

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. SIC-144

DIST. 8 REGION

W.P. No. 112-86-01 (A)

CONT. No. 91-60

W. O. No.

STR. SITE No. 16-45

HWY. No. 15

LOCATION Morton Creek Bridge

No. of PAGES -

=====
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

FOUNDATION INVESTIGATION REPORT

CONTRACT NO. 91-60



Ministry of
Transportation

Ontario

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Note: For purposes of the contract, this report supersedes all other Foundation Reports prepared by, or for the Ministry in connection with the above mentioned project.

EXPLANATION OF TERMS USED IN REPORT.

2

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
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	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m ³	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m ³	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m ³	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m ³	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m ³	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m ³	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m ³ /s	RATE OF DISCHARGE
γ_d	kN/m ³	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m ³	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m ³	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m ³	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m ³	SEEPAGE FORCE
γ'	kN/m ³	UNIT WEIGHT OF SUBMERGED SOIL						

FOUNDATION INVESTIGATION REPORT
For
Morton Creek Bridge Replacement
W.P. 112-86-01 , Site No. 16-45
Highway 15, District 8, Kingston

INTRODUCTION

This report presents the results of the final foundation investigation carried out for the construction of a replacement bridge structure at Morton Creek, Highway 15, and for the proposed detour construction on the west side of the existing structure.

A preliminary foundation investigation was carried out at this site in 1988 and the results were reported in the foundation report dated 1989 07 14. The data of the preliminary investigation are also incorporated in this report.

SITE DESCRIPTION

The site is located just north of Morton on Highway 15 at Morton Creek, in the Township of South Crosby in the County of Leeds. The site is quite hilly. Rock ridges are all around this site and are as close as 50m to the creek (on the southwest side).

The physiological region is Leeds Knobs and Clay Flats and typically consists of knobs of granite and other Precambrian rocks with areas between filled with very weak calcareous clay left by Champlain Sea (Reference: Champman and Putnam, 'The Physiography of Southern Ontario; 3rd Edition, 1984)

INVESTIGATION PROCEDURES

Earlier in 1988, a preliminary foundation investigation was carried out at this site. The investigation consisted of drilling four sampled boreholes (Boreholes 1,2,3 and 4). The locations of these boreholes are shown on the enclosed drawing 1128601B-A. *

In the earlier investigation all boreholes were advanced with hollow stem augers except at Borehole 1 (the deepest borehole), where hollow stem augers were followed by BW casings. The boreholes were advanced to depths ranging from 9.6m (B.H. 4) to 61.4m (B.H. 1) below ground surface. All boreholes were terminated either in the overburden or at presumed bedrock.

* DWG NO 2 OF THE CONTRACT DWG'S

Except in Borehole 1, where silty sand was encountered at a depth of 60.7m below the ground surface all boreholes encountered an extensive stratum of clayey silt with frequent silty clay layers and occasional sand and silt seams.

The silty clay to clayey silt was found to be underlying the surficial granular fill and ranged from 8m to 58m in thickness.

The standard penetration N - value generally ranged from 0 to 5 blows per 0.3m. Generally, the N - value was 0 and 1.

The shear strength based on unconfined compression tests ranged from 15 to 50 kPa and based on field vanes it ranged from 12 to more than 96 kPa.

Only in Borehole 1 silty sand was encountered below the silty clay to clayey silt stratum at a depth of 60.7m below ground surface. The borehole was terminated after it penetrated 0.7m in this stratum. A split spoon sample was obtained from the surface of this stratum. The N - value 32 blows per 0.3m suggested that the stratum was dense at the surface.

The final (recent) foundation investigation was carried out between 90 03 12 and 90 03 27 and comprised of drilling five boreholes. Samples from overburden were obtained from four boreholes and one borehole (Borehole 13) was advanced to the bedrock and subsequently bedrock core was obtained at this location.

The boreholes for the final investigation are identified as B.H. 11, 13, 16, 17 and 18.

Borehole 11 was advanced to 61.6m (deepest borehole). Continuous flight hollow stem auger and N-casings were used to advance this borehole. Borehole 13 was advanced by driving 'N' casings to the bedrock without taking samples in the overburden. Further, B-casings were advanced to obtain a 3.3m bedrock core.

Borehole 16, 17 and 18 were advanced by continuous flight hollow stem auger to depths ranging from 7.7m to 16.2m. These boreholes were drilled to obtain information for the analyses of settlement due to detour construction.

Survey details were provided by the Eastern Region Survey and Plans Section. The elevations given in this report are geodetic.

The sampling program consisted of split spoon and thin wall samples (shelby tubes). The Standard Penetration Tests N-values were used for the assessment of the in-situ state of compaction of the non-cohesive material. Information obtained from the Field Vane tests were used to determine the shear strength and sensitivity of the cohesive material. The shear strength was used to determine the consistency of the cohesive material. These samples also provided material for identification purposes.

The laboratory testing program for representative samples consisted of:

- grain size analyses
- natural moisture content determinations
- Atterberg Limit determinations
- unit weight determinations
- consolidation tests, and
- unconfined compression tests.

SUBSURFACE CONDITIONS

The record of Borehole Sheets in the Appendix illustrate the subsurface conditions at the borehole locations. The locations and elevations of the boreholes, along with stratigraphical profiles based on the borehole data are shown on Drawing No. 1128601B-A. *

Underlying the fill material or topsoil the soil strata consists of silty clay to clayey silt material which is underlain by silty sand with gravel and occasional boulders. The silty sand layer overlies the bedrock.

On the northwest side of the bridge (B.H. 11) the granular fill was underlain by a 1.8m thick organic soil. Organic material was also encountered in Borehole 1 (preliminary investigation, 1988) which was drilled on the northeast side of the bridge. In the recent (final) investigation continuous samples were obtained to determine the thickness and nature of the organic material.

Underlying the fill material (up to 4m thick) or the surficial topsoil, all boreholes encountered silty clay to clayey silt material. The investigation revealed that the silty clay to clayey silt layer ranges in thickness from 5m to 36m. The thickness of this stratum on the south side of the bridge is about 27m. In the centre of the creek the deposit is estimated to extend 21m below the creek bed and on the immediately northwest side of the bridge it is 36m thick. It should be noted that in the earlier investigation (1988) the borehole on the northeast side of the bridge discovered a 58m thick clayey silt layer. The great variation in stratigraphical surface in a short distance suggests that the subsurface condition at this site is quite variable and that the bedrock is steeply dipping towards the north.

* DWG NO 2 OF THE CONTRACT DWG'S

In Borehole 13 (centre of the creek) and Borehole 11 (northwest side of the bridge) a silty sand layer was encountered which was underlying the silty clay to clayey silt stratum. At Borehole 13 (centre of the creek) the silty sand layer was found to be overlying the bedrock. The thickness of this layer at this location was estimated to be 16m. At Borehole 11 the full depth of this stratum was not determined. The borehole penetrated 22m into this material after which further penetration was not possible.

The overburden thickness on the south side of the bridge therefore, is expected to be 32m, in the centre 36m and on the north side of the bridge in excess of 62m.

It has been found out that the bedrock slopes down from the south side of the bridge to the north side and at any point north of the bridge its surface rises up. It is therefore, concluded that the deepest bedrock surface at this site is not in the centre of the bridge but on the north side of the bridge.

Based on the information obtained in Borehole 11 and the assumed soil stratigraphy in Borehole 13 (since no samples were obtained from the overburden) it is expected that the boundary line between cohesive and non-cohesive material slopes to the north (in correlation to the bedrock slope) and therefore the thicknesses of the cohesive and non cohesive material increase from the south side to the north side of the bridge.

Based on the information obtained in Boreholes 16, 17 and 18 it has been determined that the thickness of silty clay to clayey silt layer decreases on the north side of the bridge. This is because the bedrock surface rises up on the north side.

Following are the detailed descriptions of the soil strata encountered.

Fill Material/Topsoil

A non-cohesive fill material was encountered in Boreholes 11, 17 and 18. The fill material was placed for the construction of approaches and shoulders of Hwy 15. The approach fill adjacent to the northwest side of the bridge (B.H. 11) was 2.1m thick and was found to be overlying a layer of organic contained soil. The soil containing organics was about 1.8m thick and consisted of dark brown clayey silt with sand and wood fragments in the upper zone and was almost like a topsoil in the lower zone.

The Standard Penetration test 'N' value of 25 blows/0.3m in the non-cohesive fill suggest that this is in compact state. The N-value in the organic soil (clayey silt with wood fragments and decomposed organic) ranged from 2 to 5 blows/0.3m, which suggest it is in soft to firm state.

Clayey Silt

This cohesive material was encountered in all boreholes. This material was immediately underlying fill material or topsoil. This layer was fully penetrated in Boreholes 11, 13 and 18.

This material is a lacustrine deposit of clayey silt, with frequent silty clay layers and contained frequent sand and silt seams and traces of sand and gravel.

The thickness of this stratum as encountered in the boreholes ranged from 5m to 36m. The stratum was only 5m thick at Borehole 18 which was located at about 100m north of the existing bridge. The overall thickness of this stratum is quite variable. On the south side of the bridge the thickness is expected to be 27m, in the centre of the bridge 21m and on the northwest side of the bridge 36m.

Typical properties of the material, as determined by laboratory tests of representative samples from the boreholes, are summarized as follows:

	<u>Range</u>	<u>Average</u>
Natural Moisture Content (w)	22-45%	30%
Liquid Limit (wP)	18-51%	29%
Plastic Limit (wL)	12-23%	15%
Unit Weight (γ)	17.3-20.4 kN/m ³	19 kN/m ³

Figure 1 illustrates a typical plasticity envelope for this material.

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

The standard penetration test in this material recorded N-values ranging from 0 to 9 blows/0.3m but generally they ranged from 1 to 2 blows/0.3m. According to the field vane, the undrained shear strength of this material ranged from 21 kPa to more than 100 kPa which suggest that the consistency of the material is soft to very stiff, but generally is firm.

The field vane was used frequently to determine the in-situ undrained shear strength of the soil. Selected samples obtained in the Shelby tubes were tested in the laboratory for unconfined shear strength. The strength characteristics of this material are as follows:

	<u>Range</u>	<u>Average</u>
Shear Strength (unconfined compression test)	15-50 kPa	32 kPa
Shear Strength (based on Field Vane)	12-100+ kPa	49 kPa

Consolidation tests were carried out in the laboratory on selected samples. The results are as follows.

	<u>Range</u>	<u>Average</u>
Initial Void Ratio (e_0)	0.769-1.18	0.96
Preconsolidation (P_c)	111-320	229 kPa
Compression Index (C_c)	0.18-0.66	0.43

The consolidation tests carried out on selected samples show that the deposit under the existing Hwy 15 alignment and along the proposed detour alignment is over consolidated.

The results of the consolidation tests are shown on Figures 3 through 7.

Silty Sand

This non-cohesive stratum underlies the silty clay to clayey silt stratum. This layer was encountered in Boreholes 11 and 13.

In Borehole 11 this material was encountered at 40.1m below the ground surface (road level). In Borehole 13 no sample was obtained from this deposit. However, based on the penetration record of the N-casings it is assumed that the sand layer was encountered at 23m below the water level in the creek.

The thickness of this layer in the centre of the bridge is estimated to be 16 m. At Borehole 11 the full depth of this stratum was not determined although the borehole was advanced 22m in this material.

The silty sand stratum contained gravel and occasional boulders. Based on the split spoon tests in Borehole 11 the silty sand layer is in compact to very dense state. The compactness increases with increasing depth.

Due to difficulty in the drilling operation the split spoon was advanced with a 63.5kg hammer (in the same way used for standard penetration test) from depths 56m to 62m below the ground surface to estimate the density of this stratum. It is determined that N-values in excess of 100 blows/0.3m could be obtained in this stratum at depths 60m or more below the existing ground surface.

Bedrock

Bedrock was encountered at Borehole 13 at a depth of 38.7m below the water level in the creek. Bedrock was proved by coring from depths 38.7m to 42.0m below the water level in the creek.

The bedrock is described as Marble of Late Precambrian Age.

As stated in our preliminary report the bedrock elevations at this site are quite variable.

It has been determined that the bedrock surface slopes down from south to north at this site. On the north side of the bridge the borehole was terminated within the overburden at a depth of 62m. Therefore, the bedrock at that location is expected to be at a depth in excess of 62m below the ground surface.

The bedrock surface on the south side of the bridge is about 32m below the ground surface. In the centre of the bridge the bedrock is expected to be encountered at 36m below the creek bottom and on the north side of the bridge the bedrock is expected to be at a depth in excess of 62m below the ground surface.

Groundwater

The groundwater was measured in open boreholes and also in a piezometer installed in Borehole 16. The groundwater in Borehole 11 and the piezometer had stabilized and was found to be matching with the water level in the creek (Elev. 92.0m). Water levels in Boreholes 17 and 18 were not stabilized at the time of measurement.

It should be noted that groundwater levels are subject to seasonal fluctuations and may therefore change as the water level in the creek changes.

MISCELLANEOUS

The field work for this project was carried out under the supervision of Ken Ahmad.

The equipment used was owned and operated by Marathon Drilling Co. Ltd.

The report was written by Ken Ahmad, Foundation Engineer, reviewed by D. Dundas, Senior Foundation Engineer and approved by M. Devata, Chief Foundation Engineer.

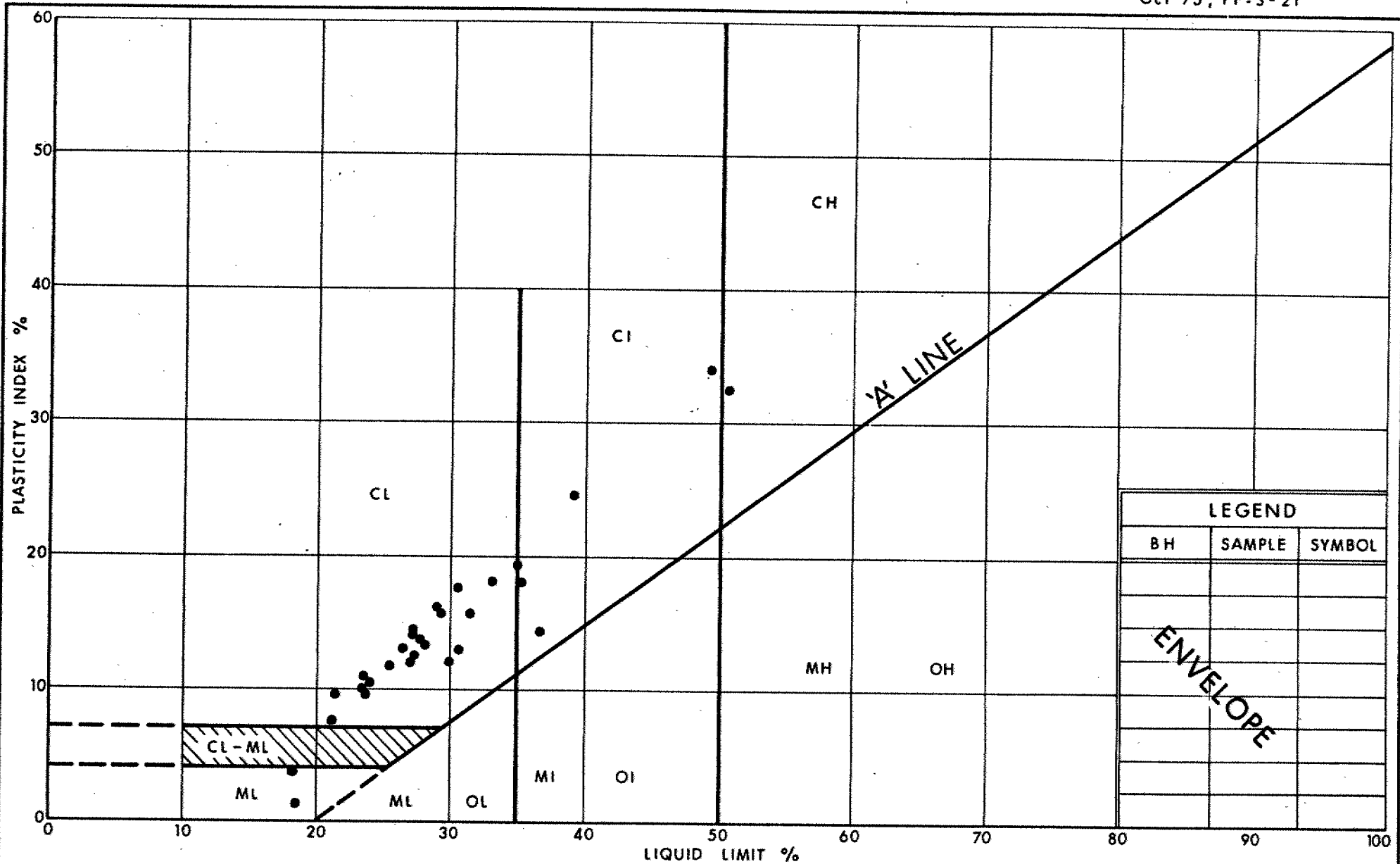


A handwritten signature in cursive script, appearing to read "Ken Ahmad".

