



FOUNDATION INVESTIGATION AND DESIGN REPORT
for
DAHMER MUNICIPAL DRAIN CULVERT REPLACEMENT
SITE NO. 25-318-C
RESURFACING 9 KM OF HIGHWAY 7 & 8
G.W.P. 335-97-00
TOWNSHIP OF PERTH EAST, ONTARIO

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PML Ref.: 05TF036A
Index No.: 016FIR and 017FDR
Geocres No.: 40P7-52
February 28, 2006



FOUNDATION INVESTIGATION REPORT

for

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Record of Borehole Sheets

Drawing 1 – Borehole Locations & Soil Strata

FOUNDATION INVESTIGATION REPORT
for
Dahmer Municipal Drain Culvert Replacement
Site No. 25-318-C
Resurfacing 9 km of Highway 7 & 8
G.W.P. 335-97-00
Township of Perth East, Ontario

1. INTRODUCTION

Planned under this project is the resurfacing a 9 km long section of Highway 7 & 8 that extends approximately from 1.7 km west of Waterloo Regional Road 1 easterly to 0.9 km west of Waterloo Regional Road 51 in the Township of Perth East, Ontario. This report was prepared for McCormick Rankin Corporation on behalf of the Ministry of Transportation of Ontario.

The resurfacing of Highway 7 & 8 will involve replacement of a culvert where the Dahmer municipal drain crosses the highway at approximate Station 25+389, Highway 7 & 8 chainage. The culvert is a single cell cast-in-place concrete rigid frame open footing structure with a size of 5.50 m wide by 1.52 m high and 21.5 m long. Replacement of the culvert with a precast concrete box culvert of similar or slightly larger size is envisaged.

This report provides a summary of the factual information obtained during the field investigation conducted at the location of the proposed culvert replacement.

2. SITE DESCRIPTION AND GEOLOGY

Highway 7 & 8 within the project limits is primarily situated in a rural setting with slightly undulating terrain. Land use along the study corridor is mainly agricultural, with municipal ditches as integral features of the landscape.

The project area lies in the physiographic region known as the Stratford Till Plain characterised by a faint knoll-and-sag relief with an overall slope towards the southwest. The principal soils along the study corridor are represented by heavy-textured calcareous silty clay / clayey silt till and sand/gravel. The till is fairly uniform and widespread in the region once covered by the Huron ice lobe. The more level parts of the plain are poorly drained (L.J.Chapman & D.F.Putnam, *The Physiography of Southern Ontario*, 3rd Edition, Ontario Research Foundation, 1984).



There is a railroad bridge structure approximately 100 m east of the culvert location. Also, a few residences are present to the west.

The culvert carries the water from the Wilmot Creek across the highway corridor.

3. INVESTIGATION PROCEDURES

The field work for this study was carried out on September 28 and 29, 2005 and comprised three boreholes advanced to depths of 6.6 to 19.0 m below existing grade. The approximate locations of the boreholes put down at both ends of the proposed culvert replacement and on the south shoulder of the highway along with a stratigraphic cross-section are shown on Drawing 1, appended.

The borehole numbers are provided with prefix code "D" to reflect the name of the Dahmer municipal drain for ease of reference.

The borehole layout was established in accordance with the requirements noted in the Request for Proposal. Peto MacCallum Ltd. selected the borehole locations in the field. The ground surface elevations at the boreholes were surveyed by Peto MacCallum Ltd. using benchmark HCM #00919743965 located at the north end of the existing culvert (elevation 351.307 m). All elevations in this report are expressed in meters.

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

Representative samples of the soil were recovered at frequent depth intervals using a conventional split spoon sampler during drilling. Standard penetration tests were conducted simultaneously with the sampling operation to assess the strength characteristics of the substrata. Penetrometer testing was performed to further assess the shear strength of the cohesive soils encountered. Dynamic cone penetration testing was also conducted in the cohesionless soil.



Soils were identified visually in the field in accordance with the MTO Soil Classification procedures. The ground water conditions at the borehole locations were assessed during drilling by visual examination of the soil, the sampler and drill rods as the samples were retrieved and, when appropriate, by measurement of the water level in open boreholes. Flowing artesian ground water conditions were encountered in one borehole (capped at the source with a grout/bentonite mix 3 hours after the flow occurrence). The remaining boreholes were backfilled with a bentonite/cement mixture in accordance with the MTO guidelines for borehole abandonment procedures.

The recovered samples were returned to our laboratory for detailed visual examination and classification. The laboratory testing program consisting of moisture content determinations as well as three Atterberg limits tests and four grain size distribution analyses was carried out on selected samples. The results of the Atterberg limits testing are presented in Figure PC-1 and grain size distribution analyses in Figures GS-1 and GS-2.

4. SUMMARISED SUBSURFACE CONDITIONS

Reference is made to the appended Record of Borehole sheets for details of the subsurface conditions including soil classifications, inferred stratigraphy, boundary elevations, standard and dynamic cone penetration as well as penetrometer test data, ground water observations and moisture content determinations. The results of laboratory Atterberg limits testing and grain size distribution analyses are also shown on the borehole logs.

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

The subsurface stratigraphy revealed in the boreholes drilled at the site comprised a surficial topsoil and/or fill underlain by cohesive clayey silt till overlying cohesionless sandy gravel at a depth of 15.8 m. Ground water was under artesian conditions in one borehole and measured in another one at 4.9 m depth (elevation 345.8). The strata encountered are summarised below.



