

G.I.-30 SEPT. 1976

GEOCRES No. 3165-133DIST. 9 REGION W.P. No. 46-78-02CONT. No. 80-14W. O. No. STR. SITE No. 3-335HWY. No. 17LOCATION Eastern Driveway Underpass
Lot R, Conc. 1, Top of GloucesterNo. of PAGES -

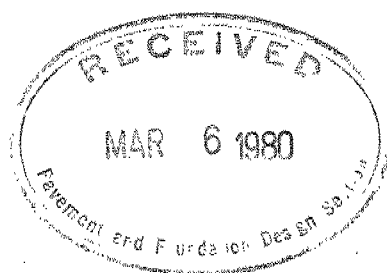
OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

FOUNDATION INVESTIGATION REPORT

CONTRACT NO 80-14



Ministry of
Transportation and
Communications



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NOTE: For purposes of the contract this report supercedes all other foundation reports prepared by or for the Ministry in connection with the above mentioned project.

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_f	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

Eastern Driveway Underpass
Lot 12, Conc. 1, Gloucester Township
Hwy. 17, District 9, Ottawa
W.P. 46-78-02, Site 3-335

INTRODUCTION

This report contains the results of a foundation investigation carried out at the above site. The fieldwork was carried out on 1978 04 24 and between 1979 01 03 to 1979 01 20 and consisted of four sampled boreholes. The borings were advanced by hollow stem augers to a depth of up to 30 metres below ground surface; thereafter washboring techniques were employed to advance the borings to bedrock at a depth of up to 54 metres below ground level. Diamond drilling techniques were then utilized in two boreholes to obtain up to 2.7 metres of BXL size rock core samples.

DETAILS OF RECENT EARTHWORKS

Recently extensive earthworks, including the Eastern Driveway approach fills plus surcharge, were carried out in this area during 1979 under the M.T.C. Contract 79-93. Accordingly topography and surficial subsoil conditions in the area have been altered since obtaining the factual data included in this report. The descriptions in this report, the Borehole Log Sheets and the Contract Drawing No. 3-335-2 reflect the ground topography and subsurface conditions at the time of the fieldwork, 1978 04 and 1979 01, prior to the grading operation.

SITE DESCRIPTION AND GEOLOGY

This site is located on Hwy. 17 approximately 1.2 km east of the Montreal Road/Hwy. 17 intersection in the Township of Gloucester, Regional Municipality of Ottawa-Carleton.

In the site vicinity the terrain is generally flat and the land presently is utilized for agriculture purposes. Existing Hwy. 17 at this location has a profile grade at approximate elevation 53 and ditch grades at elevation 51, whereas the surrounding terrain has an average ground elevation of 53.

Physiographically, the site is located within the Ottawa Valley Clay Plains. This region is characterized by extensive deposits of sensitive marine clay overlying glacial till and limestone bedrock.

SUBSURFACE CONDITIONS

General

Subsoil conditions across the site are generally uniform. Beneath a thin veneer of topsoil extending to a depth up to 47 metres below the ground surface is the dominant deposit consisting of firm to very stiff sensitive clay. The clay overlies 8 to 10 metres of a compact to very dense glacial till composed of a heterogeneous mixture of silt, sand, gravel and occasional cobbles. The upper 1.7 metres of this deposit contains cobbles and boulders up to 0.9 metres in size. Underlying the glacial till is sound dolomite bedrock.

The boundaries between the various subsoil and bedrock types are shown on the Record of Borehole Sheets. The locations and elevations of the boreholes, as well as a stratigraphical profile inferred from borehole data, is shown on Contract Drawing No. 3-335-2.

Following is a brief description of subsoil and bedrock types.

Sensitive Clay

This deposit was encountered immediately below a thin veneer of topsoil and is estimated to have a thickness ranging from 44.5 to 47.3 metres. The deposit is generally grey in colour

except for the upper 2.0 to 2.8 metres which is desiccated and has a brown-grey mottled colour. Laboratory and in-situ testing performed on representative samples from this deposit gave the following results.

<u>Geotechnical Properties</u>		<u>In the Upper 2 - 2.8 m Desiccated Zone</u>		<u>Below the Desiccated Zone</u>	
		<u>Range</u>	<u>Average</u>	<u>Range</u>	<u>Average</u>
Natural Moisture Content	(w)%	35-42	39	56-72	67
Liquid Limit	(w _L)%	51-66	60	55-69	64
Plastic Limit	(w _P)%	24-26	25	22-27	25
Plasticity Index	(I _P)%	25-42	29	32-44	39
Liquidity Index	(I _L)%	0.2-0.5	0.4	0.9-1.3	1.1
Bulk Unit Weight	(γ) kN/m ³			15-16	16
<u>Undrained Shear Strengths (c_u) kPa</u>		<u>Range</u>	<u>Sensitivity</u>	<u>Range</u>	<u>Sensitivity</u>
Field Vane Test		> 96		31 to >96	5 to 19
Unconfined Compression Tests		132		41 to 105	
<u>Consolidation Tests</u>				<u>Range</u>	
Degree of Preconsolidation (σ _p ' - σ _{vo} ') kPa				180 - 240	
Initial Void Ratio (e)				1.76 - 240	
Coefficient of Consolidation (Cc)				1.20 - 1.58	

The results of the Atterberg Limit testings are shown on the Plasticity Chart (Figure 1). These results indicate that the deposit is inorganic and of high plasticity (CH zone). The deposit may be described as a sensitive clay as evidenced by the high sensitivities (5 to 19) measured by the field vane testing. Furthermore, the Liquidity Index of the cohesive soil below the desiccated zone is generally greater than 1 confirming that the non-desiccated cohesive deposit is sensitive to remoulding.

Four consolidation tests were performed on samples from this deposit and are summarized on Figure 3. The consolidation testing indicates that the deposit has been preconsolidated by a pressure of 180 to 240 kPa in excess of the existing overburden pressure. The high initial void ratio, 1.76 to 1.98, together with the high coefficient of consolidation, 1.20 to 1.58 indicates that upon loading in excess of the preconsolidation pressure, the deposit will undergo significant consolidation.

The results of in-situ vane testings are summarized on the Shear Strength vs. Depth Profile, Figure 4. The vane testing indicates that the undrained shear strength is greater than 96 kPa in the upper 2.0 to 2.8 metre desiccated zone and thereafter decreases abruptly to a low of 31 kPa at a depth of 3.5 metres. The shear strength then gradually increases with depth to greater than 96 kPa at a depth of about 15 metres below ground surface. Based on these results the consistency is described as very stiff in the upper desiccated zone and below this it is generally firm to very stiff.

Glacial Till

The glacial till stratum was found immediately below the sensitive clay deposit and is estimated to have a thickness of 8 to 10 metres. The deposit is composed of a heterogeneous mixture of silt, sand, gravel and occasional cobbles. Furthermore, the upper 1.7 metres of this stratum contains cobbles and boulders up to 0.9 metres in size. The results of grain size distribution testing performed on samples obtained from a 2" O.D. split spoon sampler from this deposit are plotted on Figure 2.

Based on the Standard Penetration Test 'N' values ranging from 25 to 94 blows per foot, the relative density of the deposit is described as compact to very dense, generally being compact to dense.

Dolomite Bedrock

Sound bedrock was encountered immediately below the glacial till. In two boreholes bedrock was proven by obtaining up to 2.7 metres of BXL rock core. The bedrock surface was found to vary from 53.5 to 53.9 metres below ground surface which corresponds to elevation -0.9 to -1.7 metres. The bedrock is composed of hard, fine textured, medium grey dolomite which is generally sound.

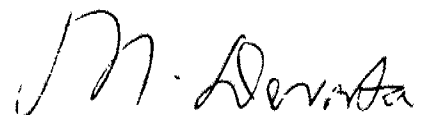
Groundwater Conditions

Groundwater conditions were observed by measuring the water level in the open boreholes 24 hours after completion of the borehole. The groundwater level was found to be 0.8 to 3.4 metres below ground surface which corresponds to elevation 49.3 to 51.0.

Soil Parameters

The following soil parameters may be used for the design of temporary earthworks or supports.

<u>Type of Material</u>	<u>Elevation</u>	<u>c kPa</u>	<u>ϕ°</u>	<u>γ kN/m³</u>
Rock Fill Material		0	35	22
Granular Fill Material		0	30	20
Clay	53 - 51	96	0	16
Clay	51 - 50	55	0	16
Clay	50 - 48	38	0	16
Clay	48 - 46	58	0	16
Clay	46 - 42	70	0	16



M. Devata, P. Eng.
Senior Foundations Eng.

January, 1980

APPENDIX

Previous B.H. 4 W.P. 911-73-00
METRIC

W P 46-78-02 LOCATION Coords. N 5 035 596.6; E 377 247.6 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger (0-30m) Washboring (30-54 m) COMPILED BY MM
DATUM Geodetic DATE 1978 04 24 CHECKED BY RS

Continued

+3, x5: Numbers refer to Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION



RECORD OF BOREHOLE No 1 Continued

Previous B.H. 4
METRIC W.P. 911-73-00

W P 46-78-02 LOCATION Coords. N 5 035 596.6; E 377 247.6 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger (0-30 m) Washboring (30-54 m) COMPILED BY MM
DATUM Geodetic DATE 1978 04 24 CHECKED BY RS

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			20	40	60	80	100					
22.0	Sensitive Clay Grey Very Stiff						20										
30.0							18										
							16										
							14										
							12										
							10										
							8										
7.5	Some Cobbles - Heterogeneous Mixture Silt, Sand and Gravel Compact to Dense (Glacial Till)						6										
44.5							4										
							2										
							0										
-2.0	End of Borehole Refusal to Tricone Probable Bedrock						-2										
54.0																	

+³, x⁵: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No2 METRIC

W P 46-78-02 LOCATION Coords. N 5 035 600.8; E 377 229.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-12 m, Washboring 12-53 COMPILED BY MM
DATUM Geodetic DATE 1979 01 09 to 19 CHECKED BY PLS

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				
52.7	Ground Level															
0.0	Very Stiff		1	SS	31		52									
			2	SS	9											
50.2							50									
2.5			3	TW	PH											
	Firm to Stiff		4	SS	V 450 mm		48									
			5	TW	PM											
			6	SS	4		46									
	Very Stiff		7	TW	PH		44									
	Sensitive		8	SS	2		42									
	Clay		9	TW	PH		40									
	Grey						38									
			10	SS	2		36									
							34									
							32									
							30									
			11	SS	4		28									
							26									
							24									
22.7																
30.0																

OFFICE REPORT ON SOIL EXPLORATION

Continued

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 2 Continued METRIC

W P 46-78-02 LOCATION Coords. N 5 035 600.8; E 377 229.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-12 m, Washboring 12-53 COMPILED BY MM
DATUM Geodetic DATE 1979 01 09 to 19 CHECKED BY RS

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	60	80	100					
22.7																	
30.0	Very Stiff Sensitive Clay Grey																
7.4																	
45.3	0.9 m Boulder Heterogeneous Mixture Silt, Sand Gravel & Occasional Cobbles Compact to Dense (Glacial Till)		12	RC	10% Rec												RQD=0%
			13	SS	34												23 37 30 10
			14	SS	25												
-1.0																	
53.7	Sound Dolomite Bedrock		15	RC BXL	91% Rec												RQD=70%
-2.7																	
55.4	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10

OFFICE REPORT ON SOIL EXPLORATION

RECORD OF BOREHOLE No 2A										METRIC			
W P 46-78-02		LOCATION Coords. N 5 035 602.8; E 377 299.4				ORIGINATED BY MM							
DIST 9 HWY 17		BOREHOLE TYPE Hollow Stem Augers, Continuous Vane Tests				COMPILED BY MM							
DATUM Geodetic		DATE 1979 01 16				CHECKED BY RS							
SOIL PROFILE			SAMPLES			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		GROUND WATER CONDITIONS	SHEAR STRENGTH kPa					
52.7	Ground Level												
0.0	Very Stiff												
	Sensitive Clay Firm to Stiff												
47.8													
4.9	End of Borehole												

OFFICE REPORT ON SOIL EXPLORATION

+3, x5: Numbers refer to Sensitivity
20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 3 Continued

METRIC

W P 46-78-02 LOCATION Coords. N 5 035 578.4; E 377 239.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-12, Tricone Ahead 12-54 COMPILED BY MM
DATUM Geodetic DATE 1979 01 20 CHECKED BY 25

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC NATURAL LIQUID LIMIT MOISTURE LIMIT CONTENT			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	W _p	W	W _L		
22.6																	
30.0																	
	Sensitive Clay Grey Very Stiff																
7.5																	
45.1	Cobbles and Boulders																
	Probable Glacial Till Heterogeneous Mixture Silt, Sand and Gravel With Occasional Cobbles																
-0.9																	
53.5	Refusal to Tricone Probable Bedrock End of Borehole																

OFFICE REPORT ON SOIL EXPLORATION

+3, x5 : Numbers refer to
Sensitivity

20
15 \div 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 3 METRIC

W P 46-78-02 LOCATION Coords. N 5 035 578.4; E 377 239.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers, 0-12, Tricone Ahead 12-54 COMPILED BY MM
DATUM Geodetic DATE 1979 01 20 CHECKED BY RS

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					
52.6	Ground Level																
0.0	Very Stiff		1	SS	15		52									17.5	
			2	TW	PH												
50.1							50										
2.5			3	SS	2												
	Firm to Stiff		4	TW	PH		48									15.6	
	Sensitive																
	Clay		5	SS	2		46										
	Grey		6	TW	PM		44										
			7	SS	7		42									16.1	
			8	TW	PM		40										
			9	SS	3		38										
							36										
	Very Stiff		10	SS	4		34										
							32										
							30										
							28										
			11	SS	6		26										
							24										
22.6																	
30.0																	

Continued

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10

RECORD OF BOREHOLE No 4

METRIC

W P 46-78-02 LOCATION Coords. N 5 035 602.8; E 377 229.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-18.3, Washboring 18.3-56.0 COMPILED BY MM
DATUM Geodetic DATE 1979 01 03 to 09 CHECKED BY RS

