



**FOUNDATION INVESTIGATION AND DESIGN REPORT
for
FARLEY'S CREEK BRIDGE REPLACEMENT
SITE NO. 35-135
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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PML Ref.: 05KF104B
Index No. 145FIR and 146FDR
Geocres No.: 40P15-37
December 29, 2006



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for
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FOUNDATION INVESTIGATION REPORT

for
Farley's Creek Bridge Replacement
Site No. 35-135
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 Farley's 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, Wellington North County, Ontario. This report was prepared for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario (MTO).

The Farley's Creek Bridge is the MTO designated Site No. 35-135 and is located at about Station 12+665.

This report summarizes the results of the foundation investigation carried out at the Farley's Creek Bridge abutments and approach embankments within 20 m of the abutments.

2. SITE DESCRIPTION

The existing single span bridge is an concrete rigid frame structure with an approximately 9.0 m span carrying the two-lane Highway 6 over the Farley's Creek.

The site is located outside the Town of Arthur about 0.1 km north of Wells Street. The local land use is mainly agricultural with residences in relative proximity of the bridge site. There is a reforested area on the south bank of Farley's Creek to the west of the bridge site otherwise the ground cover comprises agricultural crops and grasses. The banks of the Farley's Creek are also covered with grasses.

The general topography of the area is a rolling terrain typical of glacial plains intersected by creeks and drainage channels. The road grade dips about 8 m from the south towards to creek



and rises to the north about 5 m. There is a 3.0 to 4.0 m earth cut on both east and west sides of the highway about 100 m north of the bridge.

The site is located within the physiographic region known as the Dundalk Till Plain characterised by a gently undulating 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 underlying the Farley's Creek Bridge belongs to the Salina Formation of the Palaeozoic Silurian age. The bedrock is mainly composed of dolostone, shale, gypsum and salt. The estimated depth to the bedrock level is about 55 m at this site.

The frost penetration depth for the area of the Farley's Creek Bridge is 1.6 m.

3. INVESTIGATION PROCEDURES

The field work was carried out during the period from June 1 to June 13, 2006 and comprised a total of six sampled boreholes, denoted F1 to F6 which were advanced to depths of 8.2 to 18.6 m.

The borehole layout was established in accordance with the requirements noted in the Request for Proposal. 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 drillrig, 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 consisted of 47 natural moisture content determinations, grain size distribution analyses of 17 selected soil samples and determination of Atterberg plasticity limits on 13 samples. The laboratory grain size determinations are reported on Figures GS-1 to GS-4 and the plasticity limits tests on plasticity charts PC-1 and PC-2. All of the test results are summarized on the Record of Borehole sheets.

4. SUMMARIZED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred soil stratigraphy, natural moisture content determinations, results of grain size analyses, plasticity limits and groundwater observations. The borehole locations are shown on Drawing F-1. Stratigraphic profiles prepared from the borehole data are presented on Drawing F-2.

The stratigraphy revealed in the boreholes generally comprised the Highway 6 asphalt pavement and approach embankment fills overlying discontinuous deposits of sandy silt which overlays or is interbedded with clayey silt till and a stratum of sandy silt till at depth. Scattered cobbles were found in some of the boreholes. The strata encountered are described below.

4.1 Pavement

The pavement fill encountered on Highway 6 comprised 130 to 140 mm of asphalt overlying granular base materials consisting of a sand and gravel mixture. The total thickness of the pavement ranged from 800 to 1500 mm and was typically about 1000 mm thick. The granular material was in a compact condition with N-values ranging from 11 to 19 blows for 300 mm penetration of the sampler. Water content determinations were 4 and 6%.

4.2 Fill

Typically firm to very stiff mixtures of clayey silt and silty clay with variable amounts of sand, gravel and cobbles, organics and topsoil were encountered below the pavement granular materials in all boreholes except borehole F1 where the road base granular materials extend to the native soil at 1.5 m depth, elevation 442.8. The fill in boreholes F2 to F6 extended to depths



ranging from 2.2 to 3.0 m, elevations 440.9 to 441.8. It is anticipated that the fill will extend to deeper levels (inferred elevation 439.0) behind the existing abutment walls and footings. N-values in the embankment fill ranged from 3 to 21 blows, with one higher value over 50 blows due to a cobble in the sample.

The particle size distribution charts of four samples of the embankment fill are shown on Figure GS-1 and the Plasticity chart on Figure PC-1. Water content determinations ranged from 12 to 23%. The Atterberg plasticity liquid limits on the fill samples ranged from 23 to 37 and the plastic limits from 17 to 20, indicating plasticity indexes from 6 to 17.

4.3 Silt/Sandy Silt

A discontinuous layer of cohesionless compact to dense silt and sandy silt with varying amounts of clay was encountered immediately below the embankment fill in borehole F3 and extended to 3.3 m depth, elevation 440.9. 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 4.1 and 5.7 m depths (elevations 440.2 to 438.6) in borehole F1; between 3.2 m and 4.0 m depths (elevations 441.7 and 440.9) in borehole F2; and between 4.1 and 5.6 m depths (elevations 438.8 and 439.9) in borehole F4. The thickness of these layers varied from 0.8 to 1.6 m. N-values in the silt/sandy silt deposits ranged from 27 to 45 blows.

The grain size distribution charts of two silt samples are shown on the attached Figure GS-2. Natural moisture content determinations in this cohesionless material ranged from 17 to 19%.

4.4 Clayey Silt Till

A deposit of glacial till comprising very stiff to hard clayey silt till was encountered below the silt unit in borehole F3 and below the embankment /road fill in the other 5 boreholes drilled at the site. The clayey silt till extended to the 8.2 m termination depth of boreholes F1 and F6 (elevations 436.1 and 435.7), drilled 20 m away from the bridge abutments and to the 11.3 m refusal depth of borehole F4 (elevation 432.7). The unit extended to depths of 11.7, 15.0 and 15.4 m (elevations 432.3, 429.2 and 428.7) in borehole F5, F3 and F2, respectively. Locally a layer of silt/ sandy silt interbedded the glacial till as described on the previous paragraph. N-values in the unit ranged from 16 to 107 blows.



The grain size distribution charts of eight samples of the clayey silt till are shown on Figure GS-3, attached. The results of the Atterberg tests are shown on Figure PC-2. The liquid limit of the material ranged from 22 to 33, the plastic limit from 12 to 19 and the computed plasticity indexes from 8 to 18. Natural moisture content determinations ranged from 8 to 21%.

4.5 Sandy Silt Till

A cohesionless very dense glacial till consisting of sandy silt to silt trace to some clay and variable gravel content was encountered below the clayey silt till. The till occurred at depths ranging from 11.7 m (elevation 432.3) in borehole F5 and depths of 15.0 and 15.4 m (elevations 429.2 and 428.7, respectively) in boreholes F3 and F2. The deposit extended to the 15.7 to 18.6 m termination depths of boreholes F2, F3 and F5. N-values in the deposit ranged from 61 to 147 blows with typical values in excess of 100 blows.

The grain size distribution charts of three samples of the material are shown on Figure GS-4, attached. Natural moisture content determinations of 8 to 19%.

4.6 Groundwater

The boreholes encountered groundwater during and upon completion of drilling except boreholes F2 and F3 drilled behind the south abutment. During the drilling, groundwater strikes were noted at depths ranging from 1.8 to 2.7 m, elevations 441.2 to 442.5. 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	F1	5.3	439.3
South Abutment	F2, F3	No free water	-
North Abutment	F4	3.1	440.9
North Abutment	F5	2.3	441.7
North Approach	F6	3.0	440.9

It is inferred that the groundwater encountered is typically perched at the fill/native soil interface and is influenced by the water level in the Farley's Creek. The groundwater level in the Farley's Creek was recorded at elevation 441.2 in June 2006.

