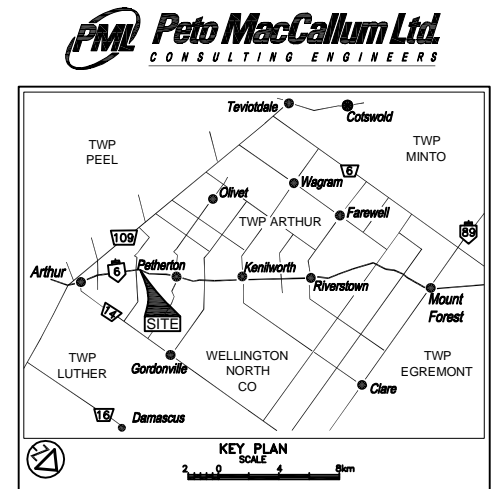
- NOTES:
- REFER TO DRAWINGS M2 FOR SECTIONS A-A, B-B AND C-C.
 - THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.



REF No S6259-335-001GA DATED AUGUST 24, 2006

METRIC
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

CONT No GWP No 342-97-00	
FOUR MILE CREEK BRIDGE Sta. 16+412 SITE 35-78 BOREHOLE LOCATIONS	SHEET



LEGEND			
	Borehole		
	Dynamic Cone Penetration Test (Cone)		
	Borehole & Cone		
N	Blows/0.3m (Std. Pen Test, 475 J / blow)		
CONE	Blows/0.3m (60 Cone, 475 J / blow)		
	W L at time of investigation May-June 2006		
	Head		
	ARTESIAN WATER		
	PIEZOMETER		
BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
M1	447.8	4 858 730	217 136
M2	447.3	4 858 747	217 125
M3	447.3	4 858 754	217 135
M4	446.7	4 858 770	217 113
M5	446.5	4 858 780	217 121
M6	446.0	4 858 796	217 111

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

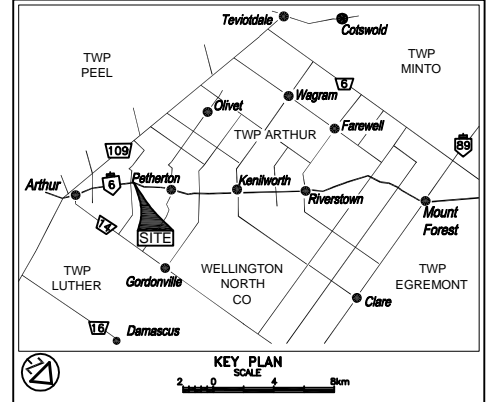
REVISIONS			
DATE	BY	DESCRIPTION	
JULY 18/07	CN	REPLACED PROPOSED BRIDGE (MRC email DATED JUNE 19, 2007)	
MAY 1/07	NK	DWG RENUMBERED FOR MRC FINAL PACKAGE	
Geocres No. 40P15-39			
HWY No	6	DIST	OWEN SOUND
SUBM'D FP	CN	DATE	JAN. 11, 2007
DRAWN	NA	CHECKED	CN
APPROVED	BRG	SITE	35-78
DWG	M1		

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

CONT No
GWP No 342-97-00
FOUR MILE CREEK BRIDGE
Sta. 16+412
Site 35-78
SOIL SECTIONS

PML Peto MacCallum Ltd.
CONSULTING ENGINEERS



LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60 Cone, 475 J / blow)
- W L at time of investigation May-June 2006
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

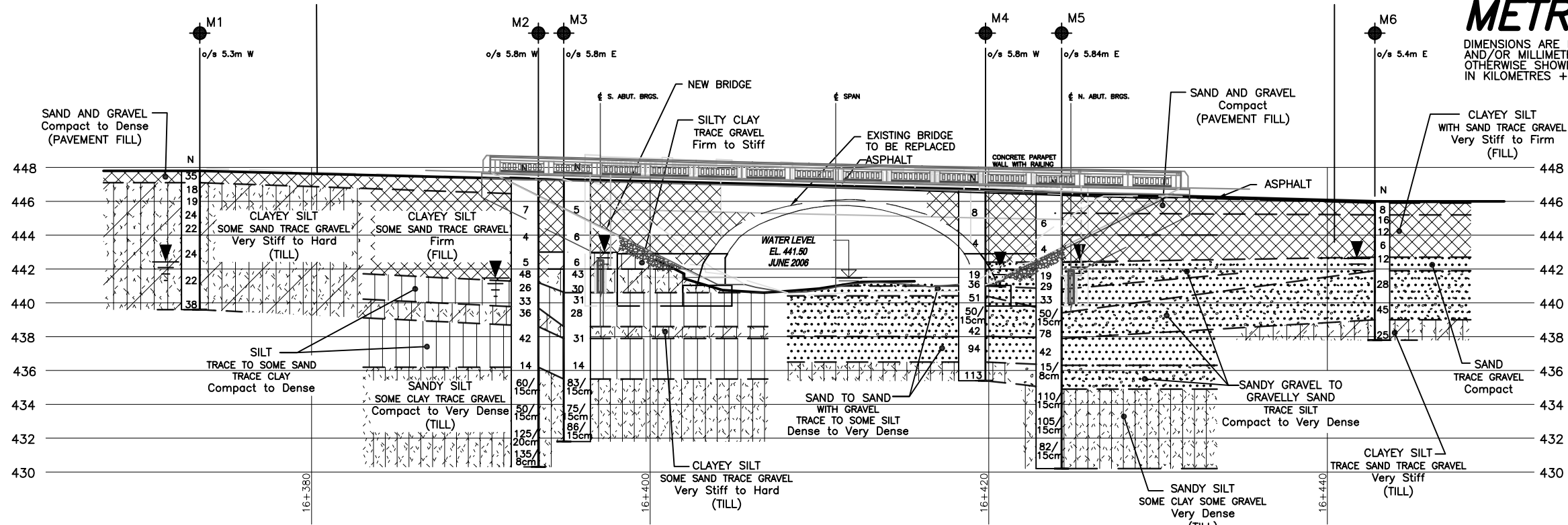
BH No	ELEVATION	CO—ORDINATES	
		NORTH	EAST
REFER DRAWING M1 FOR DETAILS			

NOTE

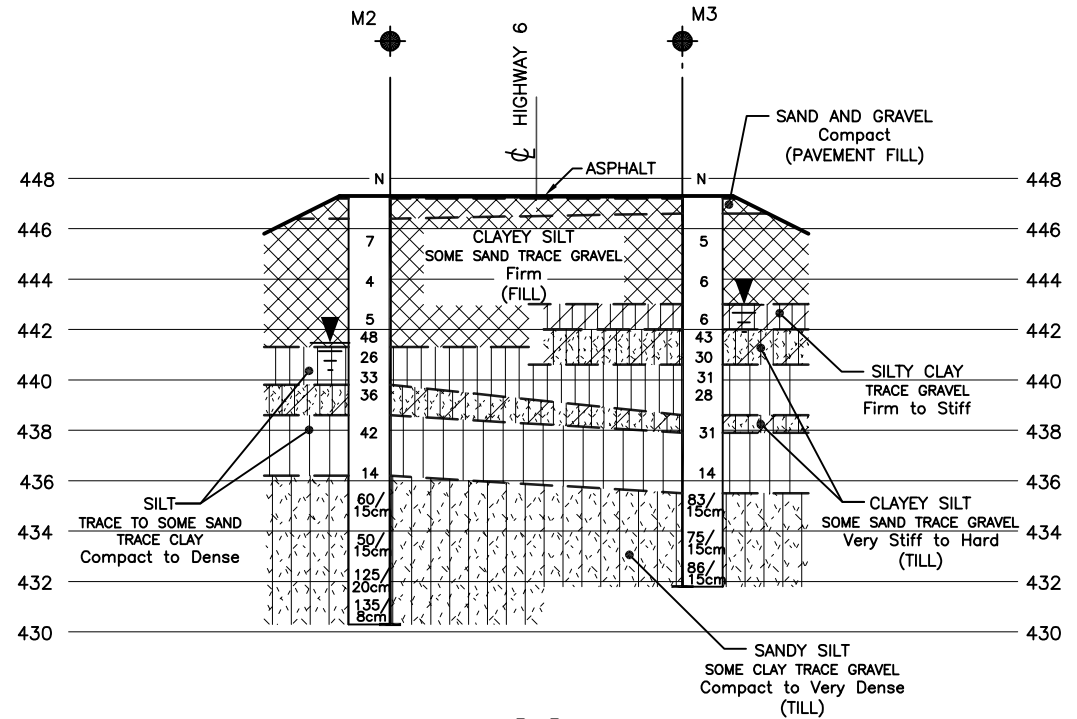
The boundaries between soil strata have been established only at Borehole locations. Between Boreholes the boundaries are assumed from geological evidence.

REVISIONS		DATE	BY	DESCRIPTION
JUL 18/07	CN			REPLACED PROPOSED BRIDGE (MRC email dated June 19, 2007)
MAY 1/07	NK			DWG RENUMBERED FOR MRC FINAL PACKAGE

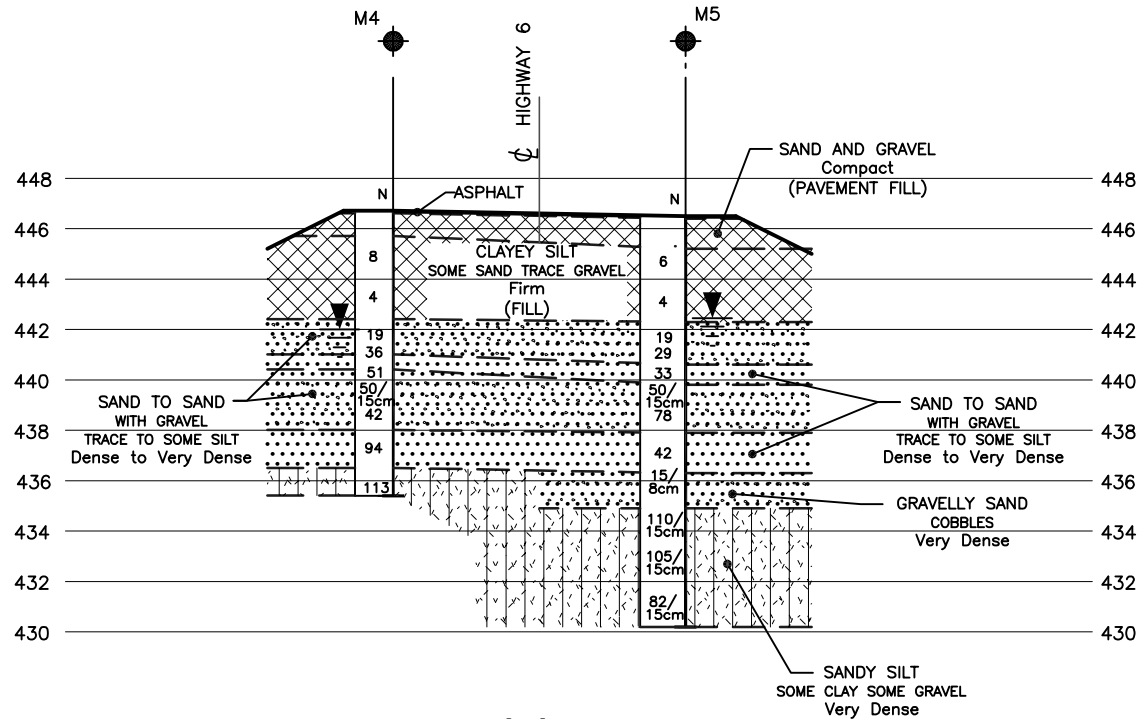
Geocres No. 40P15-39		HWY No 6		DIST OWEN SOUND	
SUBM'D FP	CHECKED CN	DATE JAN. 11, 2007	SITE 35-78		
DRAWN NA	CHECKED CN	APPROVED BRG	DWG M2		