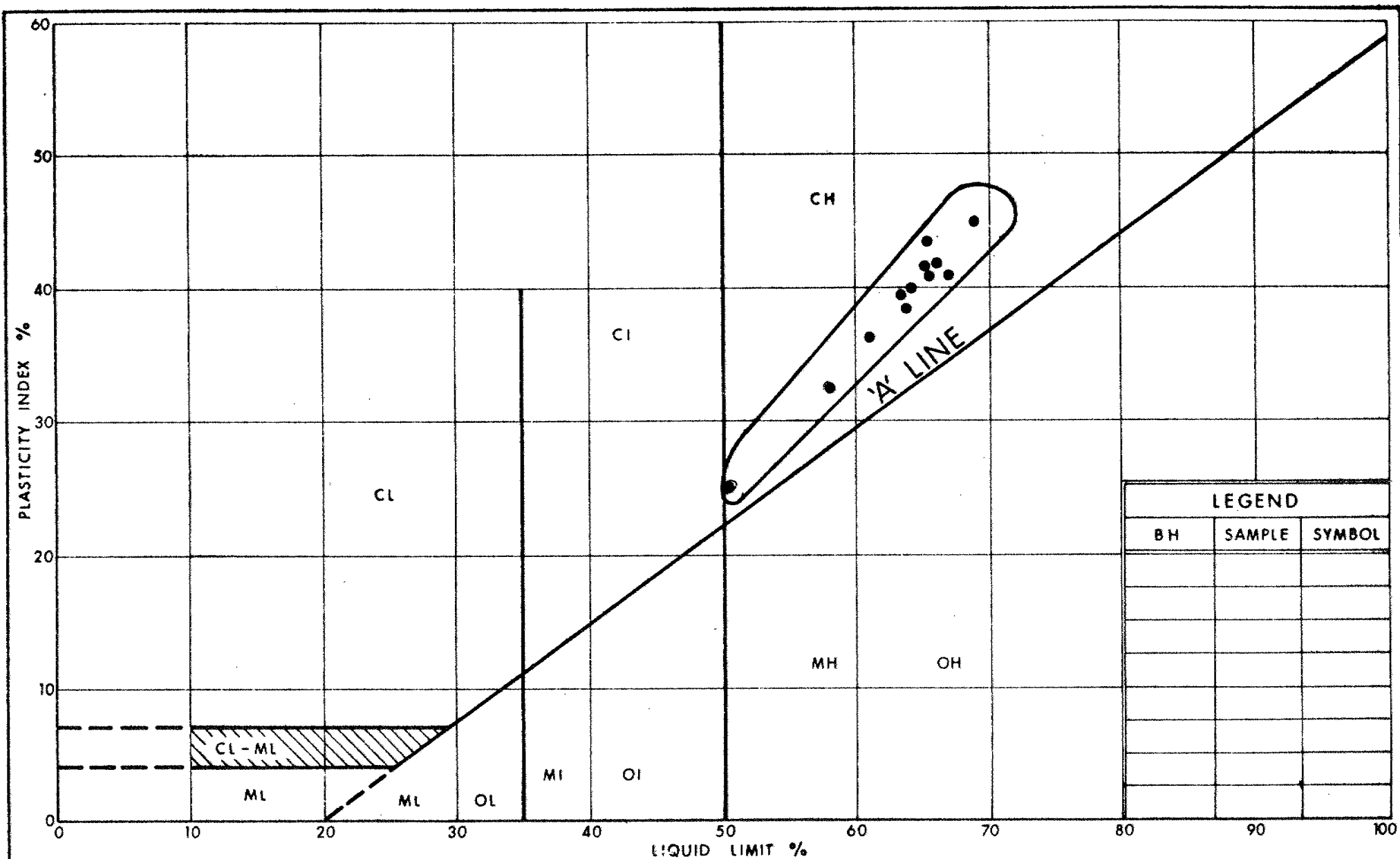
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT W _p	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W _L	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
52.0	Ground Level																
0.0	Very Stiff		1	SS	9		50										
			2	SS	6												
49.2			3	SS	3												
2.8	Firm to Stiff		4	TW	PH		48										
			5	SS	3												
			6	SS	2		46										
	Sensitive		7	TW	PH		44										
	Clay		8	SS	3		42										
	Grey		9	TW	PH		40										
			10	SS	2		38										
			11	TW	PH		36										
			12	SS	2		34										
			13	SS	2		32										
			14	SS	5		30										
							28										
							26										
							24										
22.0																	
30.0																	

OFFICE REPORT ON SOIL EXPLORATION

Continued

+3, x5: Numbers refer to
Sensitivity

20
15-5 (%) STRAIN AT FAILURE
10

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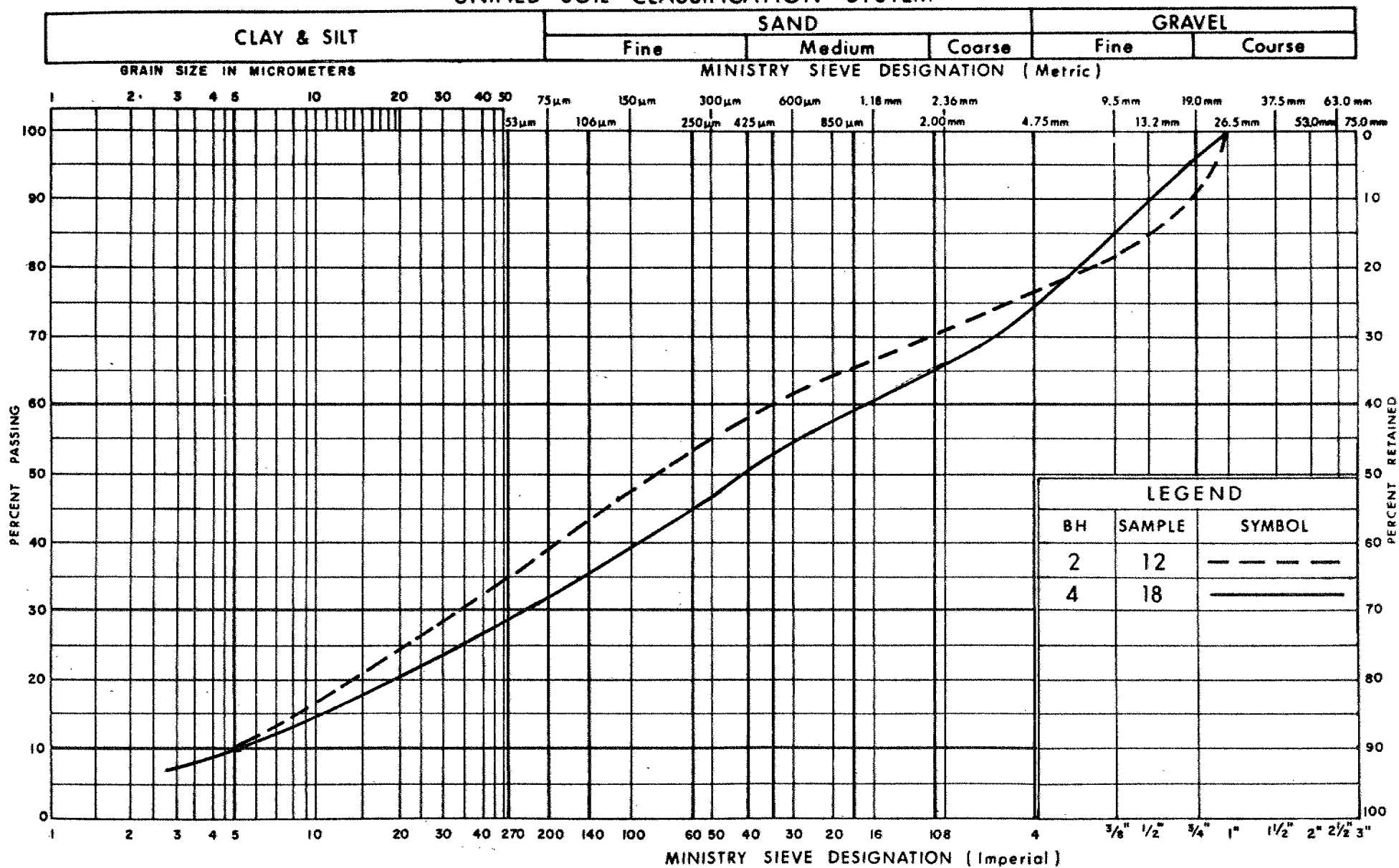
PLASTICITY CHART

SENSITIVE CLAY

FIG No 1

W P 46 - 78 - 02

UNIFIED SOIL CLASSIFICATION SYSTEM



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GRAIN SIZE DISTRIBUTION
GLACIAL TILL

FIG No 2

W P 46 - 78 - 02

VOID RATIO - PRESSURE CURVES

20

WP 46 - 78 - 02

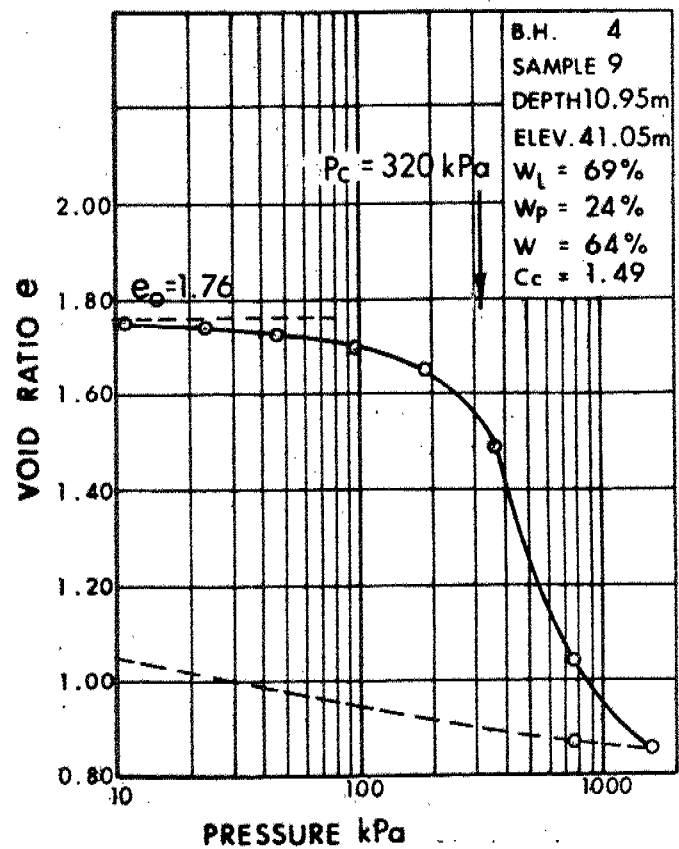
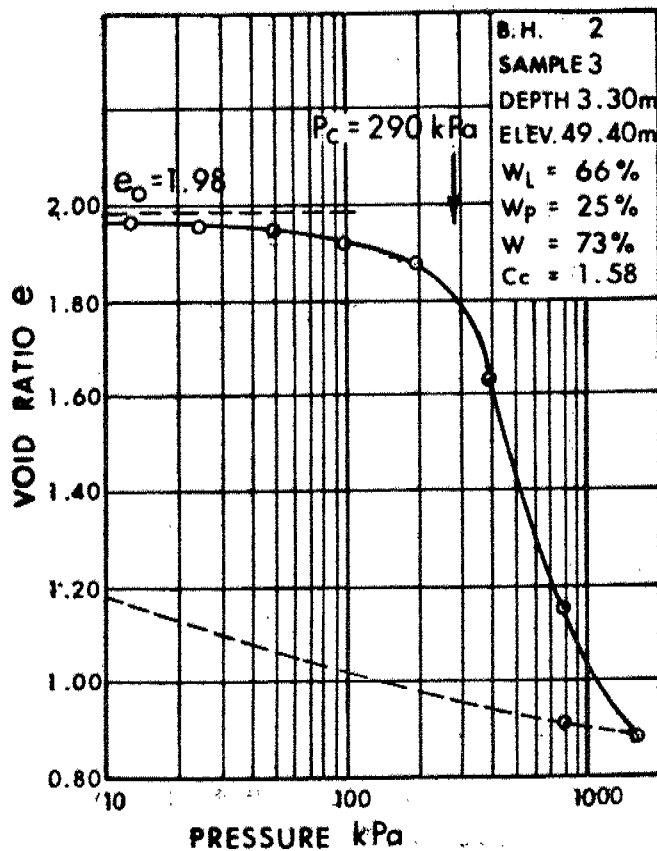
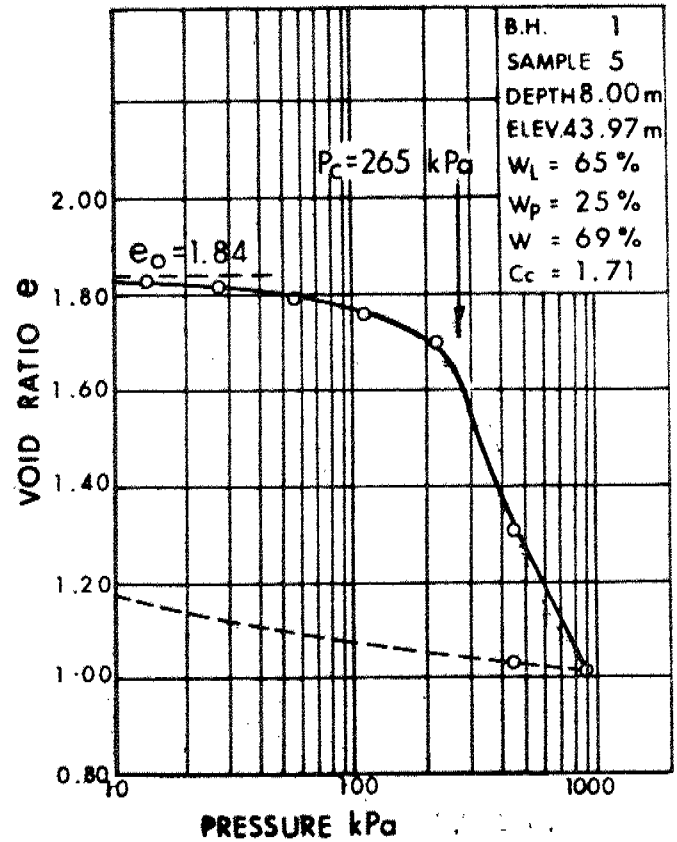
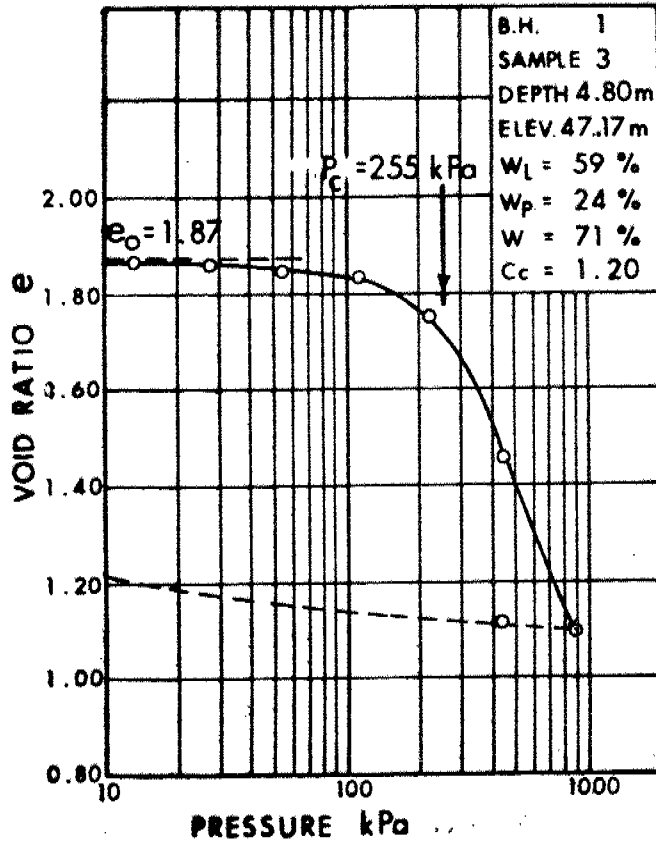
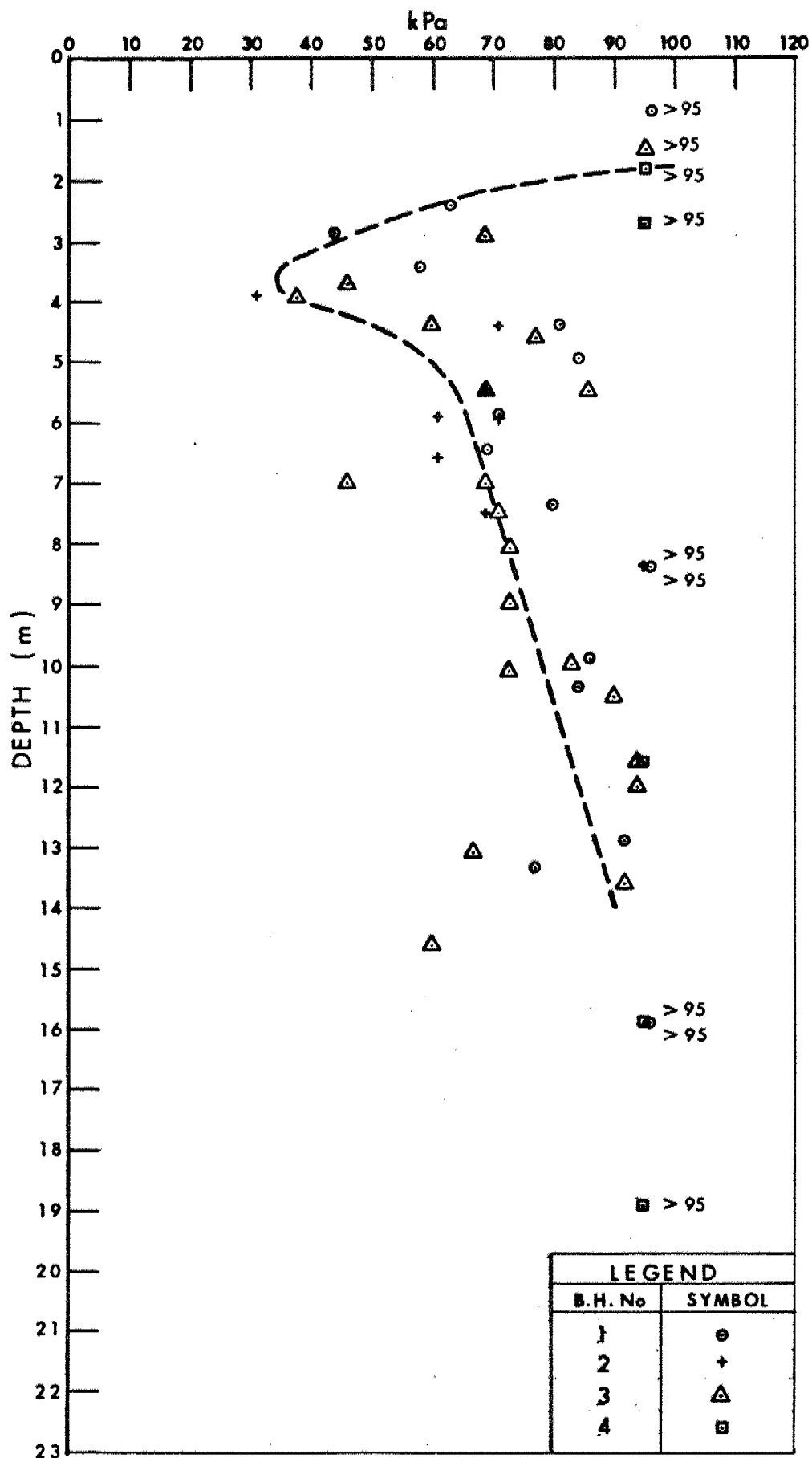