K. Ahmad, P. Eng.
Foundation Engineer

A handwritten signature in cursive script, appearing to read "P. Rayer".
for M. Devata, P. Eng.
Chief Foundation Engineer

APPENDIX



Ontario

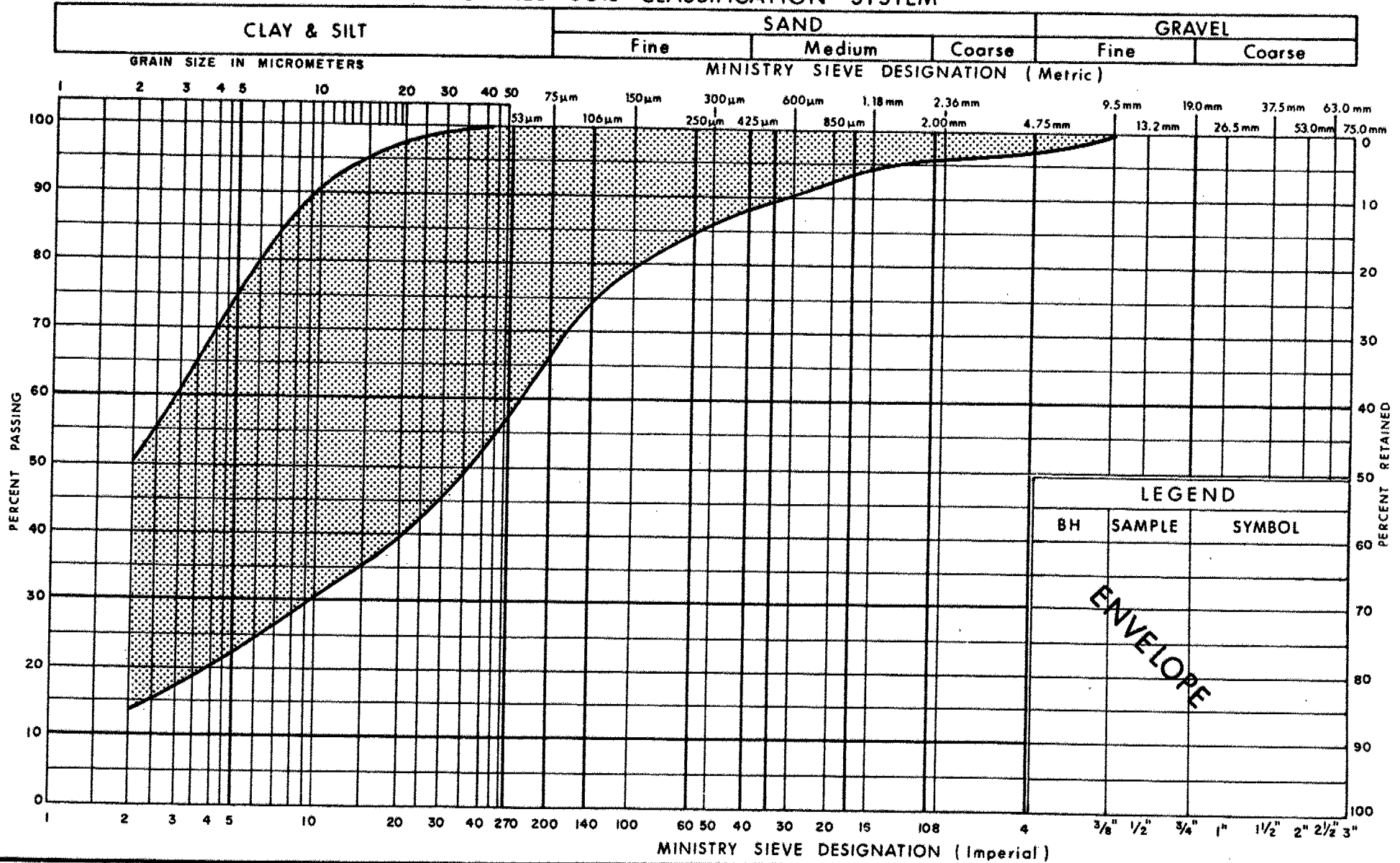
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PLASTICITY CHART
CLAYEY SILT, WITH FREQUENT LAYERS OF SILTY CLAY
AND OCCASIONAL SILT SEAMS

FIG No 1

W P 112-86-01

UNIFIED SOIL CLASSIFICATION SYSTEM



Ministry of
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GRAIN SIZE DISTRIBUTION
CLAYEY SILT, WITH FREQUENT LAYERS OF SILTY CLAY
OCCASIONAL SAND AND SILT SEAMS

FIG No 2

W P 112-86-01

VOID RATIO - PRESSURE CURVES

14

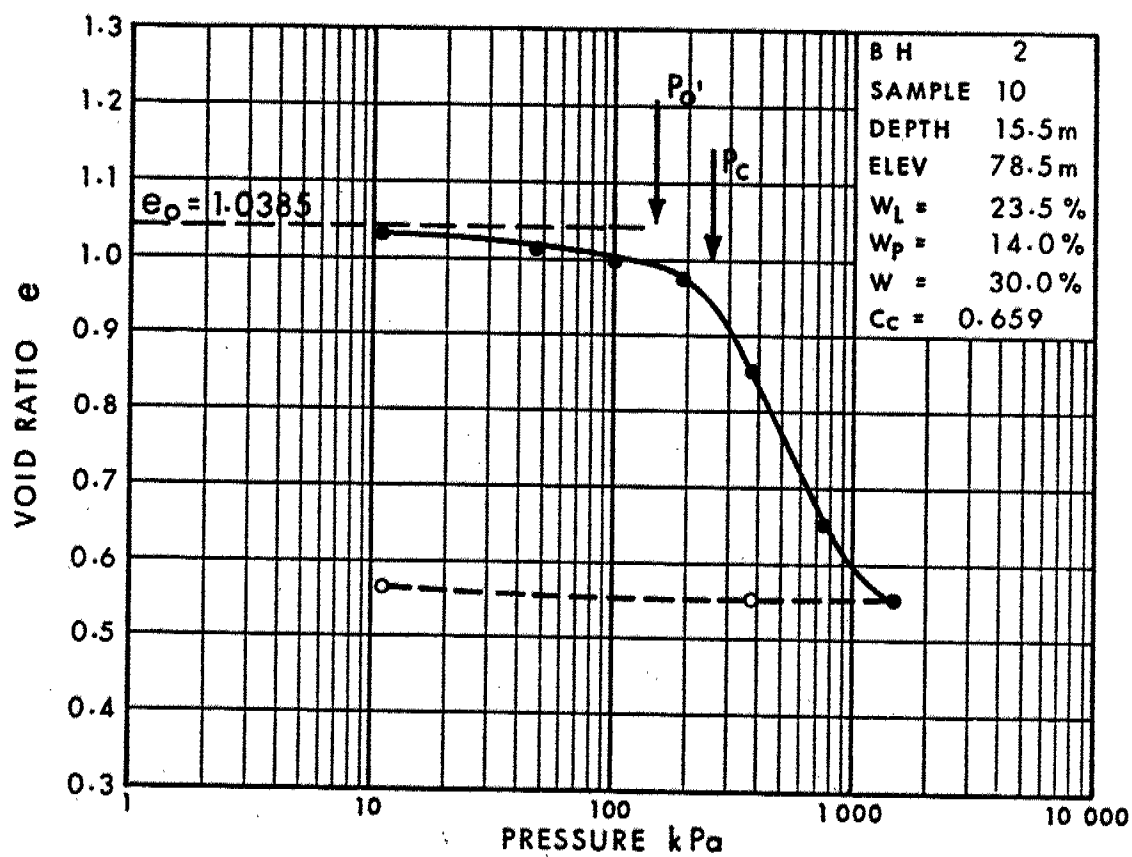
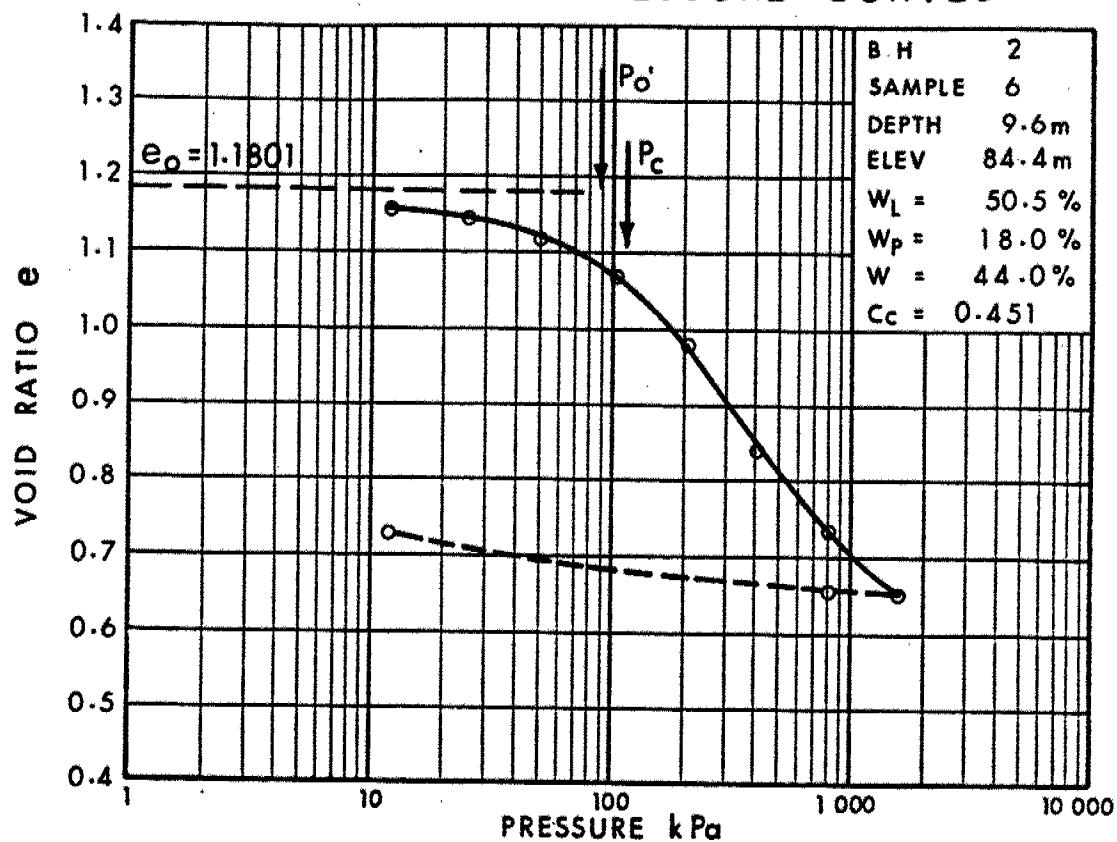