A-A



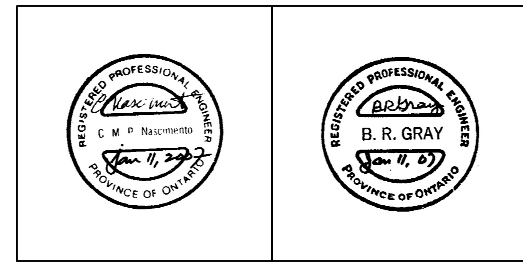
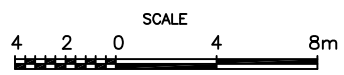
B-B



C-C

NOTES:

- REFER TO DRAWING M1 FOR PLAN AND LOCATION OF SECTIONS A-A, B-B AND C-C.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

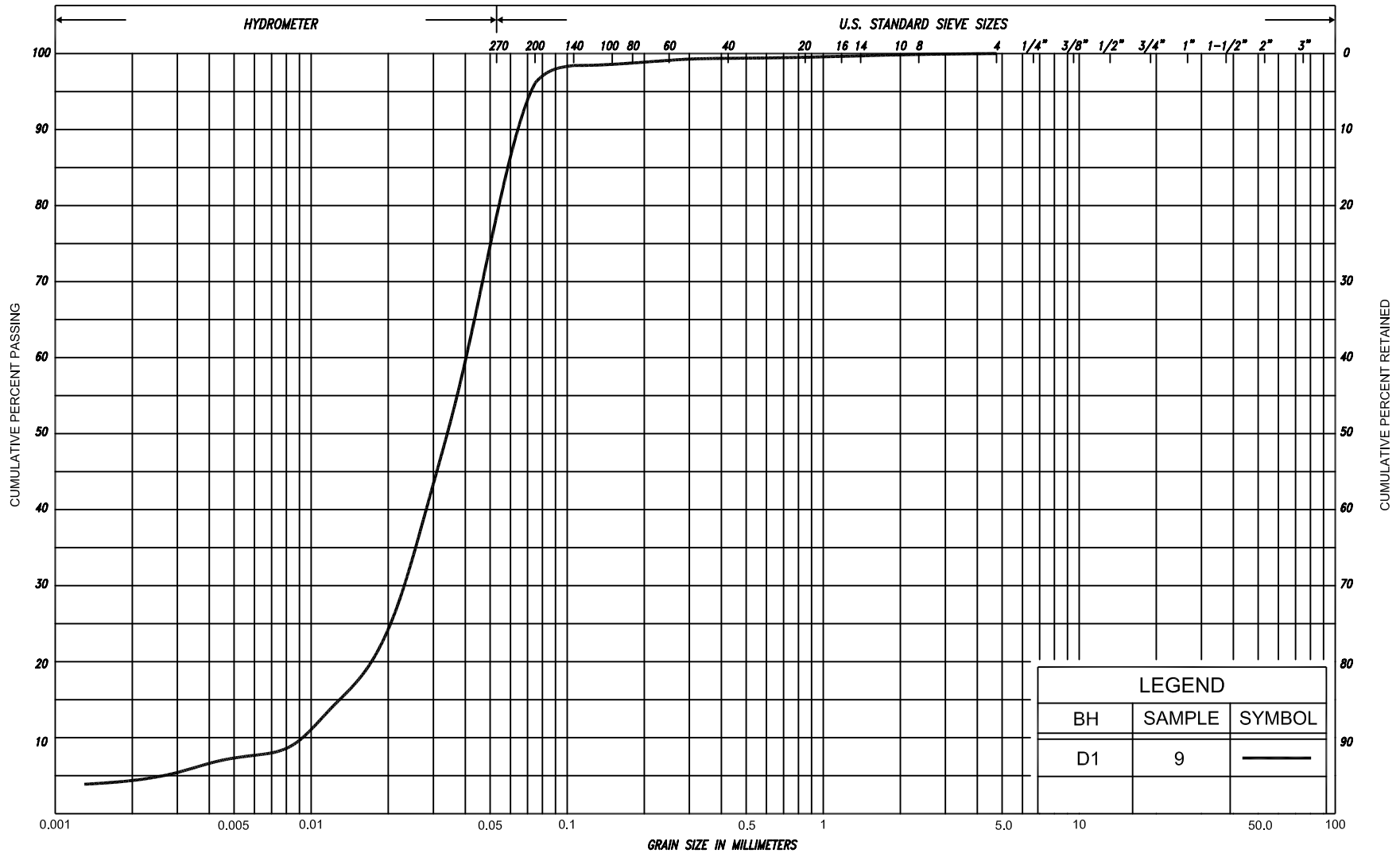


REF No S6259-335-001GA DATED AUGUST 24, 2006

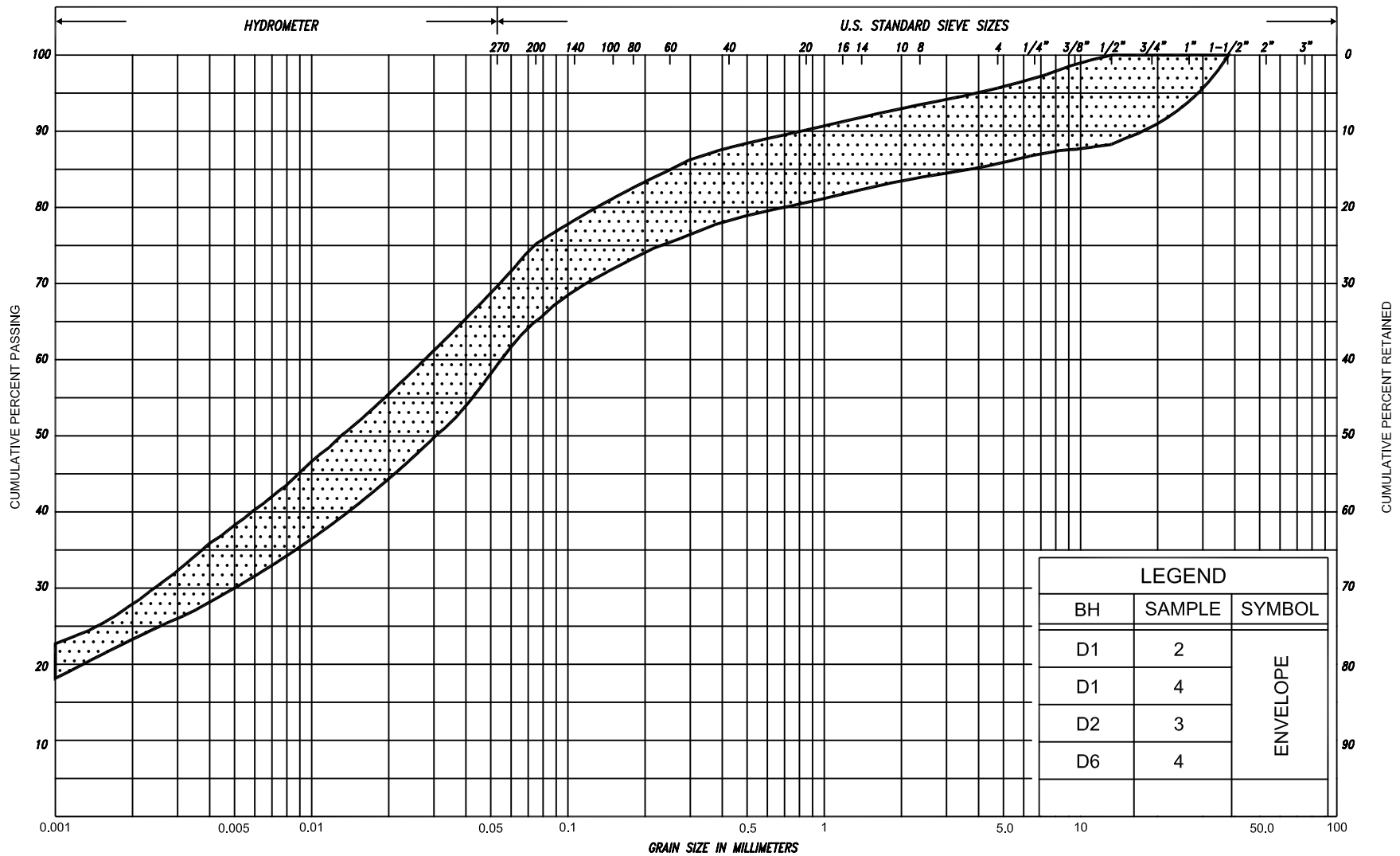
Four Mile Creek Bridge Replacement, Site No. 35-78
Improvement of Highway 6, Arthur to Mount Forest
GWP No.: 342-97-00, Index No.: 151FIR
PML Ref.: 05KF104A, January 12, 2007



