

**FOUNDATION INVESTIGATION AND DESIGN REPORT  
FOR  
WIDENING OF BLenheim ROAD OVERPASS  
W.P. 72-00-00, SITE 23-123  
HIGHWAY 401  
TOWNSHIP OF BLANDFORD-BLENHEIM, ONTARIO**

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PML Ref: 02KF137A  
Geocres No. 40P8-127

July 2003

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DRAWING 1

**FOUNDATION INVESTIGATION REPORT**

for  
Widening of Blenheim Road Overpass  
W.P. 72-00-00, Site 23-123  
Highway 401  
Township of Blandford-Blenheim, Ontario

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**INTRODUCTION**

This report summarizes the results of the foundation investigation carried out for the proposed widening of the existing overpass at Blenheim Road (East Quarter Road) and Highway 401 in the Township of Blandford-Blenheim, Ontario. The investigation was conducted for Delcan Corporation on behalf of the Ontario Ministry of Transportation.

Highway 401 passes over Blenheim Road at approximate Station 23+188, Highway 401 chainage, in the Township of Blandford-Blenheim. The existing overpass comprises twin single span rigid framed structures with a span length of 12.7 m. The structures are 16.2 m wide each and spaced 4.9 m apart.

The report pertains to the proposed overpass structure widening and approaches within about 20 m of the abutments.

**SITE DESCRIPTION**

The site is situated at the intersection of the existing Highway 401 and Blenheim Road. The structure to be widened carries Highway 401 traffic over Blenheim Road. At the location of the overpass, Highway 401 runs in an approximate west-east direction. The existing approach fill embankments are 5 to 6 m high, decreasing to about 3.0 m away from the structure.

The site is located in the Township of Blandford-Blenheim in the County of Oxford (Southwestern Ontario), approximately 9 km east of the Drumbo Road (County Road 29) interchange along Highway 401. The surrounding area is primarily rural in nature, with active agricultural operations or vacant lands adjacent to Highway 401.

The study area is part of the Waterloo Hills (Waterloo Moraine) physiographic region. The surface is composed of sandy hills and moraines with sandy outwash soils occupying the hollows, the topography gently sloping to the southwest. In general, the surficial geology in the region is fairly uniform with predominant sand and silt deposits. Bedrock consists of dolostone of the Salina Formation and is expected to exist at a depth of about 35 m.

### **INVESTIGATION PROCEDURES**

The field work for this study was carried out during the period of October 24 to 31, 2002 and comprised four boreholes advanced to depths of 8.1 to 25.0 m, as summarized in the following table, at the locations indicated on Drawing 1 (Appendix B):

Location	Borehole No.	Depth, m
West Approach	123-1	9.6
West Abutment, Median	123-2	25.0
East Abutment, Median	123-3	21.3
East Approach	123-4	8.1

The locations of and ground surface elevations at the boreholes were established in the field by Peto MacCallum Ltd. The following survey control point (HCP) and temporary benchmark (TBM) were provided by Delcan Corporation and used for vertical reference:

HCP 284: Nail in ground at northwest corner of overpass on north side of Highway 401.  
Elevation 306.239 (geodetic).

TBM: Top of northwest corner of overpass on north side of Highway 401.  
Elevation 306.339 (assumed geodetic).

The boreholes were advanced using continuous flight hollow and solid stem augers, powered by a truck-mounted CME-75 drill rig, supplied and operated by a specialist drilling contractor, working under the full-time supervision of a member of our engineering staff.

Representative 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. Dynamic cone penetration tests were also carried out to supplement the strength data. In situ pocket penetrometer testing was performed to assess the shear strength of the cohesive soils.

The groundwater conditions in the boreholes were closely monitored in the course of the field work. Upon completion of drilling, three piezometers, each consisting of 19 mm PVC pipe slotted over the bottom 600 or 900 mm, were installed in boreholes 123-1 to 123-3 to monitor groundwater conditions. The annular space around the pipe was backfilled as illustrated on the respective borehole logs. The water levels in the piezometers were measured on October 31 and December 18, 2002. The remaining borehole 123-4 was backfilled with auger cuttings to the ground surface.

All of the recovered samples were returned to our laboratory for detailed visual examination, classification and routine moisture content determinations. Atterberg Limits tests and grain size distribution analyses were carried out on selected samples, their results being presented in Figures 1 to 4 (Appendix A) and on the Record of Borehole sheets (Appendix B).

### **SUMMARIZED 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 pocket penetrometer test results, groundwater observations and moisture content determinations. The results of laboratory Atterberg Limits tests and grain size distribution analyses conducted on samples submitted for laboratory testing are also shown on the borehole logs.

The borehole locations and stratigraphic profile prepared from the borehole data are presented on Drawing 1.

The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised a surficial granular fill underlain by native deposits of sand to sand and silt, containing discontinuous layers of silt and silt till. The road embankments at this location are about 5 and 6 m high on the south and north sides of Highway 401, respectively. The strata encountered are summarized below.

#### Fill

Fill was identified in all the boreholes. In general, the fill consisted of cohesionless sand and gravel; it comprised silty sand to fine sand below 1.4 m depth in borehole 123-1 and between 0.6 and 2.2 m depth in borehole 123-4. The thickness of the fill ranged from 4.1 to 4.5 m in the approach holes and 5.6 m in the abutment holes. The fill was typically compact to very dense, locally loose between 0.6 and 1.4 m depth in borehole 123-4 (SPT-"N" values ranged from 12 to 62, locally being 4). The moisture content ranged from 4 to 9% in the granular fill, and 8 to 14% in the sand fill.

The results of a grain size distribution analysis conducted on a sample of the granular fill are presented in Figure 2.

#### Clayey Silt/Silty Clay

Cohesive clayey silt and silty clay layers were contacted below the fill in borehole 123-4 at a depth of 4.1 m (elevation 301.6). The clayey silt layer was 1.5 m thick and judged to be non to slightly plastic. The underlying silty clay layer was 0.9 m thick and penetrated at 6.5 m depth (elevation 299.2).

### Fine Sand to Fine Sand and Silt

Underlying the fills (and clay at borehole 123-4) at depths of 4.5 to 6.5 m (elevation 299.2 to 301.7) was cohesionless sand to sand and silt of various thickness and granulometric composition. The sand was compact to very dense and had a moisture content of 4 to 23%. Standard penetration test 'N' values ranged from 10 to 54, generally in a range of 15 to 35. The results of dynamic cone testing carried out in this stratum at various depths gave resistance values of 16 to 211 blows per 0.3 m penetration.

The sand/silt graded to a silt, trace clay and sand, within a 1.5 m thick zone at 10.1 m depth in borehole 123-3. In the deep boreholes, the sand/silt was interrupted by a till layer at depths of 16.2 and 15.5 m, as detailed in the following section.

The results of grain size distribution analyses conducted on the sand/silt are presented in Figure 3. The sand/silt deposits were not penetrated upon termination of all the boreholes at depths of 8.1 to 25.0 m (elevation 281.3 to 297.6).

