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DIST. 1 REGION

W.P. No. 93-78-02

CONT. No. 84-82

W. O. No.

STR. SITE No. 13-104-194

HWY. No. 2

LOCATION McDougal Drain Bridge

No of PAGES -

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OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

**ENGINEERING MATERIALS OFFICE**  
**PAVEMENT & FOUNDATION DESIGN SECTION**

WP 93-78-02 DIST 1  
HWY 2 STR SITE 13-104-194  
McDougall Drain Bridge, Chatham

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# FOUNDATION INVESTIGATION REPORT

For

W.P. 93-78-02; Site 13-104-194

McDougall Drain Bridge, Chatham

Hwy. #2, District 1, Chatham

## INTRODUCTION:

This report summarizes the results of the foundation investigation required for the proposed replacement structure at this site.

The fieldwork was conducted during the period from 83 02 17-18 utilizing a continuous-flight auger machine equipped with 82 mm I.D. hollow-stem augers.

This work consisted of 2 sampled boreholes/dynamic cone penetration tests.

## SITE DESCRIPTION

The site is located at McDougall Drain on Hwy. 2, approximately 4.5 km northeast of the Hwy. 401/Hwy. 2 interchange. (Kent County, Conc. VI, Lots 3 & 4, Tilbury East Twp.).

Physiographically the site lies in the St. Clair Clay Plains, an area of low relief in which a till plain is generally covered by a thin deposit of lacustrine clay.

## SUBSURFACE CONDITIONS

### General

The Record of Borehole Sheets, (Appendix) illustrate the conditions at the borehole locations. The locations and elevations of the boreholes, and stratigraphical profiles based on the borehole data are shown on Drawing No. 937802-A.

At this site, approximately 34 m of silty clay of low to intermediate plasticity overlies the limestone bedrock.

### Fill Material

The fill material at this site is silty clay of low to intermediate plasticity containing some sand, traces of gravel and occasional layers of silty sand. This predominantly cohesive material varies in consistency from stiff to very stiff.

The fill at the abutments will extend from the surface to below the base of the existing footings (est. elev. 173.5± m). At the borehole locations the fill thickness ranges from 3.0 to 3.4 m.

Physical properties of the material, as determined from field tests and one laboratory test, are summarized below:

Natural Moisture Content (W)	20.0%
Liquid Limit ( $W_L$ )	39.5%
Plastic Limit ( $W_p$ )	20.0%

Figure 1 illustrates a typical grain size distribution for this material.

### Overburden

The native overburden at this site is silty clay of low plasticity containing some sand and traces of gravel. It extends to approximately elevation 144± m.

Above elevation 168± m the material did not fail during field vane shear testing indicating shear strengths in excess of 107 kPa. From the results of the field vane shear tests, the unconfined compression test and the standard penetration tests, it is estimated that the average shear strength of this portion of the deposit is in excess of 105 kPa. The consistency of the deposit ranges from stiff to very stiff. Occasional very small seams of silty sand were encountered within this layer.

Below elevation 168± the average shear strength is estimated to be in excess of 40 kPa while the consistency ranges from firm to stiff.

Physical properties of the material, as determined from field and laboratory tests, are summarized below:

	<u>Range</u>	<u>Average</u>	<u>Median</u>
Natural Moisture Content (W)	17.5-23.5%	20.0%	19.5%
Liquid Limit ( $W_L$ )	29.0-32.0%	30.8%	31.0%
Plastic Limit ( $W_p$ )	16.5-18.0%	17.0%	16.8%
Unit Weight ( ) (one test)	21.1 kN/m <sup>3</sup>	N/A	N/A
Shear Strength			
- field vane (undisturbed)	44 - 107+ kPa	N/A	N/A
- field vane (remolded)	32 - 107+ kPa	N/A	N/A
- unconfined compression	151 - 267 kPa	N/A	N/A

The shear strength tests indicate that the material has low sensitivity (1 to 2).

Figure 2 illustrates a typical grain size distribution for this material.

#### Bedrock

The bedrock in this area is soft calcareous shale with interbedded limestone. It is overlain by transitional zones grading from silty clay with shaly layers and occasional layers of silty sand to weathered shale. The bedrock elevation is approximately 144.0 m.

#### Groundwater

At the time of the field investigation, the groundwater elevation was 175.4 m, approximately the same level as McDougall Drain.

## DISCUSSION AND RECOMMENDATIONS

It is proposed to replace the existing single-span structure with a single-span bridge to carry Hwy. 2 over McDougall Drain at the existing grade.

Two alternatives are proposed. The alternative which leads to the least expensive design should be adopted.