4.1 Topsoil

Surficial topsoil was present in borehole D-3 advanced near the south end of the culvert. The topsoil had a thickness of 180 mm and was penetrated at elevation 350.5.

4.2 Fill

Surficial granular material of 700 mm thickness was present in borehole D-2 put down on the south shoulder of the highway. The granular fill overlays a road embankment platform made up of medium plasticity silty clay fill 1.8 m in thickness, with topsoil and gravel inclusions. A 100 mm thick layer of sandy gravel fill was also encountered at 2.5 m depth (elevation 349.5) in the same borehole. It is noted that the asphalt road pavement was not penetrated during the investigation.

Low to medium plasticity clayey silt / silty clay fill with topsoil inclusions was present surficially in borehole D-1 and encountered directly beneath the topsoil in borehole D-3. The fill units had a total thickness of 1.2 to 2.6 m and were penetrated at depths of 1.3 to 2.6 m (elevation 349.3 to 349.4).

The results of a grain size distribution analysis performed on the clayey fill are presented in Figure GS-1. The fill units varied in moisture content between 6 and 24%.

4.3 Clayey Silt Till

Cohesive clayey silt till of low plasticity underlays the fill in all the boreholes. This deposit was 14.5 m thick in borehole D-1, at least 5.2 and 8.5 m thick in boreholes D-3 and D-2 respectively. The clayey silt till was penetrated at 15.8 m depth (elevation 334.9) in borehole D-1. Augering was terminated within the deposit in boreholes D-2 and D-3 at respective depths of 11.1 and 6.6 m (elevation 340.9 and 344.1). The clayey silt till was very stiff to approximate elevation 346 (shear strength of 100 to over 225 kPa) and stiff with firm zones at deeper levels (shear strength of 25 to 75 kPa).



The results of Atterberg limits testing and grain size distribution analyses performed on the clayey material are presented in Figures PC-1 and GS-2 respectively. The liquid and plastic limits of the clayey silt till ranged from 22 to 28 and from 13 to 17 respectively, thus giving the plasticity index of 9 to 11. The deposit had a moisture content of 12 to 30%.

4.4 Sandy Gravel

Cohesionless sandy gravel was encountered below the silty clay till at a depth of 15.8 m (elevation 334.9) in borehole D-1. This unit had a minimum thickness of 1.4 m before termination of sampling. The sandy gravel was dense (SPT-'N' value of 42) and wet (moisture content of 19%).

4.5 Ground Water

During drilling, ground water was observed in boreholes D-1 and D-3 at respective depths of 15.8 and 1.4 m (elevation 334.9 and 349.3). Water was under artesian conditions in borehole D-1, with a flow of about 50 litres per minute and an estimated piezometer head of 1.5 m above grade. Based on the depth at which the flowing artesian water was encountered, the upper limit of the aquifer is present in the sandy gravel layer encountered at 15.8 m depth (elevation 334.9) in borehole D-1. Upon completion of augering, ground water was measured in borehole D-3 to be at 4.9 m depth (elevation 345.8). No water was observed in borehole D-2 during or upon completion of drilling.

The water level in the Wilmot Creek was about 0.5 m deep at the time of the investigation.

The stabilised ground water level is expected to be near elevation 346. Ground water levels are subject to seasonal fluctuations and precipitation patterns.



5. CLOSURE

The field work was carried out under the supervision of Mr. F. Portela, Senior Technician, and direction of Mr. G. Degil, P.Eng., Senior Foundation Engineer. The equipment was supplied by Geo-Environmental Drilling Inc. The laboratory testing was performed in the PML Toronto laboratory.

This report was prepared by Mr. G.O. Degil, PhD, P.Eng., and reviewed by Mr. C.M.P. Nascimento, P.Eng., Project Manager. Mr. B.R. Gray, MEng, P.Eng., MTO Designated Contact, conducted an independent review of the report.

Yours very truly

Peto MacCallum Ltd.

A handwritten signature in blue ink, appearing to read "Grigory O. Degil".

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

A handwritten signature in blue ink, appearing to read "Carlos M.P. Nascimento".