### Silt Till

A layer of silt till was revealed at 16.2 and 15.5 m depth (elevation 290.1 and 290.3) in boreholes 123-2 and 123-3. This unit was non to slightly plastic, clayey to sandy, and very stiff to hard/dense. Moisture contents ranged from 13 to 25%. A pocket penetrometer test conducted on the cohesive silt till in borehole 123-2 gave a value of undrained shear strength exceeding 160 kPa.

The results of the Atterberg Limits tests conducted on the till material are presented in Figure 1 (Appendix A). The silt till has liquid limits of 16 and 23 and plastic limits of 13 and 18, thus giving the plasticity index of 3 and 5. The results of grain size distribution analyses are shown in Figure 4 (Appendix A).

The till layer was 3.0 and 2.2 m thick in boreholes 123-2 and 123-3 respectively.



### Groundwater

Groundwater was observed in three boreholes during or upon completion of drilling. In borehole 123-1, water was detected at a depth of 7.0 m (elevation 299.2) in the process of augering and measured at 8.9 m depth (elevation 297.3) upon completion of drilling. Water was encountered in borehole 123-2 at depths of 5.8 (elevation 300.5) and 11.6 m (elevation 294.7) during augering. Water was measured in borehole 123-3 at 15.0 m depth (elevation 290.8) upon completion. No water was observed in borehole 123-4 in the course of the field work.

Upon completion of drilling, piezometers were installed in boreholes 123-1 to 123-3. Two sets of piezometer readings subsequently taken showed water levels to be at the following depths/elevations:

Date	Borehole 123-1		Borehole 123-2		Borehole 123-3	
	Depth (m)	Elevation	Depth (m)	Elevation	Depth (m)	Elevation
October 31, 2002	8.4	297.8	11.4	294.9	15.0	290.8
December 18, 2002	8.3	297.9	11.4	294.9	18.2	287.6

Groundwater levels may fluctuate subject to seasonal variations and precipitation patterns.

### CLOSURE

The field work was carried out under the supervision of Mr. M. Rapsey and Mr. F. Portela and direction of Mr. P. Cullen, B.Eng., P.Eng. The equipment was supplied by Geo-Environmental Drilling Inc.

The report was prepared by Mr. G.O. Degil, Ph.D., Senior Project Supervisor, and Mr. M.R. Anderson, M.Eng., P.Eng., Senior Foundation Engineer. It was reviewed by Mr. D.W. Kerr, M. Eng., P.Eng., Chief Foundation Engineer. Mr. B.R. Gray, M.Eng., P.Eng., President, carried out an independent review of the report.



Yours very truly

Peto MacCallum Ltd.

A handwritten signature in black ink, appearing to read "M.R. Anderson", followed by a horizontal line.

Murray R. Anderson, M.Eng., P.Eng.  
Senior Foundation Engineer



A handwritten signature in black ink, appearing to read "D.W. Kerr", followed by a horizontal line.

Dennis W. Kerr, M.Eng., P.Eng.  
Chief Foundation Engineer



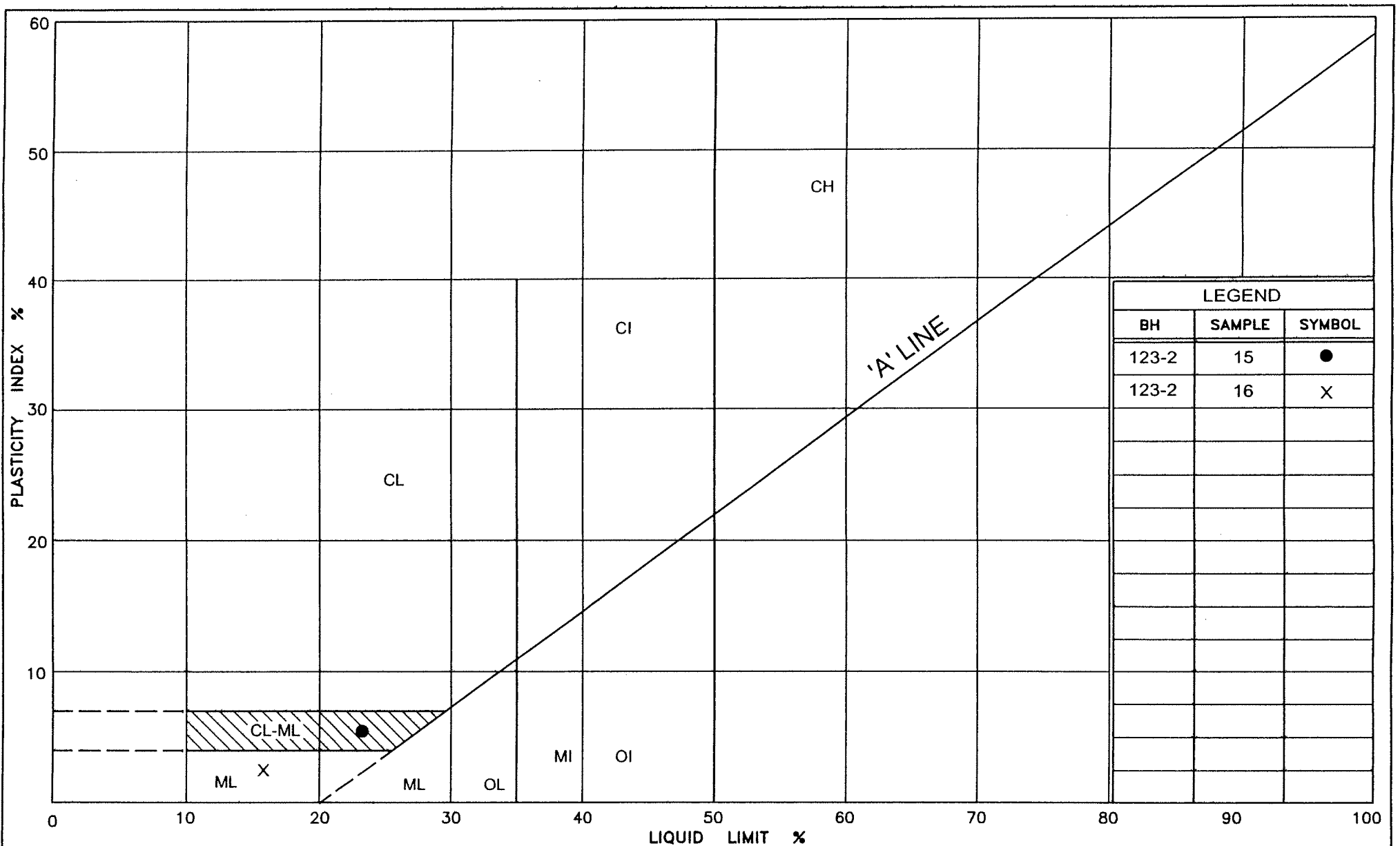
A handwritten signature in black ink, appearing to read "Brian R. Gray", followed by a horizontal line.