Fig 3

W P 112-86-01

VOID RATIO - PRESSURE CURVES

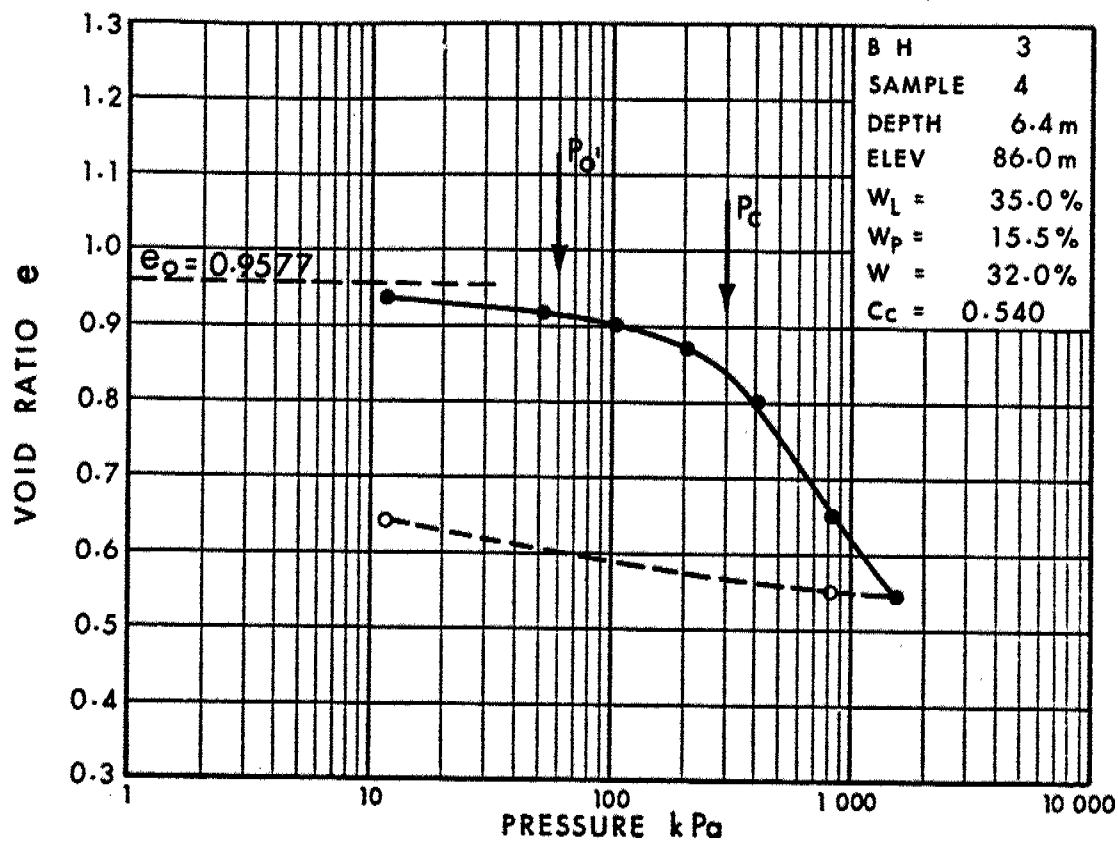


Fig 4

WP 112-86-01

VOID RATIO - PRESSURE CURVES

16

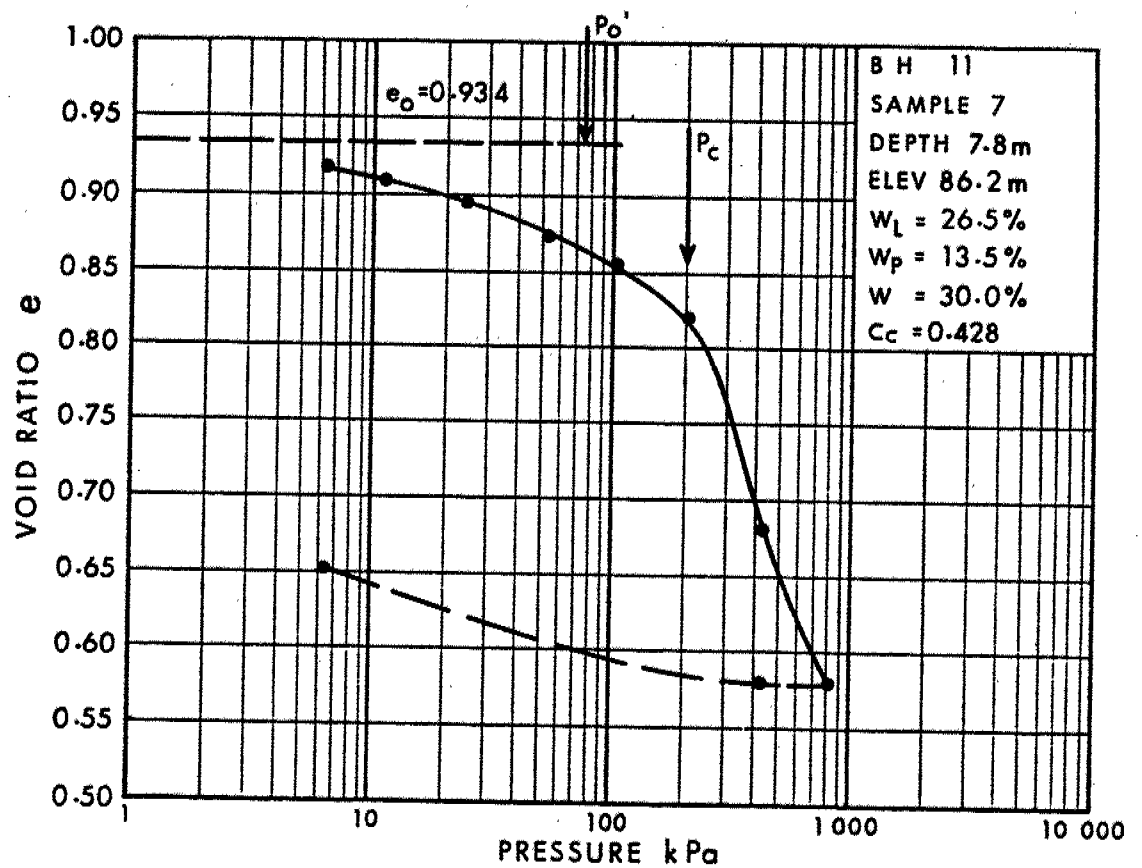
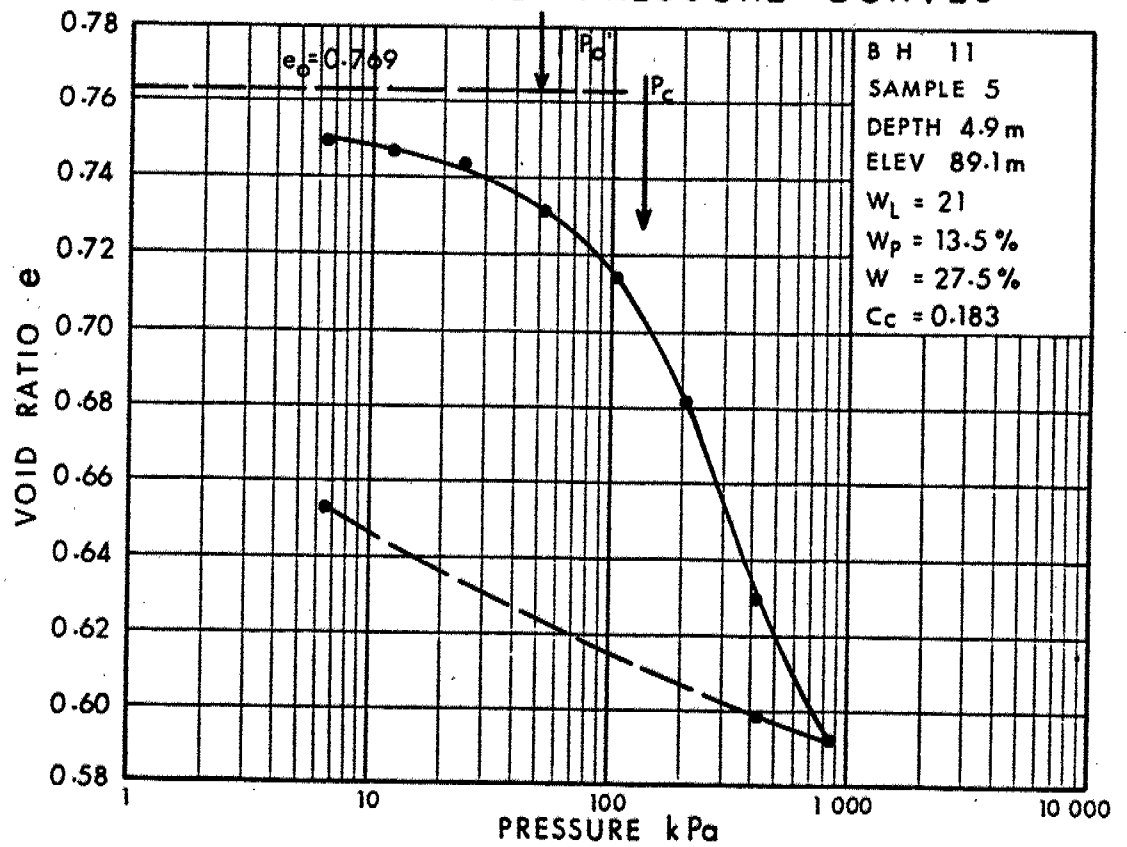


Fig 5

W P 112-86-01

VOID RATIO - PRESSURE CURVES

17

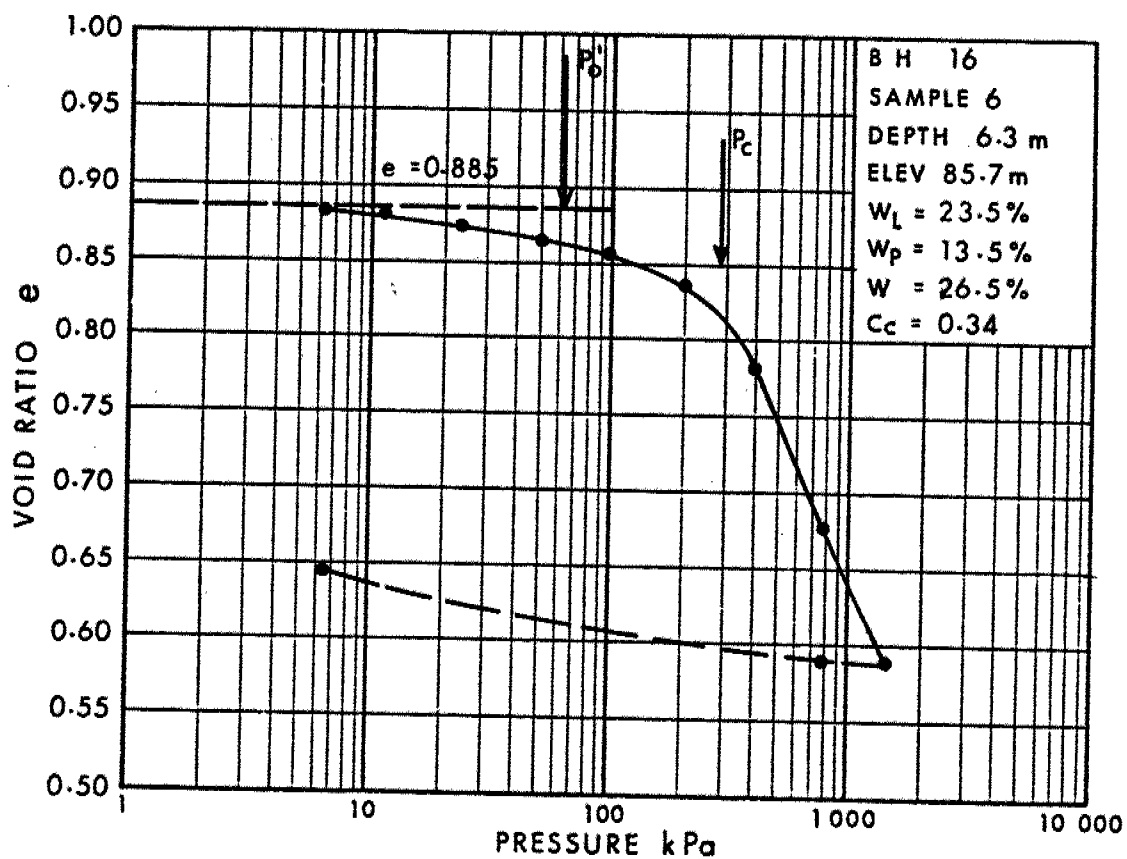
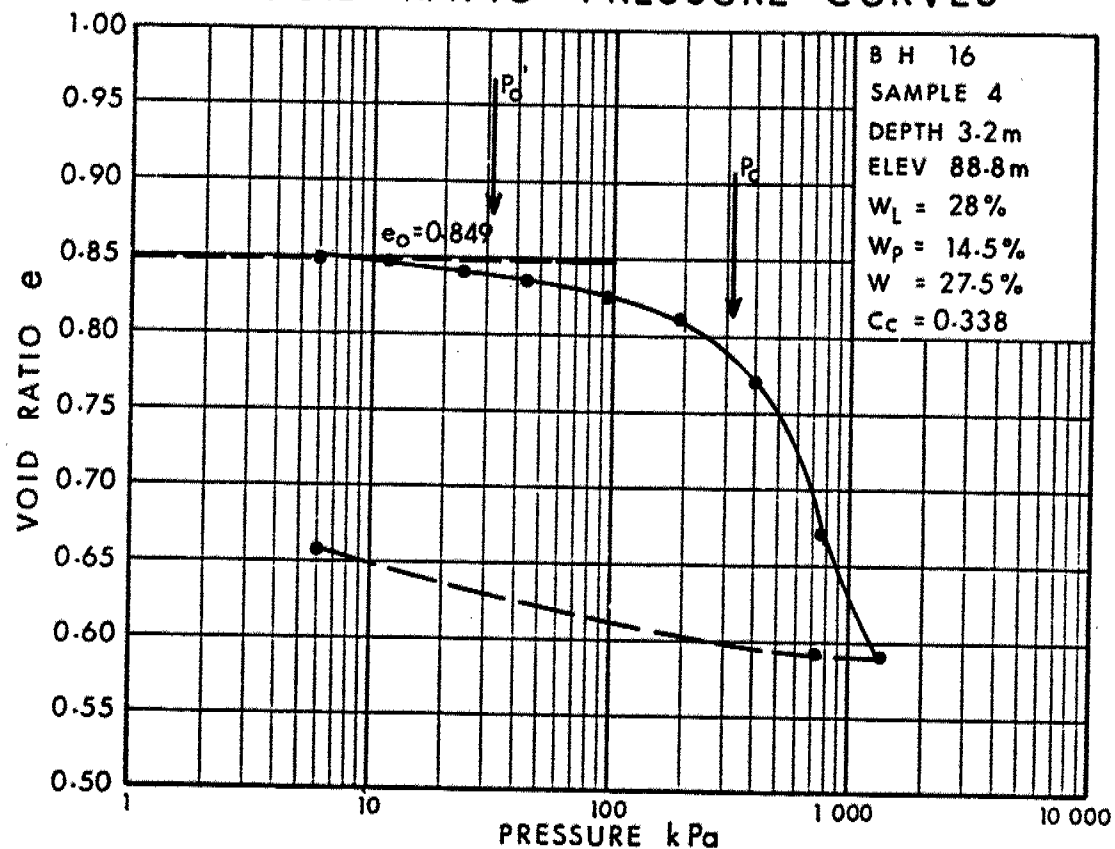


Fig 6

W P 112-86-01

VOID RATIO - PRESSURE CURVES

18

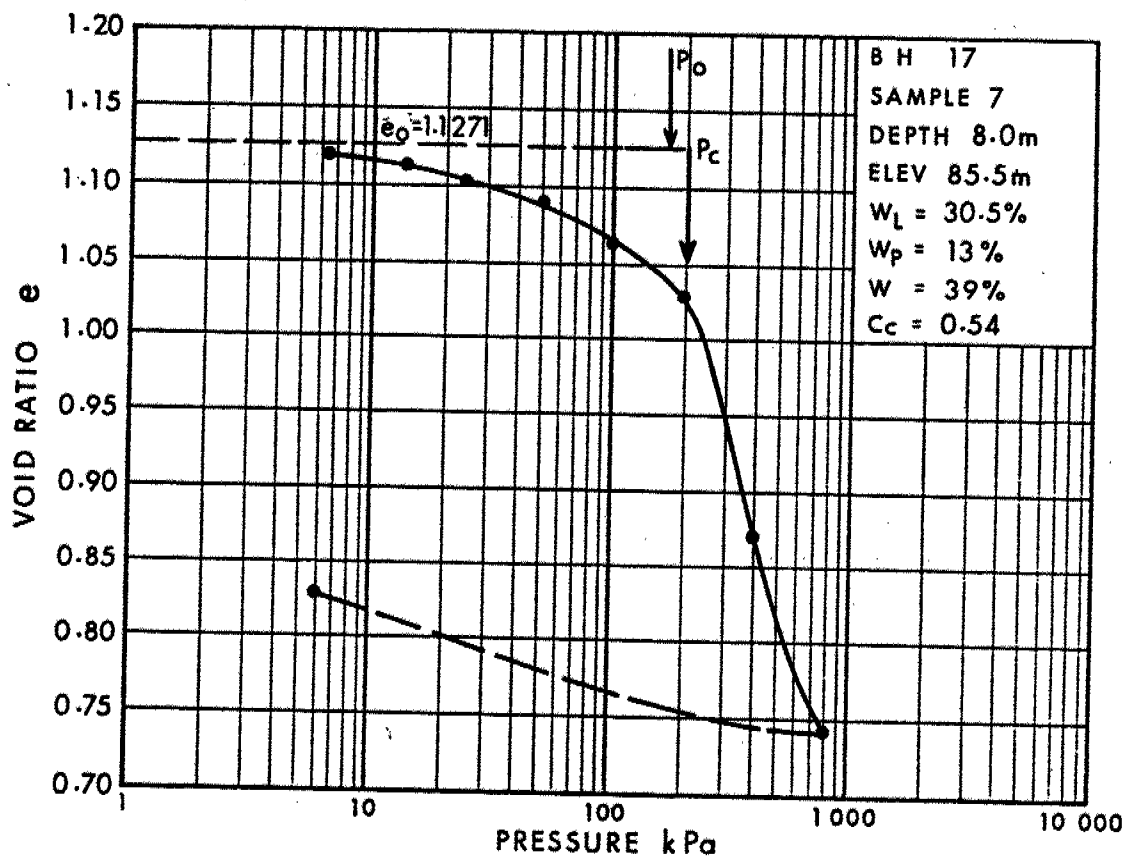
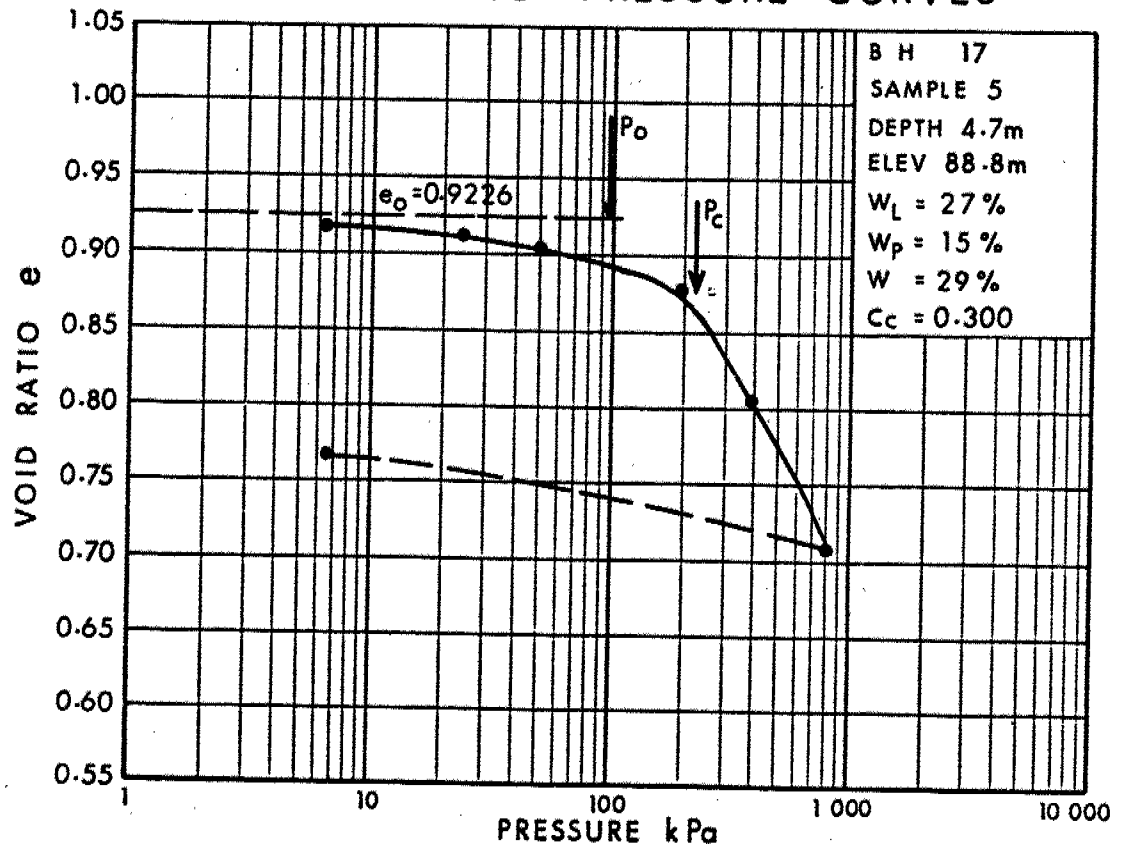