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

5. CLOSURE

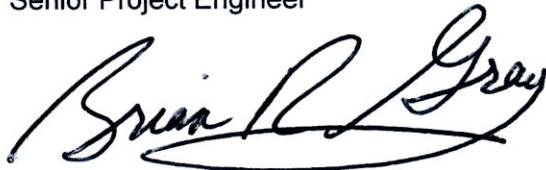
The subsurface investigation was carried out under the supervision of Mr. F. Portela and direction of Mr. C. M. P. Nascimento, P. Eng., Senior Project Engineer. Geo-Environmental Drilling Inc. supplied the drilling equipment. This report was prepared by Mr. C. M.P. Nascimento, P. Eng. and reviewed by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

Sincerely

Peto MacCallum Ltd.



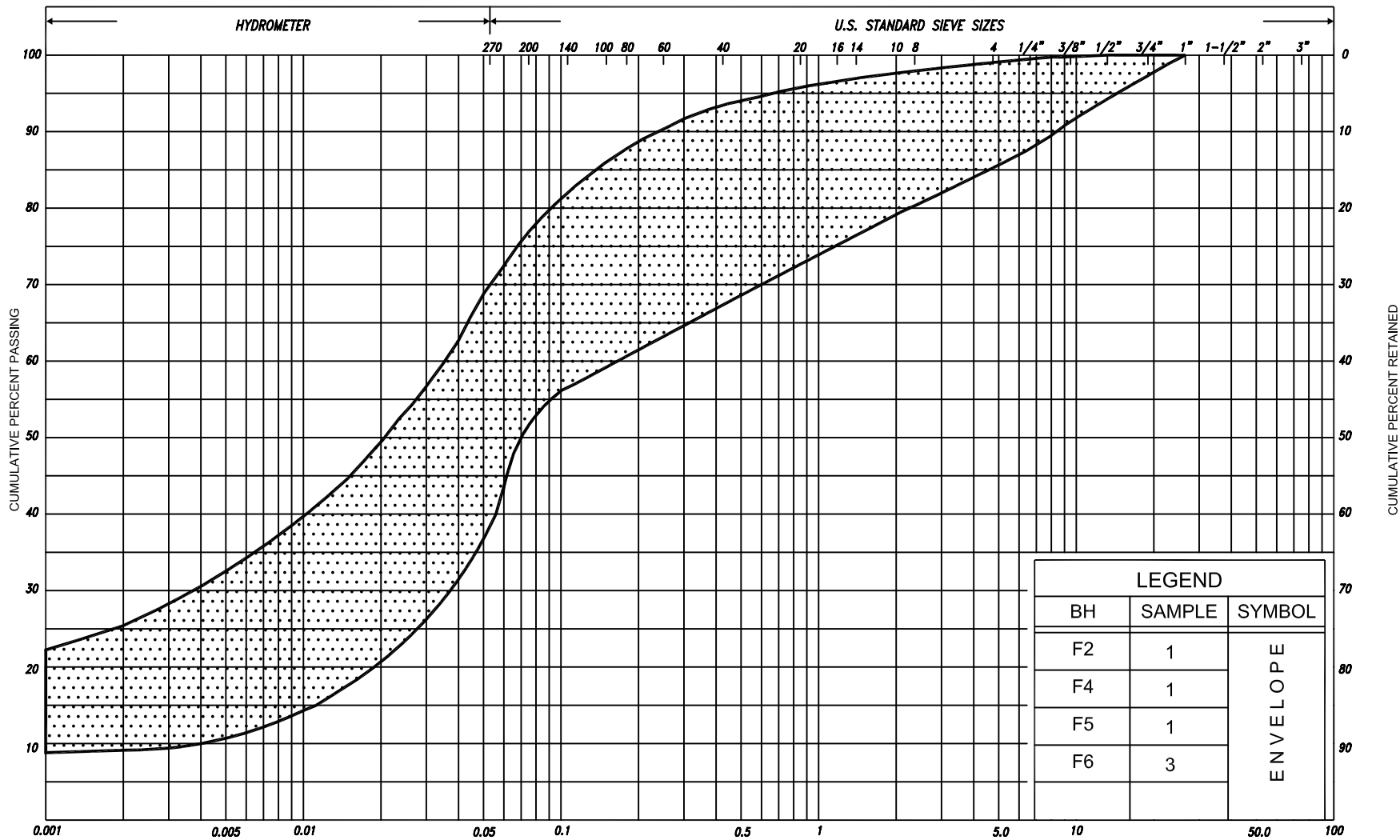
Carlos M.P. Nascimento, P.Eng.
Senior Project Engineer



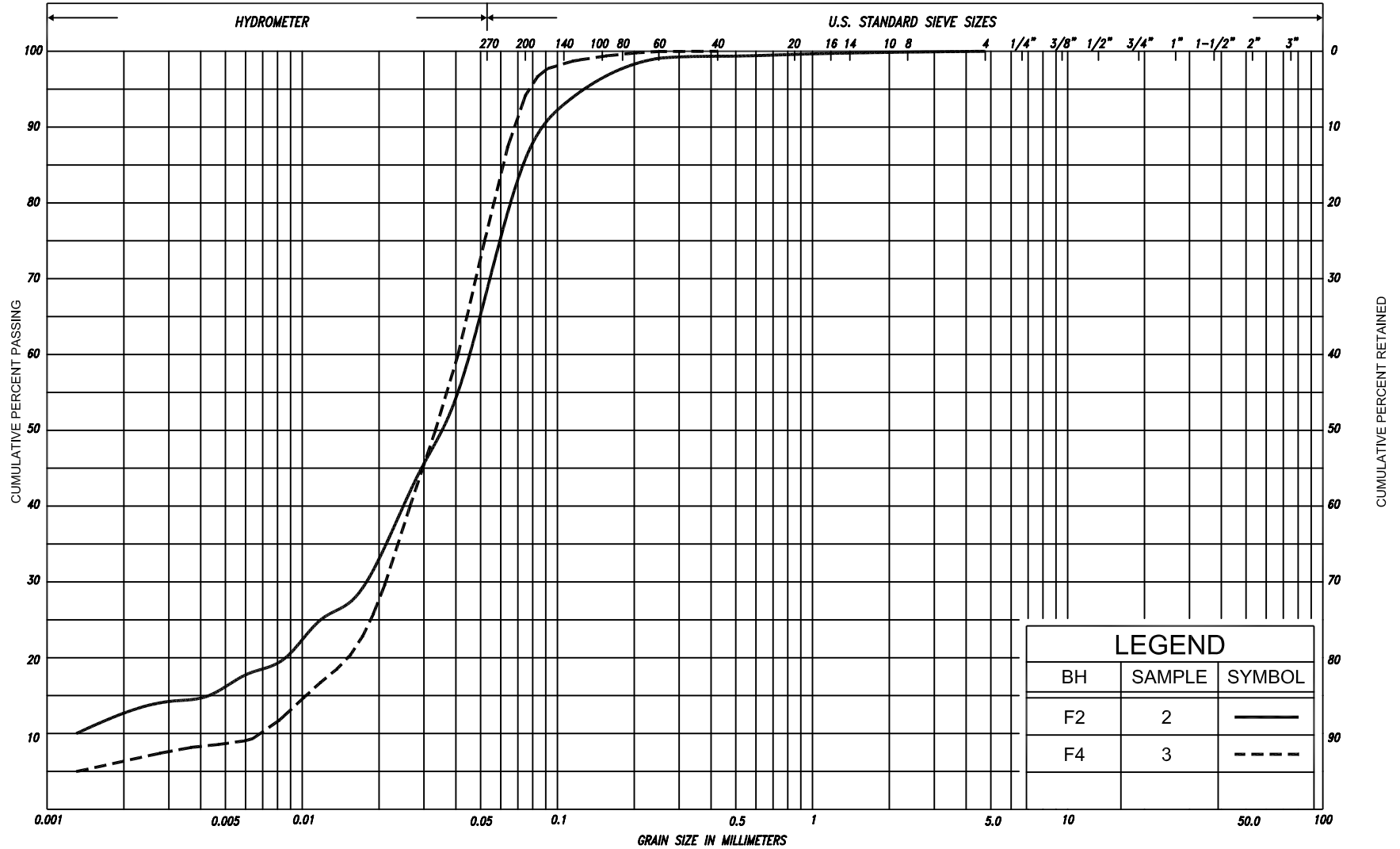
Brian R. Gray, MEng, P. Eng,
MTO Designated Contact and Project Manager