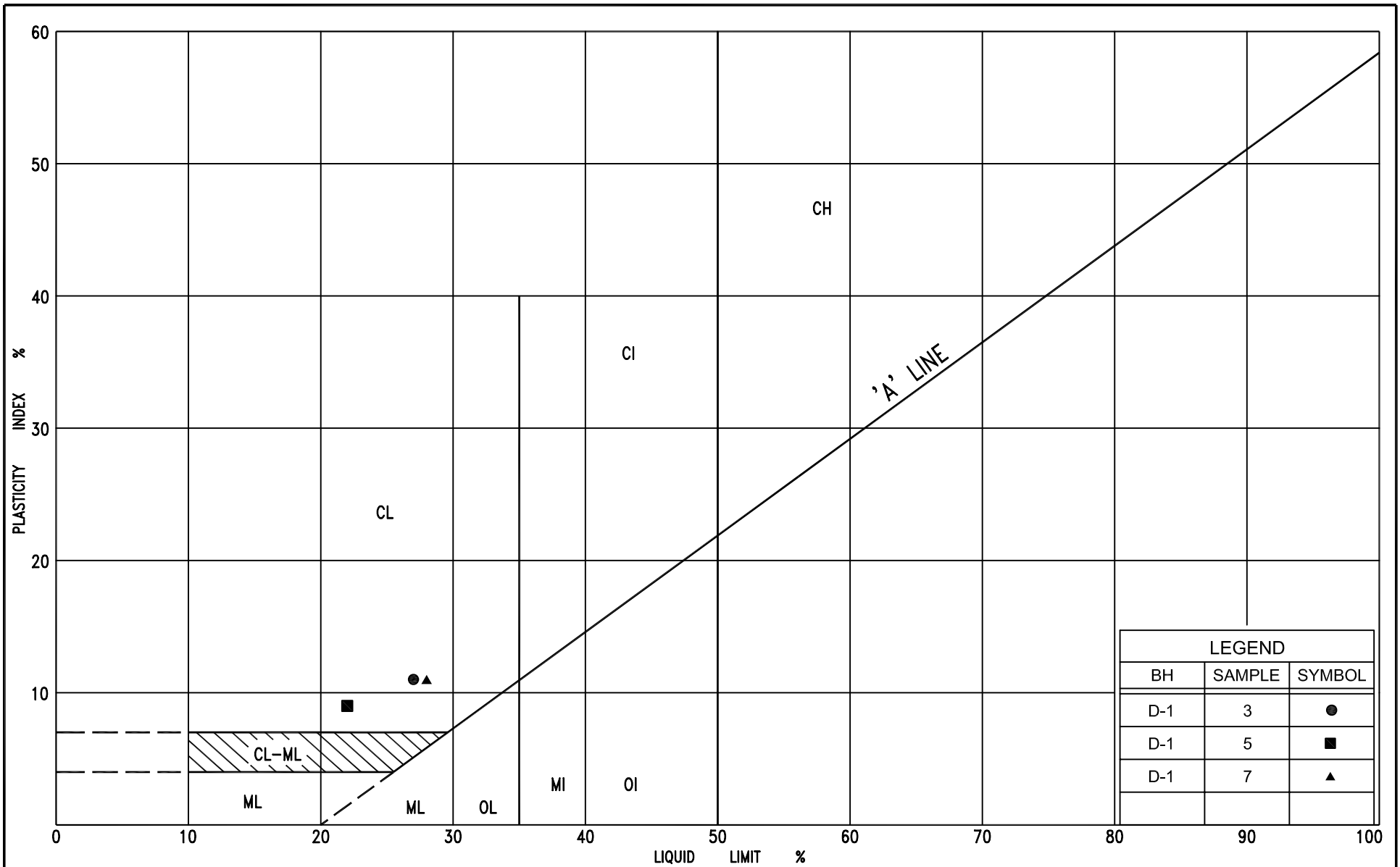
Carlos M.P. Nascimento, P.Eng.
Project Manager

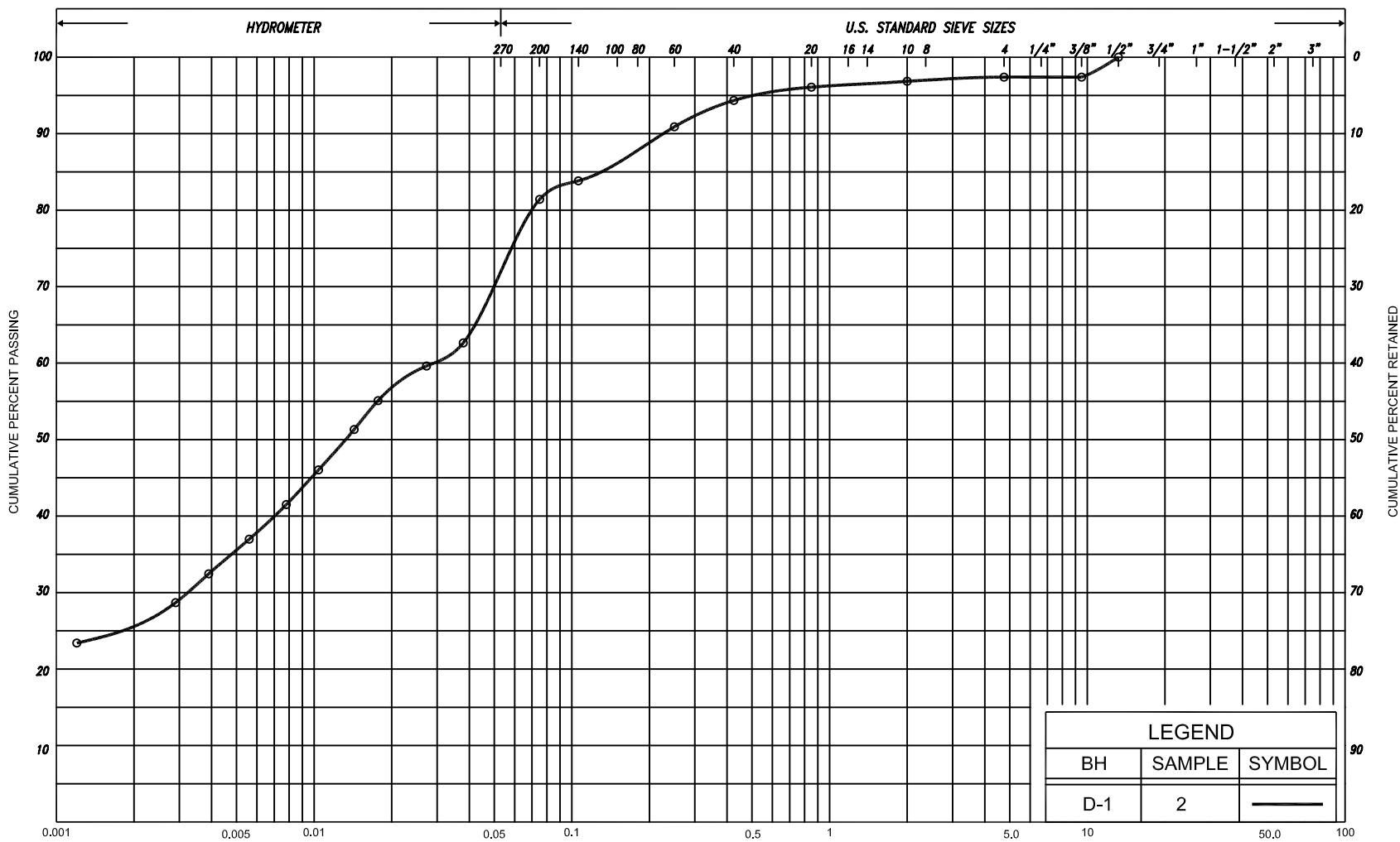
A handwritten signature in blue ink, appearing to read "Brian R. Gray".