Fig 7

W P 112-86-01

RECORD OF BOREHOLE No 1 (1 of 2)

METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 147.0; E 328 788.0 ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.); BW Casing COMPILED BY MS
 DATUM Geodetic DATE 88 06 23 to 28 CHECKED BY

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		NATURAL MOISTURE CONTENT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60 80 100	Wp	W	W _L		
94.0	Ground Surface													
0.0	Asphalt, Concrete Gravelly Sand (Fill) Compact		1	SS	20									
91.6							92							
2.4	Trace Organic		2	SS	4									0 21 63 16
			3	SS	2									
			4	SS	2		90							0 0 70 30
			5	SS	0									
			6	SS	0									
			7	SS	0		88							0 0 64 36
			8	SS	0									
			9	SS	1		86							
			10	SS	0									
			11	SS	0		84							
			12	SS	0									
			13	SS	0		82							10 5 54 31
			14	SS	2		80							
			15	SS	2		78							
			16	SS	0		76							2 10 58 30
							74							
							72							
							70							
							68							
							66							
							64							

63.8
30.2

Continued

+3, x5: Numbers refer to
sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 1 Cont'd (2 of 2) METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 089.2; E 328 775.7 ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.): BW Casing COMPILED BY MS
 DATUM Geodetic DATE 88 06 23 to 28 CHECKED BY MS

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W _p	W	W _L			
63.8	Continued		17	SS	5									
30.2														
	Clayey Silt Frequent Silty Clay Layers, Occasional Sand and Silt Seams Trace Sand		18	SS	0									
	Firm / Stiff (Lacustrine)		19	SS	5									
			20	SS	5									
48.6	Occasional Silty Clay Seams Hard													
45.4														
			21	SS	55									
33.3	Silty Sand Dense													
60.7														
32.6														
61.4	End of Borehole		22	SS	32									

[illegible]

+3, x5: Numbers refer to Sensitivity

15 ϕ 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 3 (1 of 2) METRIC														
W P <u>112-86-01</u>		LOCATION Co-ords: N 4 933 184.2; E 328 773.0				ORIGINATED BY <u>MS</u>								
DIST <u>8</u> HWY <u>15</u>		BOREHOLE TYPE <u>Continuous Flight Auger (H.S.) & Cone Test</u>				COMPILED BY <u>MS</u>								
DATUM <u>Geodetic</u>		DATE <u>88 06 30 & 88 07 04</u>				CHECKED BY <u>MS</u>								
SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100						20 40 60 80 100
92.4	Ground Surface													
0.0	Top Soil													
	Clayey Silt Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace Sand (Lacustrine) Stiff		1	SS	3								0 0 86 14	
			2	SS	1									0 1 70 29
			3	SS	2									
			4	TW	*									
			5	TW	PH									
			6	SS	0									
			7	SS	4									
			8	SS	2									
			9	SS	0									
			10	SS	0									
76.7	End of Borehole													
15.7	Presumed Clayey Silt Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace Sand (Lacustrine)													
62.2														
30.2														

Continued
 * Pushed by Self-Weight

+3, x5: Numbers refer to Sensitivity

20
 15 5 (%) STRAIN AT FAILURE
 10

Continued

RECORD OF BOREHOLE No 3 Cont'd (2 of 2) METRIC

W P 112-86-01 LOCATION Co-ords: N 4 933 184.2; E 328 773.0 ORIGINATED BY MS
DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test. COMPILED BY MS
DATUM Geodetic DATE 88 06 30 & 88 07 04 CHECKED BY M

[illegible]

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 4										METRIC			
W P 112-86-01		LOCATION Co-ords: N 4 933 063.6; E 328 766.6				ORIGINATED BY MS							
DIST 8 HWY 15		BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test				COMPILED BY MS							
DATUM Geodetic		DATE 88 07 04				CHECKED BY A1							
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT	PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES								
94.1	Ground Surface												
0.0	Gravelly Sand (Fill)												
92.6	Very Loose to Compact*												
1.5	Trace Organics		1	SS	2								
	Clayey Silt		2	SS	2								
	Frequent Silty Clay Layers												
	Occasional Sand and Silt Seams		3	SS	0								
	Trace Sand												
	(Lacustrine)		4	SS	0								
	Firm / Stiff		5	SS	0								
84.5	End of Borehole		6	SS	14								
9.6	Presumed Bedrock												
	* Trace Organics												

RECORD OF BOREHOLE No 11

1 OF 3

METRIC

W.P. 112-86-01

LOCATION Co-ords: N 4 933 142.2; E 328 779.6

ORIGINATED BY KA

DIST 8

HWY 15

BOREHOLE TYPE Hollow Stem Auger, NW Casing

COMPILED BY KA

DATUM Geodetic

DATE 90 03 12 to 90 03 20

CHECKED BY DD

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 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20 40 60 80 100	20 40 60 80 100					
94.0	Ground Level												
0.0	Gravelly Sand Fill Compact		1	SS	25								
			2	SS	3								
			3	SS	2								
90.1	Dk. Brown Clayey Silt with wood fragments/topsoil Soft to Firm		4	SS	5						w=190		
3.9			5	TW	PH							19.2	0 2 68 30
			6	SS	0								
			7	TW	PH							19.4	0 3 57 40
			8	SS	1								
			9	TW	*							19.9	0 2 68 30
			10	SS	0								
			11	TW	*								
			12	TW	*								
			13	SS	1								
			14	SS	4								
			15	SS	2								
			16	SS	0								
63.5													
30.5													

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (X) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 11

2 OF 3

METRIC 26

W.P. 112-86-01

LOCATION Co-ords: N 4 933 142.2; E 328 779.6

ORIGINATED BY KA

DIST 8 HWY 15

BOREHOLE TYPE Hollow Stem Auger, NW Casing

COMPILED BY KA

DATUM Geodetic

DATE 90 03 12 to 90 03 20

CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20					
30.5	Continued		17	SS	5								
	Clayey Silt, Grey with Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace of Sand and Gravel (Lacustrine) Soft to Stiff (Generally Firm)		18	SS	4								
54.0			19	SS	0								
40.0			20	SS	18								
	Silty Sand with Gravel Occasional Boulders Grey, Wet Compact to Very Dense		21	SS	20								
			22	SS	17								
			**	SS	16								
			**	SS	23								
			**	SS	72								
			**	SS	94								
			**	SS	93								
			**	SS	115								
			**	SS	122								

Continued

+3, x5, Numbers refer to
Sensitivity

20
15-25 (%) STRAIN AT FAILURE
10

Continued

RECORD OF BOREHOLE No 11

3 OF 3

METRIC 27

W.P. 112-86-01

LOCATION Co-ords: N 4 933 142.2; E 328 779.6

ORIGINATED BY KA

DIST 8

HWY 15

BOREHOLE TYPE Hollow Stem Auger, NW Casing

COMPILED BY KA

DATUM Geodetic

DATE 90 03 12 to 90 03 20

CHECKED BY DD

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 7 kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20	40	60	80					
32.4	Continued		**	SS	132											
61.6	End of Borehole															
	<p>* Shelby Tube sunk with the weight of the A-rad.</p> <p>**</p> <p>Split spoon sampler was advanced continuously (like cone test) to determine the density of the soil.</p> <p>The energy used to drive the spoon was the same as used in the Standard Penetration Test.</p> <p>N - values may not be representative</p>															

RECORD OF BOREHOLE No 13

2 OF 2

METRIC 29

W.P. 112-86-01 (B) LOCATION Co-ords: N 4 933 121.7; E 328 773.8
DIST 8 HWY 15 BOREHOLE TYPE NW Casing, BW Casing, BX Core
DATUM Geodetic DATE 90 03 22 to 90 03 27
ORIGINATED BY KA
COMPILED BY KA
CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
30.5	Continued															
53.3	Presumed Silty Sand with Gravel Occasional Boulders Grey, Wet Compact to Very Dense															
38.7	Bedrock MARBLE		1	RC	REC	97%										RQD 84%
50.0			2	RC	REC	98%										RQD 100%
42.0	End of Borehole															

RECORD OF BOREHOLE No 16

1 OF 1

METRIC 30

W.P. 112-86-01

LOCATION Co-ords: N 4 933 155.7; E 328 771.3

ORIGINATED BY KA

DIST 8

HWY 15

BOREHOLE TYPE Hollow Stem Auger

COMPILED BY KA

DATUM Geodetic

DATE 90 03 21

CHECKED BY DD

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w _p	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)				
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40						60	80	100	20
92.0	Ground Level																	
0.0	Topsoil 15 cm.		1	SS	2													
			2	SS	2													
			3	SS	2													
			4	TW	PH													
			5	SS	1													
			6	TW	PH													
			7	SS	2													
			8	SS	3													
			9	SS	2													
			10	SS	1													
75.8																		
16.2	End of Borehole																	
<p>• GROUND WATER CONDITIONS</p> <table border="1"> <tr> <td>PIEZO. NO.</td> <td>GROUND WATER ELEVATION (Metres)</td> </tr> <tr> <td>1</td> <td>92.0</td> </tr> </table>															PIEZO. NO.	GROUND WATER ELEVATION (Metres)	1	92.0
PIEZO. NO.	GROUND WATER ELEVATION (Metres)																	
1	92.0																	

1 OF 1

METRIC 31

LOCATION _____ Co-ords: N 4 833 203.0; E 328 779.3

ORIGINATED BY KA

DIST 8 HWY 15

BOREHOLE TYPE Hollow Stem Auger

COMPILED BY KA

DATUM Geodetic

DATE 90 03 21

CHECKED BY DD

+3, x³: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 18

1 OF 1

METRIC 32

W.P. 112-86-01

LOCATION Co-ords: N 4 933 245.6; E 328 778.5

ORIGINATED BY KA

DIST 8 HWY 15

BOREHOLE TYPE Hollow Stem Auger

COMPILED BY KA

DATUM Geodetic

DATE 90 03 21

CHECKED BY DD

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ kN/m ³	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100					
93.8	Ground Level															
0.0	Silty Sand with a Trace of Clay (Fill) Very Loose to Loose		1	SS	3											
			2	SS	1											
91.3			3	SS	11											
2.5			4	SS	9											
	Clayey Silt, Greyish Brown/Grey with Frequent Silty Clay Layers Occasional Sand and Silt Seams Trace of Sand and Gravel (Locustrine) Firm to V. Stiff		5	SS	1											
			6	SS	1											
			7	SS	1											
86.1			8	SS	50	/Acn										
7.7	End of Borehole															
	Hammer Bouncing at 7.7m depth Presumed Bedrock															



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Transportation and
Communications

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FOUNDATION DESIGN SECTION

**foundation
investigation and
design report**

ENGINEERING MATERIALS OFFICE
FOUNDATION DESIGN SECTION

CONT 91-60

WP 112-86-01 DIST 8

HWY 15 STR SITE 16-45

Morton Creek Bridge Replacement
(Preliminary)

DISTRIBUTION

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PRELIMINARY FOUNDATION INVESTIGATION REPORT
For
Morton Creek Bridge Replacement
W.P. 112-86-01, Site 16-45
Highway 15, District 8, Kingston

INTRODUCTION

This report contains the results of the preliminary foundation investigation carried out for the proposed detours and the three alignment alternatives for the replacement structure. The fieldwork was carried out during the period from 88-06-23 to 88-07-04. The fieldwork consisted of four sampled boreholes with dynamic cone penetration tests adjacent to three of them. The borings were advanced by hollow stem auger and BW casing (44 mm I.D.) using a machine mounted on a muskeg vehicle. Sampling was performed to a maximum depth of 61.4 m to an elevation of 32.6 m and the cone tests to a maximum depth of 33.3 m to elevation 59.1 m.

It should be noted that the ETR plan and chainage for the report are in imperial units with the remainder of the report being metric. This is due to both the plan and the information obtained from the Surveys and Plans, Eastern Region being in imperial units.

SITE DESCRIPTION

The site is located just north of Morton on Highway 15 at Morton Creek in the Township of South Crosby in the County of Leeds. The terrain in the immediate vicinity is hilly with marshlands adjacent to the creek between rock ridges. The physiological region is Leeds Knobs and Clay Flats and typically consists of knobs of granite and other Precambrian rocks with areas between filled with very weak calcareous clay left by the Champlain Sea.

SUBSURFACE CONDITIONS

General

The subsoil was found to consist of a soft to stiff lacustrine clayey silt, with frequent silty clay layers, occasional sand and silt seams, trace sand, gravel.

The sampling ended within this deposit expect for borehole 1, where it was underlain by silty sand. The depth of overburden is highly variable, ranging from 9.6 to 61 m+. The presumed bedrock elevation at borehole 4 was 84.5 m however approximately a metre away the cone test indicated the bedrock elevation at 86.8 m. This indicates a sloping and unpredictable bedrock topography.

The plan, location and stratigraphy of the borings are shown on drawing number 1128601-A and the obtained field and laboratory tests plotted on the Record of Borehole sheets 1 through 4 are attached.

A brief description of the different strata is given below.

Fill Material

Fill material was found at the surface to elevation 92.6 (borehole 4) and to 91.6 m (borehole 1). The fill ranged in thickness from 2.4 m along the existing road to 0 m at borehole 3 which was off the road. The fill was mainly pavement and shoulder material consisting of asphalt, concrete and sandy silt to gravelly sand. The fill also contained occasional clayey silt and there were traces of organics in the bottom 0.3 m of fill. The denseness of the fill ranged from very loose to compact.

Clayey Silt

This deposit was either immediately below the fill or the surficial deposit. The material consisted of a lacustrine deposit of clayey silt, with frequent silty clay layers, occasional sand and silt seams, trace of sand, and trace of gravel. This deposit extended to the termination of sampling in all but borehole 1 where, at approximate elevation 33.3 m, silty sand was encountered in the last sample.

The physical properties of the material as determined by field and laboratory tests are summarized below:

	<u>Range</u>	<u>Average</u>
Natural Moisture Content (w)	22-60%	33%
Liquid Limit (w_L)	18-51%	32%
Plastic Limit (w_p)	13-23%	16%
Unit Weight (γ)	17.3-20.4 kN/m ³	18.8 kN/m ³

The results above have been plotted on the Plasticity Chart, Figure 1. The majority of the tests plot as a clayey silt of low plasticity, with occasional tests indicating silt seams of low plasticity, or layers of silty clay of intermediate plasticity.