Brian R. Gray, M.Eng., P.Eng.  
President

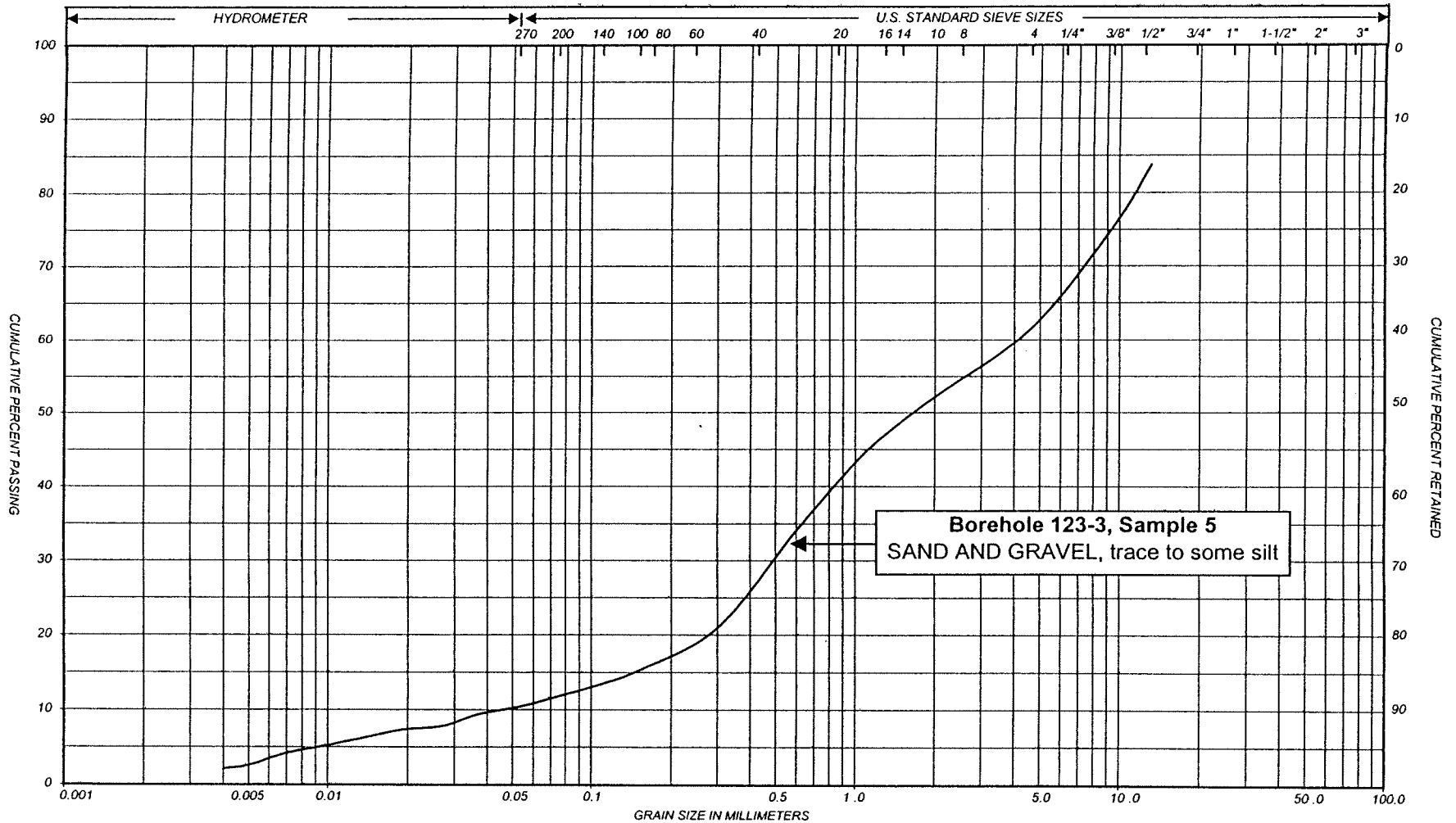
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## **APPENDIX A**