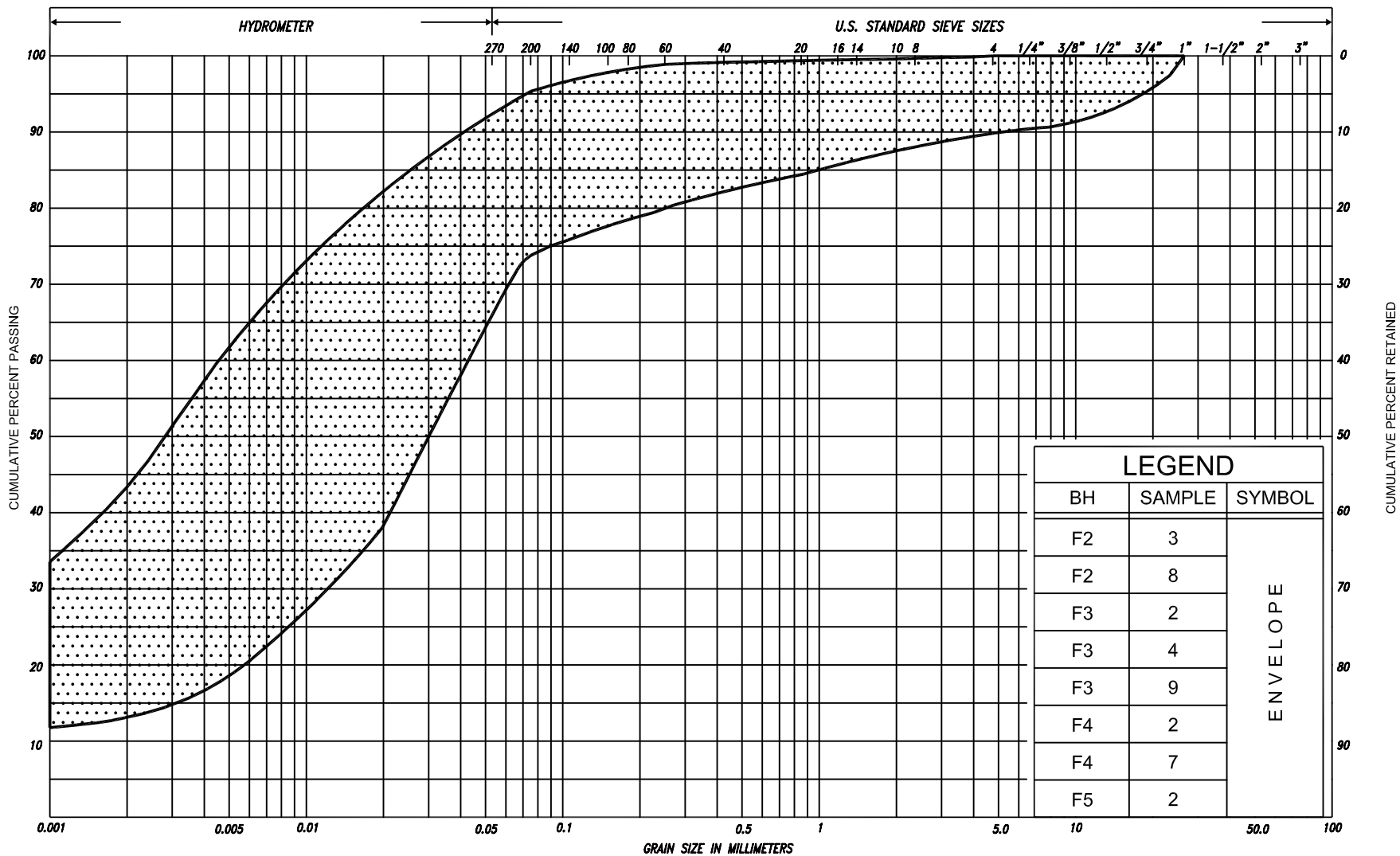
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SILT & CLAY					FINE		MEDIUM		COARSE		GRAVEL			COBBLES	UNIFIED		
					SAND												
CLAY	FINE		MEDIUM		COARSE	FINE		MEDIUM		COARSE		GRAVEL			COBBLES	M.I.T.	
	SILT																
CLAY		SILT			V. FINE	FINE	MED.	COARSE		GRAVEL							U.S. BUREAU
	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



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



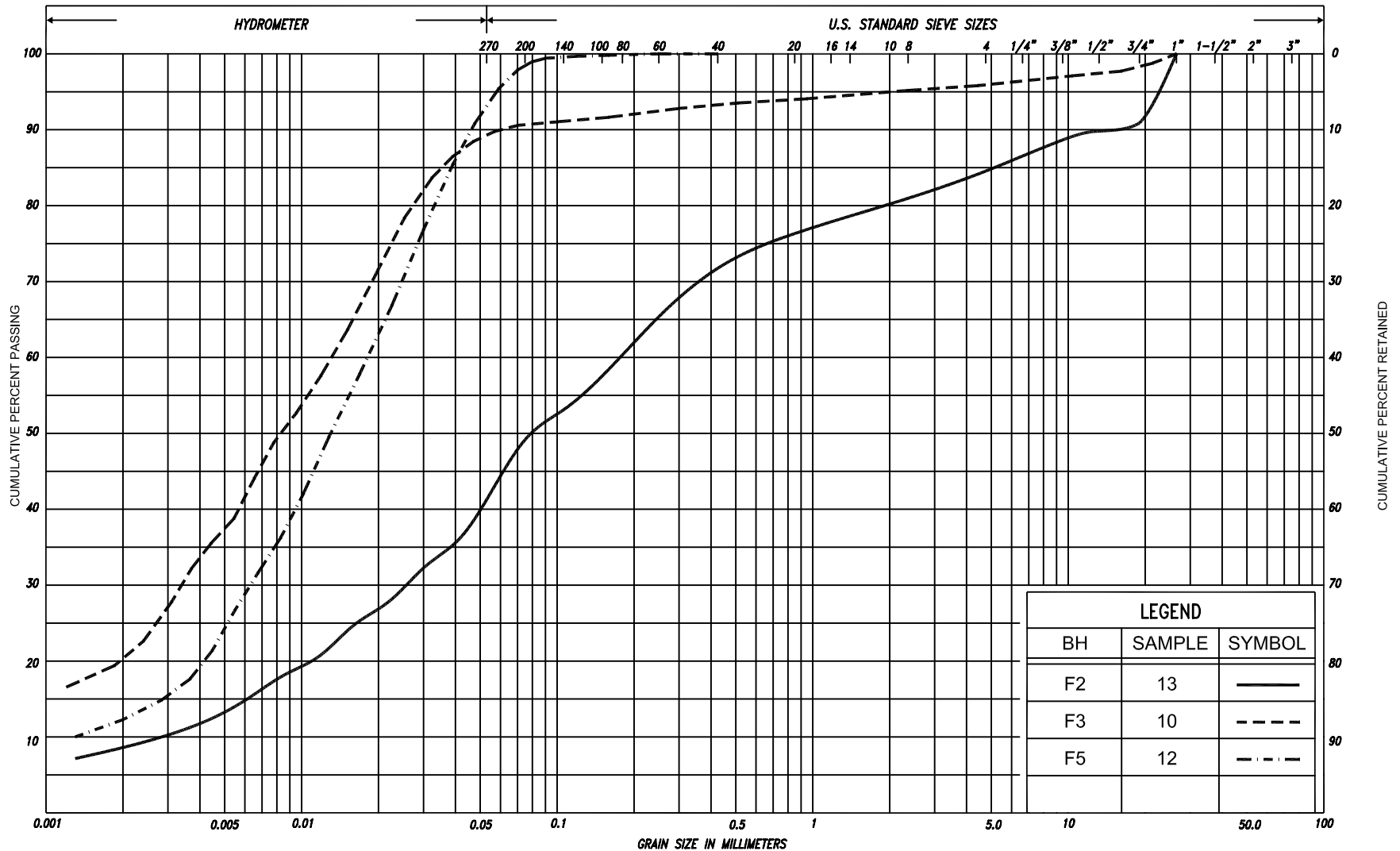
SILT & CLAY					FINE		MEDIUM		COARSE	GRAVEL			COBBLES	UNIFIED		
					SAND											
CLAY	FINE		MEDIUM		COARSE		FINE		MEDIUM		COARSE		GRAVEL		COBBLES	M.I.T.
	SILT															
CLAY		SILT			V. FINE	FINE	MED.	COARSE	GRAVEL							U.S. BUREAU
					SAND											