### General Recommendations (Applicable to All Alternatives)

- Earth pressure acting on abutments and retaining walls should be computed as per Subsection 6.6.1.2.2 of the O.H.B.D.C. assuming a yielding foundation with  $K_a = 0.33$  and  $\gamma = 21.5 \text{ kN/m}^3$  for granular backfill.
- For frost protection, cover should be greater than 1.2 m.
- No stability problems are anticipated for embankments with slopes of 2:1 or flatter.
- The creek channel slopes should be protected from erosion by suitable rip rap or other protection.
- For all alternatives differential settlements should not exceed 25 mm.
- Dewatering is not anticipated to be a major problem because of the impermeable nature of the foundation soil.
- The minimum cover required for scour protection should be determined from hydrological data.

### ALTERNATIVE 1 - Spread Footings on Glacial Till

The structure may be supported on spread footings founded on the silty clay till.

For this alternative remove the old footings, all fill material and any loose or soft material beneath the proposed footing locations, and cover (within 18 hours of exposure) the foundation soil with a 15 cm pad of mass concrete.

The elevation of the base of the existing footings is estimated at elevation 173.5± m.

The following design values are recommended for spread footings at or below the recommended footing level:

- net safe bearing pressure = 200 kPa

and for purposes of the O.H.B.D.C.:

- Factored Bearing Capacity at U.L.S. = 300 kPa
- Bearing Capacity at S.L.S. Type II = 200 kPa

#### ALTERNATIVE 2 - Steel H-Piles Driven to Bedrock

The structure may be supported on steel H-piles equipped with reinforced tips and driven to bedrock. The bedrock elevations is estimated at elevation 144.0± m.

The following design values are recommended:

<u>Pile Type</u>	<u>Safe Capacity</u>
310 HP 79	1150 kN per pile

and, for the purposes of the O.H.B.D.C.:

<u>Pile Type</u>	<u>Factored Capacity at U.L.S.</u>	<u>Capacity at S.L.S. Type II</u>
310 HP 79	1600 kN per pile	1150 kN per pile

MISCELLANEOUS

The fieldwork for this project was carried out under the supervision of Mr. D. H. Dundas, Project Foundation Engineer. The report was written by Mr. Dundas, and reviewed by Mr. K. G. Selby, Senior Foundations Engineer. The equipment used was owned and operated by Arcost Soil Drilling Inc.



*D. H. Dundas*

D. H. Dundas, P. Eng.  
Project Foundation Engineer

*K. G. Selby*

K. G. Selby, P. Eng.  
Senior Foundations Engineer

## APPENDIX

# RECORD OF BOREHOLE No 1

METRIC

W P 93-78-02 LOCATION STA 20+690 8m RT of HWY 2 ORIGINATED BY DD  
 DIST 1 HWY 2 BOREHOLE TYPE HS AUGER & CONE TEST COMPILED BY DD  
 DATUM GEODETTIC DATE 83 02 17 CHECKED BY

SOIL PROFILE			SAMPLES		GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 10 20 30	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE						
177.2	GROUND SURFACE									
0.0	SILTY CLAY (CL to CI) SOME SAND TRACE GRAVEL OCC LAYERS OF SILTY SAND Stiff to Very Stiff (FILL)		1	SS	10	176				3 18 45 34
			2	SS	13					
			3	SS	12					
174.2			4	SS	24	174				
3.0			5	SS	18					
			6	SS	15					
			7	SS	13	172				
			8	SS	15					
			9	SS	14	170				
			10	SS	13					
	Stiff to Very Stiff Firm to Stiff		11	SS	14	168				1 20 44 35
			12	SS	13					3 18 45 34
			13	SS	12	166				
			14	SS	11	164				2 20 42 36
	SILTY CLAY (CL) SOME SAND TRACE GRAVEL (TILL)		15	SS	9	162				
			16	SS	9	160				
						158				
						156				
						154				
						152				
						150				
146.7						148				
30.5										

Cont

+3, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10



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# RECORD OF BOREHOLE No 1 Cont

METRIC

W P 93-78-02 LOCATION STA 20+690 8m RT @ HWY 2 ORIGINATED BY DD  
DIST 1 HWY 2 BOREHOLE TYPE HS AUGER & CONE TEST COMPILED BY DD  
DATUM GEODETTIC DATE 83 02 17 CHECKED BY

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 (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
146.7	Cont																
30.5	SILTY CLAY (CL)																
	PROBABLE OCC LAYERS OF SILTY SAND & SHALE																
144.6																	
32.6	SHALE BEDROCK																
143.4	Weathered		17	SS	160												
33.8	END OF BOREHOLE																