FIG. 3



SHEAR STRENGTH vs DEPTH SUMMARY

WP 46 - 78 - 02

FIG 4



Golder Associates
CONSULTING GEOTECHNICAL ENGINEERS

REPORT

TO

NATIONAL CAPITAL COMMISSION

SUBSURFACE INVESTIGATION

PROPOSED EASTERN PARKWAY CROSSING
OF GREEN CREEK

TOWNSHIP OF GLOUCESTER ONTARIO

Distribution:

6 copies - National Capital Commission
Ottawa, Ontario

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Ottawa, Ontario

December, 1977

772222

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ABSTRACT

This report gives the results of a subsurface investigation carried out at the site of the proposed Eastern Parkway crossing of Green Creek. Recommendations are given for the geotechnical design of the bridge, based on our interpretation of the subsurface conditions encountered in the borings.

The proposed site is underlain by deep deposits of lacustrine and marine clays, which extend to depths of 22 to 26 metres (elevation 58 to 67 feet). This clay has a very stiff to stiff consistency and has been weathered to a grey-brown crust in the upper 2.5 to 3 metres (8 to 10 feet). The clay is underlain by some 1.5 to 3 metres (5 to 10 feet) of glacial till, followed by grey dolomite bedrock. The bedrock was found to be in generally fairly sound to sound condition and has a relatively constant surface level, generally ranging from elevation 55 to 60 feet.

The valley slopes are being actively eroded by Green Creek, leading to small scale slope failures in the area of the site. Recommendations are given in the report for slope flattening to 3 horizontal to 1 vertical and erosion protection to minimize the possibility of further slope movement.

Stability calculations indicate that the east and west abutments for the bridge can be placed at stations 137+60 and 135+10, respectively, provided that free draining granular fill is used for construction of the approach fills. A fill height of 7.5 metres (25 feet) would require 2.5 horizontal to 1 vertical side and front slopes for stability and would create some 152 millimetres (6 inches) of settlement. The amount of settlement would reduce proportionate to a decrease in fill height and would be approximately 65 millimetres (2.5 inches) for a 3 metre (10 foot) high fill.

The proposed bridge could be founded on spread footings placed on the silty clay using an allowable bearing pressure of 190 kilopascals (4,000 pounds per square foot). The settlement of footings designed using this bearing value would be about 50 millimetres (2 inches) in addition to that created by the approach fills. Alternatively, H piles driven to end bearing on the bedrock could be used. The capacity of these end bearing piles would be about 996 kilonewtons (100 tons) for vertical loading and 50 kilonewtons (5 tons) for lateral loading. The vertical load capacity would be decreased by 300 kilonewtons (30 tons) per pile in the abutment areas due to negative skin friction.

December 28, 1977
772222

1. INTRODUCTION

H.Q. Golder and Associates Limited have been retained by the National Capital Commission to carry out a subsurface investigation for the proposed Eastern Parkway crossing of Green Creek. The purpose of this investigation was to obtain an appreciation of the subsoil, bedrock and groundwater conditions at the site by means of a limited number of sampled borings. Recommendations for the geotechnical design of the proposed bridge structure, including special construction considerations which could influence design decisions, have been given based on our interpretation of this data.

2. DESCRIPTION OF PROJECT

The site of the proposed bridge is located on Green Creek some 540 metres (1,800 feet) southwest of the point where it joins the Ottawa River, as shown on Figure 1. The valley slopes in the area of the creek crossing are some 9 metres (30 feet) in height and generally have slope angles ranging from 2 horizontal to 1 vertical, to 4 horizontal to 1 vertical. The initially proposed bridge structure for this site is understood to consist of a 4 span bridge, some 120 metres (400 feet) in length. The overall length and number of spans may, however, be reduced by extending the approach embankment on the west side of the creek.

From published geologic information it is known that the bedrock in the vicinity of the site consists of dolomite and limestone of the Oxford formation. The bedrock is generally overlain by a thin layer of glacial till, followed by sensitive silty clays of marine and freshwater origin, which extend to the ground surface.

3. PROCEDURE

The field work for this investigation was carried out from October 20 to November 8, 1977. During this period five boreholes, numbered 77-1 to 77-5 inclusive, were put down at the site using a bombardier mounted hollow stem auger drill rig supplied and operated by the F.E. Johnston Drilling Co. Ltd., of Ottawa. Boreholes 77-1 to 77-4 were advanced down to the surface of the bedrock at depths of 25 metres (82 feet) to 29 metres (95 feet) below the existing ground surface. Samples of the overburden were obtained at regular intervals in these borings using a standard drive open sampler and 50 millimetre (2 inch) and 76 millimetre (3 inch) diameter thin walled shelby tube samplers. In situ vane tests were also carried out to determine the shear strength characteristics of the clay deposit. The bedrock was cored in BX size for some 3 metres (10 feet) to 4 metres (13 feet). Borehole 77-5 was put down at station 138+65 to check for indications of landslide activity. Continuous 50 millimetre (2 inch) shelby tube samples were pushed to a 6 metre (20 foot) depth in this boring. Piezometers or standpipes were installed at various levels in each of the boreholes to permit monitoring of the groundwater conditions across the site. The field work was supervised throughout by members of our engineering staff.

A detailed log of each of the borings put down at the site during the present investigation and a previous investigation (Golder Associates Report 752038) is given on the Record of Borehole sheets following the text of this report. The locations of the boreholes, together with a section of the inferred soils profile across the site are shown on Figure 2.

The overburden and bedrock samples obtained from the boreholes were brought to our laboratory for detailed examination and testing. The results of the laboratory tests are shown on the Record of Borehole sheets and on Figures 3, 4 and 5.

The locations and ground surface elevations for the boreholes were provided by the survey personnel of the National Capital Commission. The elevations are understood to be referred to Geodetic datum.

4. SUBSURFACE CONDITIONS

The detailed soil, bedrock and groundwater conditions encountered in the boreholes are given on the Record of Borehole sheets and are illustrated on Figure 2. The following is a summarized account of the subsurface conditions at the site.

4.1 Silty Clay

The principal subsoil stratum at this site consists of silty clay, which was encountered from the ground surface down to depths of 22 to 26 metres (i.e. elevation 58 feet to 67 feet). The upper portion of this clay stratum was found to have been weathered to a stiff to very stiff grey-brown crust, which extends to a depth generally ranging from 2.5 to 3 metres (8 to 10 feet). In the area of borehole 77-4, which was put down at station 134+05, the weathered crust was, however, found to extend to a depth of some 4.5 metres (15 feet). In situ vane tests carried out in the lower part of the weathered crust gave shear strength values ranging from about 85 kilopascals (1,700 pounds per square foot) to in excess of 145 kilopascals (3,000 pounds per square foot).

Atterburg limit tests carried out on samples of the weathered clay crust gave liquid limit values of about 80 and plasticity indices of about 55, indicating a highly plastic clay. The natural water content ranges from about 40 to 60 per cent, with the values increasing with depth.

Below the zone of weathering the colour of the silty clay changes from grey-brown to grey. The consistency of the grey clay immediately below the weathered crust is stiff, with measured shear strength values ranging from about 70 kilopascals (1,400 pounds per square foot) to 90 kilopascals (1,900 pounds per square foot). The in situ shear strength of the grey clay increases with depth, with values generally ranging from 80 kilopascals (1,700 pounds per square foot) to in excess of 120 kilopascals (2,500 pounds per square foot) below elevation 130 feet.

Atterburg limit tests carried out on samples of the grey clay gave liquid limit values of about 80 and plasticity indices of about 60 down to about elevation 100 feet indicating a highly plastic clay. Below elevation 100 the plasticity decreases to medium, with liquid limit values ranging from 40 to 50 and plasticity indices of about 20 to 30.

Consolidation tests were carried out on two 75 millimetre (3 inch) diameter samples from boreholes 77-1 and 77-4. The results of these tests are shown on Figures 3 and 4. The consolidation tests indicate that the clay has been subjected to an apparent preconsolidation pressure of about 268 to 311 kilopascals (2.5 to 2.9 tons per square foot) in excess of the existing overburden pressure. The compression indices, C_c , were 1.1 and 0.8 at elevations 106 and 97, respectively. The corresponding recompression indices, C_{cr} , were 0.02 and 0.03.

4.2 Glacial Till

Glacial till was encountered beneath the grey silty clay in boreholes 8 and 77-1 to 77-4. This till deposit was generally some 1.5 to 3 metres (5 to 10 feet) thick and consisted of sand and gravel, with some cobbles and boulders and a trace to some clay. The gradation of a sample of the till from borehole 77-4 is shown on Figure 5. This grading curve does not represent the cobble and boulder size fraction due to the small size of the sampler (i.e. 50 millimetres). The till was found to have a generally loose to compact relative density with 'N' values ranging from about 6 to 20 blows per foot.

4.3 Bedrock

Grey dolomite bedrock was encountered underlying the glacial till at depths generally ranging from 25 metres (elevation 55 feet) to 29 metres (elevation 60 feet). Refusal to augering was experienced at elevation 62 in borehole 8 from the preliminary investigation (Golder Associates Report 752038) but may have occurred on boulders within the glacial till rather than on the surface of the bedrock. The bedrock encountered in the borings contained dark grey shaly limestone bands and occasional calcite filled cavities and pockets.

The upper 0.3 to 1.2 metres (1 to 4 feet) of the bedrock was found to be badly fractured in the area of boreholes 77-3 and 77-4. The bedrock encountered below this fractured zone and in the remaining boreholes was in fairly sound to sound condition with 95 to 100 per cent core recovery. Boreholes 77-1 to 77-4 were terminated after coring the bedrock for some 3 to 4 metres (10 to 13 feet).

4.4 Groundwater Conditions

Standpipes and piezometers were installed at several levels within the silty clay deposit and the underlying glacial till. The details of the standpipe and piezometer installations, together with the measured water levels at the time of the investigation, are shown on the Record of Borehole sheets. The water levels measured in the shallow piezometers, i.e. at depths of 5 to 6 metres (15 to 20 feet) was generally 0.3 to 0.6 metres (1 to 2 feet) below the ground surface in the area of boreholes 77-1, 77-4 and 77-5. Artesian pressures, 0.9 metres (3 feet) to 1.2 metres (4 feet) above the ground level were, however, measured in the shallow piezometers in boreholes 8 and 77-3. Similar artesian pressures were also measured in piezometers installed within the glacial till.