FIGURE 1 – PLASTICITY CHART  
FIGURES 2 TO 4 – GRAIN SIZE DISTRIBUTION CHART



# GRAIN SIZE DISTRIBUTION CHART



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

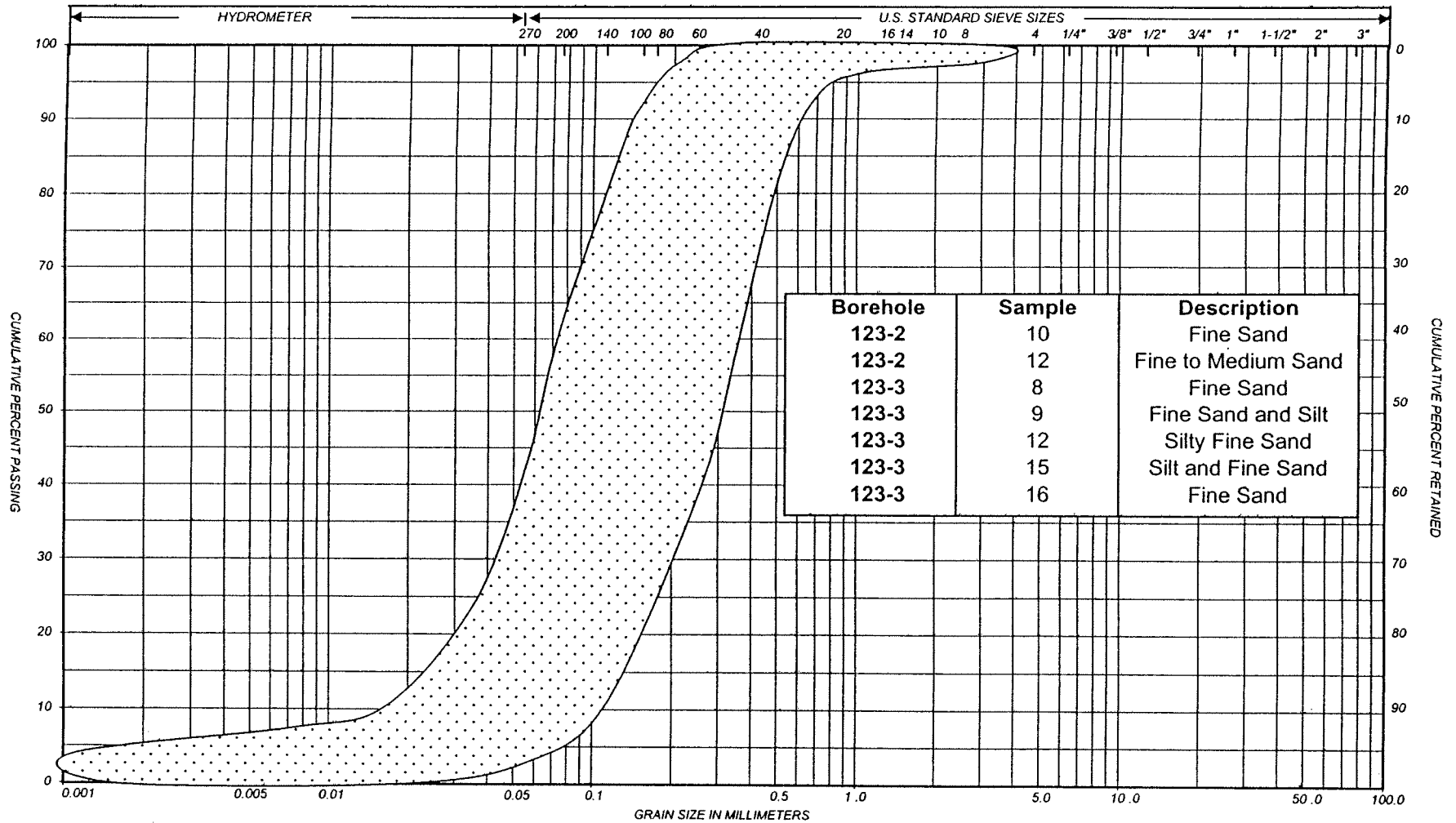
REMARKS \_\_\_\_\_

\_\_\_\_\_

\_\_\_\_\_

PML REF. 02KF137A  
G.W.P. 72-00-00  
FIGURE 3

# GRAIN SIZE DISTRIBUTION CHART



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				VERY FINE	FINE	MEDIUM	COARSE	GRAVEL								COBBLES	U.S. BUREAU	

REMARKS SAND, trace of silt, to SILT AND FINE SAND

## GRAIN SIZE DISTRIBUTION CHART

PML REF.

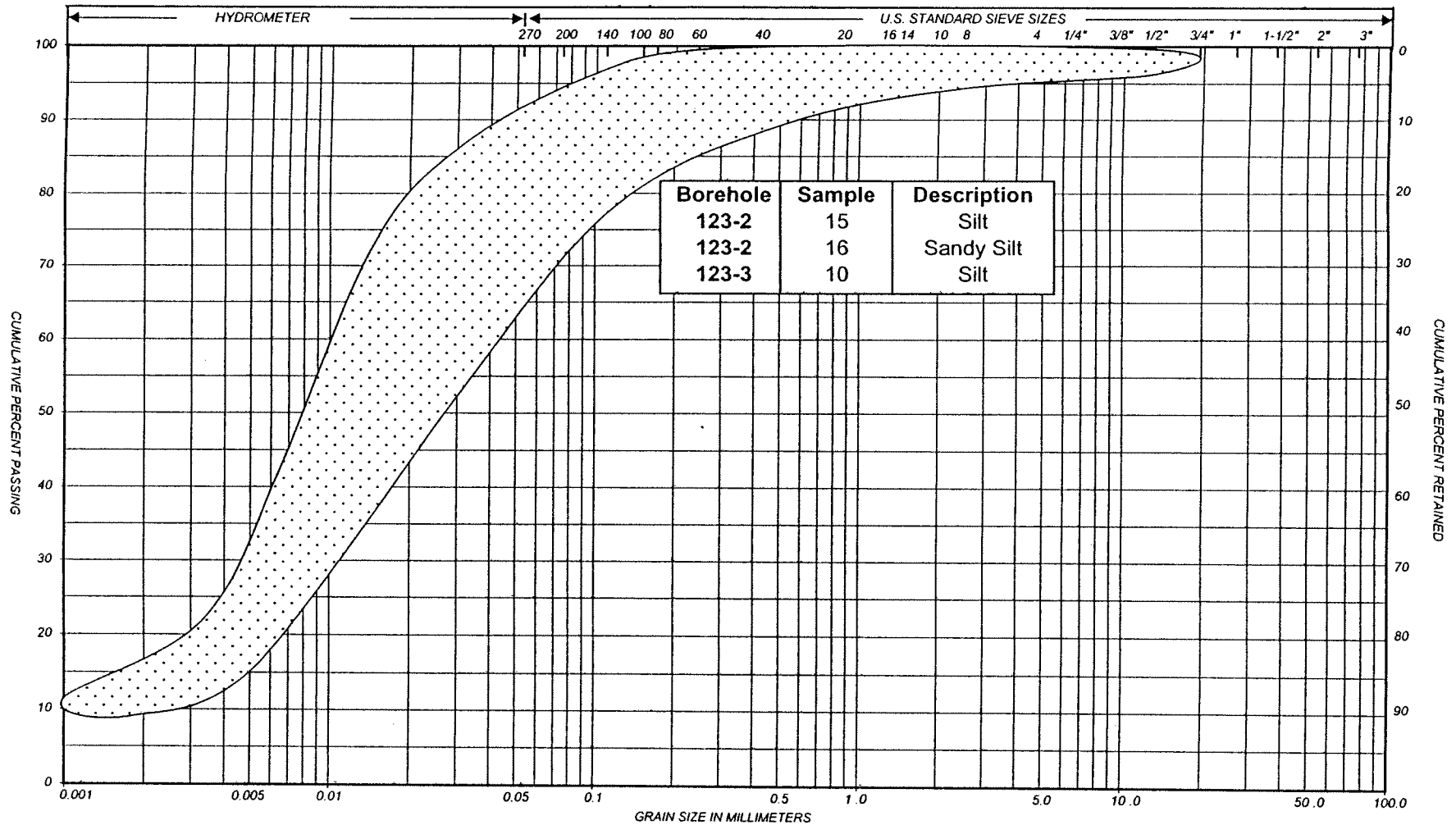
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72-00-00

FIGURE

4



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

REMARKS SILT, trace of clay to clayey, trace of sand to sandy, trace of gravel

## **APPENDIX B**

RECORD OF BOREHOLE SHEETS  
DRAWING 1



## 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 31mm 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 (31mm 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 - 30	30 - 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

SS	SPLIT SPOON	TP	THINWALL PISTON
WS	WASH SAMPLE	OS	OSTERBERG SAMPLE
ST	SLOTTED TUBE SAMPLE	RC	ROCK CORE
BS	BLOCK SAMPLE	PH	TW ADVANCED HYDRAULICALLY
CS	CHUNK SAMPLE	PM	TW ADVANCED MANUALLY
TW	THINWALL OPEN	FS	FOIL SAMPLE

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$u$	1	PORE PRESSURE RATIO
$\sigma$	kPa	TOTAL NORMAL STRESS
$\sigma'$	kPa	EFFECTIVE NORMAL STRESS
$\tau$	kPa	SHEAR STRESS
$\sigma_1, \sigma_2, \sigma_3$	kPa	PRINCIPAL STRESSES
$\epsilon$	%	LINEAR STRAIN
$\epsilon_1, \epsilon_2, \epsilon_3$	%	PRINCIPAL STRAINS
$E$	kPa	MODULUS OF LINEAR DEFORMATION
$G$	kPa	MODULUS OF SHEAR DEFORMATION
$\mu$	1	COEFFICIENT OF FRICTION