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

FIGURE GS - 3

PROJECT: FARLEY'S CREEK BRIDGE REPLACEMENT

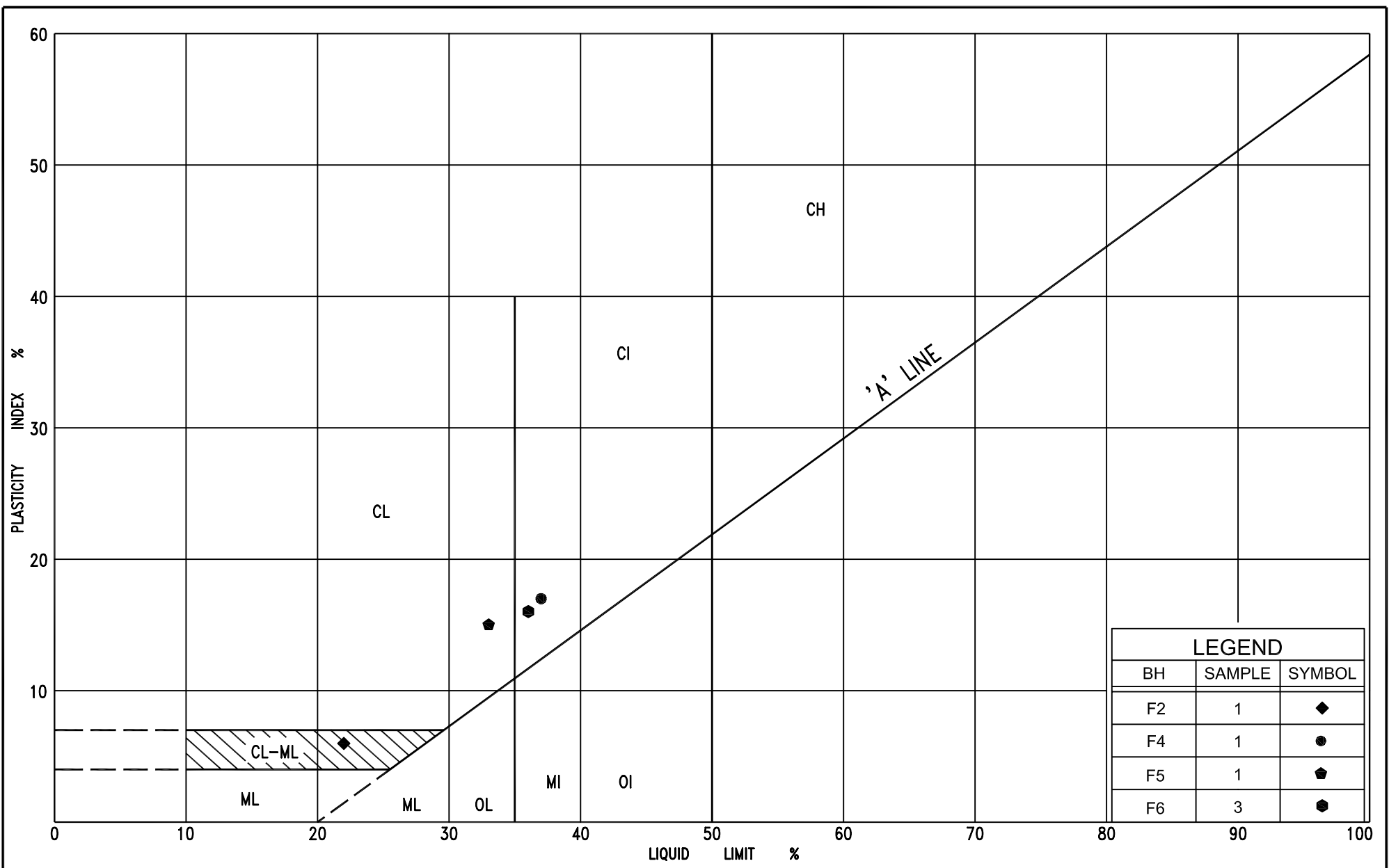
G.W.P. No. 342-97-00

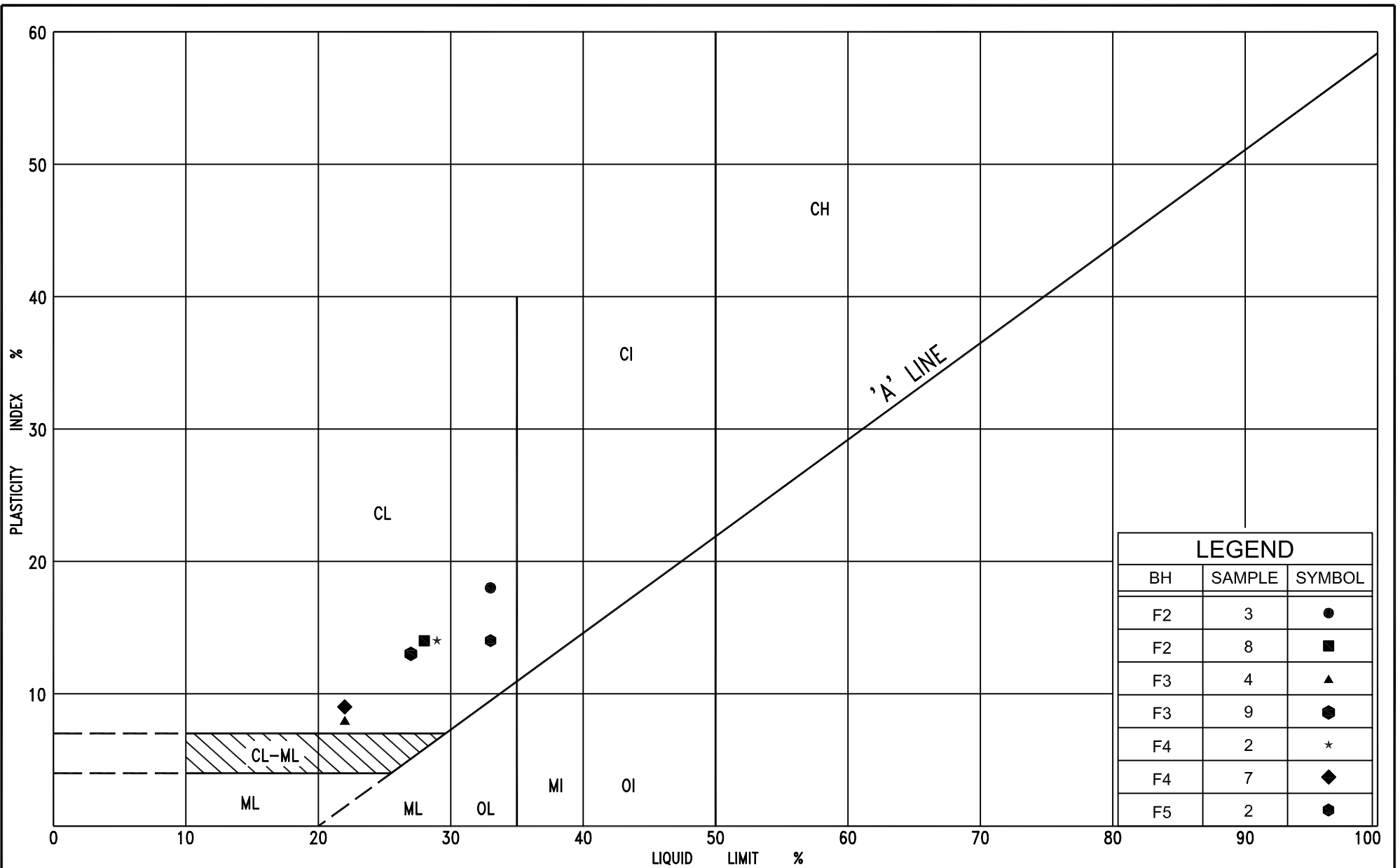


SILT & CLAY				FINE			MEDIUM			COARSE			GRAVEL			COBBLES	UNIFIED
							SAND										M.I.T.
CLAY	FINE		MEDIUM	COARSE	FINE		MEDIUM	COARSE			GRAVEL					COBBLES	
		SILT					SAND										U.S. BUREAU
CLAY					Y. FINE	FINE	MED.	COARSE			GRAVEL						
							SAND										

GRAIN SIZE DISTRIBUTION

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





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 F1

1 of 1

METRIC

G.W.P. 342-97-00 LOCATION Co-ords. 4 856 104 N; 219 729 E
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers
DATUM Geodetic DATE June 01, 2006
ORIGINATED BY F.P.
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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa									
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	× LAB VANE	WATER CONTENT (%)					
444.3	Ground surface						20	40	60	80	100						
0.0	Asphalt (130 mm)																
0.1	Sand and gravel																
	Compact Brown Dry		1	SS	19							○					
	(PAVEMENT FILL)		2	SS	11												
442.8																	
1.5	Clayey silt trace sand, trace gravel		3	SS	20								○				
	Very stiff Brown Moist																
	silt layers		4	SS	16						200	●	○				
	Very stiff to hard																
	(TILL)		5	SS	33								○				
440.2																	
4.1	Sandy silt																
	Compact Brown Wet		6	SS	27									○			
438.6																	
5.7	Clayey silt trace sand, trace gravel																
	Very stiff Brown Moist		7	SS	24						200	●	○				
	(TILL)																
	Hard Grey																
436.1			8	SS	46									○			
8.2	End of borehole																

* 2006 06 01

▽ Water level observed during drilling

▼ Water level measured after drilling

■ Penetrometer test

RECORD OF BOREHOLE No F2

1 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords. 4 856 118 N; 219 715 E
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers
DATUM Geodetic DATE June 13, 2006

ORIGINATED BY F.P.

COMPILED BY G.D.

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 (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100	W _P W W _L					
444.1	Ground surface					*	444								
0.0	Asphalt (140 mm)														
0.1	Sand and gravel														
443.2	Compact Brown Dry --- (PAVEMENT FILL) ---						443								
0.9	Sandy clayey silt some gravel topsoil inclusions, cobbles														
	Very stiff Brown Wet (FILL)		1	SS	50/ 13cm									15 33 43 9	