5. PROPOSED GREEN CREEK CROSSING

5.1 Slope Stability

As previously indicated, the valley slopes in the vicinity of the site are generally about 9 metres (30 feet) in height and have slope angles ranging from 2 horizontal to 1 vertical to 4 horizontal to 1 vertical. The silty clay soil at the site is very susceptible to erosion and Green Creek is actively eroding the sides of the valley, especially along the outside bends of the creek. Because this erosion is oversteepening the valley walls, numerous slope failures have occurred. These failures are particularly noticeable on the east bank of the creek some 15 to 90 metres (50 to 300 feet) south of the bridge site and on the west bank in the area of station 130+00. The most recent slope failures in these areas appear to be only a few years old and a tension crack is evident in the area shown on Figure 2, indicating incipient slope movement.

Stability calculations were carried out for the slopes in the vicinity of the bridge site using shear strength parameters for the clay backfigured from previous slope failures in this area, i.e. an effective angle of shearing resistance, $\phi' = 32^\circ$ and an effective cohesion intercept, $C' = 9.5$ kilopascals (200 pounds per square foot). The groundwater conditions used in these analyses approached full slope saturation, as indicated by the piezometers installed in the boreholes. The slope stability calculations generally indicated an acceptable factor of safety (1.5 or greater) for the valley slopes, except in the areas described previously. In these areas, where the valley slopes are generally about 2:1 the factor of safety was found to be about 1.0 to 1.1.

In order to minimize the possibility of slope failures, which could potentially extend back to the bridge and its approaches, it is recommended that the valley slopes be regraded to 3 horizontal to 1 vertical in the areas shown on Figure 2. This slope flattening would provide a factor of safety of 1.5 against slope movement. The flattened slopes should be protected with rip-rap as shown on Figure 6 to maintain the 3:1 slope and limit future toe erosion. It is considered that either dumped rip-rap or rock filled gabion baskets would be acceptable erosion protection. The cut slopes should be grassed by seeding or sodding as soon as possible to limit surficial erosion. Erosion protection should also be provided for the east and west banks of the creek from 23 metres (75 feet) north of the proposed bridge to 23 metres (75 feet) south of the proposed structure.

Additional slope flattening and erosion protection work should be carried out as outlined in Golder Report 752038 to protect parts of the Eastern Driveway west of the proposed bridge.

Visual observations during the course of the investigation indicated the possibility of an old, large scale slope movement covering the east bank of the creek in the area of the proposed abutment and approach fill. Borehole 77-5 was subsequently put down to check for indications of a failure. In this boring, continuous 50 millimetre (2 inch) shelby tube samplers were pushed to a depth of 6 metres (20 feet) and the samples were extruded for visual examination and laboratory testing. No positive indications of a failure surface were noted visually or by means of a moisture content profile carried out on the clay samples.

5.2 Approach Fills

Stability calculations indicated that the east abutment of the proposed bridge can be located at station 137+60, as proposed, provided that the approach fill is constructed of free-draining granular fill. A 2.5 horizontal to 1 vertical front slope and side slopes will also be required to achieve adequate stability if the fill is raised to elevation 165 feet, as is presently proposed.

The location of the west abutment could be advanced eastward about 30 metres (100 feet) from the presently proposed location (i.e. station 134+10) provided that free-draining granular fill is used together with 2.5 to 1 side and front slopes for the approach embankment. The toe of the west approach fill should not be placed closer to the creek than station 135+60.

The settlement of a 7.5 metre (25 foot) high granular fill embankment, such as would be required to raise the west approaches to elevation 165, would be about 152 millimetres (6 inches). The settlement would reduce proportionate to lowered fill heights, as shown on Figure

7. It is considered that this settlement will take place over a period of about 2 years. It is recommended that the approach fills be placed as far in advance of bridge construction as possible to permit some settlement and dissipation of excess porewater pressures in the clay subsoil to take place. Placing the fill a year in advance would permit about 60 per cent of the settlement to occur before the abutments are constructed.

It is recommended that concrete approach slabs be provided for the bridge, particularly if the approach fills are not constructed well in advance of the abutments.

5.3 Foundations

5.3.1 Spread Footings

The abutments and piers for the proposed bridge could be founded on spread footings placed on the silty clay. The allowable bearing for spread footings founded on the grey-brown silty clay at this site may be taken as 190 kilopascals (4,000 pounds per square foot). The long term settlement of footings designed using this bearing value would be about 50 millimetres (2 inches), provided that the clay at and below founding level is not excessively disturbed during construction. For the abutments this settlement would be in addition to the settlement created by the approach fill, resulting in potential differential settlement between the abutments and piers of the bridge structure.

5.3.2 Pile Foundations

An alternative foundation solution would be to support the bridge on piles driven to end bearing on the surface of the bedrock at depth. For this site, where

the valley slopes have limited stability, it is recommended that non-displacement piles (H-piles) be used to minimize disturbance and the buildup of large excess pore pressures in the clay during pile driving. For preliminary design purposes, the allowable vertical load on a 305 millimetre (12 inch) steel H-pile driven to a final set of 5 blows per inch using a hammer developing about 47,500 newton-metres (35,000 foot pounds) of energy per blow may be taken as 996 kilonewtons (100 tons). An allowance of 300 kilonewtons (30 tons) per pile must be made for negative skin friction on the abutment piles, provided that a 7.5 metre (25 foot) high fill is used. The negative skin friction would reduce to about 100 kilonewtons (10 tons) per pile for a 3 metre (10 foot) high fill.

The lateral load and uplift capacity of a 305 millimetre (12 inch) steel H-pile driven to the above criteria would be 50 kilonewtons (5 tons) and 500 kilonewtons (50 tons), respectively.

If the abutments are to be pile supported, it is recommended that the wing walls also be founded on piles to provide uniform bearing. It is further recommended that some battered piles be used to resist lateral thrust from the fill and tilting of the abutments.

5.4 Abutment Design

If closed ended abutments are used for the proposed bridge, the abutment walls should be backfilled with well compacted, clean sand and gravel fill in accordance with Ministry of Transportation and Communications (M.T.C.) Standard DD-809-C, Rev. 4. Provision should be made for drainage of this backfill to prevent hydrostatic or ice pressure buildup behind the walls. With full effective drainage of the backfill, the abutment walls should be

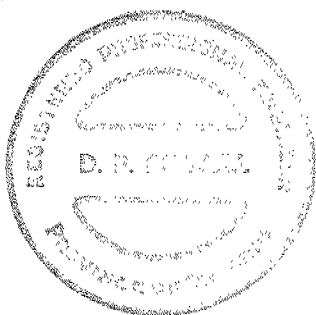
designed to resist lateral earth pressures calculated using a coefficient of lateral earth pressure at rest, K_0 , of 0.4 and a bulk unit weight of 2.1 tonnes per cubic metre (130 pounds per cubic foot).

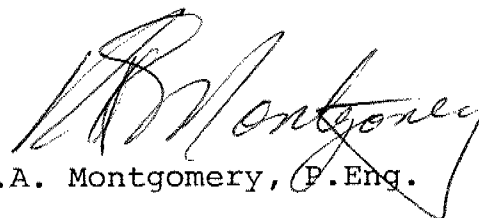
5.5 Construction Considerations

It is recommended that the construction drawings be checked by the Geotechnical Engineer after the height of the approach fills and the locations of the abutments have been finalized. It is also recommended that the piling layout and design be checked, particularly in the abutment areas.

Pile driving operations at this site should be carried out under the supervision of qualified Geotechnical personnel to ensure that the piling meets the requirements for final set and plumbness, and also to ensure that the pile driving operations are not having a detrimental effect on the stability of the valley slopes.

H.Q. GOLDER & ASSOCIATES LTD.,




R.A. Montgomery, P.Eng.



D.P. Powell, P.Eng.

DPP/RAM/rb/sl
772222

LIST OF ABBREVIATIONS

The abbreviations commonly employed on each "Record of Borehole," on the figures and in the text of the report, are as follows:

I. SAMPLE TYPES

AS auger sample
CS chunk sample
DO drive open
DS Denison type sample
FS foil sample
RC rock core
ST slotted tube
TO thin-walled, open
TP thin-walled, piston
WS wash sample

II. PENETRATION RESISTANCES

Dynamic Penetration Resistance: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch diameter, 60 degree cone one foot, where the cone is attached to 'A' size drill rods and casing is not used.

Standard Penetration Resistance, *N*: The number of blows by a 140-pound hammer dropped 30 inches required to drive a 2-inch drive open sampler one foot.

WH sampler advanced by static weight—weight, hammer

PH sampler advanced by pressure—pressure, hydraulic

PM sampler advanced by pressure—pressure, manual

III. SOIL DESCRIPTION

(a) Cohesionless Soils

<i>Relative Density</i>	<i>N, blows/ft.</i>
Very loose	0 to 4
Loose	4 to 10
Compact	10 to 30
Dense	30 to 50
Very dense	over 50

(b) Cohesive Soils

<i>Consistency</i>	<i>c_u, lb./sq. ft.</i>
Very soft	Less than 250
Soft	250 to 500
Firm	500 to 1,000
Stiff	1,000 to 2,000
Very stiff	2,000 to 4,000
Hard	over 4,000

IV. SOIL TESTS

C consolidation test
H hydrometer analysis
M sieve analysis
MH combined analysis, sieve and hydrometer¹
Q undrained triaxial²
R consolidated undrained triaxial²
S drained triaxial
U unconfined compression
V field vane test

NOTES:

¹Combined analyses when 5 to 95 per cent of the material passes the No. 200 sieve.

²Undrained triaxial tests in which pore pressures are measured are shown as \bar{Q} or \bar{R} .

LIST OF SYMBOLS

I. GENERAL

π	= 3.1416
e	= base of natural logarithms 2.7183
$\log_e a$ or $\ln a$	natural logarithm of a
$\log_{10} a$ or $\log a$	logarithm of a to base 10
t	time
g	acceleration due to gravity
V	volume
W	weight
M	moment
F	factor of safety

II. STRESS AND STRAIN

u	pore pressure
σ	normal stress
σ'	normal effective stress ($\bar{\sigma}$ is also used)
τ	shear stress
ϵ	linear strain
ϵ_{xy}	shear strain
ν	Poisson's ratio (μ is also used)
E	modulus of linear deformation (Young's modulus)
G	modulus of shear deformation
K	modulus of compressibility
η	coefficient of viscosity

III. SOIL PROPERTIES

(a) <i>Unit weight</i>	
γ	unit weight of soil (bulk density)
γ_s	unit weight of solid particles
γ_w	unit weight of water
γ_d	unit dry weight of soil (dry density)
γ'	unit weight of submerged soil
G_s	specific gravity of solid particles $G_s = \gamma_s / \gamma_w$
e	void ratio
n	porosity
w	water content
S_r	degree of saturation
(b) <i>Consistency</i>	
w_L	liquid limit
w_P	plastic limit
I_P	plasticity index
w_S	shrinkage limit
I_L	liquidity index = $(w - w_P) / I_P$
I_C	consistency index = $(w_L - w) / I_P$
e_{max}	void ratio in loosest state
e_{min}	void ratio in densest state
D_r	relative density = $(e_{max} - e) / (e_{max} - e_{min})$
(c) <i>Permeability</i>	
h	hydraulic head or potential
q	rate of discharge
v	velocity of flow
i	hydraulic gradient
k	coefficient of permeability
j	seepage force per unit volume
(d) <i>Consolidation (one-dimensional)</i>	
m_v	coefficient of volume change = $-\Delta e / (1 + e) \Delta \sigma'$
C_c	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
c_c	coefficient of consolidation
T_v	time factor = $c_v t / d^2$ (d , drainage path)
U	degree of consolidation
(e) <i>Shear strength</i>	
τ_f	shear strength
c'	effective cohesion
ϕ'	effective angle of shearing resistance, or friction
c_u	apparent cohesion*
ϕ_u	apparent angle of shearing resistance, or friction
μ	coefficient of friction
S_t	sensitivity

*For the case of a saturated cohesive soil, $\phi_u = 0$ and the undrained shear strength $\tau_f = c_u$ is taken as half the undrained compressive strength.

1-6.1-2

RECORD OF BOREHOLE 77-5

LOCATION See Figure 2

BORING DATE OCT. 25, 1977

DATUM GEODETIC

SAMPLER HAMMER WEIGHT 140 LB., DROP 30 IN.

PENETRATION TEST HAMMER WEIGHT 140 LB., DROP 30 IN.

BORING METHOD	SOIL PROFILE			SAMPLES		ELEVATION SCALE	DYNAMIC PENETRATION RESISTANCE, BLOWS/FT.		COEFFICIENT OF PERMEABILITY, k_v , CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION	
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE		BLOWS/FT.	SHEAR STRENGTH C_u , LB./SQ. FT.		WATER CONTENT, PERCENT					
								NAT. V. - + REM. V. - ●	Q. - ● U. - ○	W_p	W	W_L			
POWER AUGER 8" DIAM. (HOLLOW STEM)	149.2	GROUND SURFACE													
	0.0 148.2 1.0	TOPSOIL		1	2" DO.	6									
		VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)		2	"	5									
			3	2" TO	PH.										
			4	"	"										
			5	"	"										
	139.2 10.0	STIFF TO VERY STIFF GREY SILTY CLAY		6	"	"									
			7	"	"										
			8	"	"										
			9	"	"										
			10	"	"										
	11		"	"											
129.2 20.0	END OF HOLE														

GROUND SURFACE
SURFACE SEAL
NATIVE BACKFILL
BENTONITE SEAL
NATIVE BACKFILL
STANDPIPE
W.L. IN STANDPIPE @ ELEV. 148.4 NOV. 21, 1977