Following is a summary of strength characteristics of this material based on unconfined compression tests of shelly tube samples and on in situ field vane tests.

	<u>Range</u>	<u>Average</u>
Shear Strength (based on Compression Test)	15-50 kPa	31.5 kPa
Shear Strength (based on Field Vane)	12-96+ kPa	43 kPa

The N-blows from the standard penetration tests average at one, ranging from 0 to 5 with one test each indicating 14 and 55. These results, in addition to those from the tests shown above, indicate that the consistency of the deposit ranges from soft to hard. The majority of the material was however firm.

The following are results of consolidation tests carried out in the lab:

	<u>Range</u>	<u>Average</u>
Initial Voids Ratio (e_0)	0.96-1.18	1.06
Preconsolidation (P_C)	111-297 kPa	216 kPa
Compression Index (C_C)	0.45-0.66	0.54

The deposit under the existing alignment appears to be normally consolidated whereas the material sampled off the road is slightly overconsolidated.

The results from the grain size distribution tests are shown in envelope form in Figure 2.

Silty Sand

At the bottom of borehole 1 a deposit of dense silty sand was found at approximate elevation 33.3 m (60.7 m below the ground surface), to the bottom of the borehole at elevation 32.6 m.

Bedrock

Bedrock was not proven by rock coring. Bedrock can therefore only be presumed to be below the overburden material at or below the following estimated elevations:

<u>Borehole</u>	<u>Presumed Bedrock Elevations (m)</u>		<u>Bedrock Below Elevation (m)</u>
	<u>Borehole</u>	<u>Cone Test</u>	
1	-	-	32.6
2	63.5	65.7	63.5
3	-	-	58.8
4	84.5	86.8	84.5

It must be noted that the bedrock elevations are highly variable, as indicated by the large differences in the elevations indicated by the boreholes and their respective cone tests which are approximately a metre apart.

Groundwater Conditions

The following groundwater conditions were observed during the field investigation:

<u>Borehole</u>	<u>Elevation (m)</u>
1	93.0
2	93.0
3	92.1
4	92.0

The boreholes indicate the groundwater level is at approximate elevation 93.0 m. This level will most likely vary seasonally and with fluctuations in the creek.

DISCUSSION AND RECOMMENDATIONS

General

At this preliminary stage there are four alternatives under consideration by the Eastern Regional Structural Section for the structure replacement and detour, as follows:

- (1) To construct a new structure on the west side of existing structure with the detour using existing structure.
- (2) To construct a new structure on the east side of existing structure with the detour using existing structure.
- (3) To provide a one-lane detour using a bailey bridge on the east side of existing structure and to replace the existing structure.
- (4) To provide a two lane detour using a bailey bridge on the east side of existing structure and to replace the existing structure.

The location of piers and abutments for the structures alternatives have not been provided for this preliminary assessment.

Proposed Structure Replacement and Detour

Both the proposed structure and detour alternatives may be supported by friction piles. Granular pads were considered for supporting the weight of the detour bailey bridge however it was determined that the bearing capacity of the foundation soil was not sufficient. The design values for the pile capacities may be conservative due to the uncertainties inherent in friction piles in this subsoil. After the final investigation, it may be determined that full scale pile load testing may be required at the site and at that time pile capacities may be revised. The following foundation recommendations are provided for preliminary planning purposes and are subject to change following the foundation investigation for this project.

For the purposes of the O.H.B.D.C., and assuming an embedded length of 15 m to elevation 77 m, the following values are recommended for No. 36 timber piles. The piles should be treated with preservative.

Factored Axial Capacity at U.L.S. 150 kN

Axial Capacity at S.L.S. Type II 100 kN

Due to the concerns about differential settlement, it is imperative that the structures are not supported partially by piles end bearing on bedrock and partially by friction piles. This concern will be reviewed when actual footing locations are provided during the foundation investigation.

The pile caps should have a minimum of 1.5 m earth cover for frost protection.

Earth pressure acting on abutments, retaining and wingwalls should be computed as per subsection 6.6.1.2.2 of the O.H.B.D.C..

A yielding foundation condition should be assumed. The following properties of granular backfill may be used for computations:

Granular 'A'	$\gamma = 22.8 \text{ kN/m}^3$	$\phi = 35^\circ$	$K_a = 0.271$
Granular 'B'	$\gamma = 21.2 \text{ kN/m}^3$	$\phi = 30^\circ$	$K_a = 0.333$

Concrete should be placed in the 'dry'. Dewatering is not anticipated to be a problem although a dewatering scheme will be required for footing excavations below the groundwater level (river level).

Approach Embankments

Topsoil and surficial material should be removed prior to placing any fill. The fill should consist of well compacted acceptable material. Particle sizes in the fill immediately beneath the pile locations should not exceed 50 mm, to facilitate pile driving.

The side and forward slopes should not be steeper than two horizontal to one vertical designed and constructed in accordance with the appropriate Ministry

Standards. The safe height of fill has been estimated at 2.0 m. Although no additional fill is proposed along the existing alignment it should be noted that the existing embankment height is already 2 m and therefore the existing profile grade of the existing highway should not be raised. When the final investigation has been completed and existing and proposed profiles are received further slope stability analyses will be carried out.

Settlement

Settlement evaluations were based on two consolidation test results. The Schmertmann Method was used to correct for the effects of sample disturbance in the determination of the Preconsolidation Pressure (see Figure 3 and 4). Stress distribution was calculated using both the Purdue and Boussinesq methods. In calculating the consolidation settlements, a base width of 10 m was assumed for the imposed load and 30 m for the effective thickness of the deposit. For the time rate of settlement calculations the coefficient of consolidation was calculated using the Root Time Method (after Taylor) assuming drainage in two directions which is consistent with observations at similar sites.

Based on the above calculations, settlement is to be expected along all approaches. If the highway is re-aligned and a 2 m embankment is required, settlement in the order of 15 to 20 cm can be expected, (more settlement is expected on the east side rather than the west). It is predicted that 90% of this settlement will occur within 12 years. At least 50% of this total settlement is expected however to occur within a period of 2.5 years.

The calculations also indicated that settlement may be minimized by maintaining Highway 15 along its existing alignment, since most of the settlement has already occurred. Settlement in the order of 2 to 4 cm can however be expected along the existing alignment with no grade raise. Assuming that the existing road was constructed in the late 1950's and thus has been loaded for 30 to 35 years this settlement should occur over the next several years provided no additional loads are applied.

It has been reported by H. Kleywegt of the Eastern Region Structural Section that very large settlements in the order of 2.5 m have occurred at this site. Although our field investigation and subsequent analysis has indicated the smaller settlements indicated in this report, further analyses may be required during the foundation investigation phase of this project in order to verify these predictions.

Replacing the existing structure will not solve the settlement problems at the north approach. To eliminate settlement, subexcavation of up to 2 m and backfilling with lightweight fill should be considered on the north side. One alternative which has proven to be most cost efficient in the past is blast furnace slag. Estimates for the cost of two types of this material follow:

	Approximate Costs <u>Including Transportation</u>
'Old Clinker' slag	\$50/m ³
pelletized '3/8" Structural Coarse' slag	\$56/m ³

Settlement along a new alignment can not be eliminated without surcharging the approaches which will not be feasible in this case.

Since the request made by the Eastern Region Structural Section for a preliminary investigation indicated that the existing structure is showing no apparent distress from differential settlement, and that the main concern is excessive settlement at the approaches, consideration could be given to other options rather than replacing the structure. For example, the existing structure could be rehabilitated and the approaches could be reconstructed with lightweight material as described above.

MISCELLANEOUS

It must be noted that these recommendations are preliminary and conditions and recommendations may change when the final investigation takes place.

The fieldwork for this preliminary investigation was carried out under the supervision of M. Schnarr, Student. The equipment was owned and operated by Marathon Drilling Co. Ltd. This report was prepared by P. Marks under the supervision of D. Dundas, Senior Foundation Engineer and reviewed by M. Devata.



P. Marks
P. Marks, P.Eng.
Foundation Engineer

M. Devata
M. Devata, P.Eng.
Chief Foundation Engineer

APPENDIX

Oct 75, FF-S-21

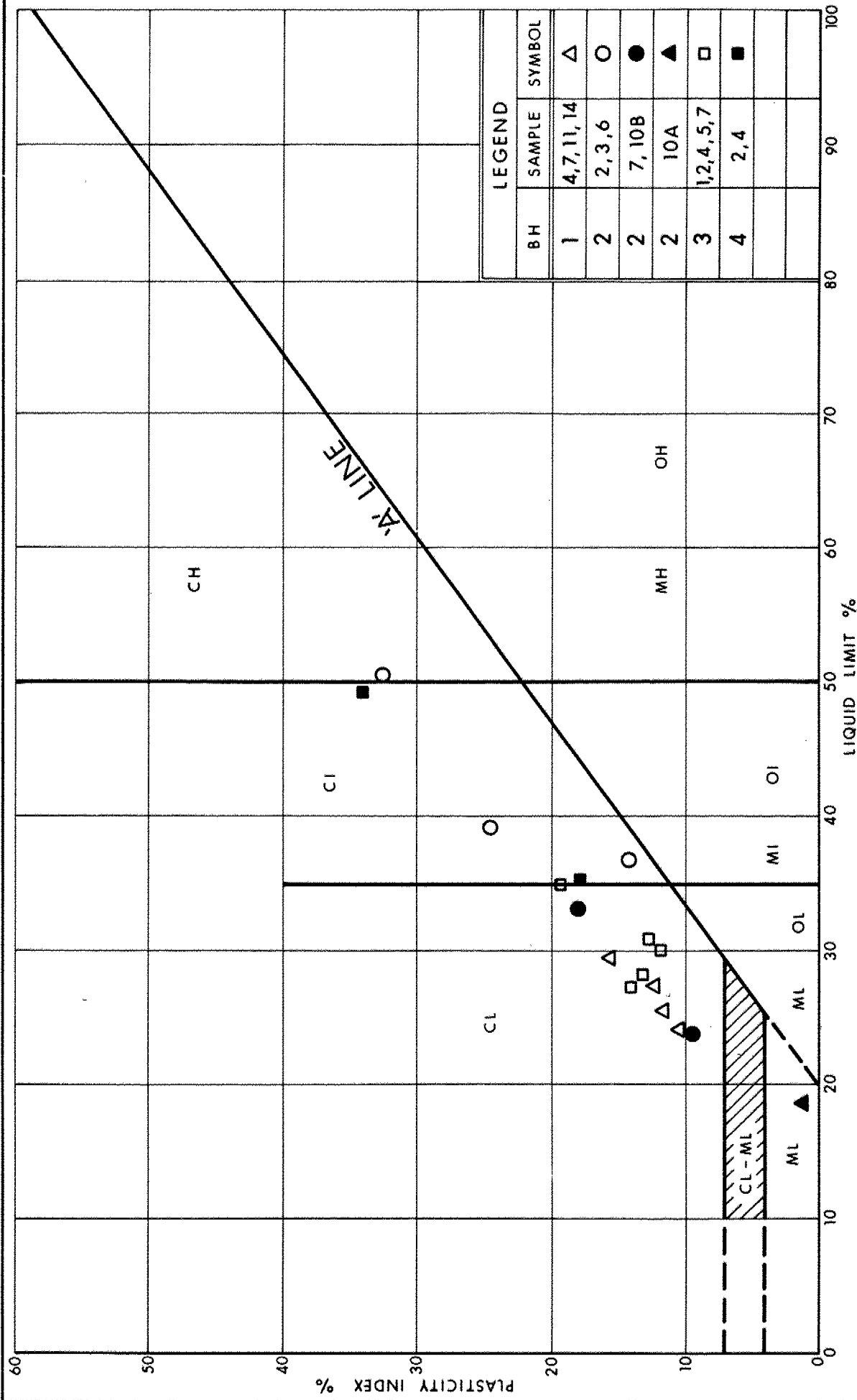


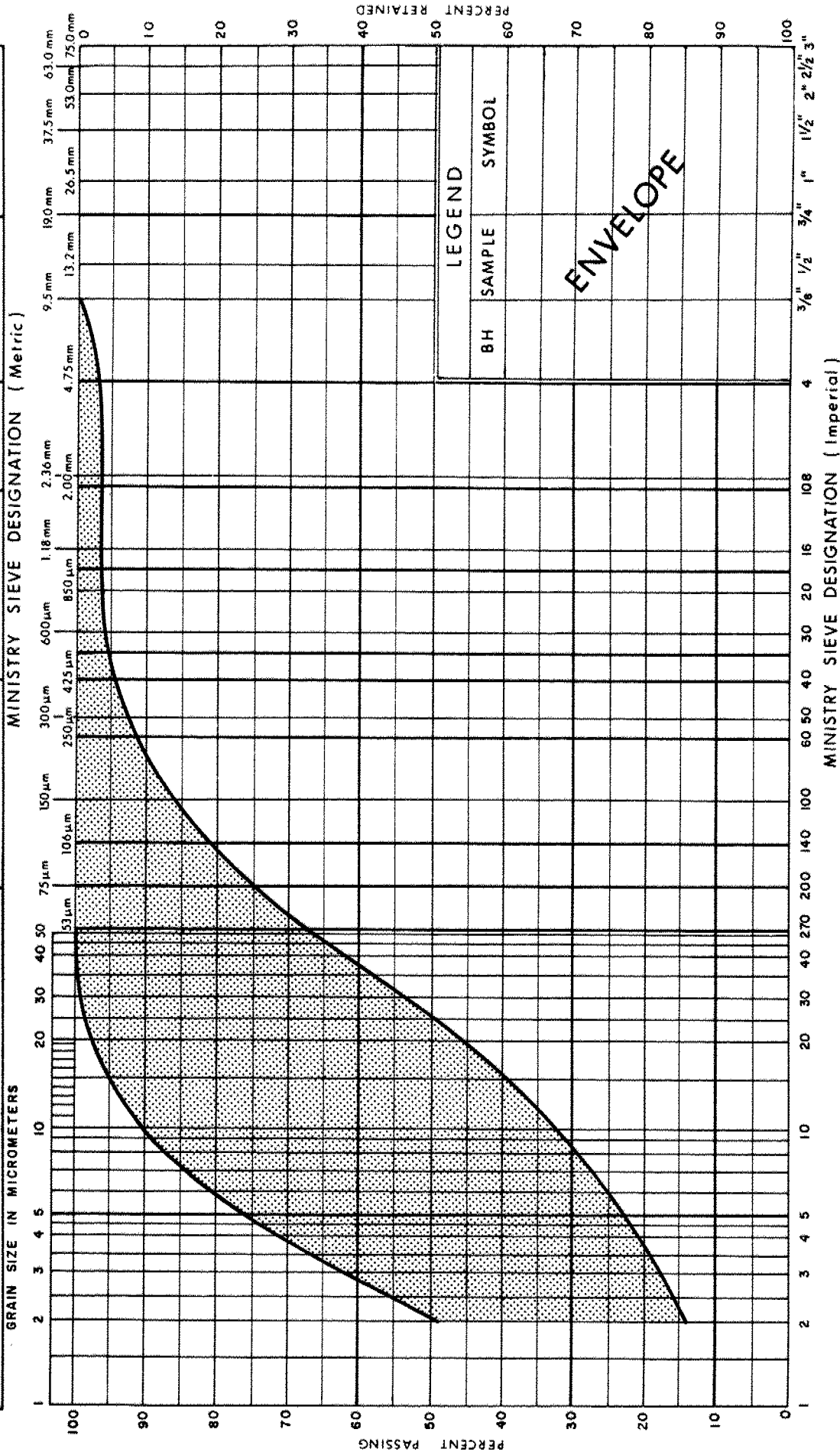
FIG No 1

PLASTICITY CHART CLAYEY SILT, WITH FREQUENT LAYERS OF SILTY CLAY AND OCCASIONAL SILT SEAMS

W P 112-86-01

UNIFIED SOIL CLASSIFICATION SYSTEM

CLAY & SILT		SAND			GRAVEL	
		Fine	Medium	Coarse	Fine	Coarse
MINISTRY SIEVE DESIGNATION (Metric)						