### MECHANICAL PROPERTIES OF SOIL

$m_v$	kPa <sup>-1</sup>	COEFFICIENT OF VOLUME CHANGE
$C_c$	1	COMPRESSION INDEX
$C_s$	1	SWELLING INDEX
$C_\alpha$	1	RATE OF SECONDARY CONSOLIDATION
$c_v$	m <sup>2</sup> /s	COEFFICIENT OF CONSOLIDATION
$H$	m	DRAINAGE PATH
$T_v$	1	TIME FACTOR
$U$	%	DEGREE OF CONSOLIDATION
$\sigma'_{vo}$	kPa	EFFECTIVE OVERBURDEN PRESSURE
$\sigma'_p$	kPa	PRECONSOLIDATION PRESSURE
$\tau_f$	kPa	SHEAR STRENGTH
$c'$	kPa	EFFECTIVE COHESION INTERCEPT
$\phi'$	-°	EFFECTIVE ANGLE OF INTERNAL FRICTION
$c_u$	kPa	APPARENT COHESION INTERCEPT
$\phi_u$	-°	APPARENT ANGLE OF INTERNAL FRICTION
$\tau_R$	kPa	RESIDUAL SHEAR STRENGTH
$\tau_r$	kPa	REMOULDED SHEAR STRENGTH
$S_t$	1	SENSITIVITY = $\frac{c_u}{\tau_r}$

### PHYSICAL PROPERTIES OF SOIL

$\rho_s$	kg/m <sup>3</sup>	DENSITY OF SOLID PARTICLES	$e$	1, %	VOID RATIO	$e_{min}$	1, %	VOID RATIO IN DENSEST STATE
$\gamma_s$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOLID PARTICLES	$n$	1, %	POROSITY	$I_D$	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
$\rho_w$	kg/m <sup>3</sup>	DENSITY OF WATER	$w$	1, %	WATER CONTENT	$D$	mm	GRAIN DIAMETER
$\gamma_w$	kN/m <sup>3</sup>	UNIT WEIGHT OF WATER	$S_r$	%	DEGREE OF SATURATION	$D_n$	mm	n PERCENT - DIAMETER
$\rho$	kg/m <sup>3</sup>	DENSITY OF SOIL	$w_L$	%	LIQUID LIMIT	$C_u$	1	UNIFORMITY COEFFICIENT
$\gamma$	kN/m <sup>3</sup>	UNIT WEIGHT OF SOIL	$w_p$	%	PLASTIC LIMIT	$h$	m	HYDRAULIC HEAD OR POTENTIAL
$\rho_d$	kg/m <sup>3</sup>	DENSITY OF DRY SOIL	$w_s$	%	SHRINKAGE LIMIT	$q$	m <sup>3</sup> /s	RATE OF DISCHARGE
$\gamma_d$	kN/m <sup>3</sup>	UNIT WEIGHT OF DRY SOIL	$I_p$	%	PLASTICITY INDEX = $w_L - w_p$	$v$	m/s	DISCHARGE VELOCITY
$\rho_{sat}$	kg/m <sup>3</sup>	DENSITY OF SATURATED SOIL	$I_L$	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	$i$	1	HYDRAULIC GRADIENT
$\gamma_{sat}$	kN/m <sup>3</sup>	UNIT WEIGHT OF SATURATED SOIL	$I_C$	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	$k$	m/s	HYDRAULIC CONDUCTIVITY
$\rho'$	kg/m <sup>3</sup>	DENSITY OF SUBMERGED SOIL	$e_{max}$	1, %	VOID RATIO IN LOOSEST STATE	$J$	kN/m <sup>2</sup>	SEEPAGE FORCE
$\gamma'$	kN/m <sup>3</sup>	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 123-1

1 of 1 METRIC

G.W.P. 72-00-00 LOCATION Co-ords. 4 793 461 N; 221 756 E ORIGINATED BY F.P.  
DIST 31 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.  
DATUM Geodetic DATE October 25, 2002 CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
306.2 0.0	Sand and gravel Dense to Very Dense Brown (Fill)		SS	1	30		306							
304.8 1.4	Silty sand, with gravel and topsoil inclusions Compact to Very Dense Brown to Dark Brown (Fill)		SS	2	55		305							
			SS	3	15		304							
			SS	4	12		303							
			SS	5	20		302							
			SS	6	54		301							
301.7 4.5	Fine sand, trace silt to silty Compact to Dense Moist to Wet		SS	7	19		300							
			SS	8	21		299							
			SS	9	17		298							
296.6 9.6	End of Borehole		SS	10	36		297							
<p>▼ Water level measured after drilling</p> <p>Piezometer Readings:</p> <p>Date Depth(m) Oct. 31/02 8.4 Dec. 18/02 8.3</p> <p>Borehole Backfill Legend:</p> <p>Native Backfill Bentonite Seal Filter Sand Screen</p>														<p>Water encountered at 7.0m</p> <p>Upon completion of augering, water at 8.9m</p>



RECORD OF BOREHOLE No 123-2

2 of 2 METRIC

G.W.P. 72-00-00 LOCATION Co-ords. 4 793 471 N; 221 768 E ORIGINATED BY F.P.  
DIST 31 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.  
DATUM Geodetic DATE October 24, 2002 CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	"N" VALUES			20 40 60 80 100	20 40 60 80 100					
305.3	Fine sand, trace silt to silty													
	Compact to Dense Brown		SS	14	25		291							
290.1	Wet (Cont'd)													
16.2	Silt, some clay to clayey, trace of sand to sandy, trace gravel						290							
	Very Stiff to Dense Brown to Grey (Till)		SS	15	24									1 9 73 17
							289							
			SS	16	45		288							5 24 57 14
287.1	Sand, trace of silt, trace gravel						287							
19.2	Very Dense Grey Wet		SS	17	54		286							
	thin layer of silty clay (till)						285							
	Compact Brown		SS	18	29		284							
281.3							283							
25.0							282							
	End of Borehole													
	<div>■ Penetrometer Test</div> <div>▼ Water level measured after drilling</div> <div>Piezometer Readings:</div> <div>Date      Depth(m)</div> <div>Oct. 31/02    11.4</div> <div>Dec. 18/02    11.4</div> <div>Borehole Backfill Legend:</div> <div> <div>Native Backfill</div> <div>Bentonite Seal</div> <div>Filter Sand</div> <div>Screen</div> </div>													

RECORD OF BOREHOLE No 123-3

1 of 2 METRIC

G.W.P. 72-00-00 LOCATION Co-ords. 4 793 490 N; 221 790 E ORIGINATED BY M.R.  
DIST 31 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.  
DATUM Geodetic DATE October 31, 2002 CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N-VALUES			20 40 60 80 100							
								SHEAR STRENGTH kPa							
								○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL × LAB VANE							
										WATER CONTENT (%)					
										20	40	60			
305.8 0.0	Fine to coarse sand and gravel, trace to some silt  Dense to Very Dense  Brown  Dry to Moist  (Fill)  occ. cobbles		SS	1	36		305								
			SS	2	54		304								
			SS	3	62		303								
			SS	4	32		302								
			SS	5	38		301								
			SS	6	47		300								
300.2 5.6	Fine sand, trace silt, to fine sand and silt  Compact to Dense  Brown  Dry		SS	7	21		299								
			SS	8	31		298								
			SS	9	31		297								
295.7 10.1	Silt, trace of clay and fine sand, non-plastic  Dense  Brown  Moist		SS	10	41		296								
			SS	11	21		295								
294.2 11.6	Silty fine sand, trace of clay  Compact  Brown  Saturated		SS	12	10		294								
							293								
							292								
	Cont'd						291								