+3, x5: Numbers refer to  
Sensitivity

20  
15 → 5 (%) STRAIN AT FAILURE  
10



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## RECORD OF BOREHOLE No 2

METRIC

W P 93-78-02

LOCATION STA 20+704 6 m RT @ HWY 2

ORIGINATED BY DD

DIST 1 HWY 2

BOREHOLE TYPE H5 AUGER & CONE TEST

COMPILED BY DD

DATUM GEODETIC

DATE 83 02 17

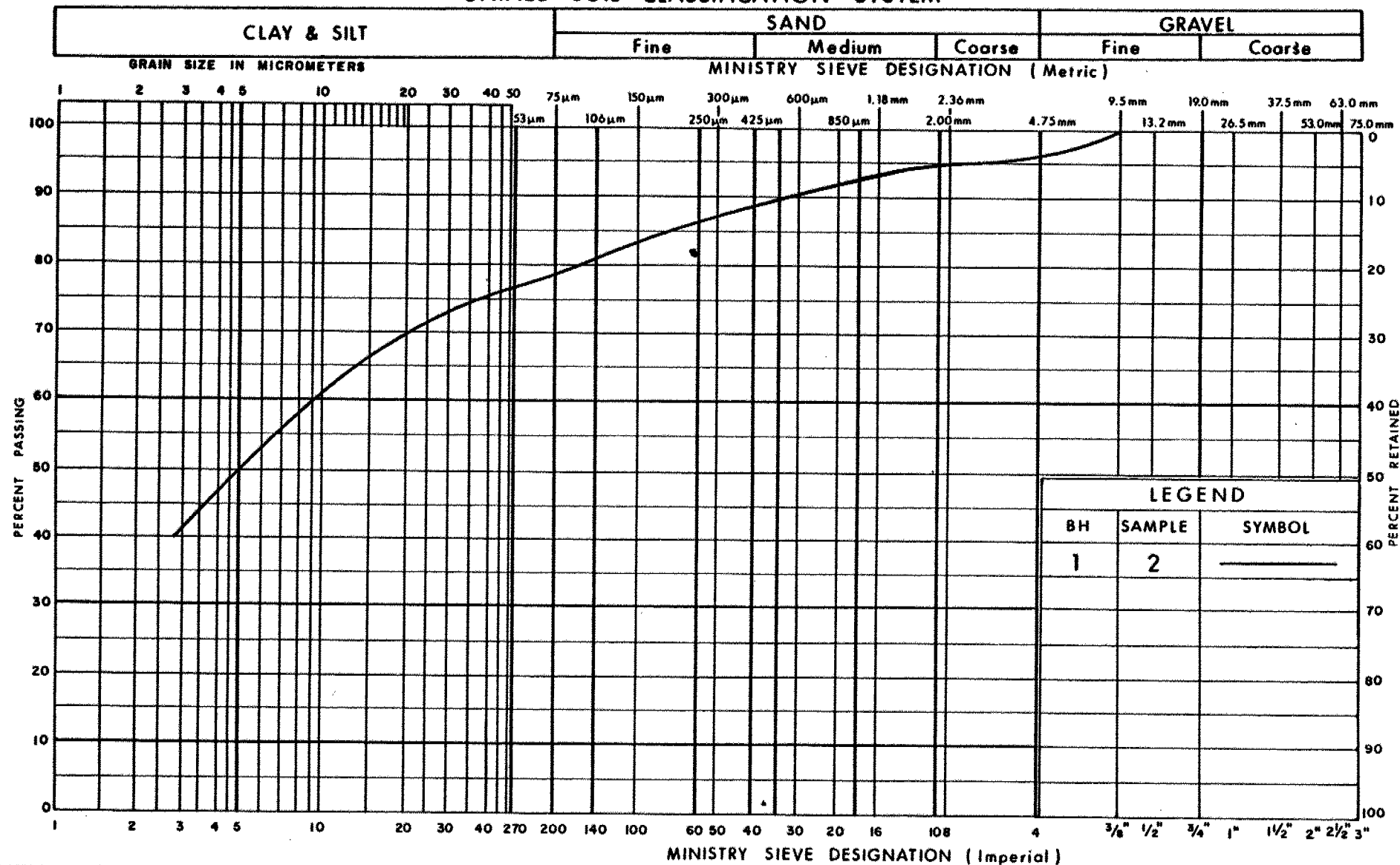
CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT 20 40 60 80 100 SHEAR STRENGTH kPa ○ UNCONFINED + FIELD VANE ● QUICK TRIAXIAL x LAB VANE 40 80 120 160 200	PLASTIC LIMIT W <sub>p</sub> NATURAL MOISTURE CONTENT W LIQUID LIMIT W <sub>L</sub> WATER CONTENT (%) 10 20 30	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						
178.0	GROUND SURFACE										
0.0	SILTY CLAY (CL to CI) SOME SAND TRACE GRAVEL OCC LAYERS OF SILTY SAND Stiff to Very Stiff (FILL)		1	SS	12		176				
			2	SS	11						
			3	SS	16						
174.6			4	SS	24						
3.4	SILTY CLAY (CL to CI) SOME SAND TRACE GRAVEL (TILL)		5	SS	28		174				
			6	TW	PH						
			7	SS	13		172				
			8	SS	12		170				
			9	SS	12		168				
	Stiff to Very Stiff Firm to Stiff		10	SS	11		166				
			11	SS	9		164				
			12	SS	8		162				
161.7			13	SS	10						
16.3	END OF BOREHOLE										* C <sub>u</sub> > 107 kPa