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

GD:mi





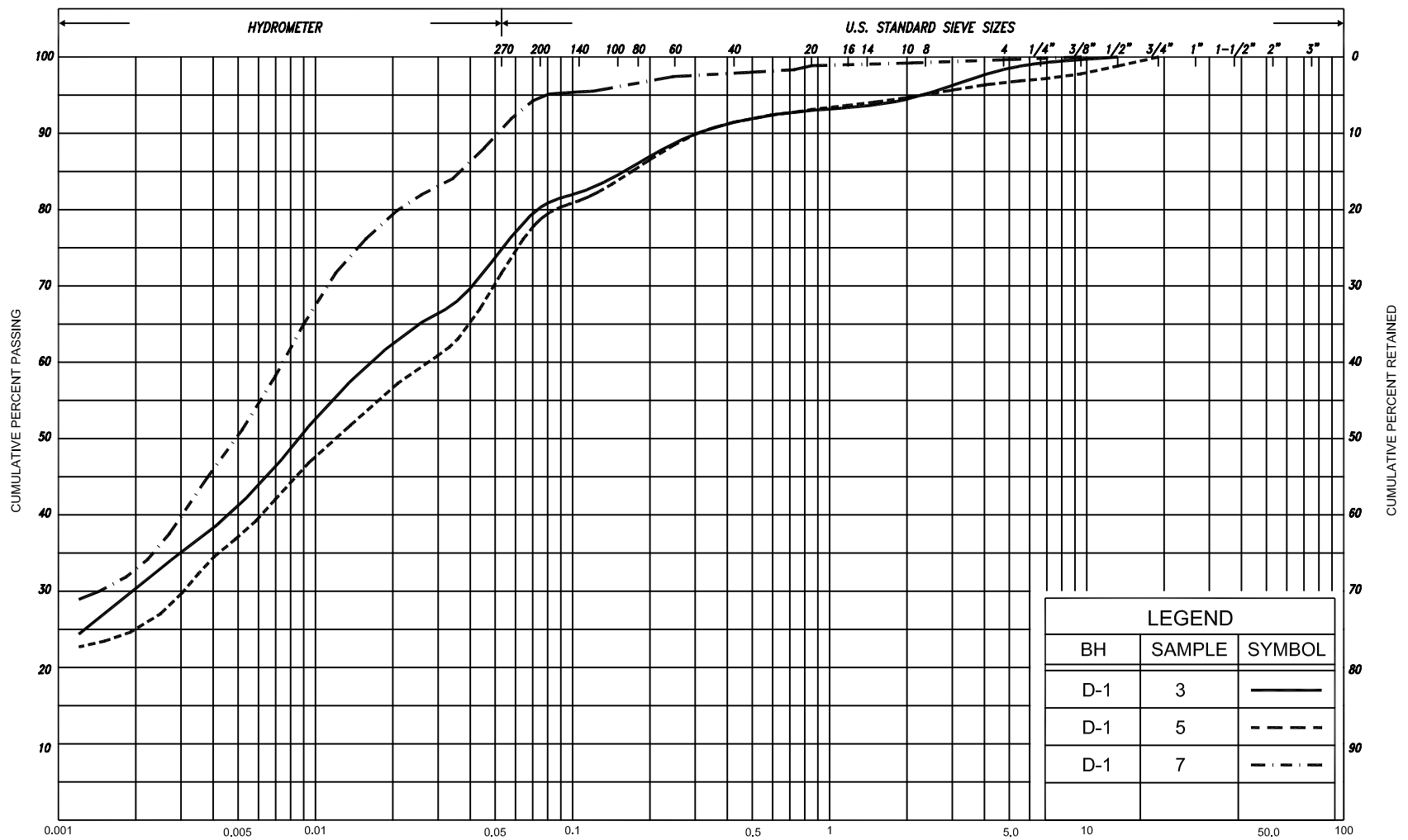


SILT & CLAY				FINE		MEDIUM		COARSE	GRAVEL		COBBLES	UNIFIED		
				SAND										
CLAY	FINE		MEDIUM	COARSE	FINE		MEDIUM		COARSE	GRAVEL		COBBLES	M.I.T.	
		SILT				SAND								
CLAY		SILT		V. FINE	FINE	MED.	COARSE	GRAVEL						U.S. BUREAU
				SAND										

GRAIN SIZE DISTRIBUTION

CLAYEY SILT, some sand, trace gravel, with topsoil inclusions
(FILL)