441.7	Clayey silt, trace sand						442								
2.4	Very stiff Brown Moist (TILL)														
440.9	Silt, some sand, some clay			2	SS		45	441							0 14 73 13
3.2	Dense Brown Wet														
440.1	Clayey silt trace sand, trace gravel						440								
4.0	Very stiff Grey Moist to hard (TILL)			3	SS	26								4 9 44 43	
	some sand			4	SS	51	439								
	silt partings														
	clayey silt lenses, cobbles			5	SS	29	438								
				6	SS	61	437								
			7	SS	76	436									
							435						10 16 41 33		
			8	SS	36	434									
							433								
			9	SS	34	432									
							431								
			11	SS	20	430									

Cont'd

Cont'd

2 of 2

METRIC

G.W.P. 342-97-00

LOCATION

Co-ords. 4 856 118 N; 219 715 E
Hwy 6, 12+654.7, o/s 5.0m 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 13, 2006

CHECKED BY C.N.

ON_MOT VER3 05KF104-FARLEYS CREEK.GPJ ON_MOT.GDT 12/21/2006 2:22:10 PM

+7, X⁵: Numbers refer to Sensitivity

(%) STRAIN AT FAILURE

RECORD OF BOREHOLE No F3

1 of 2

METRIC

G.W.P. 342-97-00 LOCATION Co-ords. 4 856 128 N; 219 718 E
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers
DATUM Geodetic DATE June 12, 2006

ORIGINATED BY F.P.
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									
								20	40	60	80	100					
								20	40	60	80	100					
444.2	Ground surface																
0.0	Asphalt (140 mm)																
0.1	Sand and gravel						444										
	Compact Brown Dry																
443.3	--- (PAVEMENT FILL) ---																
0.9	Clayey silt gravel and topsoil inclusions						443										
	Firm Brown Moist (FILL)		1	SS	5									o			
441.8							442										
2.4	Silt trace sand, trace clay																
	Compact Brown Moist																
440.9			2	SS	23		441							o			0 13 74 13
3.3	Clayey silt some sand, trace gravel																
	Very stiff Brown Moist (TILL)						440							o			
			3	SS	26									o			
			3A	SS	59		439							o			
			4	SS	24		438							Hel			1 11 60 28
	silt partings						437										
	Hard																
			5	SS	31									o			
							436										
			6	SS	34		435							o			
							434										
	trace sand cobbles																
	Grey		7	SS	35/3cm		433							o			
			8	SS	42		432							o			
							431										
			9	SS	33									Hel			1 4 59 36
							430										
429.2																	

METRIC

ON_MOT VER3 05KF104-FARLEYS CREEK.GPJ ON_MOT.GDT 12/21/2006 2:22:14 PM

+7, X⁵: Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No F4

1 of 1

METRIC

G.W.P. 342-97-00 LOCATION Co-ords. 4 856 132 N; 219 704 E
DIST Owen Sound HWY 6 BOREHOLE TYPE Continuous Flight Solid Stem Augers
DATUM Geodetic DATE June 08, 2006