15 0 5 10 Percent axial strain at failure

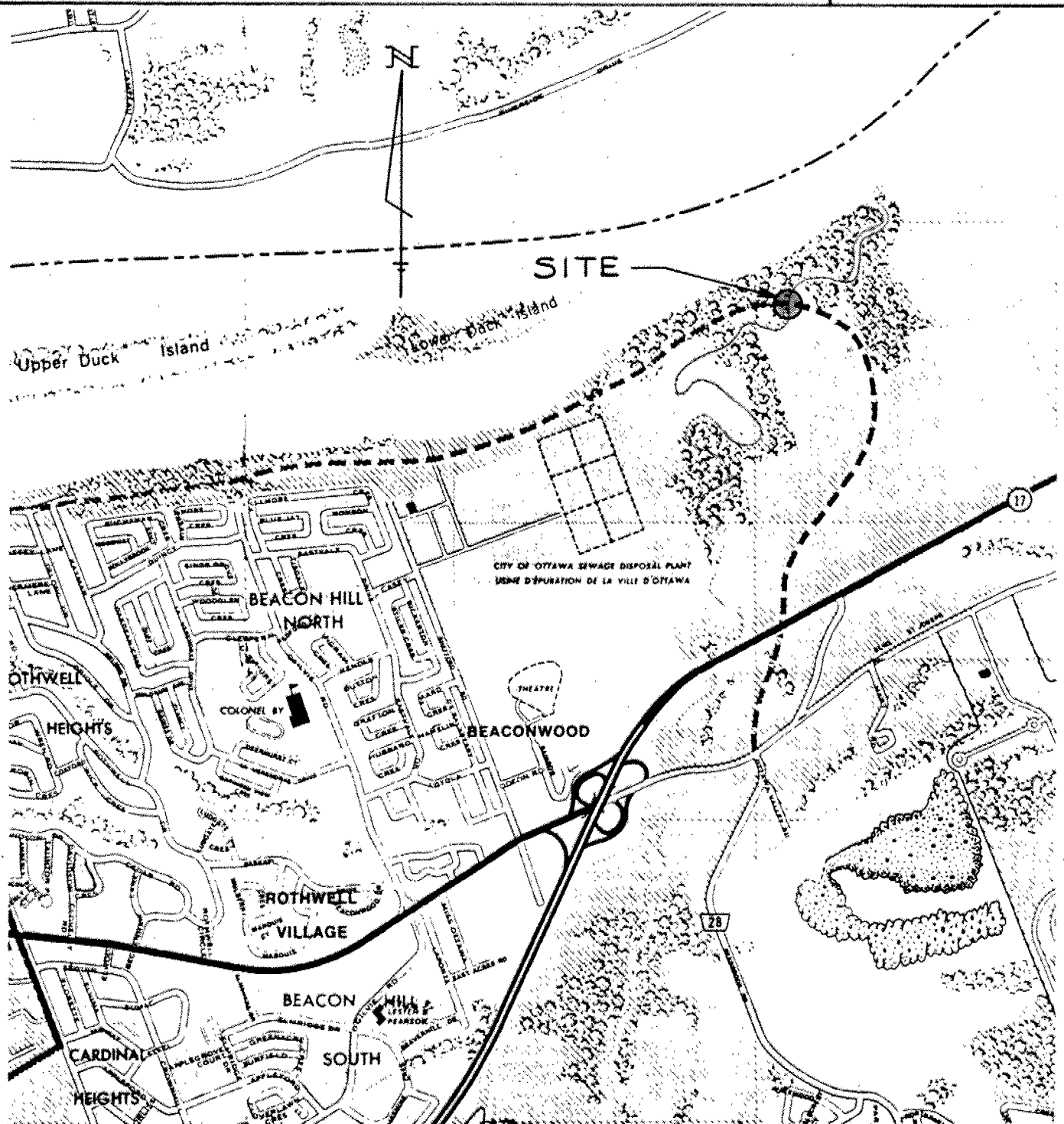
VERTICAL SCALE
1 IN. TO 5 FT.

Golder Associates

DRAWN RKB
CHECKED *[Signature]*

KEY PLAN

FIGURE 1



SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

SCALE 1:25,000

Date NOV. 29, 1977

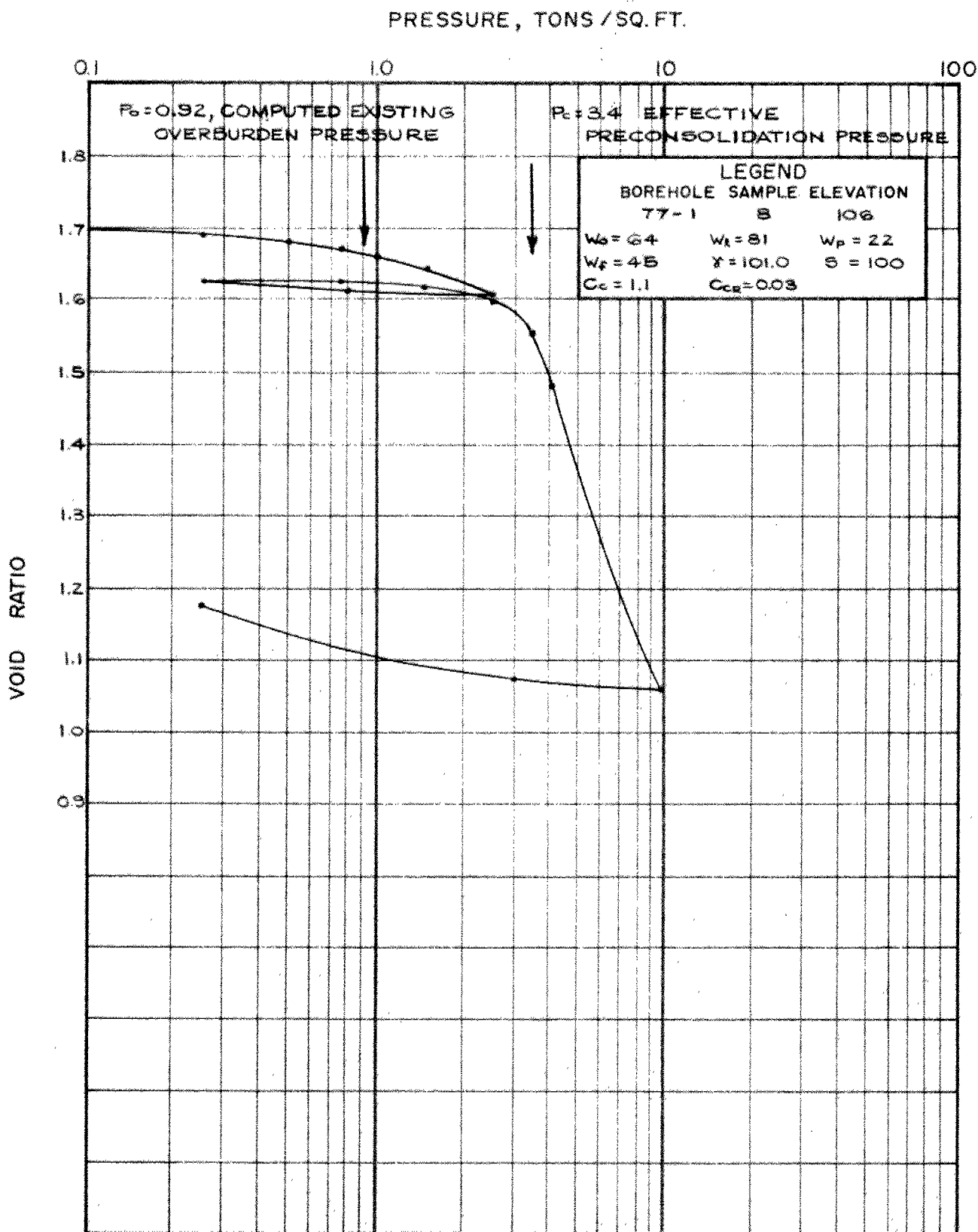
Golder Associates

Drawn *DN*
Chkd. *[Signature]*
Appd. *[Signature]*

PROJECT N 772222
G.A. 4

VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

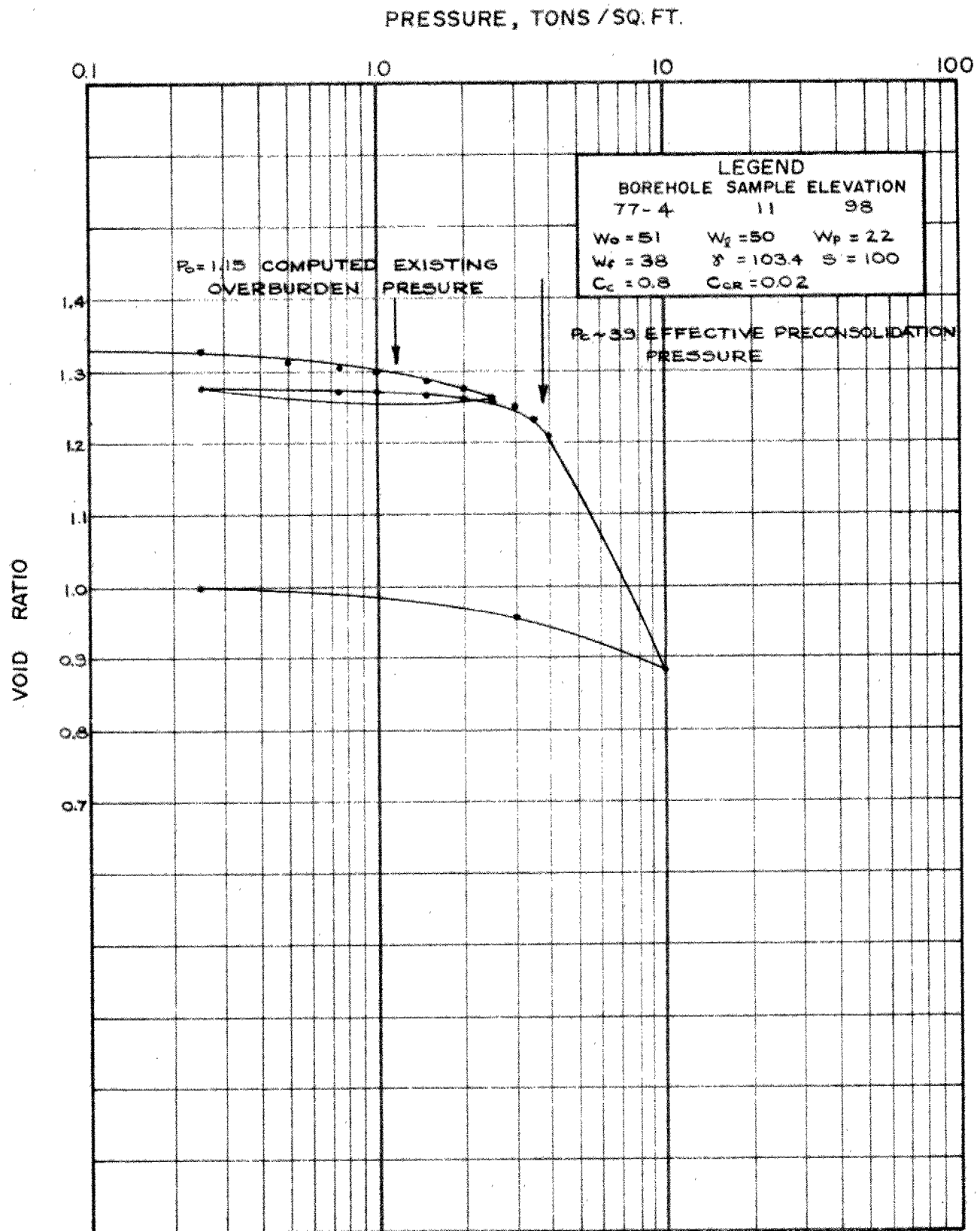
FIGURE 3



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VOID RATIO - PRESSURE CURVES CONSOLIDATION TEST

FIGURE 4



Golder Associates

M.I.T. GRAIN SIZE SCALE

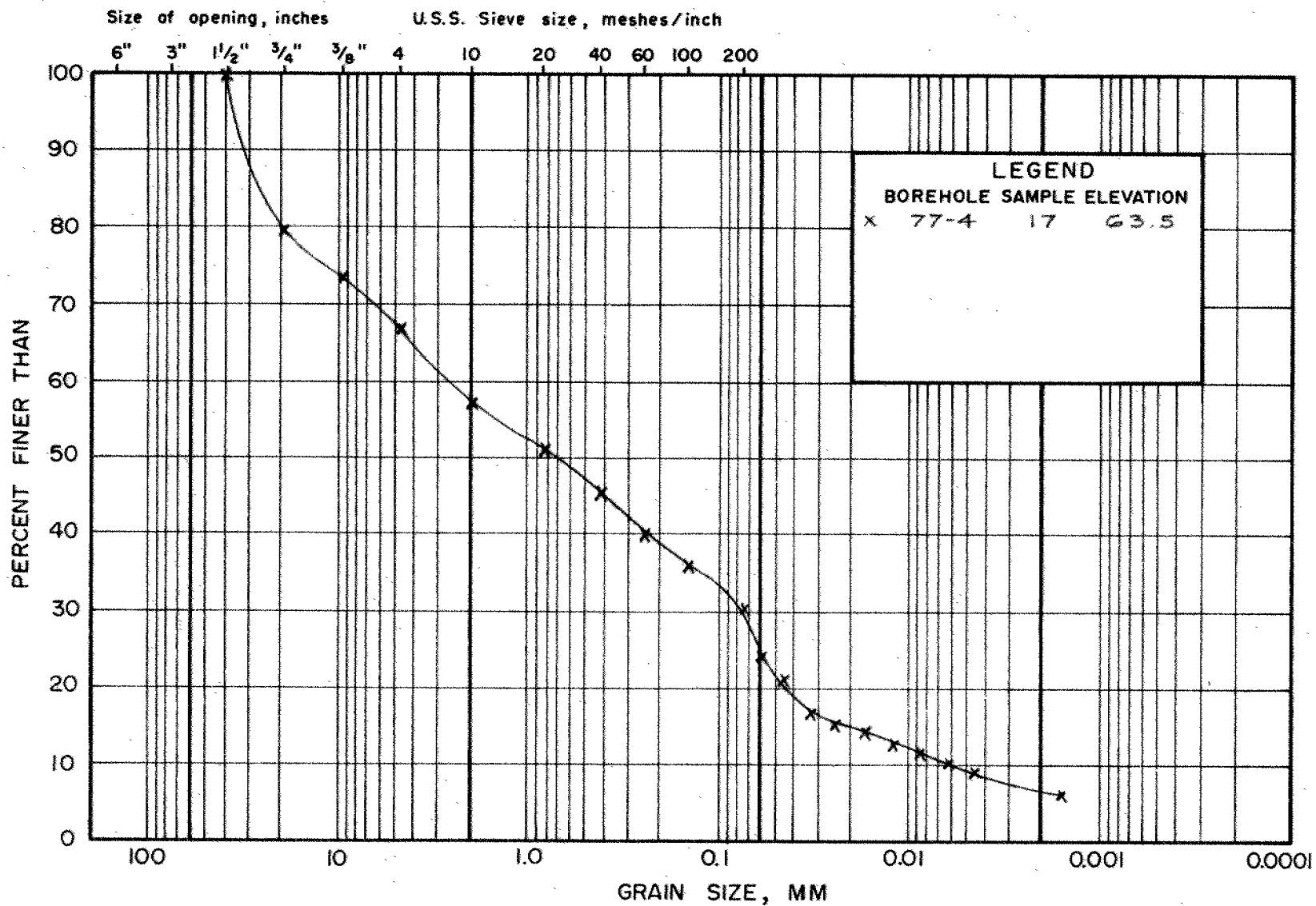
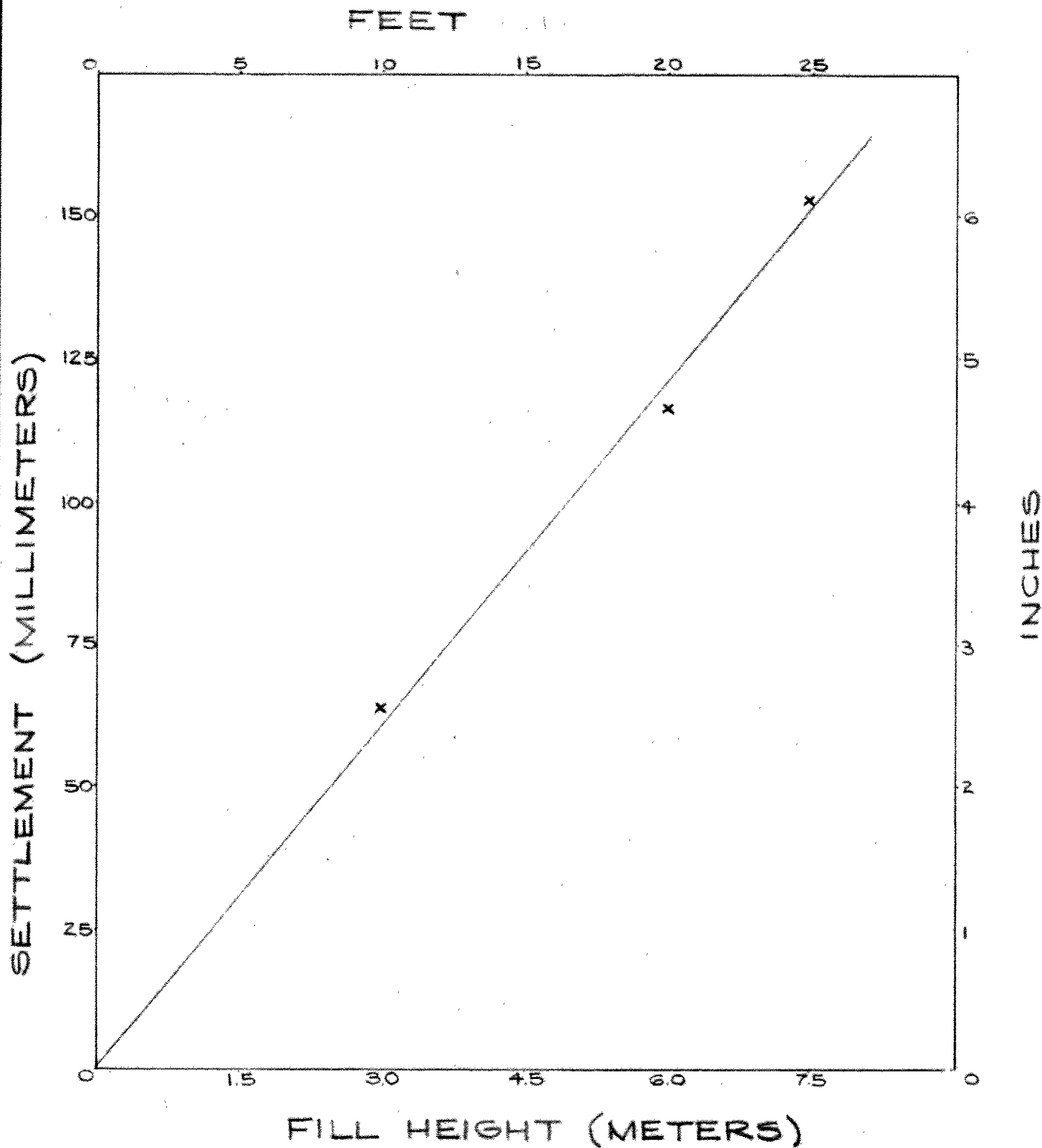
GRAIN SIZE DISTRIBUTION
GLACIAL TILL