ON\_MOT\_ONE DECIMAL 02KF0137A.GPJ ON\_MOT.GDT 2003 03 03

+7, X 5. Numbers refer to  
Sensitivity 20  
15-5 (%) STRAIN AT FAILURE  
10

RECORD OF BOREHOLE No 123-3

2 of 2 METRIC

G.W.P. 72-00-00 LOCATION Co-ords. 4 793 490 N; 221 790 E ORIGINATED BY M.R.  
DIST 31 HWY 401 BOREHOLE TYPE Continuous Flight Hollow Stem Augers COMPILED BY G.D.  
DATUM Geodetic DATE October 31, 2002 CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	T <sub>N</sub> VALUES								
305.8													
290.3 15.5	Clayey silt, trace to some sand, trace gravel, occ. layers of silt, trace clay  Very Stiff to Hard  Brown D.T.P.L. (Till)		SS	13	20		290						
			SS	14	48		289						
288.1 17.7	Silt and fine sand to fine sand, trace silt  Dense to Compact  Grey Saturated		SS	15	32		288						0 38 62 0
			SS	16	18		286						0 95 5 0
284.5 21.4	End of Borehole						285						
	<p>▼ Water level measured after drilling</p> <p><u>Piezometer Readings:</u></p> <p>Date Oct. 31/02 Depth(m) 15.0 Dec. 18/02 18.2</p>												

RECORD OF BOREHOLE No 123-4

1 of 1

METRIC

G.W.P. 72-00-00 LOCATION Co-ords. 4 793 507 N; 221 810 E ORIGINATED BY M.R.  
DIST 31 HWY 401 BOREHOLE TYPE Continuous Flight Solid Stem Augers COMPILED BY G.D.  
DATUM Geodetic DATE October 31, 2002 CHECKED BY M.R.A.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%)  GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	N° VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								○ UNCONFINED    + FIELD VANE ● QUICK TRIAXIAL    × LAB VANE										
305.7 0.0	Sand and gravel, some silt																	
305.1 0.6	Brown Moist (Fill) Fine sand, some silt to silty Loose to Compact		SS	1	4													
	Brown Moist (Fill) to Dry	SS	2	17														
303.5 2.2	Sand and gravel, trace to some silt Compact		SS	3	26													
	Brown Moist (Fill) to Dry		SS	4	25													
301.6 4.1	Clayey silt, trace of sand, non- to slightly plastic Stiff to Very Stiff		SS	5	16													
	Brown Moist		SS	6	17													
300.1 5.6	Silty clay, low to medium plastic, W.T.P.L., with thin layers of silt Stiff																	
299.2 6.5	Brown Fine sand and silt Dense		SS	7	22													
	Brown Dry																	
297.6 8.1	End of Borehole		SS	8	44													
	Borehole dry on completion of drilling																	





**FOUNDATION DESIGN REPORT  
FOR  
WIDENING OF BLENHEIM ROAD OVERPASS  
W.P. 72-00-00, SITE 23-123  
HIGHWAY 401  
TOWNSHIP OF BLANDFORD-BLENHEIM, ONTARIO**

Distribution:

2 cc: Delcan Corporation for Distribution to Ministry of Transportation  
2 cc: Delcan Corporation  
1 cc: PML Hamilton  
1 cc: PML Kitchener  
1 cc: PML Toronto

PML Ref: 02KF137A  
Geocres No. 40P8-127

July 2003

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- FIGURE 1    -    LATERAL EARTH PRESSURE DISTRIBUTION:  
                       SINGLY-BRACED CUTS IN COHESIONLESS SOILS
- FIGURE 2    -    LATERAL EARTH PRESSURE DISTRIBUTION:  
                       MULTI-BRACED CUTS IN COHESIONLESS SOILS
- FIGURE 3    -    GENERAL RECOMMENDATIONS REGARDING  
                       UNDERPINNING OF FOUNDATIONS/UTILITIES  
                       LOCATED CLOSE TO EXCAVATION

**FOUNDATION DESIGN REPORT**  
for  
Widening of Blenheim Road Overpass  
W.P. 72-00-00, Site 23-123  
Highway 401  
Township of Blandford-Blenheim, Ontario

---

**INTRODUCTION**

This report provides geotechnical comments and recommendations regarding design and construction of foundations, abutments and approaches for the proposed widening of the existing overpass at Blenheim Road (East Quarter Road) and Highway 401 in the Township of Blandford-Blenheim, Ontario. The investigation was conducted for Delcan Corporation on behalf of the Ontario Ministry of Transportation.

The existing overpass comprises twin single span rigid framed structures with a span length of 12.7 m. The structures are 16.2 m wide each and spaced 4.9 m apart (ref. Draft Structural Planning Report 'Widening of the Blenheim Road (East Quarter Road) Overpass. Highway 401, Site 23-123', Appendix C of the Preliminary Design Report Appendices prepared by McCormick Rankin Corporation in May 2002). It is proposed to widen the overpass by filling the gap in the median.

Highway 401 passes over Blenheim Road at approximate Station 23+188, Highway 401 chainage, in the Township of Blandford-Blenheim. Road grade on Highway 401 at the overpass location is near elevation 306.2, and on Blenheim Road near elevation 300.2 (interpolated from existing grade shown on Plate 113, Appendix A of the Preliminary Design Report Appendices referred to above). The approaches to the overpass comprise fill embankments of about 5 and 6 m in height on the south and north sides of Highway 401, respectively. The existing abutments of the overpass are reported to be supported on shallow spread footings; previous contract drawings (Sheet 47, Contract 97-42) indicate the footings are founded some 2.5 m below Blenheim Road grade.

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The subsurface stratigraphy revealed in the boreholes drilled at the site generally comprised a surficial granular fill underlain by native deposits of compact to very dense sand to sand and silt, containing discontinuous layers of silt and silt till.

## **FOUNDATIONS**

### **Spread Footings**

Supporting the structure widenings on conventional spread footings founded in the native sand is considered to be feasible at this site. The new footings should be founded at the same level as the existing footings. Based on the existing contract drawings, it is assumed that the existing footings are founded near elevation 298.0.