VOID RATIO - PRESSURE CURVES

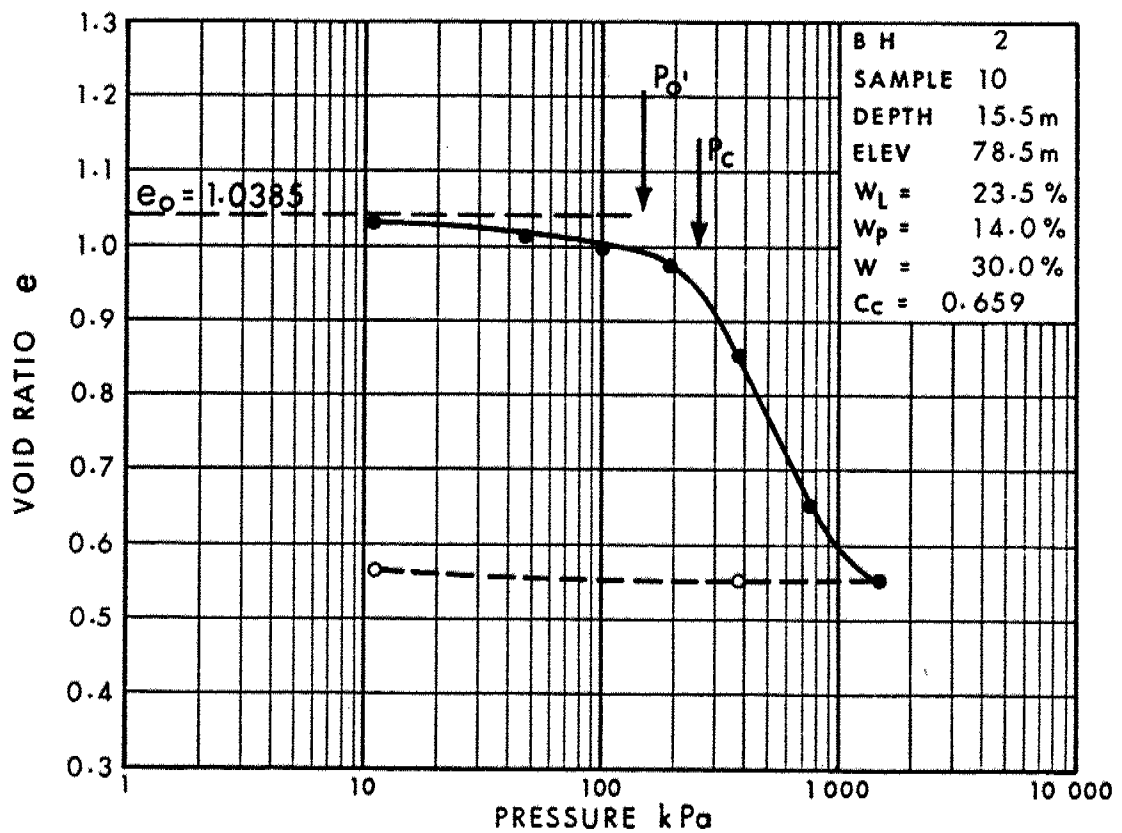
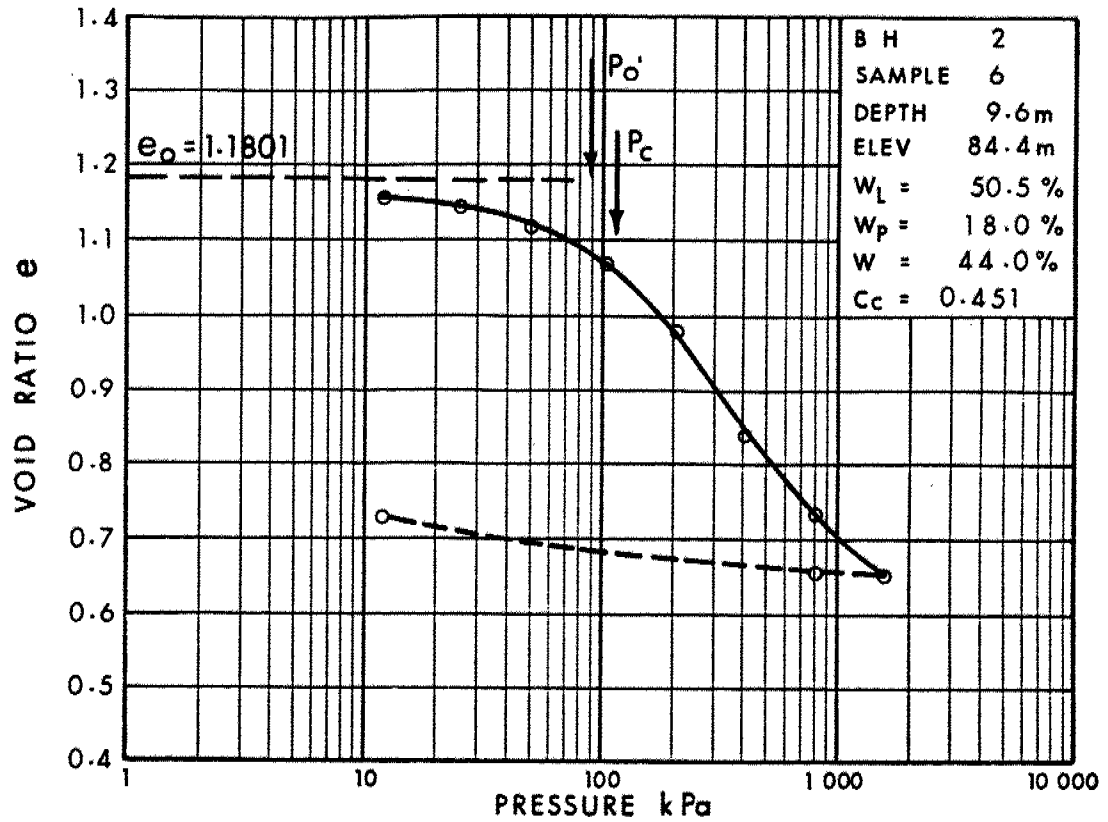


Fig 3

W P 112-86-01

VOID RATIO - PRESSURE CURVES

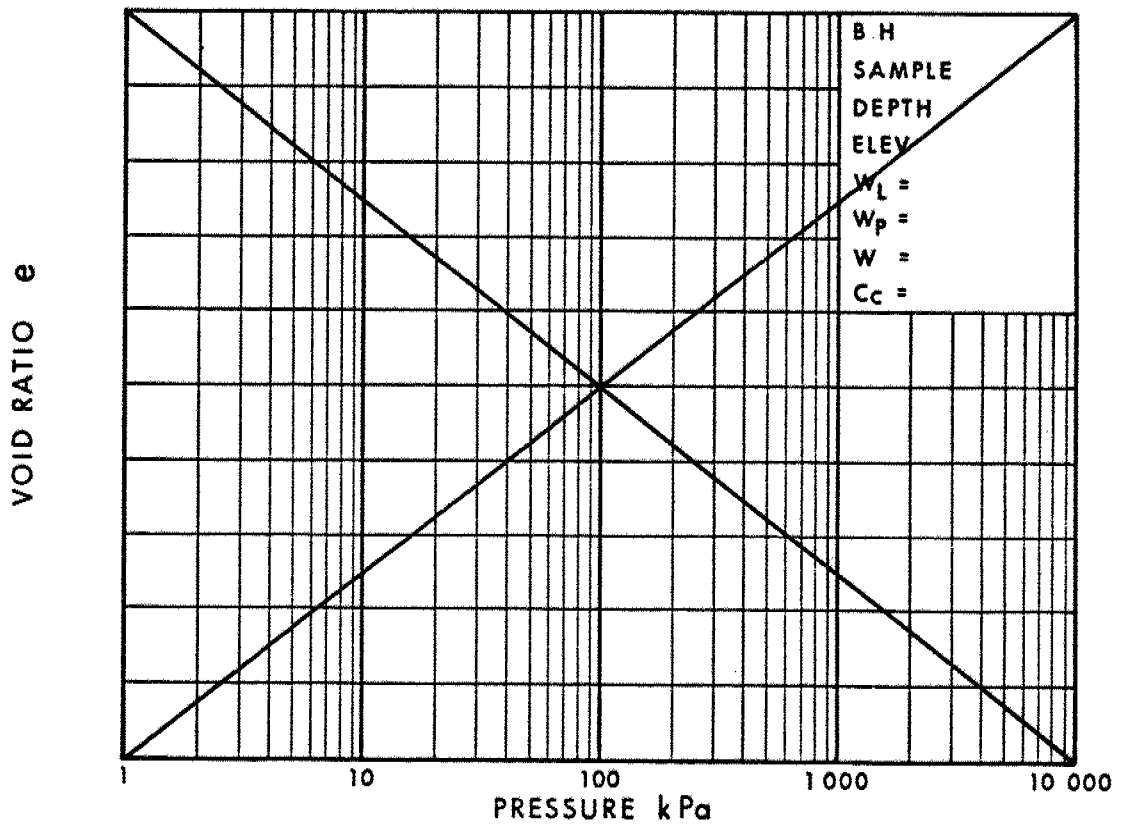
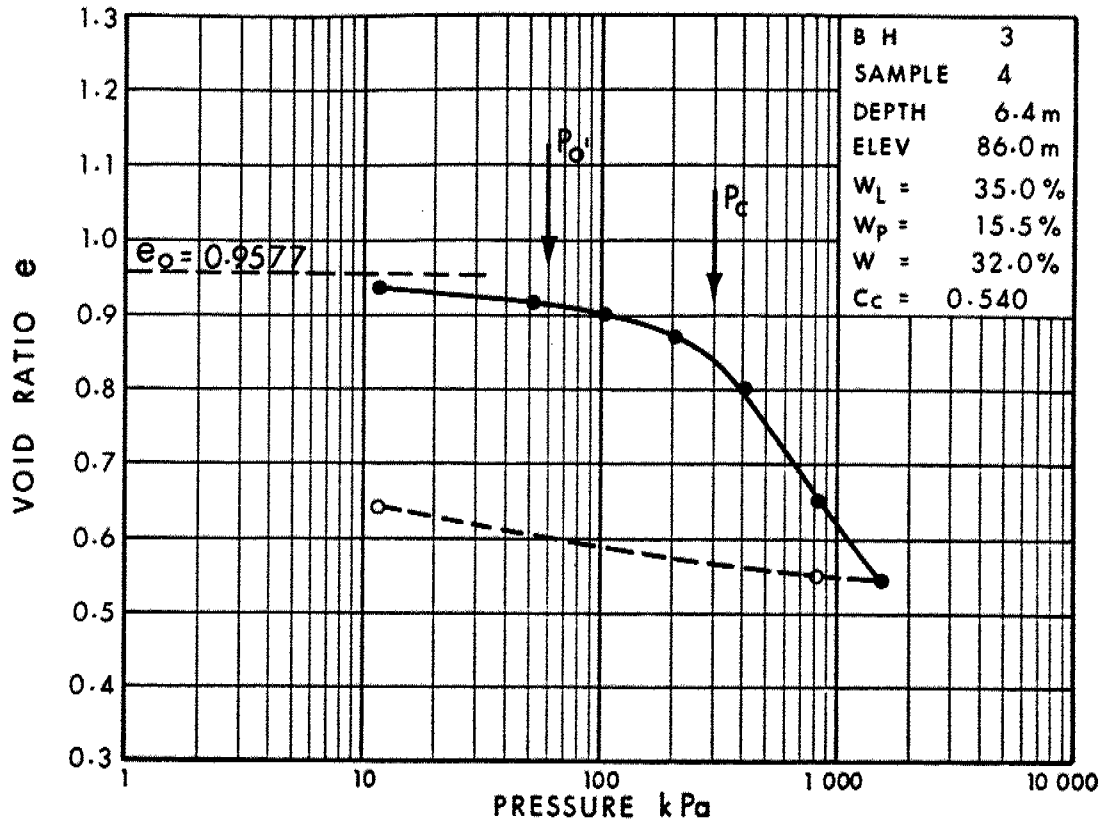


Fig 4

W P 112-86-01

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

STRESS AND STRAIN

u_w	kPa	PORE WATER PRESSURE
r_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	e	1, %	VOID RATIO	e_{min}	1, %	VOID RATIO IN DENSEST STATE
γ_s	kN/m^3	UNIT WEIGHT OF SOLID PARTICLES	n	1, %	POROSITY	I_D	1	DENSITY INDEX = $\frac{e_{max} - e}{e_{max} - e_{min}}$
ρ_w	kg/m^3	DENSITY OF WATER	w	1, %	WATER CONTENT	D	mm	GRAIN DIAMETER
γ_w	kN/m^3	UNIT WEIGHT OF WATER	S_r	%	DEGREE OF SATURATION	D_n	mm	n PERCENT - DIAMETER
ρ	kg/m^3	DENSITY OF SOIL	w_L	%	LIQUID LIMIT	C_u	1	UNIFORMITY COEFFICIENT
γ	kN/m^3	UNIT WEIGHT OF SOIL	w_p	%	PLASTIC LIMIT	h	m	HYDRAULIC HEAD OR POTENTIAL
ρ_d	kg/m^3	DENSITY OF DRY SOIL	w_s	%	SHRINKAGE LIMIT	q	m^3/s	RATE OF DISCHARGE
γ_d	kN/m^3	UNIT WEIGHT OF DRY SOIL	I_p	%	PLASTICITY INDEX = $w_L - w_p$	v	m/s	DISCHARGE VELOCITY
ρ_{sat}	kg/m^3	DENSITY OF SATURATED SOIL	I_L	1	LIQUIDITY INDEX = $\frac{w - w_p}{I_p}$	i	1	HYDRAULIC GRADIENT
γ_{sat}	kN/m^3	UNIT WEIGHT OF SATURATED SOIL	I_C	1	CONSISTENCY INDEX = $\frac{w_L - w}{I_p}$	k	m/s	HYDRAULIC CONDUCTIVITY
ρ'	kg/m^3	DENSITY OF SUBMERGED SOIL	e_{max}	1, %	VOID RATIO IN LOOSEST STATE	j	kN/m^2	SEEPAGE FORCE
γ'	kN/m^3	UNIT WEIGHT OF SUBMERGED SOIL						

RECORD OF BOREHOLE No 1 (1 of 2)

METRIC

W P 112-86-01 LOCATION Sta 126 + 19; 0/s 9.8' Rt 4 Hwy. 15 (Imperial Chainage) ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.); BW Casing COMPILED BY MS
 DATUM Geodetic DATE 88 06 23 to 28 CHECKED BY

OFFICE REPORT ON SOIL EXPLORATION

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	20 40 60 80 100					
94.0	Ground Surface												
0.0	Asphalt, Concrete Gravelly Sand (Fill) Compact		1	SS	20								
91.6	Trace Organic		2	SS	4								0 21 63 16
2.4			3	SS	2								0 0 70 30
			4	SS	2								
			5	SS	0								
			6	SS	0								
			7	SS	0								
			8	SS	0								0 0 64 36
			9	SS	1								
			10	SS	0								
			11	SS	0								
	Clayey Silt Frequent Silty Clay Layers Occasional Sand and Silt Seams		12	SS	0								10 5 54 31
			13	SS	0								
	Trace Sand Gravel (Lacustrine)												
	Firm / Stiff		14	SS	2								2 10 58 30
			15	SS	2								
			16	SS	0								
63.8													
30.2													

Continued

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

Continued



W P 112-86-01 LOCATION Sta 126 + 19; 0/s 9.8' Rt ~~E~~ Hwy. 15 (Imperial Chainage) ORIGINATED BY MS
DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.); 8W Casing COMPILED BY MS
DATUM Geodetic DATE 88 06 23 to 28 CHECKED BY 741