+3, x<sup>5</sup>: Numbers refer to  
Sensitivity

20  
15 5 (%) STRAIN AT FAILURE  
10

## UNIFIED SOIL CLASSIFICATION SYSTEM



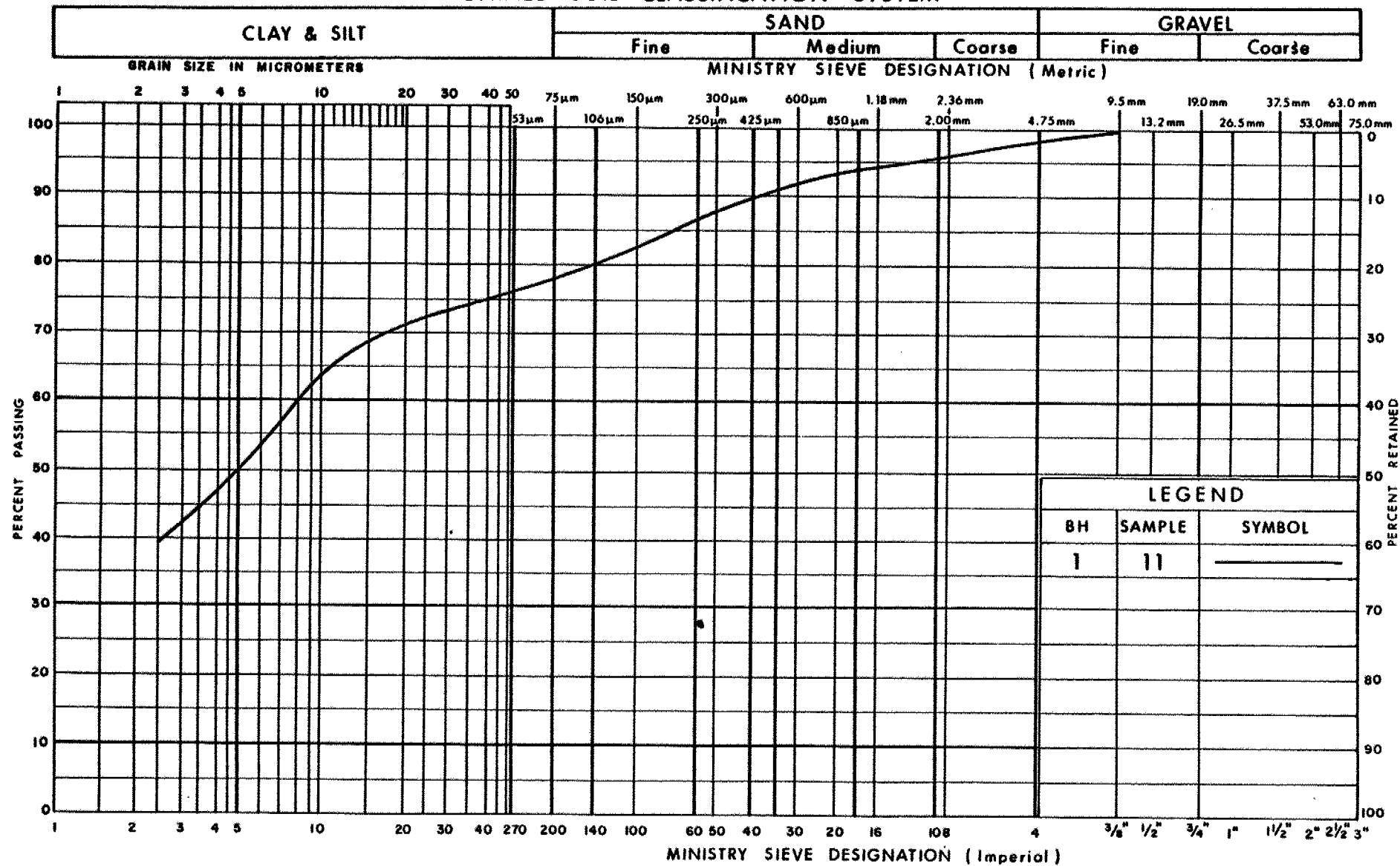
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**GRAIN SIZE DISTRIBUTION**  
**SILTY CLAY**  
**( Fill Material )**

FIG No 1

W P 93-78-02

## UNIFIED SOIL CLASSIFICATION SYSTEM



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GRAIN SIZE DISTRIBUTION  
SILTY CLAY  
(Till)

FIG No 2

W P 93-78-02

## 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:

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

### STRESS AND STRAIN

$u_w$	kPa	PORE WATER PRESSURE
$r_u$	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						

METRIC  
DIMENSIONS ARE IN METRES  
AND/OR MILLIMETRES UNLESS  
OTHERWISE SHOWN. NOTATIONS  
IN METRES - METRES.