FIG No.	GS-1
HWY	7 & 8
G.W.P. No.	335-97-00



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								

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 (RQD), FOR MODIFIED RECOVERY, IS:

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

JOINTING AND BEDDING:

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

ABBREVIATIONS AND SYMBOLS

FIELD SAMPLING

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
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 BH D-1

1 of 2

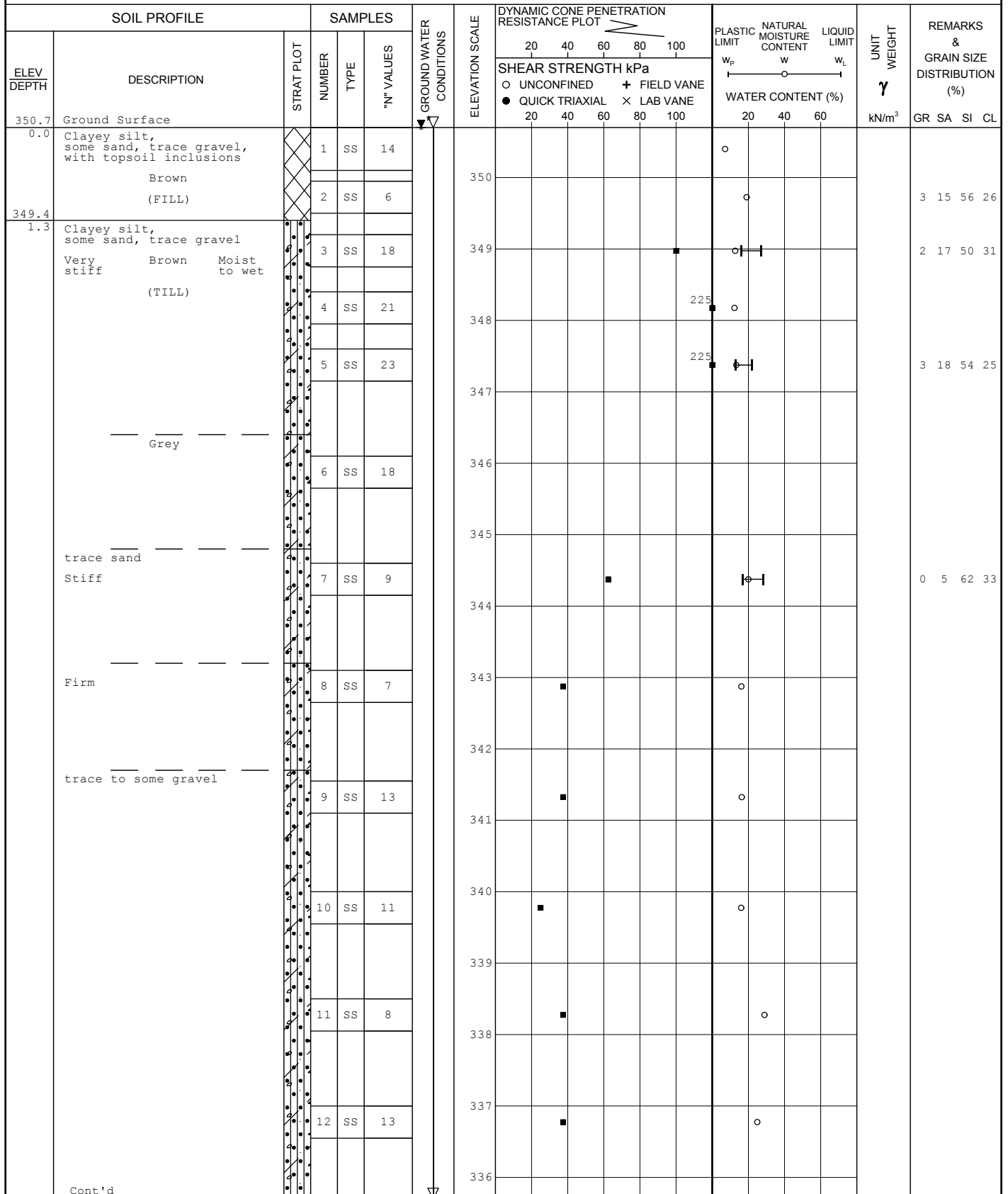
METRIC

W.P. 335-97-00 LOCATION Co-ords. 4 803 799 N; 203 187 E
DIST London HWY 7 & 8 BOREHOLE TYPE C.F.H.S.S. + Dynamic Cone Penetration Test
DATUM Geodetic DATE September 28, 2005

ORIGINATED BY F.P.

COMPILED BY G.D.

CHECKED BY



RECORD OF BOREHOLE No BH D-1

2 of 2

METRIC

W.P. 335-97-00 LOCATION Co-ords. 4 803 799 N; 203 187 E Sta. 25+381, o/s 12m Lt CL ORIGINATED BY F.P.
DIST London HWY 7 & 8 BOREHOLE TYPE C.F.H.S.S. + Dynamic Cone Penetration Test COMPILED BY G.D.
DATUM Geodetic DATE September 28, 2005 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED	+	● QUICK TRIAXIAL						×		
335.7							20	40	60	80	100	20	40	60				
334.9	Stiff		13	SS	21													
15.8	Sandy gravel																	
	Dense Grey Wet																	
333.5			14	SS	42													
17.2	End of borehole																	
	Probable sandy gravel																	
331.7																		
19.0	End of dynamic cone penetration test																	

RECORD OF BOREHOLE No BH D-2

1 of 1

METRIC

W.P. 335-97-00

LOCATION

Co-ords. 4 803 783 N; 203 204 E
Sta. 25+398, o/s 3.5m Rt CL

ORIGINATED BY F.P.