+3, x5: Numbers refer to Sensitivity

OFFICE REPORT ON SOIL EXPLORATION

METRIC

W P 112-86-01 LOCATION Sta. 124 + 27; 0° 17' Lt 4 Hwy. 15 (Imperial Chainage) ORIGINATED BY MS
DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
DATUM Geodetic DATE 88 06 29 - 30 CHECKED BY MS

[illegible]

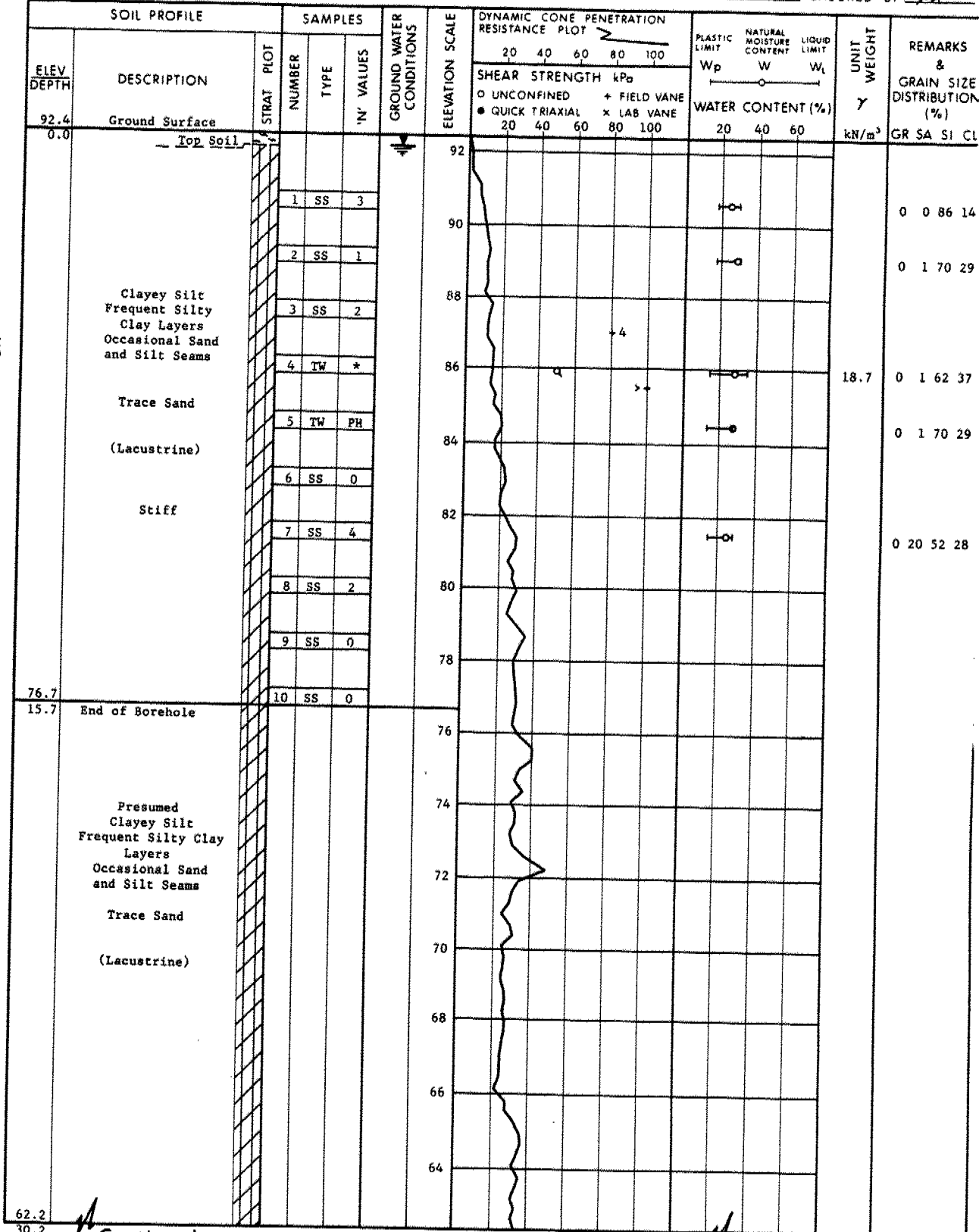
* Pushed by Self-Weight

+3, x5: Numbers refer to Sensitivity

RECORD OF BOREHOLE No 3 (1 of 2) METRIC

W P 112-86-01 LOCATION Sta 127 + 41; 0/s 43' Lt. Hwy. 15 (Imperial Chainage) ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
 DATUM Geodetic DATE 88 06 30 & 88 07 04 CHECKED BY *MS*

OFFICE REPORT ON SOIL EXPLORATION



Continued

* Pushed by Self-Weight

+3, x5: Numbers refer to Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE

Continued



W P 112-86-01 LOCATION Sta 127 + 41; 0/s 43' Lt. Hwy. 15 (Imperial Chainage) ORIGINATED BY MS
DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
DATUM Geodetic DATE 88 06 30 & 88 07 04 CHECKED BY M

[illegible]

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to Sensitivity

15 ϕ 5 (%) STRAIN AT FAILURE

RECORD OF BOREHOLE No 4

METRIC

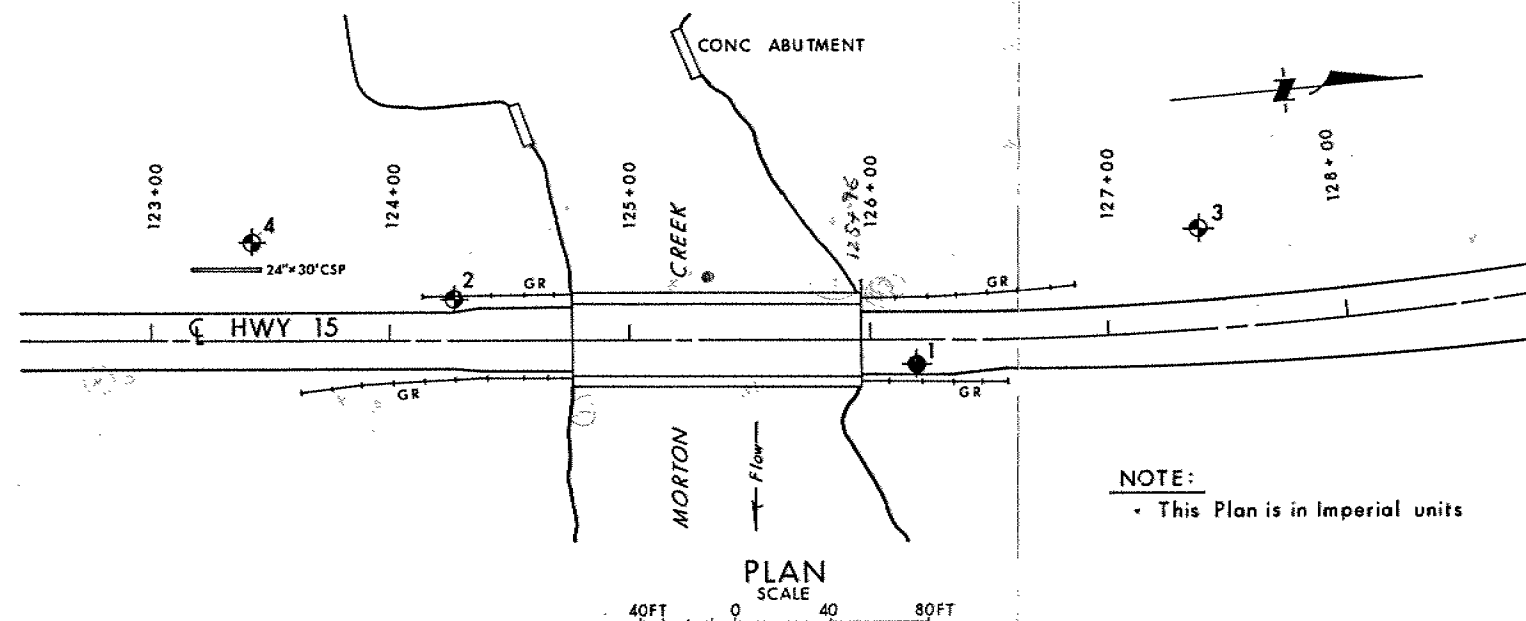
W P 112-86-01 LOCATION Sta 123 + 43.0 / s 41' Lt 6 Hwy. 15 (Imperial Chainage) ORIGINATED BY MS
 DIST 8 HWY 15 BOREHOLE TYPE Continuous Flight Auger (H.S.) & Cone Test COMPILED BY MS
 DATUM Geodetic DATE 88 07 04 CHECKED BY *AI*

OFFICE REPORT ON SOIL EXPLORATION

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 Y	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
94.1	Ground Surface													
0.0	Gravelly Sand (Fill)													
92.6	Very Loose to Compact*													
1.5	Trace Organics		1	SS	2									
	Clayey Silt		2	SS	2									0 11 58 31
	Frequent Silty Clay Layers		3	SS	0									
	Occasional Sand and Silt Seams		4	SS	0									
	Trace Sand		5	SS	0									
	(Lacustrine)		6	SS	14									0 0 77 23
	Firm / Stiff													
84.5	End of Borehole													
9.6	Presumed Bedrock													
	* Trace Organics													

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



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

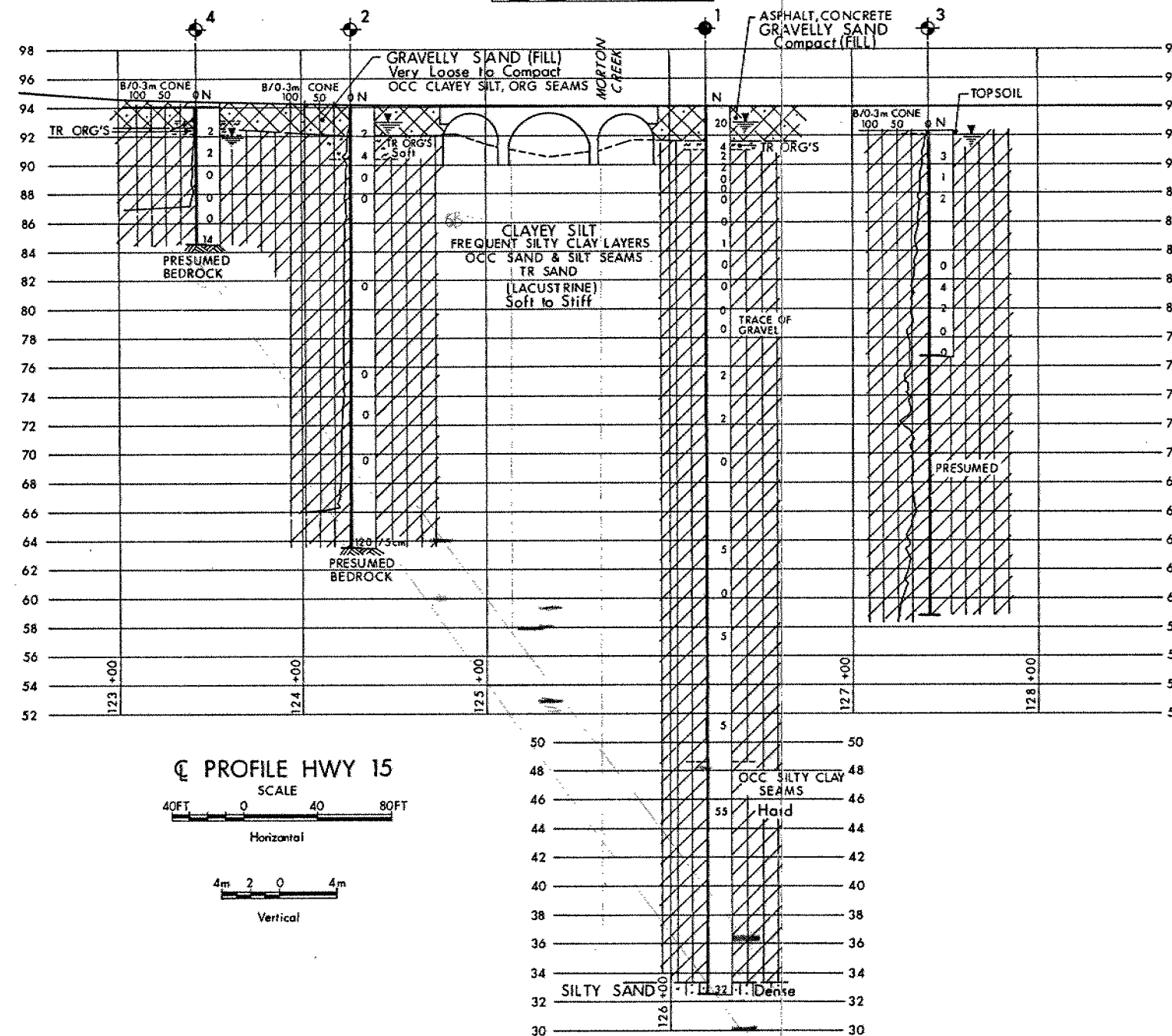
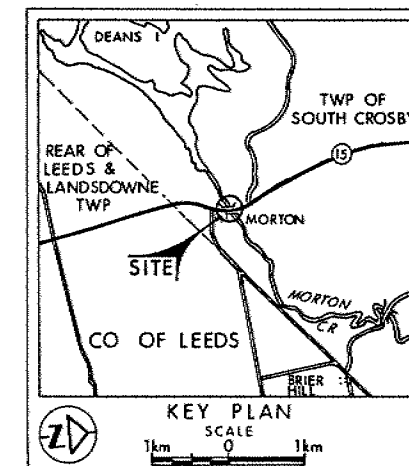
CONT No
WP No 112-86-01

MORTON CREEK BRIDGE

BORE HOLE LOCATIONS & SOIL STRATA



SHEET



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
88 06 and 88 07

No	ELEVATION (m)	IMPERIAL STATION	CHAINAGE OFFSET
1	94.0	126+19	9.8 ft RT
2	94.0	124+27	17.0 ft LT
3	92.4	127+41	43.0 ft LT
4	94.1	123+43	41.0 ft LT

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.

REV.	DATE	BY	DESCRIPTION

Geocres No 31C-144

HWY No 15	DIST 8
SUBWD PM	CHECKED 89 07 06
DRAWN DT	CHECKED 89 07 06

SITE 16-45
DWG 1128601-A

memorandum



To: E.C. Lane
Head, Structural Section
Eastern Region

Date: 89 12 18

Attn: H.S. Kleywegt

From: Foundation Design Section
Room 315, Central Building

Re: Morton Creek Bridge
W.P. 112-86-01, Site 16-45
Hwy. 15, District 8, Kingston

Further to your memo dated December 13, 1989, we understand that the current proposal for this site is to incorporate the existing foundations into the reconstructed bridge.

Based on our review of Drawing No. D 2243-1, there are few details regarding the piles. Hence, it would be extremely difficult to comment on permissible loadings for these foundations. However, as you have indicated that no discernible differential settlement has occurred at the existing structure, it would be reasonable to apply the same loading conditions for the rehabilitated bridge (loadings and distributions).

If there are any questions, please advise.

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

DHD/jb

memorandum



545-4715

To: Murty Devata
Chief Foundation Engineer
Foundation Design Section
Engineering Materials Office
Downsview, Ontario

From: Structural Section
Eastern Region
Kingston, Ontario

Re: Morton Creek Bridge, Site 16-45
W.P. 112-86-01, Highway 15
District 8 - Kingston

Date: December 13, 1989

It appears there may be considerable economy in replacing only the superstructure of the captioned bridge. Accordingly, it is desired to incorporate the existing foundations into the reconstructed bridge.

The enclosed plan details the timber pile layout for the piers and abutments. There is no record as to the diameter or length of the piles.

A recent survey to investigate differential settlement of the structure revealed that no discernable differential settlement has taken place.

Based on this limited information, plus the information in the Preliminary Foundation Report, would you please comment on the permissible loadings on the existing foundations.