DETOUR BRIDGE



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

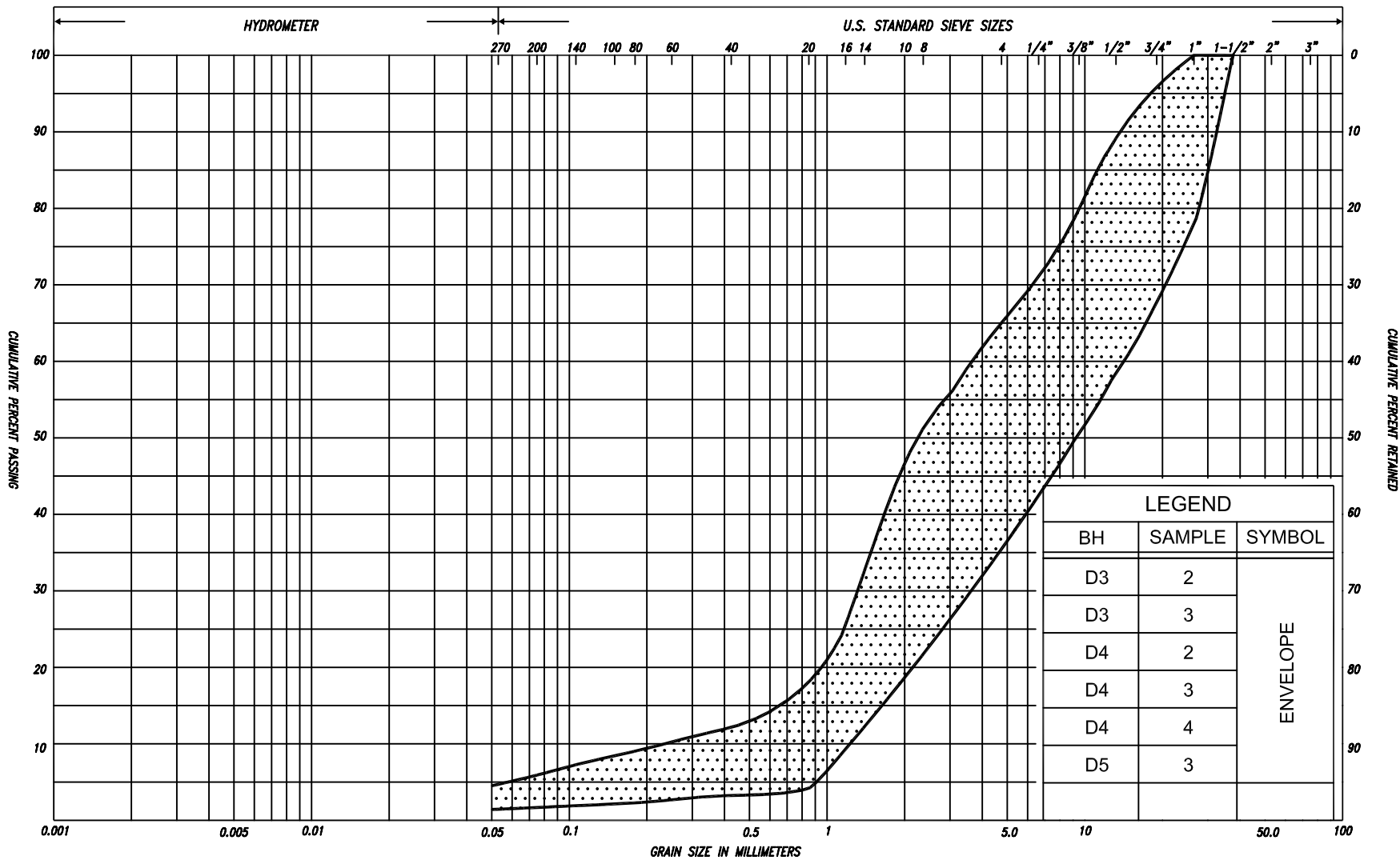


LEGEND		
BH	SAMPLE	SYMBOL
D1	2	ENVELOPE
D1	4	
D2	3	
D6	4	

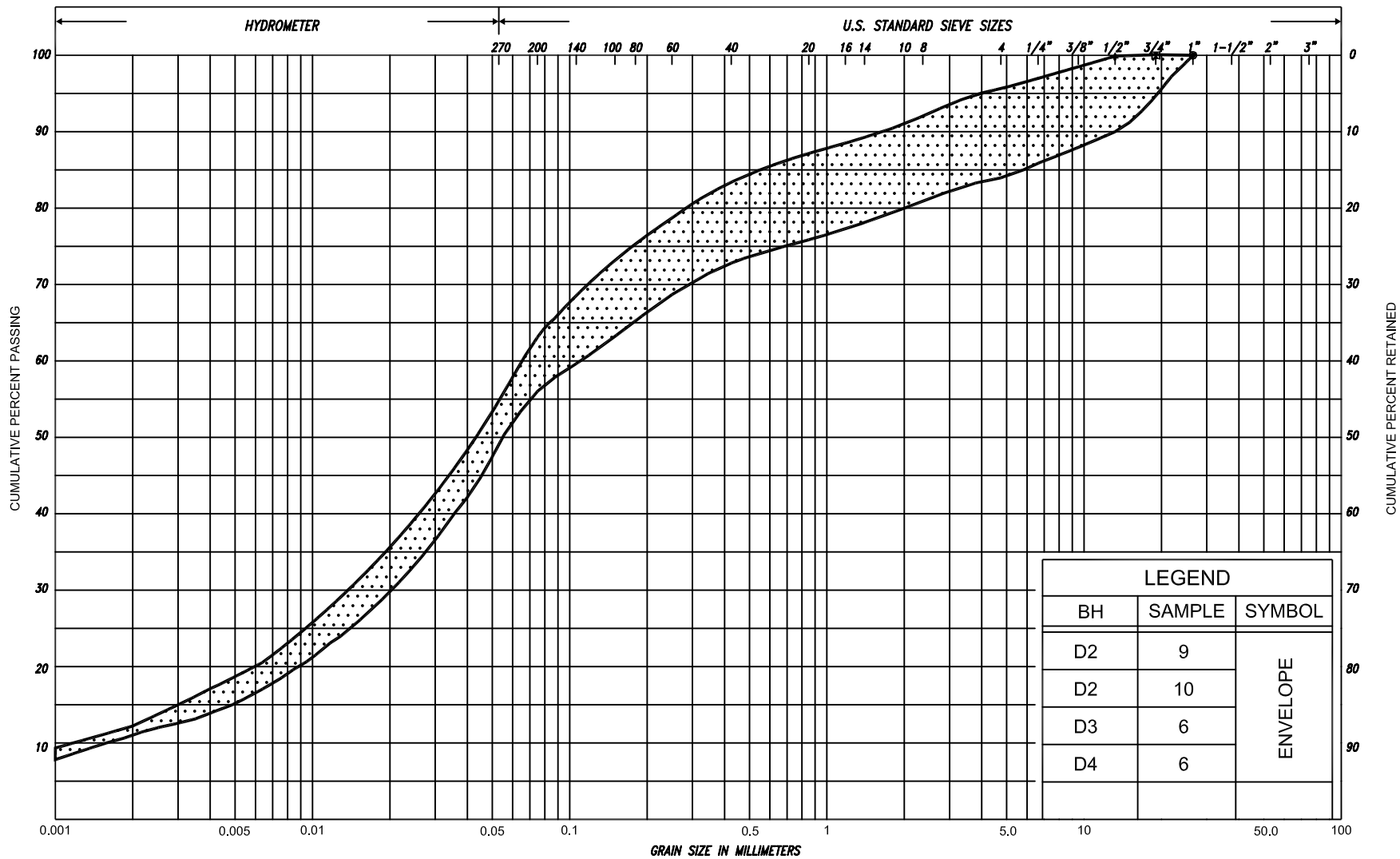
SILT & CLAY				FINE SAND			MEDIUM SAND		COARSE SAND		GRAVEL		COBBLES	UNIFIED
														M.I.T.
CLAY				FINE	MEDIUM SILT		COARSE	FINE	MEDIUM SAND		COARSE	GRAVEL		COBBLES
CLAY	SILT			V. FINE	FINE	MED.	COARSE	SAND		GRAVEL				U.S. BUREAU

GRAIN SIZE DISTRIBUTION CLAYEY SILT, with sand, trace to some gravel (TILL)

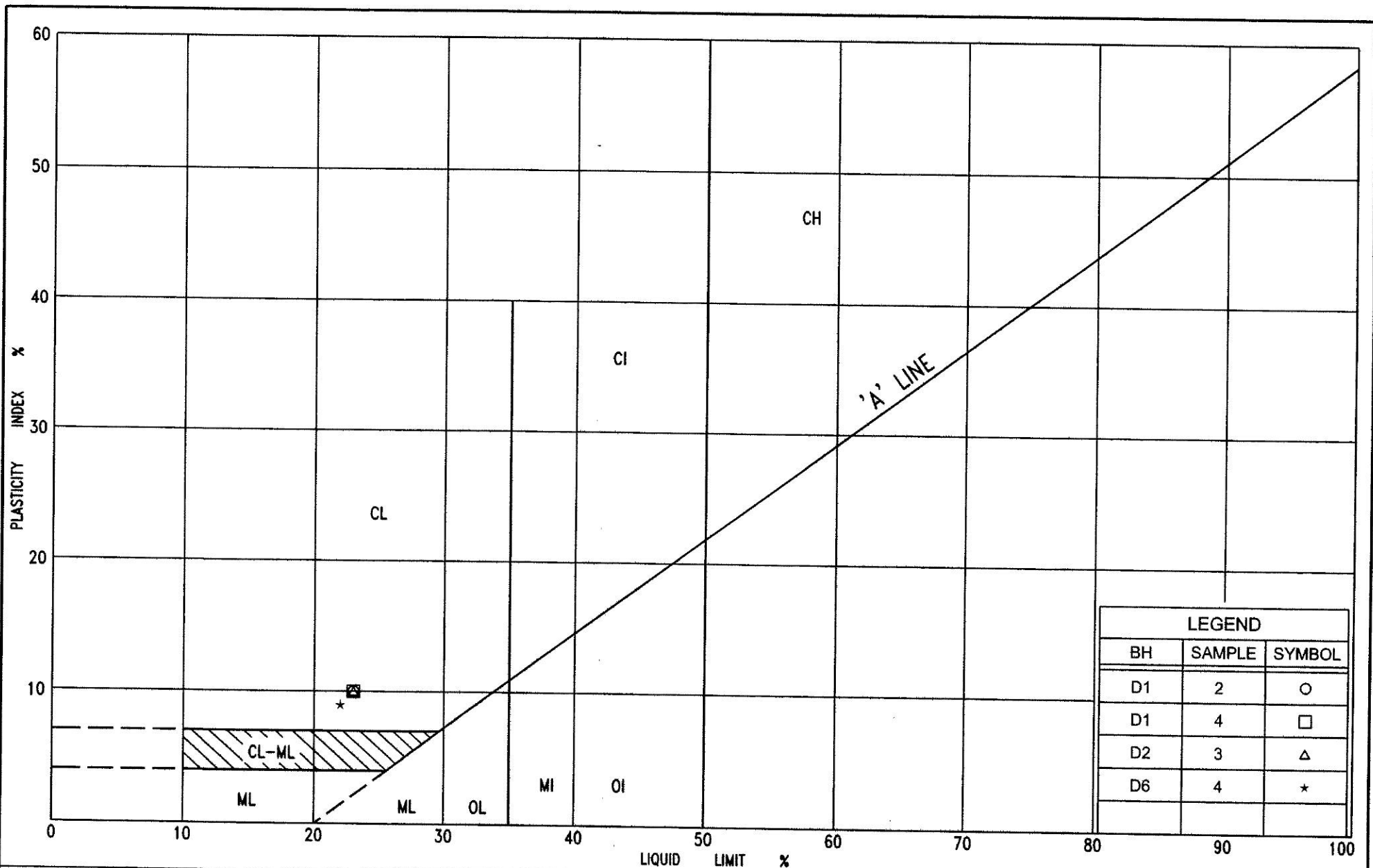
FIG No. D-GS-2
PROJECT: FOUR MILE CREEK
DETOUR BRIDGE
G.W.P. No. 342-97-00