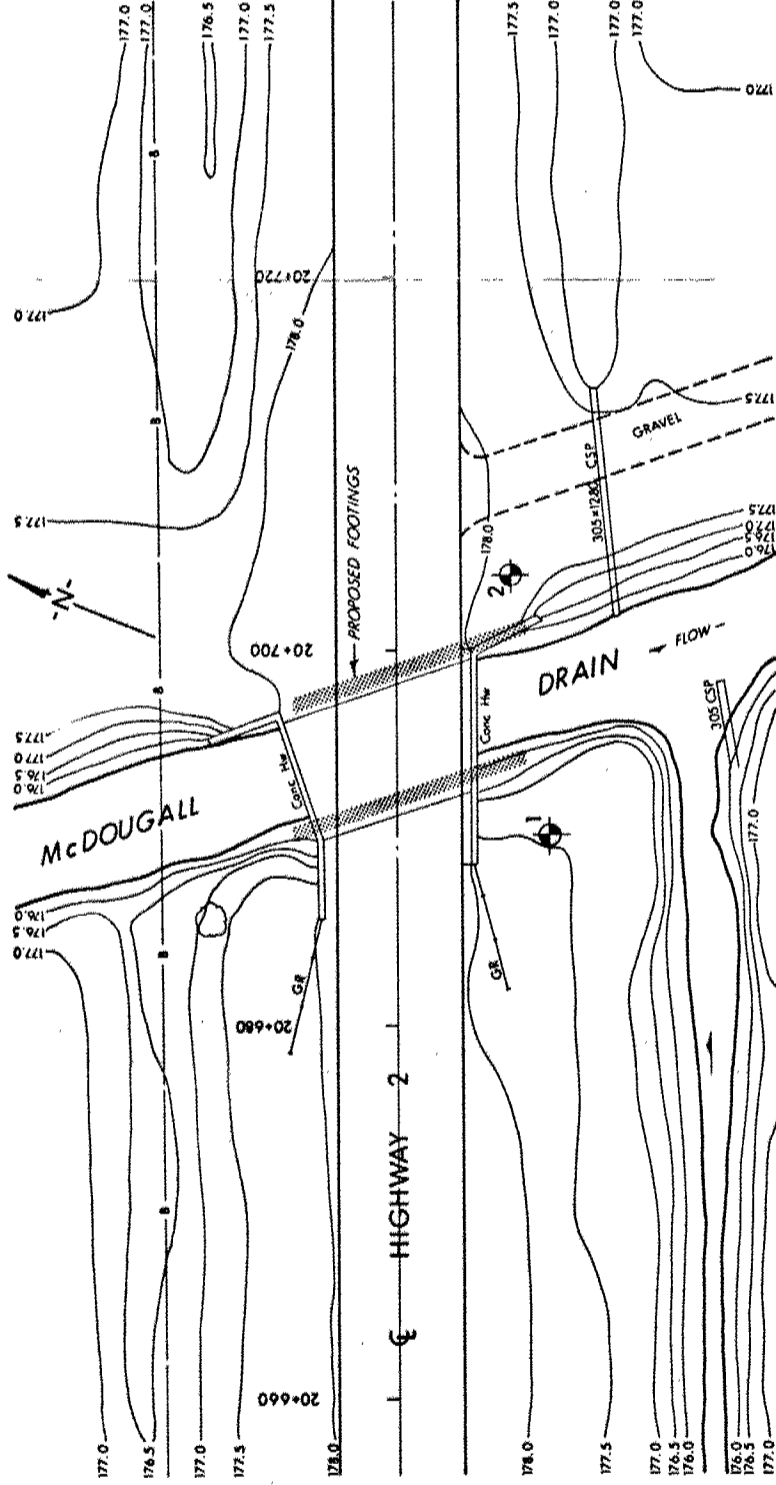
CONT No  
W P No 93-78-02

McDOUGALL DRAIN

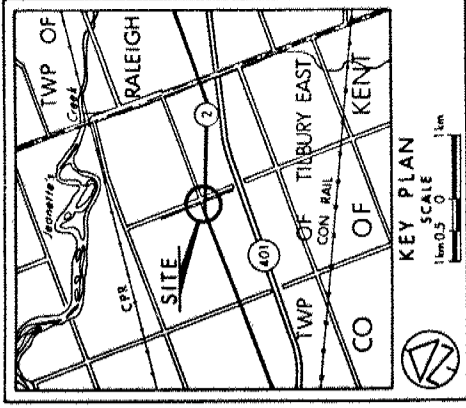
BORE HOLE LOCATIONS & SOIL STRATA



SHEET



PLAN  
SCALE  
4m 2 0 4m



KEY PLAN  
SCALE  
1:500

LEGEND

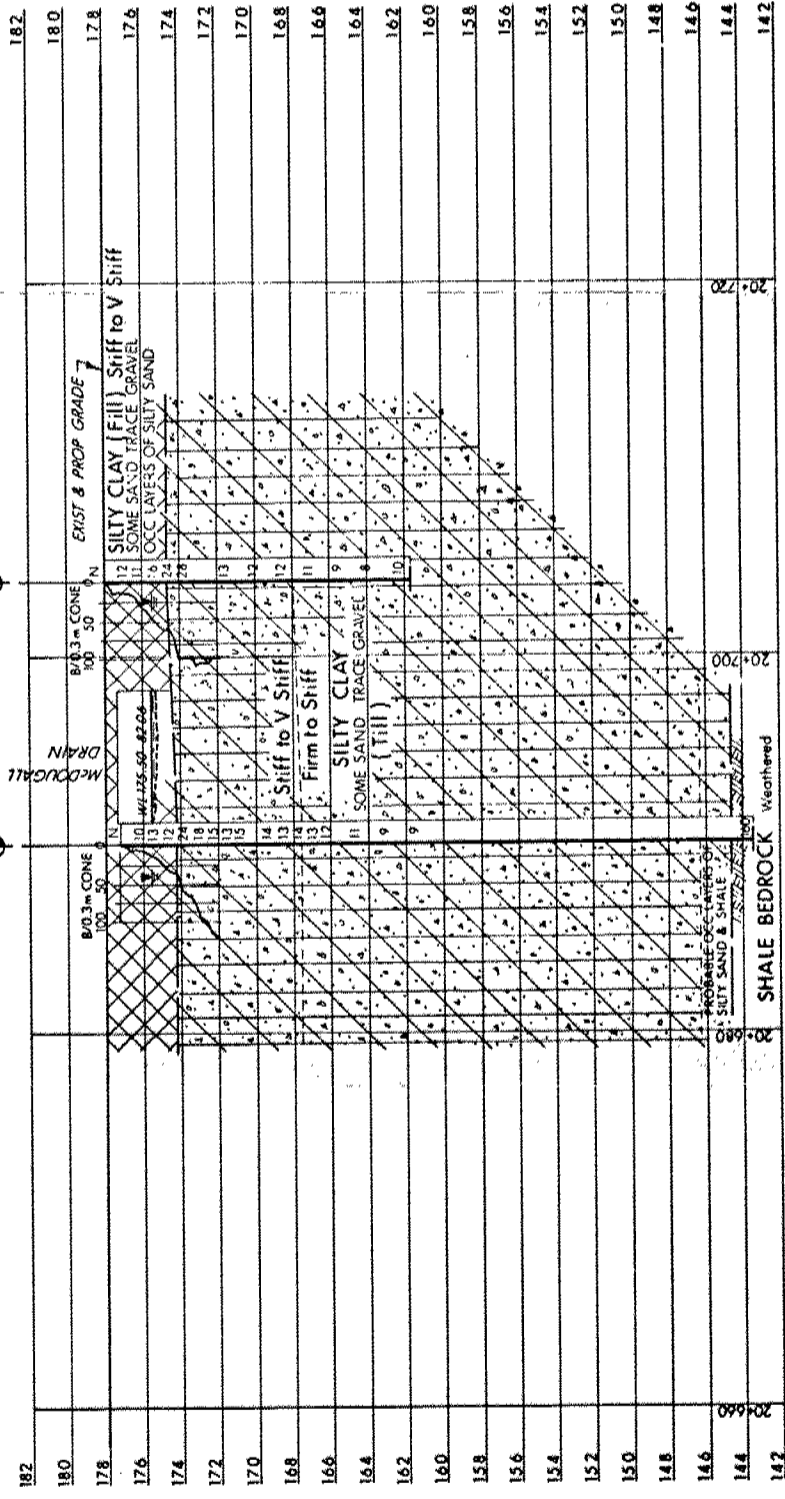
- Bore Hole
- ⊕ Dynamic Cone Penetration Test (Cone)
- ⊙ Bore Hole & Cone
- N Blows/0.3m (Std Pen Test, 475 J/blow)
- CONE Blows/0.3m (60° Cone, 475 J/blow)
- ⬇ W.L. at time of investigation 83.02

No	ELEVATION	STATION	OFFSET
1	177.2	20+690	8 m RT
2	178.0	20+704	6 m RT

NOTE:-  
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

NOTE: The complete foundation investigation and design report for this project and other related documents may be examined at the Engineering Materials Office, Downsview Information contained in this report and related documents is specifically excluded in accordance with the conditions of Section 102.2 of Form 100.