FIGURE 5

SETTLEMENT VS. HEIGHT
OF APPROACH FILL

FIGURE 7



SPECIAL NOTE

THIS DRAWING IS TO BE READ IN CONJUNCTION
WITH ACCOMPANYING REPORT.

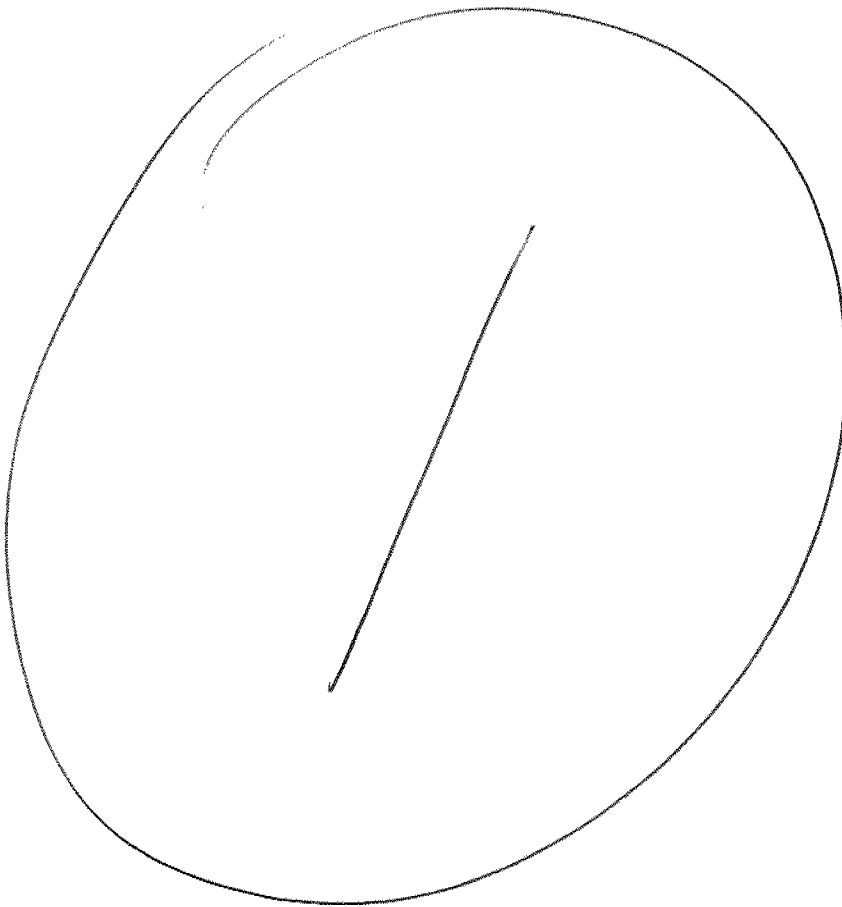
Date JAN 13 1977

Golder Associates

Drawn RKB
Chkd. *[Signature]*
Appd. *[Signature]*

35MM

DRAWING



ENGINEERING MATERIALS OFFICE
SOIL MECHANICS SECTION

WP 46-78-02

DIST 9

HWY 17

STR SITE 3-335

Eastern Driveway Underpass

DISTRIBUTION

T.C. Kingsland
W.E. Blum
C.E. Pritchard
R.W. Franks (2)

K.G. Bassi
G.A. Wrong
B.J. Giroux
R.S. Pillar

R. Hore

R. Forest)
J. Anderson) cover only
G. Sloan)

Files ✓

SAMPLE DISPOSITION NOTICE		
TYPE	DISCARD AFTER	RECOMM. BY
JARS	79 03 30	M.D.
TUBES	79 04 30	#
ROCK CORES	79 04 30 1st round of contact	11

FOUNDATION INVESTIGATION REPORT

For

Eastern Driveway Underpass
Lot 12, Conc. 1, Gloucester Township
Hwy. 17, District 9, Ottawa
W.P. 46-78-02, Site 3-335

INTRODUCTION

This report contains the results of a foundation investigation carried out at the above site. The fieldwork was carried out on April 24, 1978 and between January 3 to January 20, 1979 and consisted of four sampled boreholes. The borings were advanced by hollow stem augers to a depth of up to 30 metres below ground surface; thereafter washboring techniques were employed to advance the borings to bedrock at a depth of up to 54 metres below ground level. Diamond drilling techniques were then utilized in two boreholes to obtain up to 2.7 metres of BXL size rock core samples.

SITE DESCRIPTION AND GEOLOGY

This site is located on Hwy. 17 approximately 1.2 km east of Montreal Road/Hwy. 17 intersection in the Township of Gloucester, Regional Municipality of Ottawa-Carleton.

In the site vicinity the terrain is generally flat and the land presently is utilized for agriculture purposes. Existing Hwy. 17 at this location has a profile grade at approximate elevation 53 and ditch grades at elevation 51, whereas the surrounding terrain has an average ground elevation of 53.

Physiographically, the site is located within the Ottawa Valley Clay Plains. This region is characterized by extensive deposits of sensitive marine clay overlying glacial till and limestone bedrock.

SUBSURFACE CONDITIONS

General

Subsoil conditions across the site are generally uniform. Beneath a thin veneer of topsoil extending to a depth up to 47 metres below

the ground surface is the dominant deposit consisting of firm to very stiff sensitive clay. The clay overlies 8 to 10 metres of a compact to very dense glacial till composed of a heterogeneous mixture of silt, sand, gravel and occasional cobbles. The upper 1.7 metres of this deposit contains cobbles and boulders up to 0.9 metres in size. Underlying the glacial till is sound dolomite bedrock.

The boundaries between the various subsoil and bedrock types are shown on the Record of Borehole Sheets. The locations and elevations of the boreholes, as well as a stratigraphical profile inferred from borehole data, is shown on Drawing No. 467802-A.

Following is a brief description of subsoil and bedrock types.

Sensitive Clay

This deposit was encountered immediately below a thin veneer of topsoil and is estimated to have a thickness ranging from 44.5 to 47.3 metres. The deposit is generally grey in colour except for the upper 2.0 to 2.8 metres which is desiccated and has a brown-grey mottled colour. Laboratory and in-situ testing performed on representative samples from this deposit gave the following results.

<u>Geotechnical Properties</u>		<u>In the Upper 2.0-2.8m</u>		<u>Below the</u>	
		<u>Desiccated Zone</u>		<u>Desiccated Zone</u>	
		<u>Range</u>	<u>Average</u>	<u>Range</u>	<u>Average</u>
Natural Moisture Content	(W) %	35-42	39	56-72	67
Liquid Limit	(W _L) %	51-66	60	55-69	64
Plastic Limit	(W _p) %	24-26	25	22-27	25
Plasticity Index	(I _p) %	25-42	29	32-44	39
Liquidity Index	(I _L)	0.2-0.5	0.4	0.9-1.3	1.1
Bulk Unit Weight	(γ) kN/m ³			15.6-16.3	15.9
Undrained Shear Strengths (c _u) kPa		<u>Range</u>	<u>Sensitivity</u>	<u>Range</u>	<u>Sensitivity</u>
Field Vane Tests	> 96			31 to >96	5 to 19
Unconfined Compression Tests	132			41 to 105	

Consolidation Tests

Below the
Desiccated ZoneRange

Degree of Preconsolidation ($\sigma_p' - \sigma_{vo}'$) kPa	180 - 240
Initial Void Ratio	1.76 - 1.98
Coefficient of Consolidation	1.20 - 1.58

The results of the Atterberg Limit testings are shown on the Plasticity Chart (Figure 1). These results indicate that the deposit is inorganic and of high plasticity (CH zone). The deposit may be described as a sensitive clay as evidenced by the high sensitivities (5 to 19) measured by the field vane testing. Furthermore, the Liquidity Index of the cohesive soil below the desiccated zone is generally greater than 1 confirming that the non-desiccated cohesive deposit is sensitive to remoulding.

Four consolidation tests were performed on samples from this deposit and are summarized on Figure 3. The consolidation testing indicates that the deposit has been preconsolidated by a pressure of 180 to 240 kPa in excess of the existing overburden pressure. The high initial void ratio, 1.76 to 1.98, together with the high coefficient of consolidation, 1.20 to 1.58 indicates that upon loading in excess of the preconsolidation pressure, the deposit will undergo significant consolidation.

The results of in-situ vane testings are summarized on the Shear Strength vs. Depth Profile, Figure 4.

The vane testing indicates that the undrained shear strength is greater than 96 kPa in the upper 2.0 to 2.8 metre desiccated zone and thereafter decreases abruptly to a low of 31 kPa at a depth of 3.5 metres. The shear strength then gradually increases with depth to greater than 96 kPa at a depth of about 15 metres below ground surface. Based on these results the consistency is described as very stiff in the upper desiccated zone and below this it is generally firm to very stiff.

Glacial Till

The glacial till stratum was found immediately below the sensitive clay deposit and is estimated to have a thickness of 8 to 10 metres. The deposit is composed of a heterogeneous mixture of silt, sand, gravel and occasional cobbles. Furthermore, the upper 1.7 metres of this stratum contains cobbles and boulders up to 0.9 metres in size. The results of grain size distribution testing performed on samples obtained from a 2" O.D. split spoon sampler from this deposit are plotted on Figure 2.

Based on the Standard Penetration Test 'N' values ranging from 25 to 94 blows per foot, the relative density of the deposit is described as compact to very dense, generally being compact to dense.

Dolomite Bedrock

Sound bedrock was encountered immediately below the glacial till. In two boreholes bedrock was proven by obtaining up to 2.7 metres of BXL rock core. The bedrock surface was found to vary from 53.5 to 53.9 metres below ground surface which corresponds to elevation -0.9 to -1.7 metres. The bedrock is composed of hard, fine textured, medium grey dolomite which is generally sound.

Groundwater Conditions

Groundwater conditions were observed by measuring the water level in the open boreholes 24 hours after completion of the borehole. The groundwater level was found to be 0.8 to 3.4 metres below ground surface which corresponds to elevation 49.3 to 51.0.

DISCUSSION AND RECOMMENDATIONS

It is proposed to construct a two span (28 m, 28 m) structure to carry the Eastern Driveway over Hwy. 17, E.B.L. and W.B.L. The structure is located at about Sta. 16+685 E.B.L.

The proposed grade of the Eastern Driveway is at about elevation 60.0 m, whereas the grade of Hwy. 17 is about at elevation 53.0 m. The surrounding terrain is flat to gently undulating with a ground surface varying from elevation 51 to 53. Accordingly, fill heights of 7 to 9 metres will be required to accomplish this crossing.

Subsoil conditions across the site are uniform. Extending from the ground surface to a depth of up to 47 metres below ground is a firm to very stiff grey sensitive and compressible clay. This deposit is underlain by up to 9.5 metres of compact to very dense glacial till which is in turn underlain by bedrock. Groundwater was observed at a depth of about 0.8 to 3.4 metres below ground surface.

The extensive deposit of sensitive clay is the governing factor from the foundation point of view since it is imperative that this stratum should not be overstressed by embankment loading. In view of the importance of this matter this aspect will be discussed first.