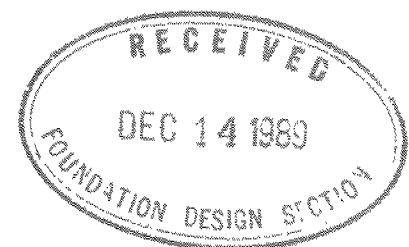
As usual, your early attention to this matter would be appreciated.

A handwritten signature in dark ink, appearing to read "H. Kleywegt".

H.S. Kleywegt
for:
E.C. Lane
Head, Structural Section

ECL:HSK:bd

Encl.



PRELIMINARY FOUNDATION INVESTIGATION REPORT

For

Morton Creek Bridge Replacement

W.P. 112-86-01, Site 16-45

Highway 15, District 8, Kingston

INTRODUCTION

This report contains the results of the preliminary foundation investigation carried out for the proposed detours and the three alignment alternatives for the replacement structure. The fieldwork was carried out during the period from 88-06-23 to 88-07-04. The fieldwork consisted of four sampled boreholes with dynamic cone penetration tests adjacent to three of them. The borings were advanced by hollow stem auger and BW casing (44 mm I.D.) using a machine mounted on a muskeg vehicle. Sampling was performed to a maximum depth of 61.4 m to an elevation of 32.6 m and the cone tests to a maximum depth of 33.3 m to elevation 59.1 m.

SITE DESCRIPTION

The site is located just north of Morton on Highway 15 at Morton Creek in the Township of South Crosby in the County of Leeds. The terrain in the immediate vicinity is hilly with marshlands adjacent to the creek between rock ridges. The physiological region is Leeds Knobs and Clay Flats and typically consists of knobs of granite and other Precambrian rocks with areas between filled with very weak calcareous clay left by the Champlain Sea.

SUBSURFACE CONDITIONS

General

The subsoil was found to consist of a soft to stiff lacustrine clayey silt, with frequent silty clay layers, occasional sand and silt seams, trace sand, gravel. The sampling ended within this deposit except for borehole 1, where it was underlain by silty sand. The depth of overburden is highly variable, ranging from 9.6 to 61 m+. The presumed bedrock elevation at borehole 4 was 84.5 m however approximately a metre away the cone test indicated the bedrock elevation at 86.8 m. This indicates a sloping and unpredictable bedrock topography.

The plan, location and stratigraphy of the borings are shown on drawing number 1128601-A and the obtained field and laboratory tests plotted on the Record of Borehole sheets 1 through 4 are attached.

A brief description of the different strata is given below.

Fill Material

Fill material was found at the surface to elevation 92.6 (borehole 4) and to 91.6 m (borehole 1). The fill ranged in thickness from 2.4 m along the existing road to 0 m at borehole 3 which was off the road. The fill was mainly pavement and shoulder material consisting of asphalt, concrete and sandy silt to gravelly sand. The fill also contained occasional clayey silt and there were traces of organics in the bottom 0.3 m of fill. The denseness of the fill ranged from very loose to compact.

Clayey Silt

This deposit was either immediately below the fill or the surficial deposit. The material consisted of a lacustrine deposit of clayey silt, with frequent silty clay layers, occasional sand and silt seams, trace of sand, and trace of gravel. This deposit extended to the termination of sampling in all but borehole 1 where, at approximate elevation 33.3 m, silty sand was encountered in the last sample.

The physical properties of the material as determined by field and laboratory tests are summarized below:

	<u>Range</u>	<u>Average</u>
Natural Moisture Content (w)	22-60%	33%
Liquid Limit (w_L)	18-51%	32%
Plastic Limit (w_p)	13-23%	16%
Unit Weight ()	17.3-20.4 kN/m ³	18.8 kN/m ³

The results above have been plotted on the Plasticity Chart, Figure 1. The majority of the tests plot as a clayey silt of low plasticity, with occasional tests indicating silt seams of low plasticity, or layers of silty clay of intermediate plasticity.

Following is a summary of strength characteristics of this material based on unconfined compression tests of shelby tube samples and on in situ field vane tests.

	<u>Range</u>	<u>Average</u>
Shear Strength (based on Compression Test)	15-50 kPa	31.5 kPa
Shear Strength (based on Field Vane)	12-96+ kPa	43 kPa

The N-blows from the standard penetration tests average at one, ranging from 0 to 5 with one test each indicating 14 and 55. These results, in addition to those from the tests shown above, indicate that the consistency of the deposit ranges from soft to hard. The majority of the material was however firm.

The following are results of consolidation tests carried out in the lab:

	<u>Range</u>	<u>Average</u>
Initial Voids Ratio (e_0)	0.96-1.18	1.06
Preconsolidation (P_c)	111-297 kPa	216 kPa
Compression Index (C_c)	0.45-0.66	0.54

The deposit under the existing alignment appears to be normally consolidated whereas the material sampled off the road is slightly overconsolidated.

The results from the grain size distribution tests are shown in envelope form in Figure 2.

Silty Sand

At the bottom of borehole 1 a deposit of dense silty sand was found at approximate elevation 33.3 m (60.7 m below the ground surface), to the bottom of the borehole at elevation 32.6 m.

Bedrock

Bedrock was not proven by rock coring. Bedrock can therefore only be presumed to be below the overburden material at or below the following estimated elevations:

<u>Borehole</u>	<u>Presumed Bedrock Elevations (m)</u>		<u>Bedrock Below Elevation (m)</u>
	<u>Borehole</u>	<u>Cone Test</u>	
1	-	-	32.6
2	63.5	65.7	63.5
3	-	-	58.8
4	84.5	86.8	84.5

It must be noted that the bedrock elevations are highly variable, as indicated by the large differences in the elevations indicated by the boreholes and their respective cone tests which are approximately a metre apart.

Groundwater Conditions

The following groundwater conditions were observed during the field investigation:

<u>Borehole</u>	<u>Elevation (m)</u>
1	93.0
2	93.0
3	92.1
4	92.0

The boreholes indicate the groundwater level is at approximate elevation 93.0 m. This level will most likely vary seasonally and with fluctuations in the creek.

DISCUSSION AND RECOMMENDATIONS

General

At this preliminary stage there are four alternatives under consideration by the Eastern Regional Structural Section for the structure replacement and detour, as follows:

- (1) To construct a new structure on the west side of existing structure with the detour using existing structure.
- (2) To construct a new structure on the east side of existing structure with the detour using existing structure.
- (3) To provide a one-lane detour using a bailey bridge on the east side of existing structure and to replace the existing structure.
- (4) To provide a two lane detour using a bailey bridge on the east side of existing structure and to replace the existing structure.

The location of piers and abutments for the structures alternatives have not been provided for this preliminary assessment.

Proposed Structure Replacement and Detour

Both the proposed structure and detour alternatives may be supported by friction piles. Granular pads were considered for supporting the weight of the detour bailey bridge however it was determined that the bearing capacity of the foundation soil was not sufficient. The design values for the pile capacities may be conservative due to the uncertainties inherent in friction piles in this subsoil. After the final investigation, it may be determined that full scale pile load testing may be required at the site and at that time pile capacities may be revised. The following foundation recommendations are provided for preliminary planning purposes and are subject to change following the foundation investigation for this project.

For the purposes of the O.H.B.D.C., and assuming an embedded length of 15 m to elevation 77 m, the following values are recommended for No. 36 timber piles. The piles should be treated with preservative.

Factored Axial Capacity at U.L.S. 150 kN

Axial Capacity at S.L.S. Type II 100 kN

Due to the concerns about differential settlement, it is imperative that the structures are not supported partially by piles end bearing on bedrock and partially by friction piles. This concern will be reviewed when actual footing locations are provided during the foundation investigation.

The pile caps should have a minimum of 1.5 m earth cover for frost protection.

Earth pressure acting on abutments, retaining and wingwalls should be computed as per subsection 6.6.1.2.2 of the O.H.B.D.C..

A yielding foundation condition should be assumed. The following properties of granular backfill may be used for computations:

Granular 'A'	$\gamma = 22.8 \text{ kN/m}^3$	$\phi = 35^\circ$	$K_a = 0.271$
Granular 'B'	$\gamma = 21.2 \text{ kN/m}^3$	$\phi = 30^\circ$	$K_a = 0.333$

Concrete should be placed in the 'dry'. Dewatering is not anticipated to be a problem although a dewatering scheme will be required for footing excavations below the groundwater level (river level).

Approach Embankments

Topsoil and surficial material should be removed prior to placing any fill. The fill should consist of well compacted acceptable material. Particle sizes in the fill immediately beneath the pile locations should not exceed 50 mm, to facilitate pile driving.

The side and forward slopes should not be steeper than two horizontal to one vertical designed and constructed in accordance with the appropriate Ministry

Standards. The safe height of fill has been estimated at 2.0 m. Although no additional fill is proposed along the existing alignment it should be noted that the existing embankment height is already 2 m and therefore the existing profile grade of the existing highway should not be raised. When the final investigation has been completed and existing and proposed profiles are received further slope stability analyses will be carried out.

Settlement

Settlement evaluations were based on two consolidation test results. The Schmertmann Method was used to correct for the effects of sample disturbance in the determination of the Preconsolidation Pressure (see Figure 3). Stress distribution was calculated using both the Purdue and Boussinesq methods. In calculating the consolidation settlements, a base width of 10 m was assumed for the imposed load and 30 m for the effective thickness of the deposit. For the time rate of settlement calculations the coefficient of consolidation was calculated using the Root Time Method (after Taylor) assuming drainage in two directions which is consistent with observations at similar sites.

Based on the above calculations, settlement is to be expected along all approaches. If the highway is re-aligned and a 2 m embankment is required, settlement in the order of 15 to 20 cm can be expected, (more settlement is expected on the east side rather than the west). It is predicted that 90% of this settlement will occur within 12 years. At least 50% of this total settlement is expected however to occur within a period of 2.5 years.

The calculations also indicated that settlement may be minimized by maintaining Highway 15 along its existing alignment, since most of the settlement has already occurred. Settlement in the order of 2 to 4 cm can however be expected along the existing alignment with no grade raise. Assuming that the existing road was constructed in the late 1950's and thus has been loaded for 30 to 35 years this settlement should occur over the next several years provided no additional loads are applied.

It has been reported by H. Kleywegt of the Eastern Region Structural Section that very large settlements in the order of 2.5 m have occurred at this site. Although our field investigation and subsequent analysis has indicated the smaller settlements indicated in this report, further analyses may be required during the foundation investigation phase of this project in order to verify these predictions.

Replacing the existing structure will not solve the settlement problems at the north approach. To eliminate settlement, subexcavation of up to 2 m and backfilling with lightweight fill should be considered on the north side. One alternative which has proven to be most cost efficient in the past is blast furnace slag. Estimates for the cost of two types of this material follow:

	Approximate Costs <u>Including Transportation</u>
'Old Clinker' slag	\$50/m ³
pelletized '3/8" Structural Coarse' slag	\$56/m ³

Settlement along a new alignment can not be eliminated without surcharging the approaches which will not be feasible in this case.

Since the request made by the Eastern Region Structural Section for a preliminary investigation indicated that the existing structure is showing no apparent distress from differential settlement, and that the main concern is excessive settlement at the approaches, consideration could be given to other options rather than replacing the structure. For example, the existing structure could be rehabilitated and the approaches could be reconstructed with lightweight material as described above.

MISCELLANEOUS

It must be noted that these recommendations are preliminary and conditions and recommendations may change when the final investigation takes place.

The fieldwork for this preliminary investigation was carried out under the supervision of M. Schnarr, Student. The equipment was owned and operated by Marathon Drilling Co. Ltd. This report was prepared by P. Marks under the supervision of D. Dundas, Senior Foundation Engineer and reviewed by M. Devata.



A handwritten signature in cursive script, appearing to read "P. Marks".

P. Marks, P.Eng.
Foundation Engineer

A handwritten signature in cursive script, appearing to read "M. Devata".

M. Devata, P.Eng.
Chief Foundation Engineer

It is anticipated that the final foundation investigation will be requested late this year when the structure configuration is decided upon.

A handwritten signature in dark ink, appearing to read 'H. Kleywegt', with a long horizontal flourish extending to the right.

H. Kleywegt
For
E.C. Lane
Head, Structural Section

ECL/HK/dka

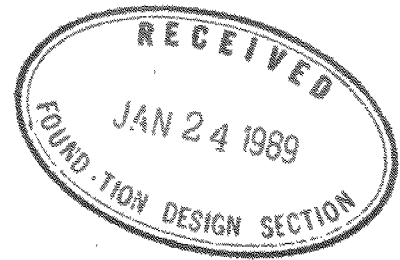
(613) 544-2220, ext. 4169

1989 01 10

Mr. H. Kleywegt
Structural Engineer
Structural Section
Eastern Region, Kingston

From: Geotechnical Section
Eastern Region, Kingston

RE: W.P. 112-86-00, Highway 15
Morton Creek Bridge
District 8, Kingston



As detailed in the project review meeting of 1988 11 23, deep organic and marine clay deposits are expected to be encountered within the project limits. This material has resulted in asphalt padding on the approaches to the structure up to 2.5 m in thickness, which is indicative of a long term settlement problem.

I have discussed with Mr. Dundas the possibility of having the Foundation Section conduct the field investigation for the structure approach fills in conjunction with their investigation for the structure footings. Mr. Dundas has agreed to extend the structure footings investigation outwards to a point where soils conditions are such that the Geotechnical Section can then continue the investigation.

Please include the Geotechnical Soils Investigation Request with your submission to the Foundation Section for the structure investigation.

Original Signed

by

R. J. Scott
Pavement Design and
Evaluation Officer

RJS/hs

c.c.: D. Dundas
T. Comfort



HWY 15
MORTON CREEK BRIDGE
SITE 16-45

PHOTO 1

LOOKING SOUTH
TOWARDS VILLAGE
OF MORTON

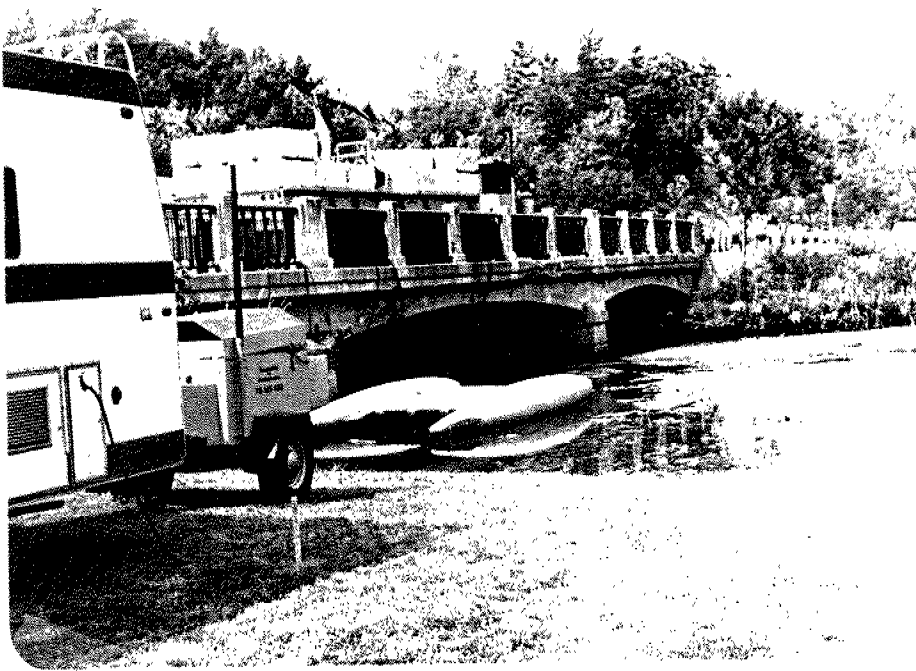


PHOTO 2

LOOKING NORTH
FROM EAST SIDE



PHOTO 3

LOOKING SOUTH
AT SOUTH WEST
BANK, OLD
BRIDGE ABUTMENTS
TO RIGHT,

HWY 15
MORTON CREEK BRIDGE
SITE 16-145



PHOTO 4

LOOKING NORTH
FROM WEST SIDE
OF BRIDGE



PHOTO 5

LOOKING EAST