For a 2.5 m wide strip footing founded in the compact sand deposit at the inferred design founding level (elevation 298.0), the geotechnical resistance at ultimate (factored) and serviceability limit states (ULS and SLS) is considered to be:

$$\begin{array}{ll} \text{Factored Geotechnical Resistance at ULS} & = 500 \text{ kPa} \\ \text{Geotechnical Resistance at SLS} & = 250 \text{ kPa} \end{array}$$

The existing foundation that supports the structure and retaining wall between the structures is reported to be a 1.5 m wide strip footing. Cognizant of the width of the existing footing, it is considered that the following geotechnical resistance is available:

$$\begin{array}{ll} \text{Factored Geotechnical Resistance at ULS} & = 400 \text{ kPa} \\ \text{Geotechnical Resistance at SLS} & = 200 \text{ kPa} \end{array}$$

The geotechnical resistance for inclined loads should be reduced in accordance with the requirements of clause 6.7.4 of the Canadian Highway Bridge Design Code (CHBDC).

The recommended resistance at SLS allows for 25 mm of total settlement; differential settlement is expected to be less than 75% of this value. A footing embedment depth of 2.0 m was assumed for computation of the ULS resistance.

Sliding would be resisted in part by the friction force developed between the underside of footing and the native sand. An unfactored friction factor of 0.4 is recommended for footings on sand.

All footings subject to frost action should be provided with the normal 1.2 m of earth cover or equivalent thermal insulation. A 25 mm thick layer of polystyrene insulation is thermally equivalent to 600 mm of soil cover.

Prior to placement of structural concrete, all foundation excavations should be examined by qualified geotechnical personnel to verify the competency of the founding surface.

### **Deep Foundations**

Supporting the structure widenings on deep foundations is not recommended at this site considering the anticipated foundation loads and the potential for incompatible settlement behaviour between the new and existing portions of the structure.

## **ABUTMENT WALLS**

The abutment walls should be designed to resist the unbalanced horizontal earth pressure imposed by the backfill adjacent to the wall. The lateral earth pressure,  $p$ , may be computed using the equivalent fluid pressures presented in Section 6.9 of the CHBDC (CAN/CSA-S6-00) or employing the following equation, assuming a triangular pressure distribution:

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

where  $K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of free-draining  
granular material ( $\text{kN/m}^3$ )

$h$  = depth below final grade (m)

$q$  = surcharge load (kPa) if present

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

	<u>Granular "A"</u>	<u>Granular "B"</u>
Angle of Internal Friction, degrees	35	32
Unit weight, $\text{kN/m}^3$	22.8	21.2
Coefficient of Active Earth Pressure $K_a$	0.27	0.31
Coefficient of Earth Pressure At Rest $K_o$	0.43	0.47
Coefficient of Passive Earth Pressure $K_p$	3.69	3.25

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.

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 geotextile 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.

Use of a retained soil system (RSS) is not likely to be compatible with the existing wall design.

### **APPROACH EMBANKMENTS**

Backfilling adjacent to the structure should be carried out in conformance with Ontario Provincial Standards specifications for granular backfill (OPSD 3501.00).

The proposed widening will be carried out within the confined area between the twin structures. Therefore, the embankment slopes will not be impacted by the widening. Settlement due to placement of additional fill within the median is expected to be negligible ( $\pm 5$  mm).

### **EXCAVATION AND GROUNDWATER CONTROL**

Excavation for construction of footings is expected to extend through the embankment fill and some 2.0 m into the native sand. Based on the inferred design founding elevation of 298.0, the depth of excavation is expected to be about 8.0 m. Cognizant of the fill present at the site, the materials are classified as Type 3 soils according to Occupational Health and Safety Act (Ontario Regulation 213/91) criteria. Temporary cut slopes inclined at 45° to the horizontal should generally be stable.

Flatter side slopes may be required if excessively soft/wet materials or concentrated seepage zones are encountered locally. In particular, seepage and sloughing of the wet silty sand layers encountered at about 6 m depth on the west side of the overpass should be anticipated.

It is anticipated that shoring will be required to support the walls of the excavation and adjacent traffic lanes during construction.

The magnitude and distribution of the lateral earth pressures acting on a braced excavation wall is dependent upon the support system used, the number of supports, the allowable movements and the construction sequence. The recommended design earth pressure distribution for singly and multi-braced walls, for the conditions that exist at the site, are presented in Figures 1 and 2 respectively. Recommendations concerning design and construction of the braced excavation support systems are provided in the figures.

A soldier pile and lagging system may be considered. Provided the spacing between soldier piles is at least five pile diameters, the unfactored lateral passive resistance developed on the face of the soldier pile below the base of the excavation may be taken as the passive earth pressure developed over an equivalent wall area of width three times the pile diameter and depth of six times the pile diameter. A passive earth pressure coefficient  $K_p$  of 3.0 is recommended for this computation.

Additional lateral resistance could be provided by installing tiebacks anchored in the compact sand. The unfactored pull-out resistance (R) of anchors grouted in cohesionless material can be estimated using the following equation:

$$R = K_f \sigma'_z L_s A_s$$

where

$K_f$	=	anchorage coefficient
	=	0.8 for compact sand/silt
$\sigma'_z$	=	effective vertical stress at midpoint of anchor
	=	$\gamma' z$
$\gamma'$	=	effective unit weight of overburden soil
	=	20 kN/m <sup>3</sup> above groundwater level
	=	(elevation 295.0 for design purposes)
	=	10.2 kN/m <sup>3</sup> below groundwater level
$z$	=	depth to midpoint of anchor (m)
$L_s$	=	fixed length of anchor (m)
$A_s$	=	circumference of cross-section of
	=	fixed length of anchor (m <sup>2</sup> /m)



A resistance factor of 0.4 should be applied to the computed anchor capacity to determine the ULS resistance.

The ground surface adjacent to the excavation is expected to experience some inward movement and vertical settlement. The magnitude of movements adjacent to a braced cut can be limited by selection of an appropriate lateral earth pressure coefficient (see Figures 1 and 2) provided good quality workmanship and construction practice is employed. The anticipated magnitude of movements is as follows:

<u>Movement (% of Excavation Depth)</u>	
Lateral Movement	
Braced Excavation	0.2
Anchored Wall	0.1
Vertical Movement	0.05

Construction procedures should be specifically suited to limit any consequent settlement of the pavement subgrade behind the excavation face.

Foundations of heavily loaded/settlement sensitive structures and/or utilities, if located within close proximity to the excavation, may require underpinning to preserve the integrity of these structures. Further comments and general recommendations in this regard are provided in Figure 3.

Groundwater was measured at depths of 8.3 to 18.2 m below existing grade in the piezometers installed in three boreholes. The measured water levels are below the anticipated excavation depth. It is anticipated that any groundwater seepage or surface water that enters the excavation will be handled by conventional sump pumping techniques.

A wet silty sand layer was identified at about 6 m depth on the west side of Blenheim Road. The moisture condition in this unit probably reflects partially perched water draining through the overlying granular fill. Some sloughing and seepage should be anticipated at this level.

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

**CLOSURE**

The report was prepared by Mr. G.O. Degil, Ph.D., Senior Project Supervisor, and Mr. M.R. Anderson, M.Eng., P.Eng., Senior Foundation Engineer. It was reviewed by Mr. D.W. Kerr, M.Eng., P.Eng., Chief Foundation Engineer. Mr. B.R. Gray, M.Eng., P.Eng., President, carried out an independent review of the report.