Approach Fills - Stability Considerations

Analysis in terms of total stress have been carried out to determine the stability of fills including surcharge requirements. In this method of analysis, stability is governed by the undrained shear strength properties of the foundation and fill materials. The following data and values were used in carrying out the stability analysis.

Fill Material

Rock Fill Material	$\gamma = 22 \text{ kN/m}^3$	$\phi = 35^\circ$
	$\gamma' = 12.2 \text{ kN/m}^3$	$C = 0$
Granular Fill Material	$\gamma = 20.2 \text{ kN/m}^3$	$\phi = 30^\circ$
For Granular Core	$\gamma' = 10.4 \text{ kN/m}^3$	$C = 0$

Subsoil Conditions

Groundwater Level Elevation 51.0

<u>Elevation (M)</u>	<u>γ (kN/m³)</u>	<u>γ' (kN/m³)</u>	<u>ϕ°</u>	<u>c_u (kPa)</u>
53-51	16.16	6.8	0	96
51-50	15.7	5.9	0	55
50-48.5	15.7	5.9	0	38
48.5-46.0	15.7	5.9	0	58
46.0-42.0	15.7	5.9	0	70

The analysis was undertaken assuming forward and/or side slopes of 1½:1 constructed of rock fill and 2:1 constructed of granular type fill. The results of the analysis are outlined below.

1. Fill heights up to 10.5 metres high (ie. fill height of 9.0 metres + 1.5 metres of surcharge) will be stable with slopes of not less than 2 horizontal to 1 vertical. (Reference Fig. 5)
2. Fill heights up to 9.0 metres high (ie. fill height of 9.0 metres only) will be stable with slopes of not less than 1½ horizontal to 1 vertical if constructed of rock fill. (Reference Fig. 6)
3. Fill heights up to 10.5 metres high (ie. fill height of 9.0 metres + 1.5 metres of surcharge) will be stable with slopes of not less than 1½:1 if constructed of rock fill (Reference Fig. 6).

The proposed fill heights for the north approach are in the order of 7 metres high or 8.5 metres with a 1.5 metre surcharge and hence, no stability problems are anticipated at final grade plus a 1.5 metre surcharge with side and/or forward slopes of 2:1. At final grade plus a 1.5 metre surcharge with side and/or forward slopes of 1½:1 constructed of rock fill, no transverse stability problems are anticipated. However, in the longitudinal direction a reduction in counterbalancing moment is effected by the existing ditch and it is recommended that this ditch be filled in by constructing Hwy. 17 W.B.L. to final grade for a distance of 40 metres east and west of the Eastern Driveway ϕ .

The south approach will require fill heights up to 9 metres high. The additional surcharge of 1.5 metres would require a 10.5 metre temporary fill height. No stability problems are anticipated at this fill height with slopes of 2:1 if constructed of granular type fill. However, if it is desired to proceed with a rock fill embankment employing 1½:1 side and/or forward slopes and a 1.5 metre surcharge, it will be necessary to provide a counterbalancing berm in the transverse section where the temporary fill height exceeds 9.0 metres. This could be accomplished by filling in the ditch to an elevation of 53.0 for a minimum distance of 10 metres beyond the toe of the approach fill.

Approach Fills - Settlement Considerations

Settlement will occur due to the consolidation of the underlying compressible clay deposit. Settlement analysis indicates that the maximum settlement due to consolidation of the sensitive clay deposit beneath a 7 metre fill height will be in the order of 0.45 metres. It is estimated that 80% of this settlement would occur within the first 18 to 24 months after construction.

In order to minimize post construction maintenance costs due to settlements and in order to reduce the negative skin friction forces on piles, it is recommended that the embankment be surcharge loaded for 1.5 metres for a period of up to 6 months prior to driving piles. Attached on Figure 7 are the minimum dimensional requirements for the temporary surcharge.

Structure

The structure piers and abutments should be supported on end bearing steel 'H' piles driven to the bedrock surface. For estimating purposes the following pile tip elevations may be used.

Estimated Pile Tip Elevation (M)

North Abutment	-1.98
Pier	-1.98
South Abutment	-1.67

Because of the presence of boulders and cobbles in the glacial till, it is suggested that the pile tips be reinforced. It is important

to note that piles stopped by the boulders and cobbles will not perform satisfactorily upon loading. It is imperative that the piles be driven past boulders to sound bedrock.

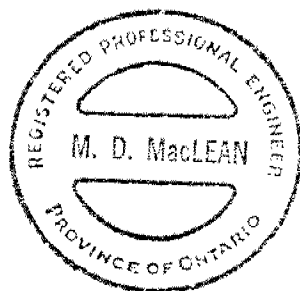
Considerable negative skin friction loads may be imposed on the piles supporting the abutments due to settlement of the roadway embankment. In view of this, it is recommended that the pile capacities for the abutments and wing walls should be reduced by 15%; i.e. 85% of their maximum allowable loads. For example, a HP 12X74 steel 'H' pile may be designed for 750 kN per pile. Piles elsewhere may be designed for their maximum allowable loads, i.e. a HP 12X74 steel 'H' pile may be designed for 900 kN per pile.

In addition to the negative skin frictional forces, movement of subsoil due to strain imposed by the embankment loading will generally tend to displace the piles laterally and can cause rotation of the abutments. In view of this, we recommend that consideration be given to supporting the extreme ends of the wing walls on end bearing piles founded as aforementioned. It is considered that this will improve the stability in the longitudinal direction. It is understood that the approaches will be constructed of rock fill, however, no rock fill material should be placed in areas where piles are to be driven. A composite granular core should be provided in these areas. A recommended composite section is shown on Figure 8.

MISCELLANEOUS

The fieldwork for this investigation was carried out under the supervision of Mr. M. MacLean, Project Engineer, using equipment rented from Dominion Soil Investigation Ltd.

This report was written by Mr. M. MacLean and reviewed by Mr. M. Devata, Supervising Engineer.



M MacLean
M. MacLean, P. Eng.
Project Engineer

M. Devata
M. Devata, P. Eng.
Supervising Engineer

APPENDIX

RECORD OF BOREHOLE No 1

Previous B.H. 4 W.P. 911-73-00
METRIC

W P 46-78-02 LOCATION Coords. N 5 035 596.6; E 377 247.6 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger (0-30m) Washboring (30-54 m) COMPILED BY MM
DATUM Geodetic DATE 1978 04 24 CHECKED BY RS

[illegible]

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+3, x5: Numbers refer to Sensitivity


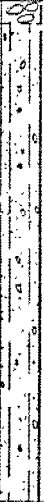

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 1 Continued

Previous B.H. 4
METRIC W.P. 911-73-00

W P 46-78-02 LOCATION Coords. N 5 035 596.6; E 377 247.6 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Auger (0-30 m) Washboring (30-54 m) COMPILED BY MM
DATUM Geodetic DATE 1978 04 24 CHECKED BY RS

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			20	40	60	80	100						SHEAR STRENGTH		
22.0	Sensitive Clay Grey Very Stiff						ELEVATION SCALE	○ UNCONFINED + FIELD VANE					● QUICK TRIAXIAL x LAB VANE			WATER CONTENT (%)				
30.0																				
7.5	Some Cobbles Heterogeneous Mixture Silt, Sand and Gravel Compact to Dense (Glacial Till)						ELEVATION SCALE	○ UNCONFINED + FIELD VANE					● QUICK TRIAXIAL x LAB VANE			WATER CONTENT (%)				
44.5																				
-2.0	End of Borehole Refusal to Tricone Probable Bedrock						ELEVATION SCALE	○ UNCONFINED + FIELD VANE					● QUICK TRIAXIAL x LAB VANE			WATER CONTENT (%)				
54.0																				



RECORD OF BOREHOLE No2 METRIC

W P 46-78-02 LOCATION Coords. N 5 035 600.8; E 377 229.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-12 m. Washboring 12-53 COMPILED BY MM
DATUM Geodetic DATE 1979 01 09 to 19 CHECKED BY RS

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					
52.7	Ground Level																
0.0	Very Stiff		1	SS	31		52										
			2	SS	9												
50.2							50										
2.5			3	TW	PH												
	Firm to Stiff		4	SS	V 450 mm		48										
			5	TW	PM												
			6	SS	4		46										
			7	TW	PH		44										
	Very Stiff		8	SS	2		42										
	Sensitive Clay		9	TW	PH		40										
	Grey						38										
							36										
			10	SS	2		34										
							32										
							30										
			11	SS	4		28										
							26										
							24										
22.7																	
30.0																	

Continued

+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 2 Continued METRIC

W P 46-78-02 LOCATION Coords. N 5 035 600.8; E 377 229.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-12 m, Washboring 12-53 COMPILED BY MM
DATUM Geodetic DATE 1979 01 09 to 19 CHECKED BY RS

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			20	40	60	80	100					
22.7																	
30.0																	
	Very Stiff						22										
	Sensitive						20										
	Clay						18										
	Grey						16										
							14										
							12										
							10										
							8										
7.4							6										
45.3	0.9 m Boulder		12	RC	10% Rec												RQD=0%
	Heterogeneous		13	SS	34												23 37 30 10
	Mixture Silt, Sand						4										
	Gravel & Occasional						2										
	Cobbles						0										
	Compact to Dense		14	SS	25												
	(Glacial Till)																
-1.0																	
53.7	Sound Dolomite		15	RC	91% Rec												RQD=70%
	Bedrock			BXL			-2										
-2.7																	
55.4	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 2A

METRIC

W P 46-78-02 LOCATION Coords: N 5 035 602.8; E 377 299.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers, Continuous Vane Tests COMPILED BY MM
DATUM Geodetic DATE 1979 01 16 CHECKED BY R-S

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			SHEAR STRENGTH kPa										WATER CONTENT (%)
								20 40 60 80 100										
52.7	Ground Level																	
0.0	Very Stiff																	
	Sensitive Clay Firm to Stiff																	
47.8																		
4.9	End of Borehole																	


+3, x5: Numbers refer to
Sensitivity

20
15
10
5 (%) STRAIN AT FAILURE



RECORD OF BOREHOLE No 3 METRIC

W P 46-78-02 LOCATION Coords. N 5 035 578.4; E 377 239.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers, 0-12, Tricone Ahead 12-54 COMPILED BY MM
DATUM Geodetic DATE 1979 01 20 CHECKED BY RS

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	20 40 60 80 100	20 40 60 80 100					
52.6	Ground Level															
0.0	Very Stiff		1	SS	15		52									
			2	TW	PH											
50.1							50									
2.5			3	SS	2											
	Firm to Stiff		4	TW	PH		48									
	Sensitive															
	Clay		5	SS	2		46									
	Grey		6	TW	PM											
			7	SS	7		44									
			8	TW	PM		42									
			9	SS	3		40									
							38									
							36									
	Very Stiff		10	SS	4		34									
							32									
							30									
			11	SS	6		28									
							26									
							24									
22.6																
30.0																

Continued

+3, x5: Numbers refer to
Sensitivity

20
15  5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 3 Continued

METRIC

W P 46-78-02 LOCATION Coords. N 5 035 578.4; E 377 239.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-12, Tricone Ahead 12-54 COMPILED BY MM
DATUM Geodetic DATE 1979 01 20 CHECKED BY RS

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			SHEAR STRENGTH										WATER CONTENT (%)		
								○ UNCONFINED	+ FIELD VANE	● QUICK TRIAXIAL	x LAB VANE	W _p						W	W _L	
22.6							22													
30.0							20													
	Sensitive Clay Grey Very Stiff						18													
							16													
							14													
							12													
							10													
							8													
7.5							6													
45.1	Cobbles and Boulders						4													
	Probable Glacial Till Heterogeneous Mixture Silt, Sand and Gravel With Occasional Cobbles						2													
							0													
-0.9																				
53.5	Refusal to Tricone Probable Bedrock End of Borehole																			

+3, x5: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 4

METRIC

W P 46-78-02 LOCATION Coords. N 5 035 602.8; E 377 229.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-18.3, Washboring 18.3-56.0 COMPILED BY MM
DATUM Geodetic DATE 1979 01 03 to 09 CHECKED BY RS

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					
52.0	Ground Level																
0.0	Very Stiff		1	SS	9		50										
			2	SS	6												
49.2			3	SS	3												
2.8	Firm to Stiff		4	TW	PH		48		+7								
			5	SS	3					+8							
			6	SS	2		46			+8							
	Sensitive		7	TW	PH		44										
	Clay		8	SS	3					+19							
	Grey		9	TW	PH		42			+7							
			10	SS	2		40				+11					15.9	
			11	TW	PH		38			+6							
			12	SS	2		36				+ > 96						
							34				+ > 96						
	Very Stiff		13	SS	2		32										
							30										
							28				+ > 96						
			14	SS	5		26										
							24										
22.0																	
30.0																	