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												



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



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

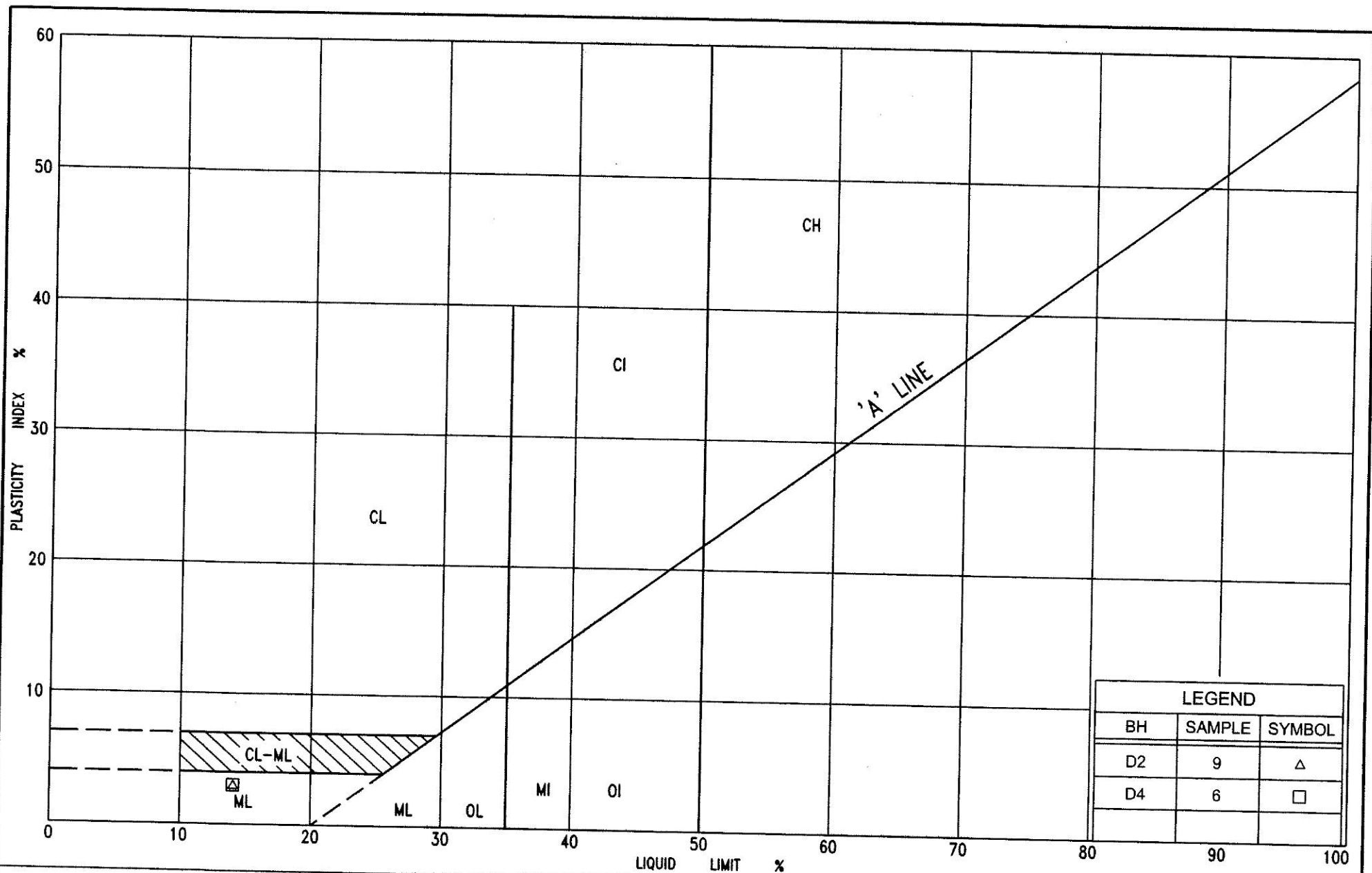


Ministry of
Transportation
Ontario

FIG No. D-PC-1

PROJECT: FOUR MILE CREEK
DETOUR BRIDGE

G.W.P. No. 342-97-00



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.

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_t	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^2	SEEPAGE FORCE
e	1, %	VOID RATIO						

RECORD OF BOREHOLE No D1

1 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 728 N; 217 119 E
Hwy 6 Detour, Sta. 26+380.3, o/s 3.0m Lt. ORIGINATED BY F.P.
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.
DATUM Geodetic DATE June 15, 2006 CHECKED BY C.N.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20	40	60	80	100		
447.7	Ground surface													
0.0	Topsoil													
0.2	Clayey silt with sand, some gravel Very stiff Brown Moist (TILL)		1	SS	22		447							
			2	SS	24		446							14 21 42 23
	Grey		2A	SS	36		445							
			3	SS	23		444							
	trace gravel		3A	SS	35		443							4 21 48 27
			4	SS	21		442							
			5	SS	25		441							
			6	SS	19		440							
439.3	Silt trace clay, trace sand Compact Grey Wet						439							
438.2	Clayey silt with sand, trace gravel Very stiff Grey Moist (TILL)		7	SS	28		438							
437.5	Silt trace sand, trace clay Dense Grey Moist to wet		8	SS	37		437							
10.2			9	SS	36		436							0 4 92 4
434.5	Silt trace sand, trace clay Very dense Grey Moist (TILL)		10	SS	131		434							
14.0	End of borehole													

Cont'd

RECORD OF BOREHOLE No D1

2 of 2

METRIC

G.W.P. 342-97-00

LOCATION

Co-ords: 4 858 728 N; 217 119 E

Hwy 6 Detour, Sta. 26+380.3, o/s 3.0m Lt.

ORIGINATED BY F.P.

DIST Owen Sound HWY 6

BOREHOLE TYPE

Continuous Flight Hollow Stem Augers

COMPILED BY G.D.

DATUM Geodetic

DATE _____

June 15, 2006

CHECKED BY C.N.

SOIL PROFILE					
ELEV DEPTH	DESCRIPTION	STRAT PLOT	SAMPLES	GROUND WATER CONDITIONS	ELEVATION SCALE
432.7			NUMBER TYPE "N" VALUES		DYNAMIC CONE PENETRATION RESISTANCE PLOT <div><div>20406080100</div><div></div></div>
	* 2006 06 15 ▽ Water level observed during drilling ▼ Water level measured after drilling ■ Penetrometer test				SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE <div>20406080100</div>
					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT W _p W W _L WATER CONTENT (%)
					UNIT WEIGHT γ kN/m³
					REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL

METRIC



ON_MOT VER3 05KF104-FOUR MILE CREEK-DETOUR.GPJ ON_MOT.GDT 1/15/2007 4:33:16 PM

RECORD OF BOREHOLE No D2

2 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 733 N; 217 123 E ORIGINATED BY F.P.
Hwy 6 Detour, Sta. 26+382.3, o/s 3.0m Rt.
 DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.
 DATUM Geodetic DATE June 16, 2006 CHECKED BY C.N.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			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					W _p	W	W _L		
432.6							20 40 60 80 100									
432.1			11	SS	123											
15.5	End of borehole					432										
	* 2006 06 16  Water level observed during drilling  Penetrometer test															

METRIC

ON_MOT VER3 05KF104-FOUR MILE CREEK-DETOUR.GPJ ON_MOT.GDT 1/15/2007 4:33:19 PM

+ , X⁵ : Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

METRIC

CHECKED BY C.N.

ON_MOT VER3 05KF104-FOUR MILE CREEK-DETOUR.GPJ ON_MOT.GDT 1/15/2007 4:33:22 PM

+ , X⁵ : Numbers refer to Sensitivity

15 20 5 10 (%) STRAIN AT FAILURE

METRIC

CHECKED BY C.N.

ON_MOT VER3 05KF104-FOUR MILE CREEK-DETOUR.GPJ ON_MOT.GDT 1/15/2007 4:33:24 PM

+ , X⁵ : Numbers refer to Sensitivity

2.0
1.5 — 5
1.0

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No D6

1 of 1

METRIC

G.W.P. 342-97-00 LOCATION Co-ords: 4 858 718 N; 217 132 E
Hwy 6 Detour, Sta. 26+365.3, o/s 4.0m Rt. ORIGINATED BY F.P.
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.
DATUM Geodetic DATE June 21, 2006 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/m³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
448.3	Ground surface																
0.0	Topsoil																
0.2	Clayey silt trace sand, trace gravel		1	SS	16		448										
	Very stiff Brown/ Moist stiff grey (TILL)		2	SS	15							125					
			3	SS	13		447					175					
			4	SS	18		446					163					8 21 46 25
		5	SS	16		445					125						
						444											
443.1	Grey		6	SS	15						125						
5.2	End of borehole																
	* Borehole dry on completion of drilling																
	■ Penetrometer test																

METRIC

DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

CONT No
GWP No 342-97-00

FOUR MILE CREEK DETOUR BRIDGE

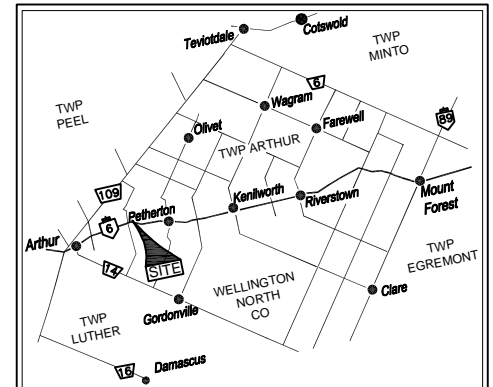
Sta. 16+412

Site 35-78

BOREHOLE LOCATIONS



SHEET



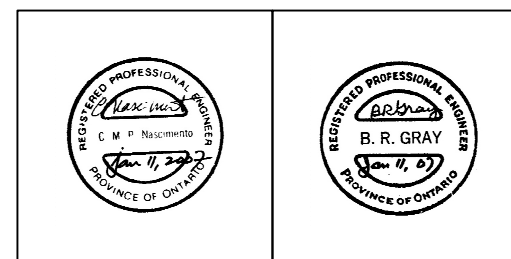
LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60 Cone, 475 J / blow)
- W L at time of investigation June 2006
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
D1	447.7	4 858 728	217 119
D2	447.6	4 858 733	217 123
D3	443.0	4 858 768	217 104
D4	443.0	4 858 764	217 098
D5	443.2	4 858 783	217 091
D6	448.3	4 858 718	217 132

NOTE

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



REF No S6258 DETOUR DATED AUGUST 24, 2006

NOTES:

- REFER TO DRAWINGS D2 FOR SECTIONS A-A, B-B and C-C.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.

REVISIONS	DATE	BY	DESCRIPTION
JUL 19/07	CN		TITLE BLOCK CHANGED (MRC email dated July 18, 2007)
MAY 1/07	NK		DWG RENUMBERED FOR MRC FINAL PACKAGE