Yours very truly

Peto MacCallum Ltd.

A handwritten signature in black ink, appearing to read "M. R. Anderson", followed by a horizontal line.

Murray R. Anderson, M.Eng., P.Eng  
Senior Foundation Engineer



A handwritten signature in black ink, appearing to read "D. W. Kerr", followed by a horizontal line.

Dennis W. Kerr, M.Eng., P.Eng  
Chief Foundation Engineer



A handwritten signature in black ink, appearing to read "Brian R. Gray", followed by a horizontal line.

Brian R. Gray, M.Eng., P.Eng.  
President

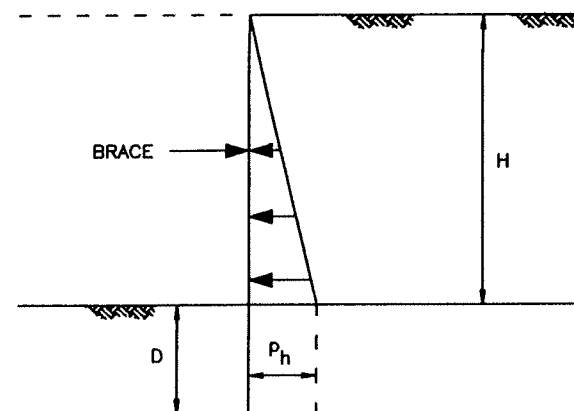
GD:lad

## **APPENDICES**

- FIGURE 1    -    LATERAL EARTH PRESSURE DISTRIBUTION:  
SINGLY-BRACED CUTS IN COHESIONLESS SOILS**
- FIGURE 2    -    LATERAL EARTH PRESSURE DISTRIBUTION:  
MULTI-BRACED CUTS IN COHESIONLESS SOILS**
- FIGURE 3    -    GENERAL RECOMMENDATIONS REGARDING  
UNDERPINNING OF FOUNDATIONS/UTILITIES  
LOCATED CLOSE TO EXCAVATION**

NOTES

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, the earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

EARTH PRESSURE DIAGRAM

$$p_h = \text{design lateral earth pressure} \\ p_h = K\gamma H$$

$$K = \text{lateral earth pressure coefficient}$$

$$\gamma = \text{unit weight of soil}$$

$$H = \text{depth of excavation}$$

$$D = \text{depth of embedment of soldier piles (if used).}$$

RECOMMENDED DESIGN PARAMETERS

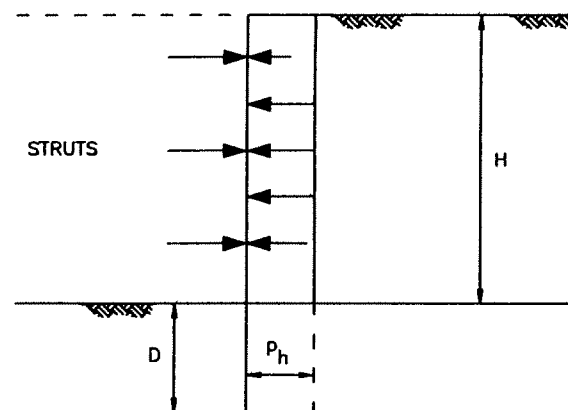
$$\gamma = 20.0 \text{ kN/m}^3$$

$$K = 0.35$$

**LATERAL EARTH PRESSURE DISTRIBUTION****SINGLY-BRACED CUTS IN COHESIONLESS SOILS**

**NOTES**

1. The actual magnitude and distribution of the horizontal earth pressures which will act on the bracing system are dependent upon the permissible lateral/vertical movements adjacent to the excavation, the soil type, groundwater conditions, drainage provisions, temporary/permanent surcharge loads, the type of bracing system adopted, weather conditions, quality of workmanship and length of time the excavation will be supported. Hence, the recommended pressure diagram and design parameters should be reviewed when construction details, schedule and type of support system are established.
2. Stability of base of excavation must be confirmed when bracing system design, excavation geometry and surcharge loads are established.
3. Earth pressure diagram is applicable to maximum depth of cut of 12m (40 ft.).
4. Structural components of bracing system should be confirmed adequate for each level of excavation.
5. If sheeting will not permit drainage, bracing system must be designed to resist water pressure.
6. Surcharge loads such as street/construction traffic, supported utilities, adjacent foundations, temporary stockpiles and other loads carried by bracing system are not included in earth pressure diagram.
7. Temporary surcharge loading should not be closer to the face of the excavation than half the depth of excavation unless accounted for in bracing design.
8. If settlement sensitive structures are located near the excavation, special measures should be undertaken to control settlements. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction.
9. Earth pressure diagram is applicable for relatively short construction periods. If excavation is to be open for long periods, monitoring of deformation is essential, the earth pressure diagram must be reviewed, and remedial works may be required.
10. Earth pressure diagram does not account for extended periods of exposure of the excavation to freezing temperatures.
11. Bracing system should be regularly examined for signs of distress.
12. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
13. This sheet should be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

**EARTH PRESSURE DIAGRAM**

$$p_h = \text{design lateral earth pressure} \\ = 0.65K\gamma H$$

$K$  = lateral earth pressure coefficient

$\gamma$  = unit weight of soil

$H$  = depth of excavation

$D$  = depth of embedment of soldier piles (if used).

**RECOMMENDED DESIGN PARAMETERS**

$$\gamma = 20.0 \text{ kN/m}^3$$

$K = 0.35$  (movement of retained soil acceptable)  
 $0.50$  (movement of adjacent structures/facilities unacceptable)

NOTES

1. The need to underpin existing footings/utilities is dependent upon soil type, proximity of the existing facility to the face of the excavation, loads imposed on the foundation and permissible movements.

ZONE A:

Foundations of relatively heavy and/or settlement sensitive structures/utilities located in Zone A generally require underpinning.

ZONE B:

Foundations of structures located within Zone B generally do not require underpinning. Consideration should be given to underpinning of settlement sensitive utilities or heavy foundation units located in this zone.

ZONE C:

Utilities and foundations located within Zone C do not normally require underpinning.

Underpinning of foundations located in Zones A and B should extend at least into Zone C.

2. As an alternative to underpinning, it may be possible to control movement of existing utilities and foundations by supporting the face of the excavation with bracing/tiebacks or a rigid (caisson) wall. Horizontal and vertical earth pressures imposed on the excavation wall by non-underpinned foundations must be considered in the design of the support system.
3. A condition survey should be conducted prior to construction and appropriate monitoring (surface and insitu) carried out during construction to monitor any movement which may occur.
4. All work should be carried out in accordance with the Occupational Health and Safety Act and local regulations. Good quality workmanship and construction practices are to be employed.
5. This sheet is to be read in conjunction with text of report for this project. Additional comments and recommendations concerning these general guidelines will be provided if required.