ORIGINATED BY F.P.
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			SHEAR STRENGTH kPa									
								20	40	60	80	100					
								○ UNCONFINED	+	FIELD VANE							
								● QUICK TRIAXIAL	×	LAB VANE							
								WATER CONTENT (%)									
								20	40	60	80	100					
444.0	Ground surface																
0.0	Asphalt (130 mm)																
0.1	Sand and gravel																
	Compact Dark Dry																
	brown/ brown																
442.8	--- (PAVEMENT FILL) ---						443										
1.2	Clayey silt to silty clay with sand, trace gravel organic inclusions																
	Soft Brown Moist		1	SS	3		442						○	├─			4 29 49 18
	(FILL)																
441.5	Clayey silt trace sand, trace gravel																
2.5	Very stiff Brown Moist						441						┐├─				4 7 52 37
	(TILL)		2	SS	27												
							440										
439.9	Silt trace sand, trace clay																
4.1	Dense Brown Wet						439						○				0 6 87 7
			3	SS	43												
438.4	Clayey silt trace sand, trace gravel						438						○				
5.6	Hard Brown Moist																
	(TILL)		4	SS	52		437										
			5	SS	45		436						○				
			6	SS	37		435										
	some sand						434										
			7	SS	107		433						○	├─			6 19 49 26
			8	SS	101												
432.7	End of borehole												○				
11.3																	

RECORD OF BOREHOLE No F5

1 of 2

METRIC

G.W.P. 342-97-00

LOCATION

Co-ords. 4 856 140 N; 219 707 E
Hwy 6, 12+675.7, o/s 5.5m 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 12, 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		
444.0	Ground surface													
0.0	Asphalt (130 mm)													
0.1	Sand and gravel													
	Compact Brown Damp (PAVEMENT FILL)													
442.9							443							
1.1	Clayey silt to silty clay some sand, trace gravel organic inclusions													
	Firm Brown Moist (FILL)		1	SS	5		442							1 23 55 21
441.8														
2.2	Clayey silt trace sand, trace gravel cobbles													
	Very stiff to hard Brown Moist (TILL)		2	SS	18		441					225		2 10 50 38
							440							
			3	SS	31		439							
			4	SS	49		438							
			5	SS	38									
			6	SS	50/ 15cm		437							
			7	SS	29		436					162		
	Grey						435							
	Brown		8	SS	45		434							
							433							
	Grey		9	SS	83		432							
432.3							431							
11.7	Silt, trace sand trace to some clay clayey silt seams						430							
	Very dense Brown Moist (TILL)		10	SS	121									
	Grey		11	SS	147									
429.0														

METRIC

ON_MOT VER3 05KF104-FARLEYS CREEK.GPJ ON_MOT.GDT 12/21/2006 2:22:20 PM

+7, X⁵: Numbers refer to Sensitivity

20
15—○—5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No F6

1 of 1

METRIC

G.W.P. 342-97-00 LOCATION Co-ords. 4 856 153 N; 219 693 E
Hwy 6, 12+694.7, o/s 4.6m 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 01, 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			20	40	60	80	100					
443.9	Ground surface																
0.0	Asphalt (130 mm)																
0.1	Sand and gravel																
443.1	Compact Brown Dry (PAVEMENT FILL)		1	SS	15												
0.8	Clayey silt to silty clay with sand, trace gravel topsoil and organic inclusions		2	SS	21												
	Very Dark Moist stiff brown		3	SS	6												
	Grey		4	SS	9												
440.9	(FILL)																
3.0	Clayey silt some sand, trace gravel		5	SS	20												
	Very Brown Moist stiff																
	Hard Grey																
	(TILL)		6	SS	35												
			7	SS	51												
			8	SS	51												
435.7																	
8.2	End of borehole																

METRIC

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

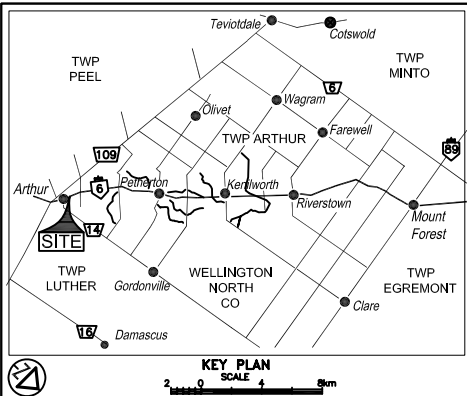
CONT No
WP No 342-97-00

HIGHWAY 6
FARLEY'S CREEK BRIDGE
BOREHOLE LOCATIONS



SHEET

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 June 2006
- Head
- ARTESIAN WATER Encountered
- PIEZOMETER

BH No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
F1	444.3	4 856 104	219 729
F2	444.1	4 856 118	219 715
F3	444.2	4 856 128	219 718
F4	444.0	4 856 132	219 704
F5	444.0	4 856 140	219 707
F6	443.9	4 856 153	219 693

- 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 DRAWING F2 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 S6258-340-001GA; H6258xb01; H6258xn01
DATED AUGUST 24, 2006