Geocres No. 40P15-39

HWY No	6	DATE	JAN. 11, 2007	SITE	OWEN SOUND
SUBM'D FP	CN	CHECKED	CN	APPROVED	BRG
DRAWN	NA	CHECKED	CN	DWG	D1

METRIC

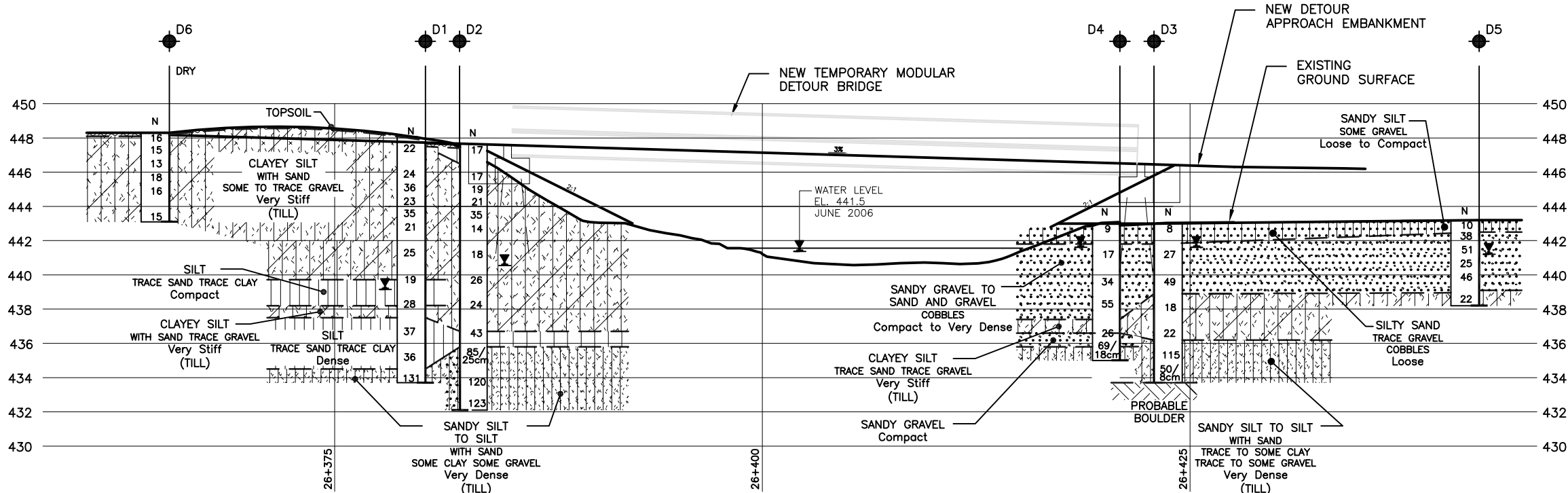
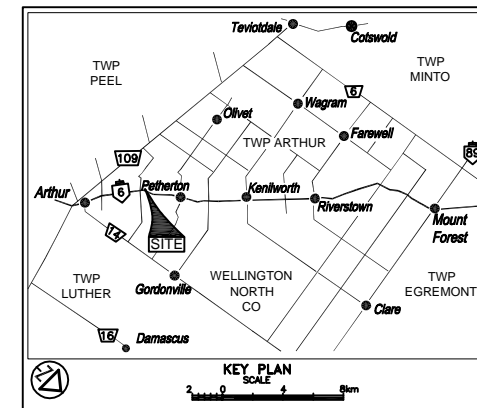
DIMENSIONS ARE IN METRES
AND/OR MILLIMETRES UNLESS
OTHERWISE SHOWN. STATIONS
IN KILOMETRES + METRES

CONT No
GWP No 342-97-00

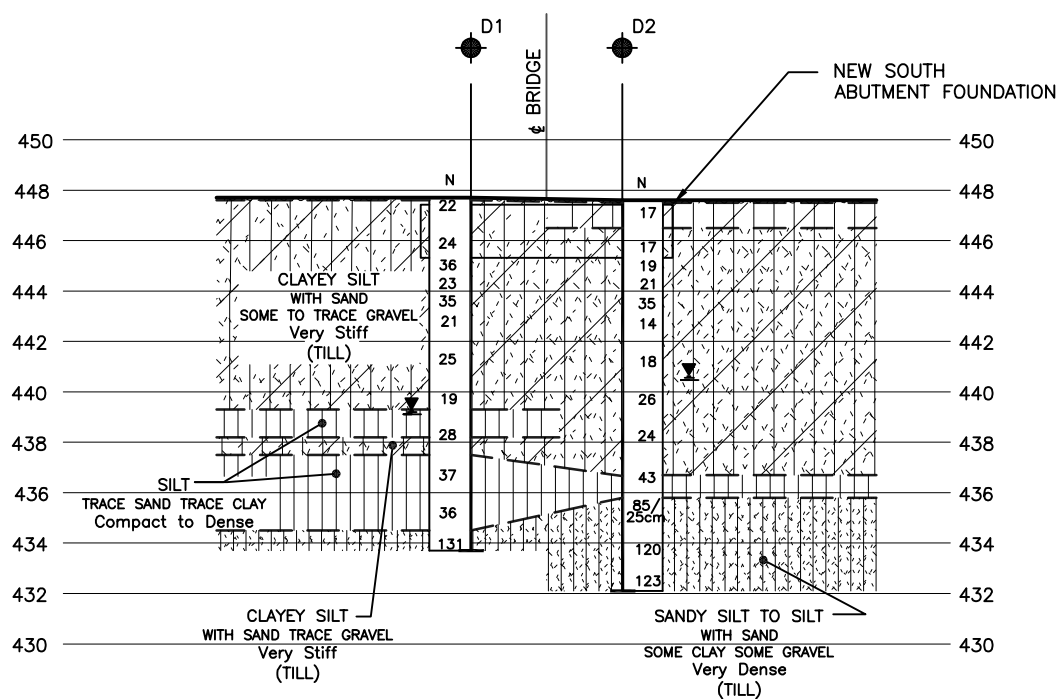
FOUR MILE CREEK DETOUR BRIDGE
Sta. 16+412
Site 35-78
SOIL SECTIONS

SHEET

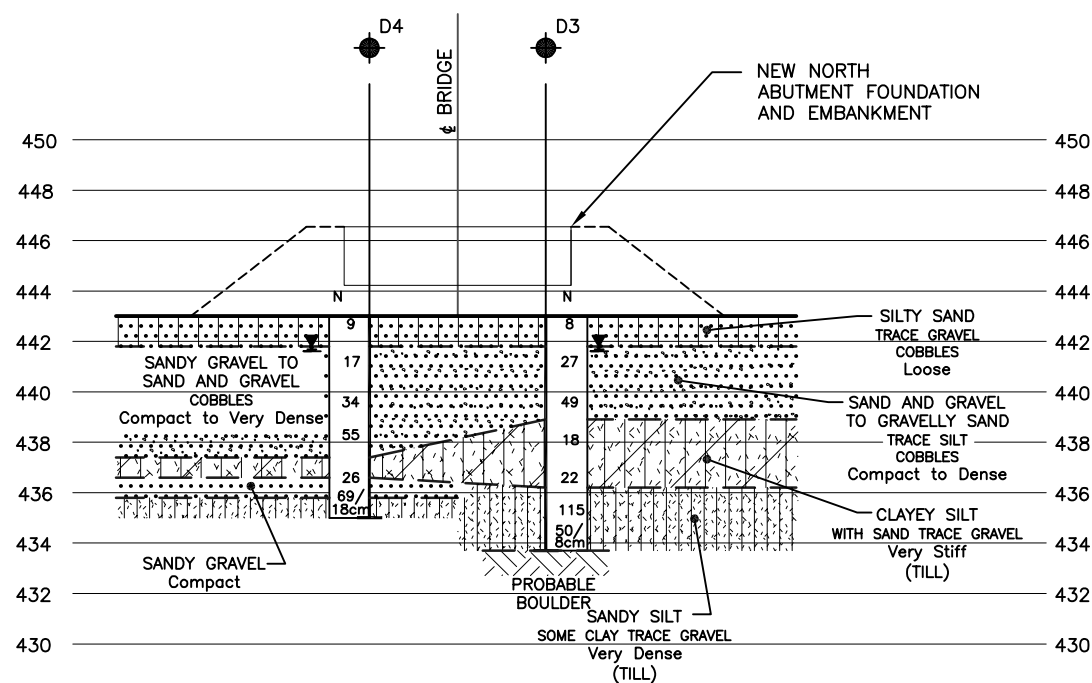
PMI Peto MacCallum Ltd.
CONSULTING ENGINEERS



A-A



B-B



C-C

LEGEND

- Borehole
- Dynamic Cone Penetration Test (Cone)
- Borehole & Cone
- N Blows/0.3m (Std. Pen Test, 475 J / blow)
- CONE Blows/0.3m (60 Cone, 475 J / blow)
- W L at time of investigation June 2006
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

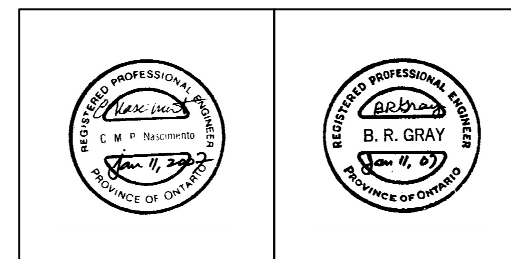
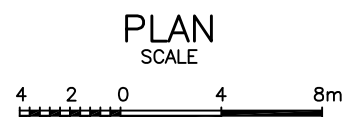
BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
FOR	DETAILS	REFER	DRAWING D1

NOTE

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

NOTES:

- REFER TO DRAWINGS D1 FOR PLAN AND LOCATION OF BOREHOLES.
- THIS DRAWING IS FOR SUBSURFACE INFORMATION ONLY. SURFACE DETAILS AND FEATURES ARE FOR CONCEPTUAL ILLUSTRATION.



REF No S6258 DETOUR DATED AUGUST 24, 2006

REVISIONS	DATE	BY	DESCRIPTION
MAY 1/07	NK		DWG RENUMBERED FOR MRC FINAL PACKAGE

Geocres No. 40P15-39

HWY No	6	DIST	OWEN SOUND
SUBM'D FP	CHECKED CN	DATE	SEPT. 14 2006
DRAWN	NA	CHECKED	CN
APPROVED	BRG	DWG	D2



FOUNDATION DESIGN REPORT

for

**FOUR MILE CREEK BRIDGE REPLACEMENT
AND TEMPORARY DETOUR BRIDGE**

SITE NO. 35-78

HIGHWAY 6 IMPROVEMENTS

FROM ARTHUR (WELLS STREET) TO SOUTH OF MOUNT FOREST

AGREEMENT NUMBER 3005-E-0036

GWP NO. 342-97-00

TOWNSHIP OF ARTHUR

WELLINGTON NORTH COUNTY, ONTARIO

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

Distribution:

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(PDF format), and Drawings (AutoCAD format)
- 2 cc: McCormick Rankin Corporation
+ one digital copy (PDF format)
- 1 cc: PML Kitchener
- 1 cc: PML Toronto

PML Ref.: 05KF104A
Index No.: 152FDR
Geocres No.: 40P15-39
January 12, 2007
Re-issued July 24, 2007
with Revised Foundation Drawings



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Table 1 - List of Standard Specifications Referenced in Report

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

Figure 1 - Abutment on Compacted Fill Showing Granular 'A' Core

FOUNDATION DESIGN REPORT
for
Four Mile Creek Bridge Replacement
And Temporary Detour Bridge
Site No. 35-78
Highway 6 Improvements
From Arthur (Wells Street) to South of Mount Forest
Agreement Number 3005-E-0036
G.W.P. 342-97-00
Township of Arthur
Wellington North County, Ontario

1. INTRODUCTION

This report provides foundation engineering comments and recommendations regarding design and construction of foundations, abutments and the approach embankments for the proposed construction of a replacement bridge at the Highway 6 crossing of Four Mile Creek and associated Temporary Detour Bridge about 0.7 km south of Arthur Road No. 9. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario.

Highway 6 crosses Four Mile Creek at about Station 16+410, in the Township of Arthur. According to the RFP and to the General Arrangement (GA) Drawing S6259-335-001 GA dated August 24, 2006, prepared by MRC the existing bridge is a single span concrete arch bridge with a span of about 14.0 m. The existing abutments are founded on spread footings with the base established at an estimated elevation 440.0, as shown on the referenced GA drawing.

It is planned to replace the existing structure with a new bridge with a longer span of about 27.8 m. The existing highway pavement grades over the bridge will be raised about 0.5 m on the north side.

The Temporary Detour Bridge will be a single-span and single-lane modular bridge with a length of about 36.5 m that will be constructed about 18.0 m to the west of the existing bridge. The temporary bridge is proposed between about Stations 26+385 and 26+421, Highway 6 Detour chainage, according to GA Drawing S6258-Detour dated August 24, 2006.

The Highway 6 asphalt and gravel shoulder pavement (off the bridge deck) and approach embankment fills were encountered at the Four Mile Creek Bridge location. Since the existing abutments are founded on spread footings, the embankment fills are likely to extend to the founding



levels of the existing footings (elevation 440.0) immediately behind the abutments. Topsoil was encountered at the ground surface south of the creek at the Temporary Detour Bridge location.