DIST London HWY 7 & 8

BOREHOLE TYPE

Continuous Flight Solid Stem Augers

COMPILED BY G.D.

DATUM Geodetic

DATE

September 29, 2005

CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS *	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE CONTENT			UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			SHEAR STRENGTH kPa					W _p	W	W _L		
352.0	Ground Surface							20	40	60	80	100					
0.0	Sand and gravel		1	SS	19												
	Brown (FILL)																
	Silty clay with topsoil inclusions		2	SS	6		351										
	Grey																
	with topsoil and gravel inclusions		3	SS	6		350										
349.4	Sandy gravel		4	SS	10												
2.6	Clayey silt, some sand, trace gravel																
	Very stiff Grey Moist to wet (TILL)		5	SS	19		349					225					
							348										
			6	SS	23		347					225					
	Stiff		7	SS	12		346										
							345										
			8	SS	13		344										
							343										
			9	SS	12		342										
340.9			10	SS	14		341										
11.1	End of borehole																
	* Borehole dry on completion of drilling																
	■ Penetrometer Test																

RECORD OF BOREHOLE No BH D-3

1 of 1

METRIC

W.P. 335-97-00

LOCATION

Co-ords. 4 803 772 N; 203 206 E
Sta. 25+400, o/s 15m Rt CL

ORIGINATED BY F.P.

DIST London **HWY** 7 & 8

BOREHOLE TYPE Continuous Flight Solid Stem Augers

COMPILED BY G.D.

DATUM Geodetic

DATE September 29, 2005

CHECKED BY _____

[illegible]



FOUNDATION DESIGN REPORT

for

DAHMER MUNICIPAL DRAIN CULVERT REPLACEMENT

SITE NO. 25-318-C

RESURFACING 9 KM OF HIGHWAY 7 & 8

G.W.P. 335-97-00

TOWNSHIP OF PERTH EAST, ONTARIO

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February 28, 2006



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Table 1 - List of Standard Specifications Referenced in Report

FOUNDATION DESIGN REPORT

for

Dahmer Municipal Drain Culvert Replacement

Site No. 25-318-C

Resurfacing 9 km of Highway 7 & 8

G.W.P. 335-97-00

Township of Perth East, Ontario

1. INTRODUCTION

This report provides foundation engineering comments and recommendations for the proposed replacement of one culvert while resurfacing a 9 km long section of Highway 7 & 8 that extends approximately from 1.7 km west of Waterloo Regional Road 1 easterly to 0.9 km west of Waterloo Regional Road 51 in the Township of Perth East, Ontario. This report was prepared for McCormick Rankin Corporation on behalf of the Ministry of Transportation of Ontario (MTO).

The resurfacing of Highway 7 & 8 will involve replacement of a culvert where the Dahmer municipal drain crosses the highway at approximate Station 25+389, Highway 7 & 8 chainage. The culvert is a single cell cast-in-place concrete rigid frame open footing structure with a size of 5.50 m wide by 1.52 m high and 21.5 m long. Replacement of the culvert with a precast concrete box culvert of similar or slightly larger size is envisaged.

Staged construction is considered for the culvert replacement and will require temporary culvert extensions, embankment platform widening and paving for the on-site detour to permit two-way traffic at all times.

This report pertains to the design and construction of the proposed culvert replacement and associated bedding/backfill zones.

The subsurface stratigraphy revealed in the boreholes drilled at the site comprised a surficial topsoil and/or fill underlain by cohesive clayey silt till (very stiff to approximate elevation 346 and stiff with firm zones further down) overlying cohesionless sandy gravel. Ground water was measured in one borehole at elevation 345.8. Flowing artesian ground water conditions encountered in another borehole at a depth of 15.8 m will not influence the progress of work for this project. However, the structures for temporary road protection should be installed at least 1 m above the level where the artesian conditions were encountered.



A list of MTO documents used in subsequent sections of the report is given in Table 1 for ease of reference. All elevations in this report are expressed in meters.

2. FOUNDATIONS

The invert of the existing culvert is indicated to be at elevation 349.4 (ref.: Plate 166-788/37-0 of 'Highway 7 & 8. Preliminary Design' drawings). It is assumed that the new culvert will be constructed at the same level. The design subgrade level of the granular bedding for the replacement culvert is interpreted to be near elevation 348.9. The subgrade material below this level revealed in all the boreholes comprised very stiff clayey silt till.

It is considered that the very stiff clayey silt till exposed in the boreholes at and below the design subgrade is capable of supporting the stress imposed by the embankment and culvert foundations.

The culvert foundations constructed on the very stiff clayey silt till should be designed using the following geotechnical resistance at ultimate and serviceability limit states (ULS and SLS) for the 5.5 m span box culvert:

Factored Geotechnical Resistance at ULS = 300 kPa
Geotechnical Resistance at SLS = 200 kPa

The resistance at SLS allows for 25 mm of settlement of the culvert founding medium. However, negligible settlements are expected since no additional loading is to be placed at the replacement culvert location as compared to the existing embankment loading.

For preparation of the new founding subgrade, the footings of the existing culvert should be removed and backfilled with concrete or unshrinkable fill. Alternatively, the existing footings may be left in place and trimmed to allow for installation of the replacement culvert.