REVISIONS	DATE	BY	DESCRIPTION

Geocres No. 40P15-37

HWY No	6	DIST	OWEN SOUND
SUBM'D	FP	CHECKED	CN
DRAWN	NA	CHECKED	CN
DATE	DEC. 20, 2006	APPROVED	BRG
SITE	35-135	DWG	F1

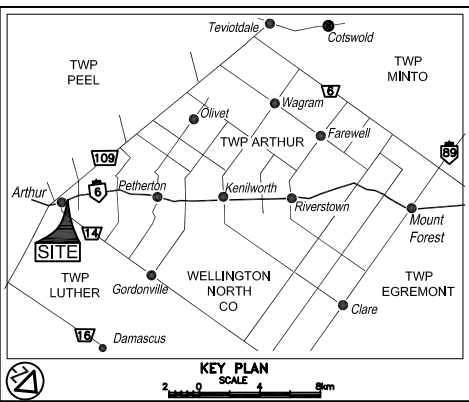
METRIC

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

CONT No
WP No 342-97-00

HIGHWAY 6
FARLEY'S CREEK BRIDGE
SOIL SECTIONS

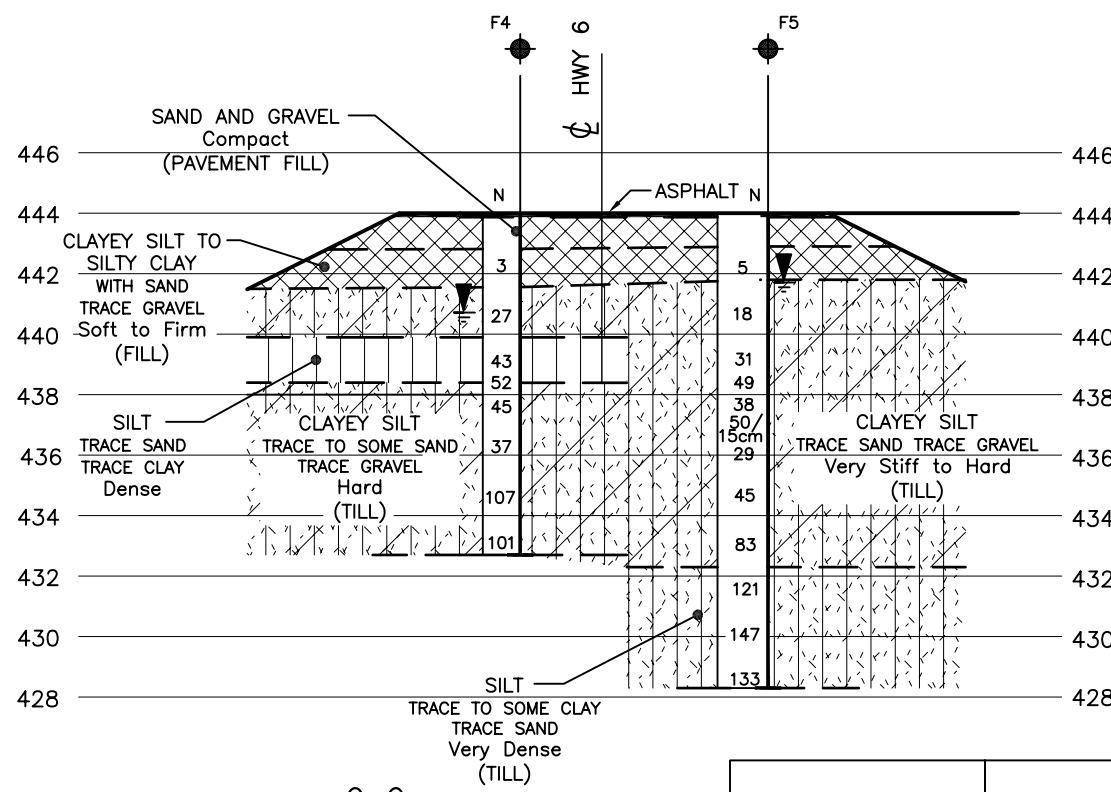
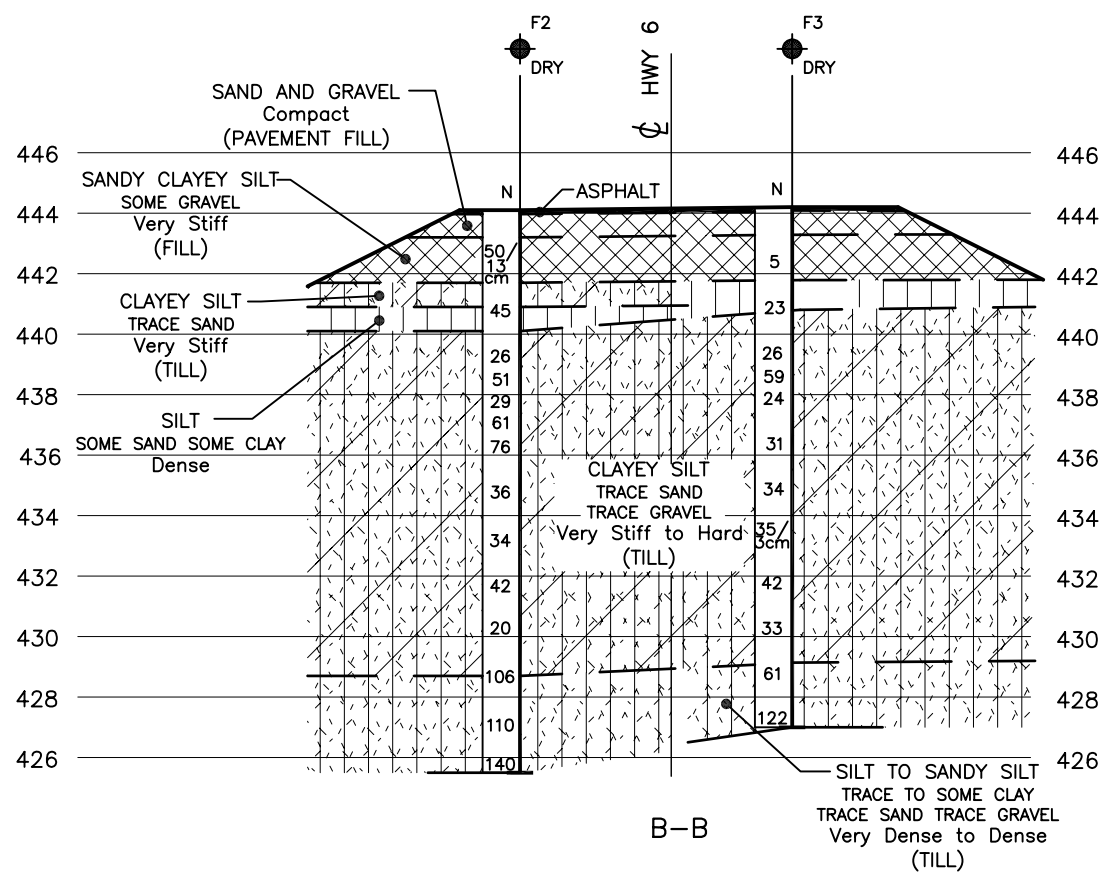
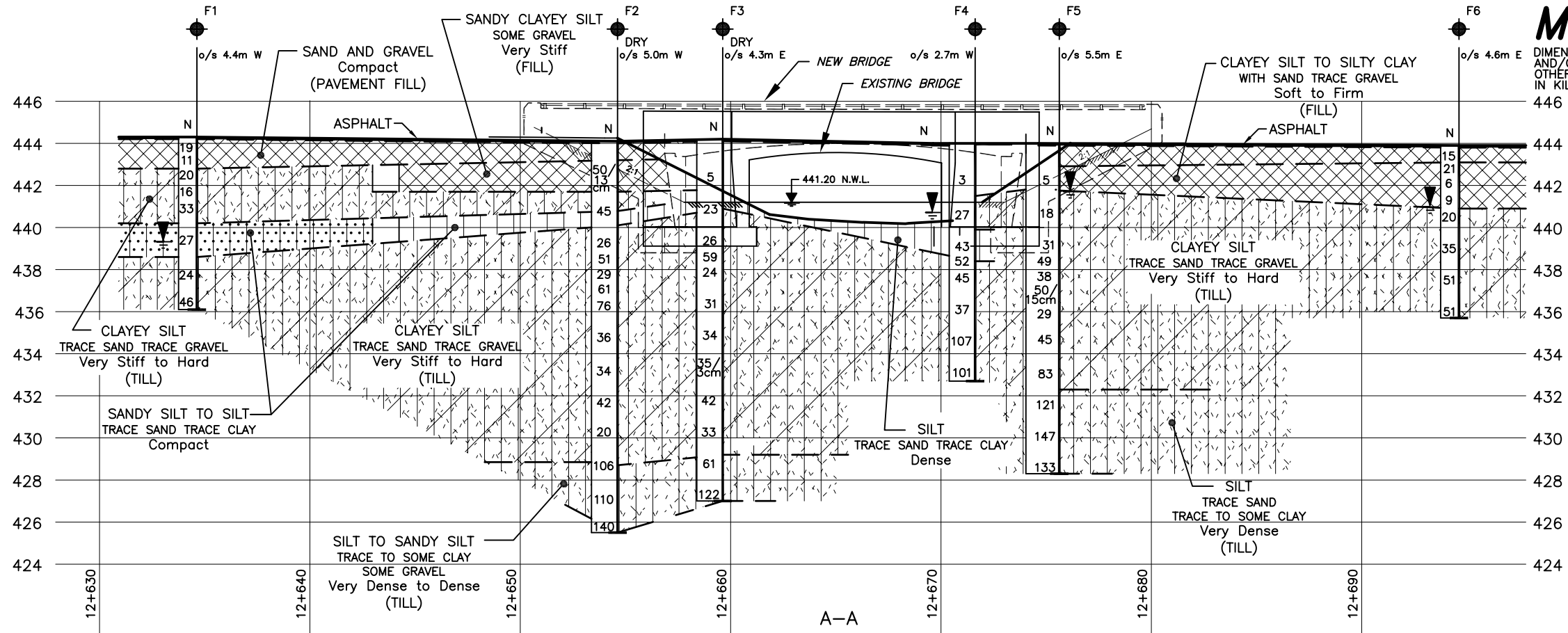
SHEET