DATE	BY	DESCRIPTION
13-10-1988	1	DIST 1
13-10-1988	2	DIST 2
13-10-1988	3	DIST 3
13-10-1988	4	DIST 4
13-10-1988	5	DIST 5
13-10-1988	6	DIST 6
13-10-1988	7	DIST 7
13-10-1988	8	DIST 8
13-10-1988	9	DIST 9
13-10-1988	10	DIST 10
13-10-1988	11	DIST 11
13-10-1988	12	DIST 12
13-10-1988	13	DIST 13
13-10-1988	14	DIST 14
13-10-1988	15	DIST 15
13-10-1988	16	DIST 16
13-10-1988	17	DIST 17
13-10-1988	18	DIST 18
13-10-1988	19	DIST 19
13-10-1988	20	DIST 20
13-10-1988	21	DIST 21
13-10-1988	22	DIST 22
13-10-1988	23	DIST 23
13-10-1988	24	DIST 24
13-10-1988	25	DIST 25
13-10-1988	26	DIST 26
13-10-1988	27	DIST 27
13-10-1988	28	DIST 28
13-10-1988	29	DIST 29
13-10-1988	30	DIST 30
13-10-1988	31	DIST 31
13-10-1988	32	DIST 32
13-10-1988	33	DIST 33
13-10-1988	34	DIST 34
13-10-1988	35	DIST 35
13-10-1988	36	DIST 36
13-10-1988	37	DIST 37
13-10-1988	38	DIST 38
13-10-1988	39	DIST 39
13-10-1988	40	DIST 40
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13-10-1988	42	DIST 42
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13-10-1988	44	DIST 44
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13-10-1988	48	DIST 48
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13-10-1988	52	DIST 52
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13-10-1988	54	DIST 54
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13-10-1988	58	DIST 58
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13-10-1988	66	DIST 66
13-10-1988	67	DIST 67
13-10-1988	68	DIST 68
13-10-1988	69	DIST 69
13-10-1988	70	DIST 70
13-10-1988	71	DIST 71
13-10-1988	72	DIST 72
13-10-1988	73	DIST 73
13-10-1988	74	DIST 74
13-10-1988	75	DIST 75
13-10-1988	76	DIST 76
13-10-1988	77	DIST 77
13-10-1988	78	DIST 78
13-10-1988	79	DIST 79
13-10-1988	80	DIST 80
13-10-1988	81	DIST 81
13-10-1988	82	DIST 82
13-10-1988	83	DIST 83
13-10-1988	84	DIST 84
13-10-1988	85	DIST 85
13-10-1988	86	DIST 86
13-10-1988	87	DIST 87
13-10-1988	88	DIST 88
13-10-1988	89	DIST 89
13-10-1988	90	DIST 90
13-10-1988	91	DIST 91
13-10-1988	92	DIST 92
13-10-1988	93	DIST 93
13-10-1988	94	DIST 94
13-10-1988	95	DIST 95
13-10-1988	96	DIST 96
13-10-1988	97	DIST 97
13-10-1988	98	DIST 98
13-10-1988	99	DIST 99
13-10-1988	100	DIST 100



PROFILE - HWY 2

SCALE  
4m 2 0 4m

# memorandum



To: V.F. Boehnke  
Head, Structural Section  
Southwestern Region

Date: 1983 12 15

Atten: A.S. P. Ma

From: Foundation Design Section  
Room 315, Central Building

Re: Calculation of Passive Force

As per your request for information concerning the calculation of passive forces — it is our understanding that you propose to mobilize passive pressure in a retaining wall design.

Following is an example calculation (working stress system) of passive force for your proposed conditions, assuming saturated subsurface conditions, and given that front of retaining wall is immediately adjacent to cohesive subsoil. If fill is placed in front of wall, the passive force would change.

Conditions:

- subsoil is silty clay
- fill material is Granular B
- footing depth in front of wall is 1.35 m

$$P_p = \gamma \frac{H^2}{2} + 2cH$$

Where:

$P_p$  = passive force

$\gamma$  = unit weight of subsoil (silty clay)  
= 21 kN/m<sup>3</sup>

$H$  = depth of footing in front of wall = 1.35 m

$c$  = undrained shear strength of silty clay = 100 kPa  
(assumed)

$$P_p \sim 289 \text{ kN/m}$$

Please note that passive pressure should only be utilized in design if the following conditions are met.

- 1) the depth of footing in front of the wall ( $H$ ) is maintained (i.e. no scouring)
- 2) passive pressure is actually mobilized. Mobilization of passive pressure requires some movement of the subsoil. This may be achieved by ensuring the degree of backfill compaction required to mobilize passive pressure, immediately after the wall is poured. The mobilization of passive pressure would be more difficult to achieve with pre-cast or gabion structures.

In any case, the design passive force should be adjusted by a safety factor (minimum recommended S.F. = 3).

Enclosed are 2 pages from Introductory Soil Mechanics and Foundations by Sowers and Sowers, for your information.

If there are any questions, please do not hesitate to contact this office.

*D. H. Dundas*  
D.H. Dundas  
Foundations Engineer

DHD/mmj

Encl.

# memorandum



To: V.F. Boehnke  
Head, Structural Section  
Southwestern Region

Date: 1983 12 14

Atten: A.S.P. Ma

From: Foundation Design Section  
Room 315, Central Building

Re: McDougall Drain Bridge Replacement  
W.P. 93-78-02, Site 13-104-194  
Highway 2, District 1, Chatham

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This section has reviewed the revised preliminary drawing  
(C/L moved 1.7 m west, gabion retaining wall).

There are no comments.