The topsoil and/or fill revealed at the borehole locations should be excavated to expose the founding subgrade where temporary culvert extensions are required. Any other deleterious soils encountered below the subgrade level should be excavated prior to placement of the granular base below the culvert extensions and replaced with engineered fill.

Preparation of the subgrade for construction of the culvert replacement and temporary extensions should be performed and monitored in accordance with OPSS 902 and SP 902S01. This should include site review by qualified geotechnical personnel during preparation of the subgrade as well as during placement and compaction of the engineered fill, if required.

Engineered fill placed under the culvert to accommodate any variation in the level of the native surface and/or replace any deleterious soils extending below the design founding level should comprise Granular A material compacted to at least 95% of the target density with conformance to OPSS 501 and SP 105S10. The limit of the granular fill zone should extend sideways a minimum 0.3 m beyond the culvert base and down to the subgrade at 45° to the horizontal and be established by a site specific survey.

The geometry of the subgrade preparation, cover backfill and frost taper treatment for the culvert should be carried out in accordance with OPSD 803.010 and OPSS 422. The bedding material for a precast box culvert should comprise a minimum 150 mm thick layer of Granular A.

A frost penetration depth of 1.6 m should be employed.

Based on the road grade (elevation 352.2) and ground surface elevation at the toe of slope (elevation 350.7), the embankment fill height at the existing culvert location is assessed to be 1.5 m.

The depth of excavation for the temporary culvert extensions will be about 2 m beyond the toe of the existing embankment and less than 3.5 m within the existing embankment fill. The excavation for the culvert replacement will be about 3.5 m deep from the road level if the existing footings remain in place or up to 4.5 m if they are removed.



Where the excavation will extend into the existing embankment, road protection will necessitate bracing to support the cut slopes. It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of ground water into the excavation. Further comments in this regard are provided in subsequent sections of the report.

3. CULVERT BACKFILL

Backfill adjacent to the culvert should be placed in accordance with OPSD 803.010, OPSD 3121.150 and OPSS 422.

Backfill should be brought up simultaneously on each side of the culvert and operation of heavy equipment within 0.5 times the height of the culvert (each side) restricted to minimise the potential for movement and/or damage of the culvert due to the lateral earth pressure induced by compaction.

The culvert must be designed to support the stress imposed by the overlying fill as well as to resist the unbalanced lateral earth pressure and compaction pressure imposed by the backfill adjacent to the culvert walls.

The lateral earth and water pressure, p (kPa), should be computed using the equivalent fluid pressures presented in Section 6.9 of the Canadian Highway Bridge Design Code (CHBDC), CAN/CSA-S6-00, March 2001, or employing the following equation assuming a triangular pressure distribution:

$$p = K (\gamma h_1 + \gamma' h_2 + q) + \gamma_w h_2 + C_p$$

where p = lateral earth pressure (kPa)

K = lateral earth pressure coefficient

γ = unit weight of backfill material above design water level (kN/m³)

γ' = unit weight of submerged backfill material below design water level (kN/m³)

$$= \gamma - \gamma_w$$

γ_w = unit weight of water

$$= 9.8 \text{ kN/m}^3$$

h_1 = depth below final grade (m), above design water level

h_2 = depth below design water level (m)

q = any surcharge load (kPa)

C_p = compaction pressure (refer to clause 6.9.3 of CHBDC)



The following parameters are recommended for design:

PARAMETER	GRANULAR A	GRANULAR B TYPE II	EXCAVATED MATERIAL
Angle of Internal Friction, degrees	35	35	30
Unit Weight, kN/m ³	22.8	22.8	20.0
Coefficient of Active Earth Pressure (K_a)	0.27	0.27	0.33
Coefficient of Earth Pressure At Rest (K_o)	0.43	0.43	0.50
Coefficient of Passive Earth Pressure (K_p)	3.69	3.69	3.00

The design should consider both the maximum water level in the stream and the stabilised ground water level condition. The ground water level measured at the culvert location was at elevation 345.8, some 3 m below the design subgrade. The maximum stream water level will be dictated by flood flow conditions and should be defined by the project hydraulic engineer.

The coefficient of earth pressure at rest should be employed to design rigid and unyielding walls.

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

4. EXCAVATION AND GROUND WATER CONTROL

Excavation to the anticipated founding level of the replacement culvert and culvert extensions is expected to extend through the topsoil and fill into the clayey silt till. Excavation of the soils should be feasible using conventional equipment. According to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria, the in situ materials are classified as Type 3 soils necessitating temporary cut slopes to be inclined at 1H:1V (horizontal to vertical). The need to excavate flatter sideslopes if excessively soft/wet materials or concentrated seepage zones are encountered locally during construction should be considered.



It is anticipated that a suitable roadway protection scheme in accordance with SP 105S19 will be required to support the walls of the excavation and adjacent traffic lanes during construction. A performance level 1a system such as soldier piles and lagging with anchored tiebacks or rakers is recommended to prevent movement of the existing embankment. The contractor is responsible for preparing a detailed design for the road protection system.

The ground water level measured at the site was at elevation 345.8, some 3 m below the design subgrade. The flowing artesian water found at 15.8 m depth (elevation 334.9) in borehole D-1 is beyond the expected excavation depths and will not influence the progress of work for this project. The structures for the temporary road protection system(s) should be installed at least 1 m above the level of the flowing artesian aquifer to prevent potential flow of the ground water above grade.

It is anticipated that conventional sump pumping techniques will be sufficient to control seepage of ground water into the excavation.