The native soil stratigraphy revealed in the boreholes was generally consistent at both existing and proposed bridge locations. On the south side of the Four Mile Creek, the native soils at shallow depths comprised discontinuous deposit of variable soils (firm silty clay, very stiff clayey silt, loose to compact sandy silt/silty sand) overlaying discontinuous compact to dense silt/sandy silt layers which overlaid or interbedded very stiff to hard clayey silt till which in turn overlaid very dense sandy silt till units. North of the Four Mile Creek, the native soil stratigraphy comprised compact to very dense gravelly sand/sand and gravel deposits overlying very stiff to hard clayey silt till and very dense sandy silt till deposits. Scattered cobbles and boulders were found in the boreholes.

The bedrock underlying the Four Mile Creek Bridge and Temporary Detour Bridge sites is mainly composed of dolostone, shale, gypsum and salt of the Salina Formation. The estimated depth to the bedrock level is about 50 m at this site.

The stabilized groundwater encountered at widely varying depths ranging from 1.2 to 8.8 m, however within a narrower range of elevations 441.8 to 443.0 at the Four Mile Creek Bridge and elevations 438.9 to 441.8 at the Detour Bridge site. The groundwater at both sites is considered to be typically perched within the fill and relatively pervious silt and sandy/gravelly soils and is controlled by the water level in the Four Mile Creek that was recorded at elevation 441.5 in June 2006.

It is considered feasible to establish the foundations of the abutments and wing walls of the replacement and temporary bridges on spread footings or deep foundations. The founding levels of the spread footings should be designed to minimize undermining problems from the existing footings to be removed. In view of the presence of discontinuous silt layers that are susceptible to erosion, construction and future performance of the footings will require the placement of a sheetpile cofferdam to protect the excavation and subgrade during and after construction.

It is understood that the planned construction staging entails the demolition of the bridge for subsequent replacement and will require the diversion of traffic to a single lane temporary detour



bridge while the existing bridge is being replaced. This construction staging will not require temporary road protection installations along (or near) the centreline of the alignment behind both bridge abutments. For shallow foundations where excavations will extend to levels about 1.5 m below the water level in the Four Mile Creek, sheetpiling and groundwater control is likely required to adequately establish the founding subgrade.

The tips of the sheetpiles should be equipped with driving shoes to prevent damage from cobbles and boulders potentially present in the native till soils.

It is noted that the consultants assume no responsibility or liability for alerting the contractor and to “red-flag” all critical issues. The requirement to deliver acceptable construction quality remains the responsibility of the contractor.

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

2. FOUNDATIONS

2.1 General

Based on the inferred site conditions it is considered feasible to establish the foundations of the abutments and wing walls of the replacement and temporary bridges on spread footings or deep foundations. The foundation recommendations for the new replacement bridge and temporary detour bridge are provided separately in the following sections of this report.

It is inferred that the Four Mile Creek channel will be shaped to about elevation 441.5.

All footings or pile caps subject to frost action should be provided with 1.6 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 0.6 m 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), CAN/CSA-S6-00, clause 4.4.6].

Based on the grain size and relative density/consistency of the soil cover at the site, it is considered that liquefaction of the soil is unlikely to occur (refer to clause 4.6.2 of the CHBDC).

2.2 Four Mile Creek Replacement Bridge

The existing abutment spread footings are founded at about elevation 440.0, according to the reference drawings. The abutments of the new replacement bridge are proposed about 2.5 m behind the existing abutments (existing face of abutment to new face). The native soils are considered suitable to support the new structure foundations on spread footings on the native soils at the existing elevation (elevation 440.0) that is limited by the local frost depth of 1.6 m. Construction of footings at a higher level on engineered fill is not considered feasible due to frost penetration and erosion control limitations.

It is also considered feasible to support the abutment foundations on piles driven to refusal on the very dense till soils encountered at 11 to 12 m depths at the south abutment and at 10 to 12 m at the north abutment. The presence of cobbles and possible boulders within the glacial soils may damage the piles and/or cause the piles to reach false refusal. Drilled caissons to support the foundations are not considered suitable for this site due to the presence of cohesionless native soils in wet condition that would cause construction difficulties.

Use of steel H-piles to support the abutment foundation loads will be dictated by structural design considerations. The feasibility of employing integral or semi-integral abutments supported on steel H-piles will also be subject to structural design considerations as defined on the MTO Report Ref. No. SO-96-01.

2.2.1 Spread Footings

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



boulders should be removed and the geometry of the excavation should be designed to prevent undermining from the excavations required to remove the existing footings. In view of the presence of discontinuous silt layers that are susceptible to erosion, construction and future performance of the footings will require the placement of a sheetpile cofferdam to protect the excavation and subgrade during and after construction.

Spread footings should be constructed on the native soils comprising typically hard clayey silt till, compact to dense silt or dense/very dense gravel/sand deposits at the highest elevation 440.0 provided that the above recommendations are followed. The recommended bearing resistance for minimum 2.0 m wide footings constructed on the native soils is as follows:

Factored Bearing Resistance at ULS	=	500 kPa
Bearing Resistance at SLS	=	300 kPa

A footing embedment depth of 1.6 m and groundwater level about 1.5 m above the founding depth (creek level at approximate elevation 441.5) was assumed for computation of the ULS resistance. The resistance at SLS normally allows for 25 mm of total compression of the founding medium. Differential settlement is expected to be less than 75% of the total settlement value.

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

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the native soils. Refer to Section 2.4 for further recommendations in this regard.

2.2.2 Deep Foundations

2.2.2.1 *General*

Conventional or integral/semi-integral abutment designs are considered feasible at the Four Mile Creek bridge site.



The general pile foundation design recommendations are provided on the following section followed by additional geotechnical recommendations for integral abutment foundations.

2.2.2.2 Conventional Abutment Considerations

Piles for the north and south abutments should be driven to refusal into the very dense sandy silt till encountered at the estimated range of reference levels that are provided on the following table:

LOCATION	STRATUM DEPTH (m)	STRATUM ELEVATION	RELEVANT BOREHOLES
North Abutment	10.2 to 11.6	434.9 to 436.5	M4 and M5
South Abutment	11.8	435.5	M2 and M3

The reference depths and elevations are taken from the existing ground surface at the borehole locations to the top of the founding stratum. About 1.5 to 2.0 m for pile embedment at refusal should be allowed.

The presence of cobbles/boulders was identified above the founding soils at depth in some of the boreholes. Since these deposits appear to be typically very stiff to hard or compact to dense, the risk of damage during driving is considered to be low and, as a consequence, application of a reduction factor is not employed. Nevertheless, an NSSP should be prepared to advise the contractor of the presence of boulders at this site. The pile tips should be protected with driving shoes such as the Titus "H" Bearing Pile Point, Standard Model to minimize potential damage due to the presence of cobbles and/or boulders in the native till soils. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in SP 903S01.

The NSSP should include specific direction for the contractor to provide experienced full time foundation engineering supervision to monitor the driving operations over the complete length of the pile. This should involve assessment of the performance of the hammer, recording of the number of blows required to advance the pile during each 300 mm of penetration over the total length of the pile, interpretation of the penetration data as the pile is driven for evidence of unusual conditions that



could be indicative of damage, ensuring the piles have been driven to refusal and the need to drive replacement piles if evidence of damage is detected.

On this basis the above recommendations the factored axial resistance at ultimate limit states (ULS) for the two pile sections noted is considered to be appropriate:

PILE SECTION	FACTORED AXIAL RESISTANCE AT ULS (kN)
HP 310 x 79	1300
HP 310 x 110	1800

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the pile length required (about 8 m below the abutment), the design is not expected to be governed by settlement since the required loads causing appreciable deformation of the pile and/or native glacial till soils are much larger than the ULS factored capacity.

The compacted granular fill pad used for the installation of the abutment piles should comprise OPSS Granular A or Granular B Type II materials to allow installation of the piles without damage. The modest increase of 0.5 m to the road grade level over the Creek and the consistency of the native soils indicate that negative skin friction on the piles will be negligible.

The piles will be driven through 2 to 3 m of compacted granular fill and the underlying native soils that typically comprise very stiff to hard or compact to dense glacial tills with cobbles and/or boulders. It is considered, based on our experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed.

The pile sets should be prepared based on the section of steel pile to be selected, the energy of the hammer and setup that will be employed. Overdriving the piles on the sandy silt till should be avoided by allowing for dissipation of the high pore water pressures that will develop within the



founding stratum during initial driving to the design founding depths. Retapping of the all piles should be included in the contract, as indicated in SP 903S01.

2.2.2.3 Integral Abutments on Piles

The design of integral abutments should allow for the required 5.0 m free pile length between the founding soil and the base of the abutment stem.

To accommodate movement of the integral abutment system, two concentric CSPs that extend at least 3 m below the bottom of the abutment should be placed around the pile to create an annular space. The inner CSP should be filled with sand meeting the gradation requirements of Granular B Type 1. Alternatively, a single CSP or auger hole filled with loose uniform sand meeting the requirements shown in the attached Table 2 may be used. Refer to MTO Report SO-96-01 for further details.

The installation of the piles, including geotechnical resistances and recommendations on the driving of the piles should follow the recommendations provided in the previous section of this report.

2.3 Temporary Detour Bridge

The new abutment foundations are proposed at about elevations 444.2 and 445.5 on the north and south sides of the Four Mile Creek, according to the preliminary GA drawing prepared by MRC.

The native soil at the south abutment comprises very stiff clayey silt till (boreholes D1 and D2) that is considered suitable to support the new structure foundations on spread footings at the proposed elevation 445.5.

The base of the north abutment foundation is proposed at about elevation 444.2 that is about 1.2 m higher than the existing ground surface level at the boreholes D3 and D4 locations. It is considered that the north abutment may be constructed on spread footings placed on an engineered fill pad constructed over the existing native soils at this location.



The footing should be protected against frost action by providing adequate earth cover or the equivalent insulation protection. The local frost depth is 1.6 m.

It is also considered feasible to support the abutment foundations on piles driven to refusal on the very dense till soils encountered at 7 m depth at the north abutment and at 12 to 13 m depths at the south abutment. The presence of cobbles and possible boulders within the glacial soils may damage the piles and/or cause the piles to reach false refusal. Drilled caissons to support the foundations are not considered suitable for this site due to the presence of cohesionless native soils in wet condition that would cause construction difficulties.

Use of steel H-piles to support the abutment foundation loads will be dictated by structural design considerations. It is considered that employing integral or semi-integral abutments is not applicable to modular bridge foundations.

2.3.1 Spread Footings on Native Soil

Construction of the spread footings on native soil for the south abutment should be performed and monitored in accordance with OPSS 902 and SP 902S01 to verify the competency of the founding surface. All loose soil and/or boulders should be removed from the footing subgrade proposed at about elevation 445.5.

Spread footings should be constructed on the native soils comprising typically very stiff clayey silt till, (boreholes D1 and D2) at the proposed elevation 445.5 provided that the above recommendations are followed. The recommended bearing resistance for minimum 2.0 m wide footings constructed on the native soils is as follows:

Factored Bearing Resistance at ULS	= 400 kPa
Bearing Resistance at SLS	= 250 kPa

A footing embedment depth of 1.6 m and groundwater level below the founding depth was assumed for computation of the ULS resistance. The resistance at SLS normally allows for 25 mm of total compression of the founding medium. Differential settlement is expected to be less than 75% of the total settlement value.