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
REFER TO DRAWING F1 FOR DETAILS			

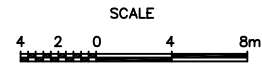
— NOTE —

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



NOTES:

- REFER TO DRAWING F-1 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 S6258-340-001GA; H6258xb01; H6258xn01
DATED AUGUST 24, 2006



REVISIONS						
	DATE	BY	DESCRIPTION			

Geocres No. 40P15—37

HWY No	6					DIST	OWEN SOUND
SUBM'D	FP	CHECKED	CN	DATE	DEC. 20, 2006	SITE	35—135
DRAWN	NA	CHECKED	CN	APPROVED	BRG	DWG	F2



**FOUNDATION DESIGN REPORT
for
FARLEY'S CREEK BRIDGE REPLACEMENT
SITE NO. 35-135
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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Distribution:

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PML Ref.: 05KF104B
Index No.: 146FDR
Geocres No.: 40P15-37
December 29, 2006



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Table 1 – List of Standard Specifications Referenced in Report

Table 2 - Gradation Requirements for Sand Fill at Integral Abutments

FOUNDATION DESIGN REPORT

for
Farley's Creek Bridge Replacement
Site No. 35-135
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 Farley's Creek about 0.1 km north of Wells Street in Arthur. The investigation was conducted for McCormick Rankin Corporation (MRC) on behalf of the Ministry of Transportation of Ontario.

Highway 6 crosses Farley's Creek at about Station 12+665, in the Township of Arthur. According to the RFP and to General Arrangement Drawing Reference S6258-340-001 GA dated August 24, 2006, prepared by MRC, the existing bridge is a single span rigid frame structure with a span of about 9.0 m. The existing abutments are founded on spread footings with the base established at an estimated elevation 439.0, as shown on the reference drawing.

It is planned to replace the existing structure with a new wider bridge with a longer span of about 15.0 m. The existing highway pavement grades over the bridge will be raised about 0.5 m.

The upper layers of the soil stratigraphy revealed in the boreholes generally comprised the Highway 6 asphalt pavement and approach embankment fill. Since the existing abutments are founded on spread footings, the embankment fills are likely to extend to levels as deep as the existing footings (elevation 439.0) at locations immediately behind the abutment. The fills overlay a discontinuous deposit of compact to dense silt/sandy silt that locally overlays or is interbedded within glacial deposits of typically very stiff to hard clayey silt till and very dense sandy silt till. Cobbles were found in some of the boreholes. The bedrock underlying the Farley's Creek Bridge is mainly composed of dolostone, shale, gypsum and salt of the Salina Formation. The estimated depth to the bedrock level is about 55 m at this site.



The groundwater encountered at levels ranging from 1.8 to 5.3 m depths, elevations 439.0 to 442.5 is typically perched at the fill/native soil interface and is influenced by the water level in the Farley's Creek. The groundwater level in the Farley's Creek was recorded at elevation 441.2 in June 2006.

It is considered feasible to establish the foundations the new bridge abutments and wing walls 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 consists of replacing the east and west halves of the bridge separately and will require the diversion of traffic to a single lane over one half of the bridge deck while the other half of the bridge is being replaced. This construction staging will require road protection installed along (or near) the centreline of the alignment behind both bridge abutments. Road protection with performance level 1a including sheetpiling will likely be required in view of the requirement to retain the granular pavement of the highway. Where sheetpiling is used the tips should be equipped with driving shoes in view of the potential presence of boulders in the till soils. For shallow foundations where excavations will extend to levels about 2.1 m below the water level in the Farley's Creek, sheetpiling and groundwater control is likely required to adequately establish the founding subgrade.

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

It is inferred that the new wider channel of Farley's Creek will be shaped to about elevation 441.2.

The existing abutment spread footings are founded at about elevation 439.0, according to the reference drawings. The abutments of the new replacement bridge are proposed about 3.0 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 elevations or slightly higher levels (maximum elevation 439.4) 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 very dense till soils encountered at 15 to 16 m depths at the south abutment and at 9 to 10 m at the north abutment. The presence of cobbles and possible boulders within the glacial soils should be noted since these particles may damage the piles and/or cause the piles to reach false refusal. Drilled caissons to support the foundations are not considered practical for this site due to the presence of cobbles/boulders and the wet condition of the native cohesionless soils 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.

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).



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.

2.2 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 very stiff clayey silt till or locally dense silt (borehole F4) at the highest elevation 439.4 provided that the above recommendations are followed. The recommended bearing resistance for minimum 2.5 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 about 1.8 m above the founding depth (creek level at elevation 441.2) 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 Deep Foundations

2.3.1 General

Conventional or integral/semi-integral abutment designs are considered feasible at the Farley's Creek Bridge site.

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

2.3.2 Conventional Abutment Considerations

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

LOCATION	STRATUM DEPTH (m)	STRATUM ELEVATION	RELEVANT BOREHOLES
North Abutment	9.0 to 11.7	432.3 to 435.0	F4 and F5
South Abutment	15.4 to 17.0	427.2 to 428.7	F2 and F3

The depths and elevations are taken from the existing ground surface at the borehole locations to the top of the founding soils. 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. The piles should, however be equipped with driving shoes, such



as the Titus "H" Bearing Pile Point, Standard Model. 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 below 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 relatively short 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 as outlined in SP 903S01 should be included in the contract.

2.3.3 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.



2.3.4 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.

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 very stiff clayey silt till. 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/m³) should be computed using the following equation for the cohesive very stiff to hard clayey silt till below elevation ± 440 .

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

$$C_u = \text{undrained shear strength of the clayey silt till} \\ = 150 \text{ kPa}$$

$$b = \text{pile width, m}$$

For design purposes, the groundwater should be considered at elevation 441.2, that is approximately the current water level of the Farley's 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	GRANULAR B TYPE II	SILTY CLAY TILL
Friction Angle, degrees	35	35	0
Cohesion, kPa	0	0	150
Unit Weight, kN/m ³	22.8	22.8	20.0

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

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 this site 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 Farley's 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 (about elevation 441.2 at the abutments) 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

Should construction and traffic staging required traffic adjacent to the excavations it is anticipated that a suitable roadway protection scheme following OPSS 538 and SP 105S19 will be required to support the walls of the excavation and adjacent traffic lanes during construction.

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 is recommended 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 Farley's Creek at the time of the field investigation (elevation 441.2) was up to 2.2 m above the anticipated deepest level of excavation (elevation 439.0). Cognisant of the low permeability characteristics of the clayey silt and sandy silt layers and the relatively high hydraulic head, it is anticipated that vigorous pumping from sumps installed within the cofferdams will be required to control seepage of water into the excavations.



The perched groundwater that was observed within the fill in the abutment boreholes (F1 and F6) 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. It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of groundwater into the general embankment excavations.

Surface water flow should be diverted away from the excavation that will be required for demolition and for the construction of new foundation structures to help maintain the founding subgrade in the dry.

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 Farley's Creek Bridge.



ADVANTAGES AND DISADVANTAGES – FARLEY’S CREEK BRIDGE REPLACEMENT

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.

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.

A handwritten signature in cursive script, reading "C. M. P. Nascimento", is positioned to the left of the professional engineer's seal.



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

A handwritten signature in cursive script, reading "Brian R. Gray", is positioned to the left of the professional engineer's seal.



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 208.010	Benching of Earth Slopes	November 2003
OPSD 3101.150	Minimum Granular Backfill Requirements - Abutments	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