It will be necessary to implement measures to control water flow in the stream. Conventional procedures such as draining and/or diversion of the stream should be sufficient. Observed water levels are subject to seasonal fluctuations and precipitation patterns.

It is recommended that the work be carried out during the dry summer months to minimise the amount of ground water inflow to be handled and the volume of surface water, if any, to be diverted from the construction area.

All construction work should be carried out in accordance with the Occupational Health and Safety Act and with local/MTO regulations.



5. EMBANKMENT FILL

The height of embankment at the culvert location and proposed embankment widening for temporary detours is about 1.5 m.

The anticipated subgrade for the new embankment and widening comprises clayey silt till (very stiff to elevation 346 and stiff with firm zones below). Topsoil was encountered in the borehole drilled beyond the toe of the existing embankment on the south side of the highway. The construction specifications for grading in OPSS 206 should be followed. In particular, the topsoil and other excessively loose, soft, organic or otherwise deleterious materials encountered in the ditch beyond the culvert inlet within the limits of the embankment fill should be subexcavated prior to fill placement. The benching of the earth slopes should follow the OPSD 208.010 procedures and geometry. The new embankment fill should be placed and compacted in accordance with OPSS 501 and SP 105S10. The material should comprise native soils compacted to at least 95% of the target density.

It is considered that the prepared subgrade soil is capable of adequately supporting the new 1.5 m high embankment of the proposed widening. Provided that the new fill is placed and compacted as specified, the estimated settlement of the detour embankment platform surface is assessed to be within 25 mm (5 to 10 mm from the settlement of the new fill and 15 to 20 mm from the settlement of the subsoil). The settlement is expected to be essentially complete within three months following fill placement.

The embankment side slopes should be inclined no steeper than 2H:1V. A vegetation cover or other measures should be established to control surface runoff and minimise erosion of the temporary and permanent embankment slopes.



6. EROSION CONTROL

The protective measures noted in the OPSD 800 series to deal with erosion (inlet/outlet treatment, headwalls, cut-off walls) are considered to be appropriate. The backfill should comprise OPSS Granular A or Granular B Type II. The cut-off walls should extend below the upper edge of the footings of the existing culvert (if they are not removed) and to a depth at least equal to the fluctuation of the water level at the culvert location to prevent flow below the culvert that could erode the bedding material as well as extend laterally to protect the granular material. The requirements of CHBDC clauses 1.10.5.6 and 1.10.11.6.5 should be applied.

Inlet protection in accordance with OPSS 511 and 1004 is recommended to prevent erosion adjacent to the culvert as well as scour that could undermine the culvert and/or embankment foundation. The actual design requirements (length and width of the aprons at the inlet of the culvert as well as the rock size, apron thickness and height of erosion protection on the embankment slope) will be dictated by stream hydraulics, stream configuration, the water level in the stream and should be established by a hydraulic engineer. A non-woven, Class II geotextile with an FOS of 75-150 μm , according to OPSS 1860, should be placed below the rip-rap to minimise the potential for erosion of fine particles from below the treatment.

All newly constructed embankment slopes and retained soils behind the head walls and wing walls (if provided) should be covered with topsoil and seeded (as per OPSS 570 and 572) as soon after grading as possible to prevent erosion. Where slopes are inclined at 2.5H:1V or steeper, the permanent slopes should be protected with erosion control blankets. Also, sod (as per OPSS 571) shall be placed where it currently exists with a view to aesthetics. Additional appropriate erosion control measures for the project should be assessed using the following erodibility K factor:

<u>SOIL TYPE</u>	<u>K FACTOR</u>
Clayey Silt Till	0.5



7. CLOSURE

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

Yours very truly

Peto MacCallum Ltd.

A handwritten signature in black ink, appearing to read "Grigory O. Degil", is positioned above the printed name.

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

A handwritten signature in blue ink, appearing to read "Carlos M.P. Nascimento", is positioned above the printed name.

Carlos M.P. Nascimento, P.Eng.
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A handwritten signature in blue ink, appearing to read "Brian R. Gray", is positioned above the printed name.

Brian R. Gray, MEng, P.Eng.
MTO Designated Contact
GD:mi





TABLE 1
LIST OF STANDARD SPECIFICATIONS REFERENCED IN REPORT

TITLE	DOCUMENT	DATE
Construction Specification for Grading	OPSS 206	November 2000
Construction Specification for Precast Reinforced Concrete Box Culverts and Box Culverts in Open Cut	OPSS 422	April 2004
Construction Specification for Compacting	OPSS 501	November 2005
Construction Specification for Rip-Rap, Rock Protection, and Granular Sheeting	OPSS 511	November 2004
Construction Specification for Topsoil	OPSS 570	August 1990
Construction Specification for Sodding	OPSS 571	November 2001
Construction Specification for Seed and Cover	OPSS 572	November 2003
Excavation and Backfilling of Structures	OPSS 902	November 2002
Material Specification for Aggregates - Miscellaneous	OPSS 1004	November 2005
Material Specification for Geotextiles	OPSS 1860	November 2004
Construction Specification for Compaction	SP 105S10	November 2004
Construction Specification for Protection Systems	SP 105S19	March 2005
Excavation and Backfilling of Structures	SP 902S01	September 2003
Benching of Earth Slopes	OPSD 208.010	November 2003
Backfill and Cover for Concrete Culverts	OPSD 803.010	November 1999
Walls, Retaining, Backfill, Minimum Granular Requirements	OPSD 3121.150	November 2005