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

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the native soils. Refer to Section 3.2 for further recommendations in this regard.

2.3.2 Spread Footings on Engineered Fill

The north abutment foundation is considered feasible on a structural fill pad placed on the native soil encountered at the north abutment location. Particular care is required to prepare the structural fill founding subgrade. All deleterious soil and boulders under the engineered fill pad should be removed. The loose sand identified at the surface of boreholes D3 and D4 should be proof rolled and compacted from the surface removing any unsuitable soils as directed by the geotechnical engineer. The structural fill pad should be at least 1.5 m thick and founded on the compacted silty sand. In view of the proximity of the creek bed the contract should allow for the removal and replacement of a surficial layer of the silty sand at least 0.5 m thick.

The structural fill should comprise OPSS Granular A material placed in maximum 200 mm thick lifts, compacted to 100% maximum dry density determined by the MTO test method LS-706 (standard Proctor) 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 limits of the required fill pad should be clearly marked and surveyed in the field.

The recommended bearing resistance for minimum 2.0 m wide footings constructed on structural fill at least 1.5 m thick is as follows:

$$\begin{aligned}\text{Factored Bearing Resistance at ULS} &= 900 \text{ kPa} \\ \text{Bearing Resistance at SLS} &= 350 \text{ kPa}\end{aligned}$$

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



of 1.6 m and groundwater level below the founding depth was assumed for computation of the ULS resistance.

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

The horizontal force imposed on the foundations will be resisted in part by the friction force developed between the underside of the footing and the structural fill. Further recommendations in this regards are provided in Section 3.2 of this report. An unfactored friction factor of 0.7 is recommended for footings on the structural fill.

2.3.3 Deep Foundations

Conventional abutment designs are considered feasible at the temporary bridge detour site. Integral and semi-integral abutment designs are not considered suitable for the proposed modular bridge.

Piles for the north and south abutments should be driven to refusal into the very dense sandy silt till encountered at the estimated range of reference levels that are provided on the following table:

LOCATION	STRATUM DEPTH (m)	STRATUM ELEVATION	RELEVANT BOREHOLES
North Abutment	6.8 to 7.2	435.8 to 436.2	D3 and D4
South Abutment	11.8 to 13.2	434.5 to 435.8	D1 and D2

The reference depths and elevations are taken from the existing ground surface at the borehole locations to the top of the founding stratum. About 1.5 to 2.0 m for pile embedment at refusal should be allowed.

The presence of cobbles/boulders was identified above the founding soils at depth in some of the boreholes. Since these deposits appear to be typically very stiff or compact to dense, the risk of



damage during driving is considered to be low and, as a consequence, application of a reduction factor is not employed. Nevertheless, an NSSP should be prepared to advise the contractor of the presence of boulders at this site. The NSSP is required to ensure that more comprehensive engineering supervision is required than is called for in SP 903S01.

The NSSP should include specific direction for the contractor to provide experienced full time foundation engineering supervision to monitor the driving operations over the complete length of the pile. This should involve assessment of the performance of the hammer, recording of the number of blows required to advance the pile during each 300 mm of penetration over the total length of the pile, interpretation of the penetration data as the pile is driven for evidence of unusual conditions that could be indicative of damage, ensuring the piles have been driven to refusal and the need to drive replacement piles if evidence of damage is detected.

On this basis the above recommendations the factored axial resistance at ultimate limit states (ULS) for the two pile sections noted is considered to be appropriate:

PILE SECTION	FACTORED AXIAL RESISTANCE AT ULS (kN)
HP 310 x 79	1300
HP 310 x 110	1800

The resistance at SLS normally allows for 25 mm of compression of the pile and founding medium. Considering the pile length required (about 8 m below the abutment), the design is not expected to be governed by settlement since the required loads causing appreciable deformation of the pile and/or native glacial till soils are much larger than the ULS factored capacity.

The compacted granular fill pad used for the installation of the abutment piles should comprise OPSS Granular A or Granular B Type II materials to allow installation of the piles without damage. The modest increase of 0.5 m to the road grade level over the Creek and the consistency of the native soils indicate that negative skin friction on the piles will be negligible.



The piles will be driven through 2 to 3 m of compacted granular fill and the underlying native soils that typically comprise very stiff to hard or compact to dense glacial tills with cobbles and/or boulders. It is considered, based on our experience with pile driving under similar conditions, that a hammer transferring at least 40 kJ of energy to the pile should be employed to drive the piles. The rated energy of the hammer should therefore be 50 to 55 kJ depending on the type of equipment employed.

The pile sets should be prepared based on the section of steel pile to be selected, the energy of the hammer and setup that will be employed. Overdriving the piles on the sandy silt till should be avoided by allowing for dissipation of the high pore water pressures that will develop within the founding stratum during initial driving to the design founding depths. Retapping of the all piles should be included in the contract, as indicated in SP 903S01.

2.4 Lateral Resistance

The soil adjacent to the upper section of the piles is expected to comprise the typically cohesive very stiff to hard clayey silt till or compact to very dense gravels and sands.

Resistance to lateral loads may be provided in part by mobilization of passive resistance along the pile. The pile length providing resistance for integral abutment piles should be considered the dimension below the annular space that will be mostly embedded into the very stiff clayey silt till (south abutment) or compact to dense gravel/sand (north abutment). The recommended lateral resistance for the pile sections noted previously is as follows:

PILE SECTION	FACTORED RESISTANCE AT ULS (kN)	RESISTANCE AT SLS (kN)
HP 310	200	110

The lateral resistance values assume that the piles are driven through the native undisturbed soils or through compacted granular materials placed as recommended in the section titled "Approach Embankments". If greater resistance is required, batter piles should be installed.



To evaluate the point of contraflexure, the coefficient of horizontal subgrade reaction, k_s (MN/m^3) for granular backfill and cohesionless materials and the cohesive clayey silt till should be computed using the following equations:

Granular/Cohesionless Soils: Embankment fill, sand/gravel at north abutment and silt at south abutment.

$$k_s = n_h z/b$$

where n_h = coefficient related to soil density

$$= 14.0 \text{ MN}/\text{m}^3 \text{ for granular backfill}$$

$$= 2.0 \text{ MN}/\text{m}^3 \text{ for native cohesionless soils above groundwater table}$$

$$= 1.3 \text{ MN}/\text{m}^3 \text{ for native cohesionless soils below groundwater table}$$

z = depth, m

b = pile width, m

Cohesive Soils: Clayey silt till at south abutment.

$$k_s = \frac{67C_u}{b}$$

C_u = undrained shear strength of the clayey silt till

$$= 150 \text{ kPa}$$

b = pile width, m

For design purposes, the groundwater should be considered at elevation 441.5, at the current water level of the Four Mile Creek.

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



Pile Spacing in Direction of Loading d = Pile Diameter or Width	Subgrade Reaction Reduction Factor, R
8d	1.00
6d	0.70
4d	0.40
3d	0.25

3. ABUTMENT WALLS

3.1 Earth Pressures

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.

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

where K = coefficient of lateral earth pressure (dimensionless)

γ = unit weight of free-draining granular material, kN/m³

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 Granular B Type II)

δ = angle of friction between the soil and wall (23.5° for Granular A or Granular B Type II)

The seismic site coefficient for the conditions at this site was provided in Section 2.1.



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

PARAMETERS	GRANULAR A OR GRANULAR B TYPE II
Internal Friction Angle, ϕ (degrees)	35
Unit weight, γ (kN/m ³)	22.8
Coefficient of Active Earth Pressure, K_a	0.27
Coefficient of Earth Pressure At Rest, K_o	0.43
Coefficient of Passive Earth Pressure, K_p	3.69

The assigned geotechnical parameter values are the same for both granular materials in view of their similar physical characteristics.

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 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).

The magnitude of the passive resistance is dependent on the actual lateral movement of the structure toward the retained soil. We refer to Figure C6.9.1(a) of the CHBDC for this computation. The subsoil/backfill should be considered as medium dense sand for the project.

A weeping tile system (SP 405F03) and/or weep holes should be installed to minimize the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or geotextile to prevent migration of fines into the system. The drainage pipe should be installed on a positive grade and lead to a frost-free outlet.



Backfilling adjacent to retaining structures should be carried out in conformance with Ontario Provincial Standard Drawings for granular backfill at abutments (OPSD 3101.150 and 3121.150).

Operation of compaction equipment adjacent to retaining structures should be restricted to limit the compaction pressure noted in clause 6.9.3 of the CHBDC. Refer to SP 105S10 for additional information in this regard.

3.2 Sliding Resistance

Where wing walls are utilised, the previous recommendations and geotechnical parameters for abutment foundations and backfill should be utilized for the design of the foundations. The wall founding levels should match those of the respective abutments.

The design of the walls should be checked for sliding resistance using the following geotechnical parameters for cast-in-place concrete foundations.

PARAMETER	GRANULAR A OR GRANULAR B TYPE II	SILT	CLAYEY SILT TILL	GRAVEL/SAND
Friction Angle, degrees	35	28	0	32
Cohesion, kPa	0	0	150	0
Unit Weight, kN/m ³	22.8	21.0	20.0	21.0

A resistance factor of 0.8 is to be applied in calculating the factored horizontal resistance in accordance with the CHBDC Section 6.7.5.

A weeping tile system and/or weep holes should be installed to minimise the build-up of hydrostatic pressure behind the wall. The weeping tiles should be surrounded by a properly designed granular filter or non-woven Class II geotextile (with an FOS of 75-150 µm according to OPSS 1860) placed to prevent migration of fines into the system. The drainage pipe should be placed on a positive grade and lead to a frost-free outlet.



3.3 RSS Wall Considerations

it is considered that a retained soil system (RSS) is not suitable at these sites since the exposed height of the retaining wall will be less than 3 m high.

4. APPROACH EMBANKMENTS

4.1 General

It is anticipated that the approach embankments will be slightly widened and/or reconstructed with earth borrow or granular materials. The north and south approach fill embankments will be raised about 0.5 m to a height of about 3.0 m near the structures and taper down to match the existing road grades away from the structure. Construction of the fill on the native very stiff to hard clayey silt till or compact to dense sandy silt is considered feasible.

4.2 Embankment Design and Construction Considerations

The embankments should be designed and constructed in accordance with OPSD 200.010, 202.010, 208.010 and SP 206S03. The side slopes of the approach embankments should be inclined no steeper than 2H:1V for earth fill.

Where slope flattening is proposed, a drainage gap should be provided in accordance with OPSD 202.020. OPSS Granular B Type II should be used for the drainage gaps.

The earth fill slopes, if employed, should be protected against surface erosion by sodding and suitable vegetation. Refer to OPSS 571 or 572 for time constraints and the type of seed and mulch required. The slopes should also be protected against erosion from the effects of high water conditions in the Four Mile Creek (OPSS 511).



4.3 Embankment Settlements

It is considered that the approach embankments widened and reconstructed in accordance with the foregoing recommendations will be stable. Settlement under the new embankment fill due to consolidation of the underlying native very stiff to hard/compact to dense soils is considered to be negligible.