DHD/mmj

*D.H. Dundas*  
D.H. Dundas  
Foundations Engineer

# memorandum



To:

Mr. J.L. Keen  
Design Engineer, S-W. Region,  
Structural Office  
3501 Dufferin Street  
Downsview, Ontario.

Date:

Re: Mc.Dougall Drain Bridge Replacement  
Highway 2, Site 13-104-194  
W.P. 93-78-02  
District 1 - Chatham



With reference to your comments, sent to us on 83-12-05, we have revised our General Arrangement Drawing 13-104-194-P1. The revisions are as follows:

- 1) The centreline of span was moved westward by 1.7 m to avoid cutting into the steep east bank. The span length remains unchanged.
- 2) The concrete retaining walls at four corners of the structure were replaced with gabion retaining walls. The gabion retaining walls are extended long enough (6.6 m) to allow proper channel improvement/widening to meet the hydraulic requirement. The founding level of the widening gabion wall was given by Foundation Design Section.

In order to place an 80 mm thick asphalt on the approach slab, as per your comment, the approach slab is to be 10 mm higher than the deck slab. However, the standard, DD-1209-A & B, shows that deck and approach slab are at level to facilitate the waterproofing membrane to be carried onto the approach slab by 300 mm. The step (10 mm high) would be a construction problem for extending waterproofing membrane onto the approach slab and also would trap moisture at the end of deck. Because of the above reasons, we would like to keep the same thickness (90 mm) of asphalt on both deck and approach slab. Since the grade over the structure is relatively flat, we would like to have your agreement to retain four deck drains as shown on the General Arrangement Drawing.

/2

# memorandum



To: Mr. V.F. Boehnke  
Head, Structural Section  
Southwestern (London) Region

Date: 83 02 28

Attn: Mr. A.P. Watt

From: Pavement & Foundation Design Section  
Room 315, Central Building  
Downsview

Re: Foundation Investigations:

W.P. 91-67-01, Site 13-104-193  
Baptiste Creek Bridge

and

W.P. 93-78-02, Site 13-104-194  
McDougall Drain Bridge

Hwy. 2, District #1 (Chatham)

Fieldwork for the above-noted projects has been completed. This memo contains recommendations pertaining to the design and construction of the foundations for the proposed structures. These recommendations are intended to be sufficient to allow the design of the structures to proceed. Our complete foundation investigation and design report will be submitted in the near future.

Two alternatives are proposed. The alternative which leads to the least expensive design should be adopted.

## General Recommendations (Applicable to All Alternatives)

- Earth pressure acting on abutments and retaining walls should be computed as per Subsection 6.6.1.2.2 of the O.H.B.D.C. assuming a yielding foundation with  $K_a = 0.33$  and  $\gamma = 21.5 \text{ kN/m}^3$  for granular backfill.
- For frost protection, cover should be greater than 1.2 m.
- No stability problems are anticipated for embankments with slopes of 2:1 or flatter.
- The creek channel slopes should be protected from erosion by suitable rip rap or other protection.

.../2

- For all alternatives differential settlements should not exceed 25 mm.
- Dewatering is not anticipated to be a major problem because of the impermeable nature of the foundation soil.
- The minimum cover required for scour protection should be determined from hydrological data.

#### ALTERNATIVE 1 - Spread Footings on Glacial Till

The structure may be supported on spread footings founded on the silty clay till.

For this alternative remove the old footings, all fill material and any loose or soft material beneath the proposed footing locations and cover (within 18 hours of exposure) the foundation soil with a 15 cm pad of mass concrete.

The elevation of the base of the existing footings is estimated at:

Location	Elevation
W.P. 91-67-01 Site 13-104-193 Baptiste Creek Bridge	174.0± m
W.P. 93-78-02 Site 13-104-194 McDougall Drain Bridge	173.5± m

The following design values are recommended for spread footings at or below the recommended footing level:

- net safe bearing pressure = 200 kPa  
and for purposes of the O.H.B.D.C.:
- Factored Bearing Capacity at U.L.S. = 300 kPa
- Bearing Capacity at S.L.S. Type "II" = 200 kPa

#### ALTERNATIVE 2 - Steel H-Piles Driven to Bedrock

The structure may be supported on steel H-piles equipped with reinforced tips and driven to bedrock. The bedrock elevation is estimated at elev. 144.0± m.

The following design values are recommended:

<u>Pile Type</u>	<u>Safe Capacity</u>
310 HP 79	1150 kN per pile

and for the purposes of the O.H.B.D.C.:

<u>Pile Type</u>	<u>Factored Capacity at U.L.S.</u>	<u>Capacity S.L.S. Type II</u>
310 HP 79	1600 kN per pile	1150 kN per pile

If there are any questions, please contact this Office.

*D. H. Dundas*  
D.H. Dundas, P. Eng.  
Project Foundations Engineer

DHD:syc

McDougall Sideroad Drain Bridge  
Hwy. 2  
Site No. ~~13-194~~  
13-104-194



LOOKING EAST



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