If the embankments are constructed with granular materials, some settlement of the road surface adjacent to the abutments should also be expected due to "consolidation" of the backfill. The granular backfill placed adjacent to the abutments will be about 5 m high. The magnitude of the "consolidation" of these fills 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 less than 10 mm. These estimated total settlements of the approach fill surface near the abutments should be essentially complete within 1 to 2 months after placement of the fill.

5. EXCAVATION AND GROUNDWATER CONTROL

5.1 General Considerations

Excavation for construction of the abutment foundations on spread footings or pile caps will extend through embankment fills and the native very stiff to hard/compact to dense soils to about 5.5 m depth below existing grades. Cobbles and boulders should be expected at the site.

The very stiff to hard clayey silt till is classified as Type 2 soil according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. The fills and compact to dense sandy silt are classified as Type 3 soils above the water table. The excavations should be carried in accordance with the soils in the slopes having the highest number. Consequently the temporary cut slopes over the full depth of the excavation should be inclined at 45° to the horizontal to meet Type 3 soil criteria. The need to excavate flatter side slopes if excessively soft/wet materials or concentrated seepage zones are encountered locally should be considered.



The cohesionless soils below the groundwater as encountered during construction should be reclassified as classified as Type 4 soil if groundwater is not adequately controlled. For this condition, side slopes should be cut at 3H:1V.

In view of the presence of discontinuous silt layers that are susceptible to erosion, construction and future performance of the footings will require the placement of a sheetpile cofferdam to protect the excavation and subgrade during and after construction.

Care should be employed to excavate the existing footings located in front of the new foundations and prevent affecting the founding subgrade of the new footings.

5.2 Road Protection Considerations

Since the construction and traffic staging consists of diverting traffic over a temporary bridge while the existing bridge is being replaced, road protection requirements are not anticipated. However, should construction and traffic staging requires traffic adjacent to the excavations, a suitable roadway protection scheme following OPSS 538 and SP 105S19 should be provided to support the walls of the excavation and adjacent traffic lanes.

Several protection scheme alternatives such as sheet piling, sheeting supported by rakers or bracing, cantilever or anchored soldier piles and lagging may be considered. It is noted however that soldier pile and lagging schemes are not considered adequate where the excavation will be carried out through sand with gravel fills or native sandy silt materials in particular below the water table. The schemes should be designed for performance level 1a system to prevent movement of the existing embankment. The contractor is responsible for the selection, preparation and performance of a detailed design for the road protection scheme.

5.3 Groundwater Control Considerations

The water level observed in the Four Mile Creek in June 2006 (elevation 441.5) was up to 1.5 m above the anticipated deepest level of excavation (elevation 440.0). Cognisant of the relatively high permeability characteristics of the sand, gravel and silt layers and the relatively high hydraulic head,



it is anticipated that vigorous pumping from sumps installed within sheetpile cofferdams will be required to control seepage of water into the excavations and fill areas.

The perched groundwater that was observed within the fill in the abutment boreholes (M1, M6, D5 and D6) during or upon completion of drilling should be considered when excavating for the construction of the new embankments. Groundwater levels are subject to seasonal fluctuations and rainfall patterns. Seepage should be anticipated locally at the fill/native soil interface. All surface water runoff should be diverted away from the excavations to maintain the founding subgrade in the dry. It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the general embankment excavations and fill areas.

The contract documents should clearly state that dewatering of the excavations is the contractor's responsibility. Where groundwater control is required it should be designed to prevent affecting existing water wells.

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

6. DISCUSSION OF FOUNDATION ALTERNATIVES

6.1 Advantages and Disadvantages of Foundation Alternatives

The following table summarizes the advantages and disadvantages and inferred risks/consequences of each of the foundation alternatives for the replacement of the Four Mile Creek replacement and temporary bridges.



ADVANTAGES AND DISADVANTAGES – FOUR MILE CREEK REPLACEMENT BRIDGE

SPREAD FOOTINGS ON NATIVE SOIL		DRIVEN PILES	
ADVANTAGES	DISADVANTAGES	ADVANTAGES	DISADVANTAGES
Less costly than deep foundation alternative Conventional design and construction of foundations	Requires sheetpiling cofferdam for construction Requires ground water control to establish founding subgrade in the dry. Requires permanent sheetpiling for post-construction erosion control	Foundation is established below levels of potential scour from creek flow.	More costly than shallow foundation alternative. Requires erosion protection against scour if pile cap is placed at an elevation higher than creek bed level.

- Notes: 1. Spread footings on engineered fill are considered not applicable at this site.
2. Driven piles include conventional and integral/semi-integral abutment designs.
3. Caisson foundations were not considered practical at this site.

ADVANTAGES AND DISADVANTAGES – TEMPORARY DETOUR BRIDGE

SPREAD FOOTINGS ON NATIVE SOIL OR ENGINEERED FILL		DRIVEN PILES	
ADVANTAGES	DISADVANTAGES	ADVANTAGES	DISADVANTAGES
Less costly than deep foundation alternative Conventional design and construction of foundations	Requires ground water control to establish founding subgrade in the dry. Requires sheetpiling for post-construction erosion control	Foundation is established below levels of potential scour from creek flow.	More costly than shallow foundation alternative. Requires erosion protection against scour if pile cap is placed at an elevation higher than creek bed level.

- Notes: 1. Spread footings on engineered fill are applicable to the north abutment at this site.
2. Driven piles include conventional abutment design.
3. Caisson foundations were not considered practical at this site.



6.2 Preferred Foundation Option Considerations

From the foundation perspective both spread footings and driven pile foundations are considered feasible. The spread footing foundation are considered to be the least costly alternative and therefore the preferred option.

It is noted that the selected foundation alternative also depends on other considerations which are being evaluated separately by MRC.

7. CLOSURE


The report was prepared by Mr. C. M. P. Nascimento, P.Eng., Senior Project Engineer and reviewed by Mr. B. R. Gray, MEng, P.Eng., MTO Designated Contact.

Yours very truly,

Peto MacCallum Ltd.


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Senior Project Engineer




Brian R. Gray, MEng, P.Eng.
MTO Designated Contact and Project Manager



CN/BRG:cn-lr



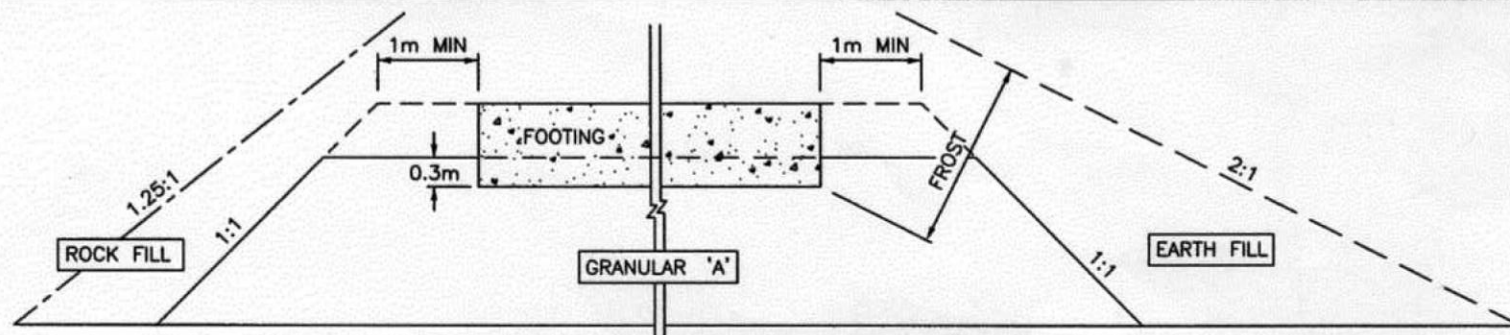
TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

DOCUMENT	TITLE	DATE
OPSS 501	Construction Specification for Compacting	November 2005
OPSS 511	Construction Specification for Rip-Rap, Rock Protection and Granular Sheeting	November 2004
OPSS 538	Construction Specification for Support Systems	November 2005
OPSS 571	Construction Specification for Sodding	November 2001
OPSS 572	Construction Specification for Seed and Cover	November 2003
OPSS 902	Excavation and Backfilling of Structures	November 2002
OPSS 1860	Material Specification for Geotextiles	November 2004
OPSD 200.010	Earth/Shale Grading – Undivided Rural	November 2005
OPSD 202.010	Slope Flattening Using Excess Material on Earth or Park Embankment	November 2005
OPSD 202.020	Drainage Gap for Slope Flattening on Rock or Granular Embankment	November 2005
OPSD 208.010	Benching of Earth Slopes	November 2003
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments	November 2005
OPSD 3121.150	Minimum Granular Backfill Requirements - Retaining Walls	November 2005
SP 105S10	Construction Specification for Compaction	November 2004
SP 105S19	Construction Specification for Protection Systems	March 2005
SP 206S03	Construction Specification for Grading	January 2004
SP 405F03	Construction Specification for Pipe Subdrains	May 2004
SP 902S01	Excavation and Backfilling of Structures	September 2003
SP 903S01	Construction Specification for Piling	September 2004



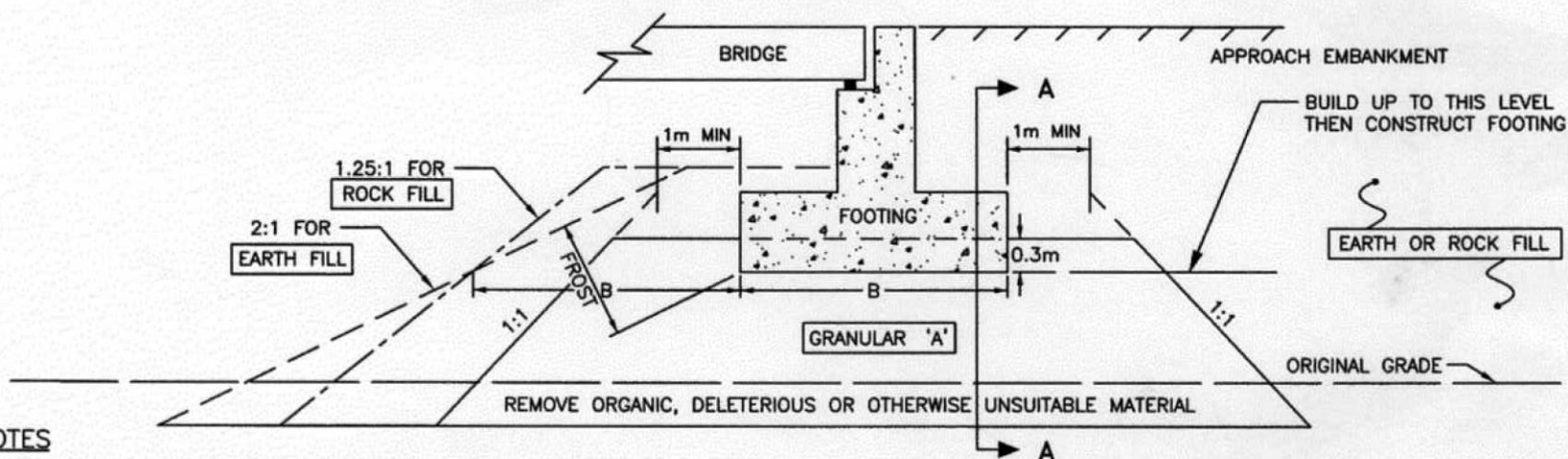
TABLE 2
GRADATION SPECIFICATION FOR SAND FILL IN
PRE-AUGERED HOLES AT INTEGRAL ABUTMENTS

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



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