Continued

+³, x⁵: Numbers refer to
Sensitivity

20
15 ϕ 5 (%) STRAIN AT FAILURE
10



RECORD OF BOREHOLE No 4 Continued

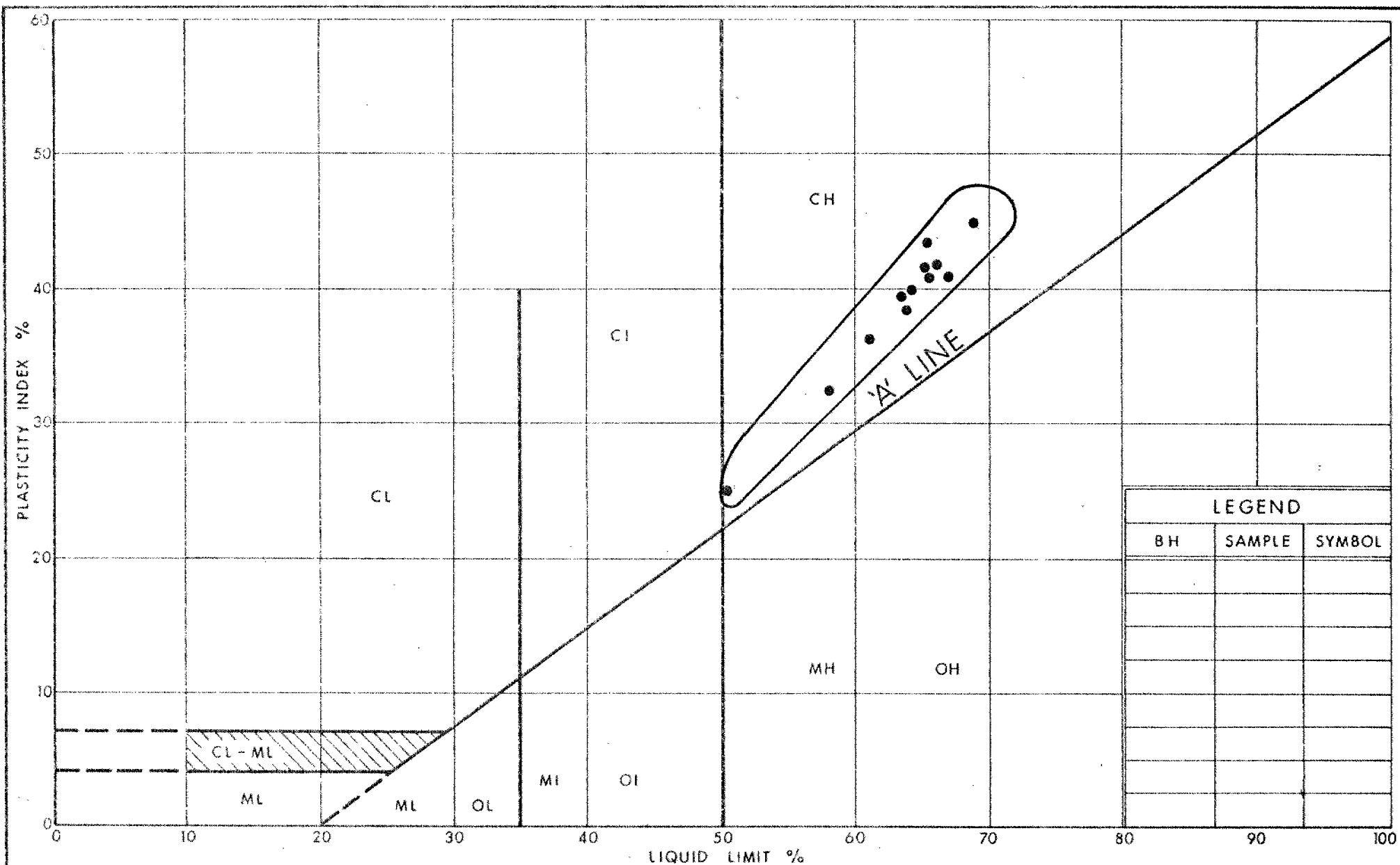
METRIC

W P 46-78-02 LOCATION Coords. N 5 035 602.8; E 377 229.4 ORIGINATED BY MM
DIST 9 HWY 17 BOREHOLE TYPE Hollow Stem Augers 0-18.3, Washboring 18.3-56.0 COMPILED BY MM
DATUM Geodetic DATE 1979 01 03 to 09 CHECKED BY RS

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 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					
22.0							22										
30.0	Sensitive Clay Grey		15	SS	6		20										
							18										
	Sand and Gravel						16										
	Very Stiff		15	SS	5		14										
							12										
			17	SS	18		10										
							8										
							6										
4.8							4										
47.2	Cobbles & Boulders		18	SS	44		2										
	Heterogeneous Mixture Silt, Sand & Gravel (Glacial Till) Occasional Cobbles Compact to Very Dense						0										
-1.7			19	SS	94		-2										
53.7	Dolomite Bedrock Sound		20	RC BXL	73% Rec		-4										RQD=67%
-4.4			21	RC BXL	77% Rec												RQD=65%
56.4	End of Borehole																

+3, x5: Numbers refer to
Sensitivity

20
15 5 (%) STRAIN AT FAILURE
10



LEGEND		
BH	SAMPLE	SYMBOL



Ontario

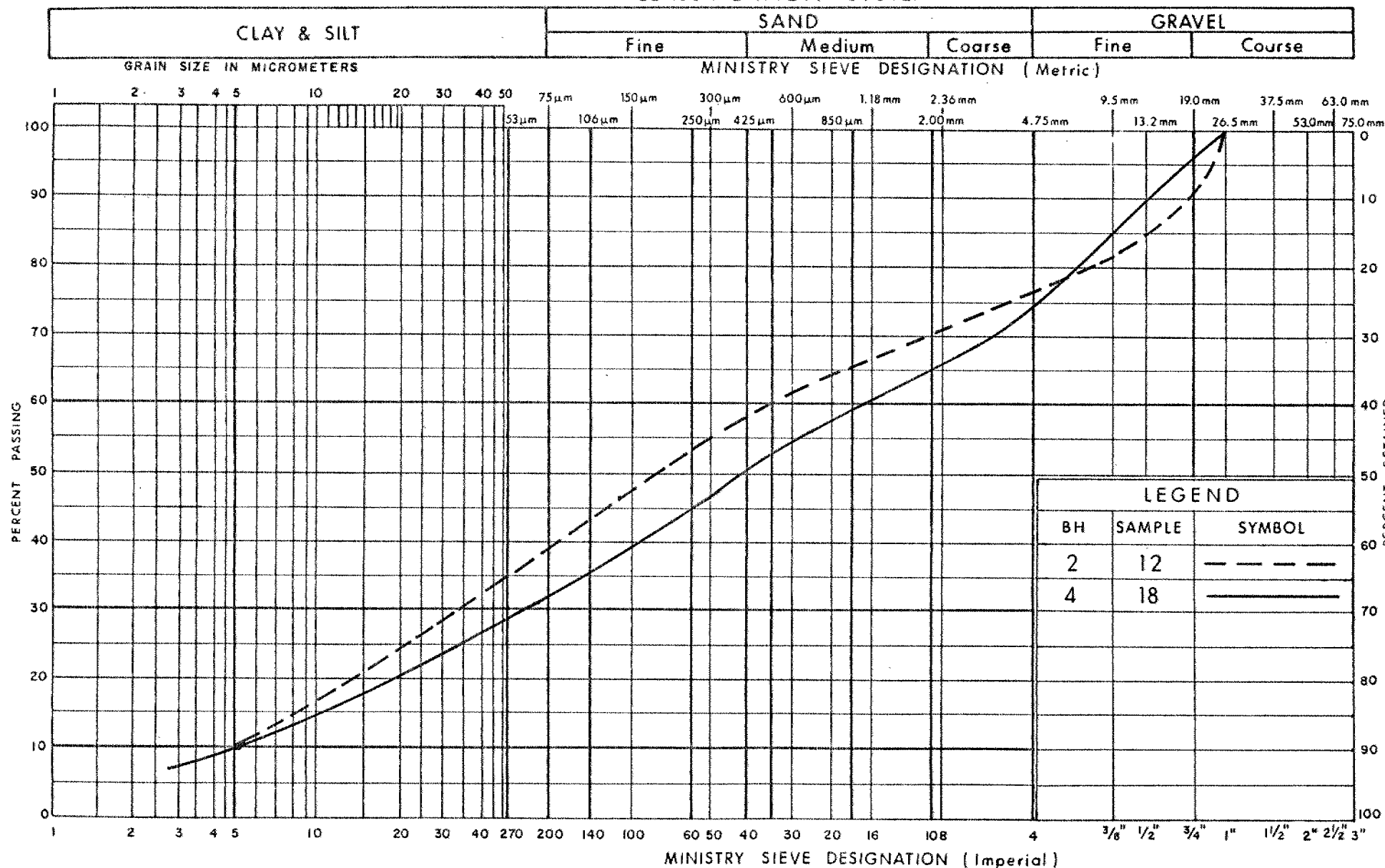
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PLASTICITY CHART SENSITIVE CLAY

FIG No 1

W P 46 - 78 - 02

UNIFIED SOIL CLASSIFICATION SYSTEM



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Communications

GRAIN SIZE DISTRIBUTION GLACIAL TILL

FIG No 2

W P 46 - 78 - 02

VOID RATIO - PRESSURE CURVES

WP 46 - 78 - 02

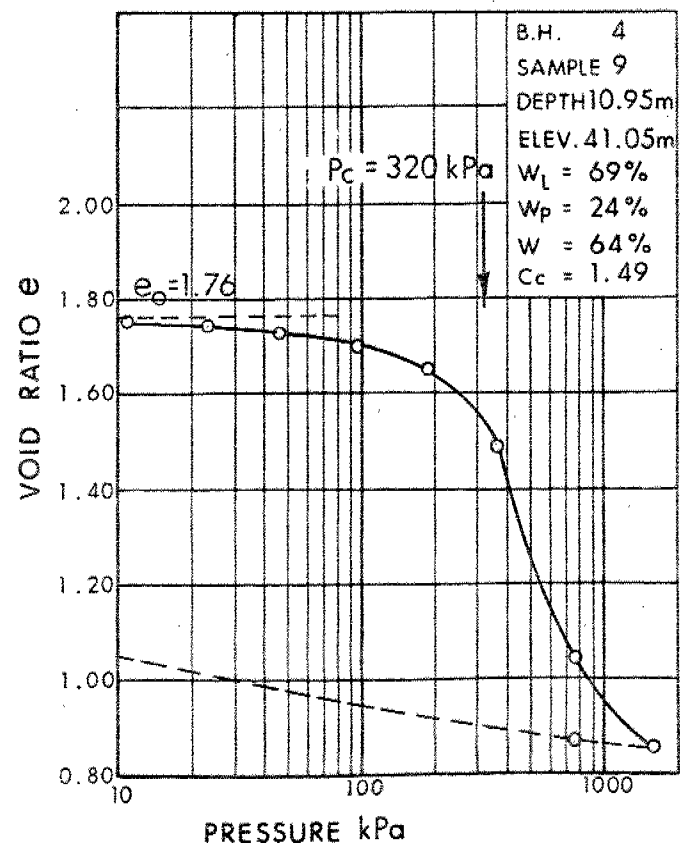
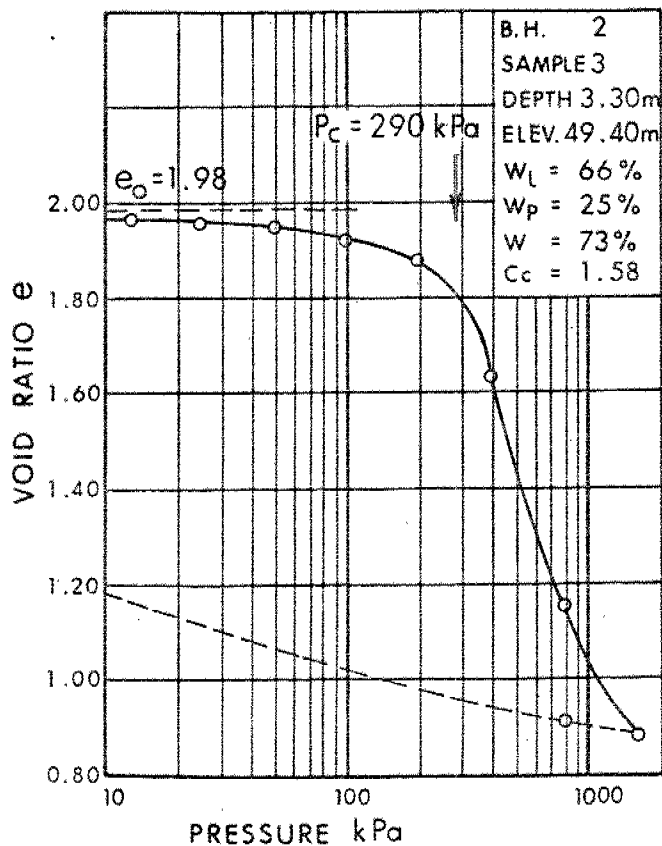
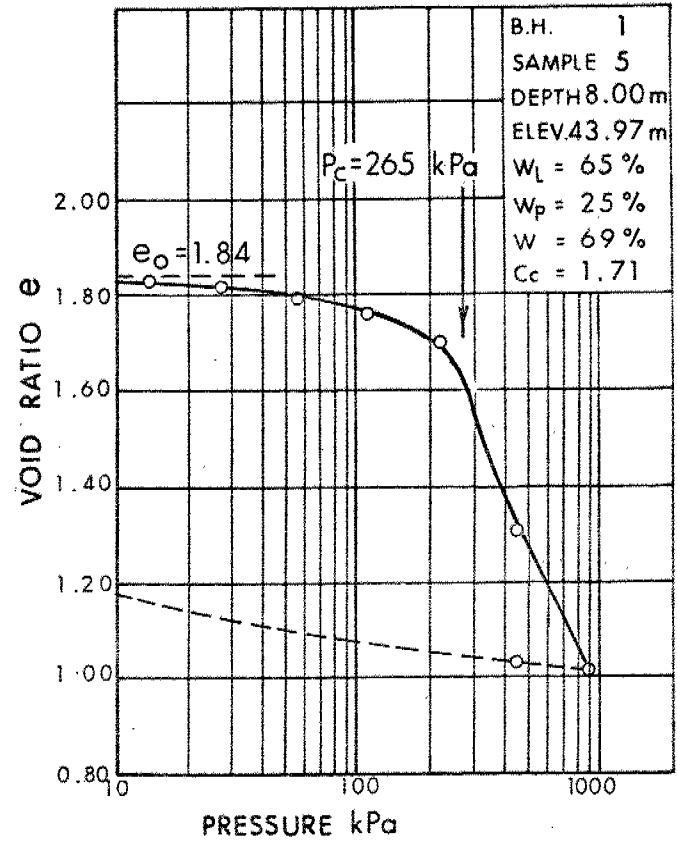
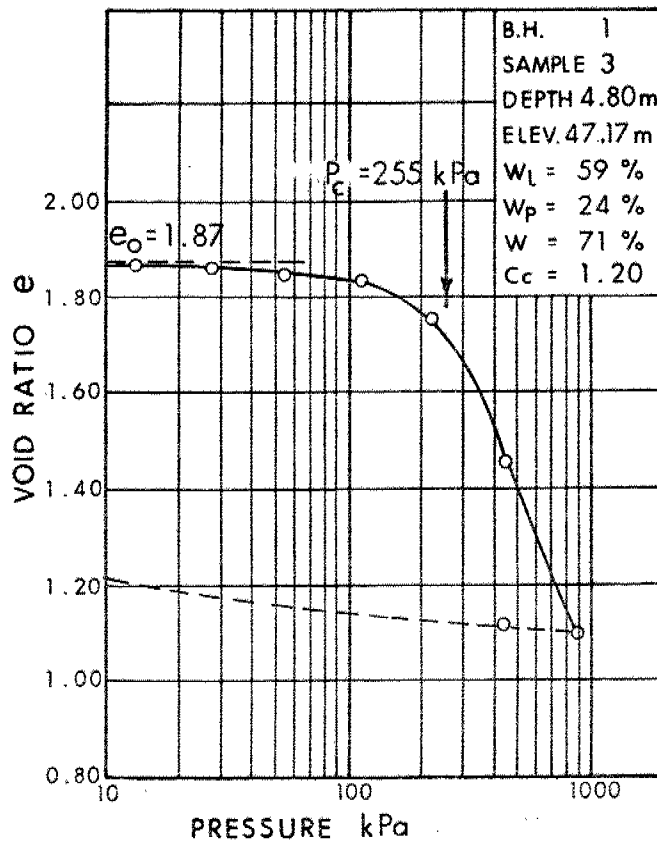
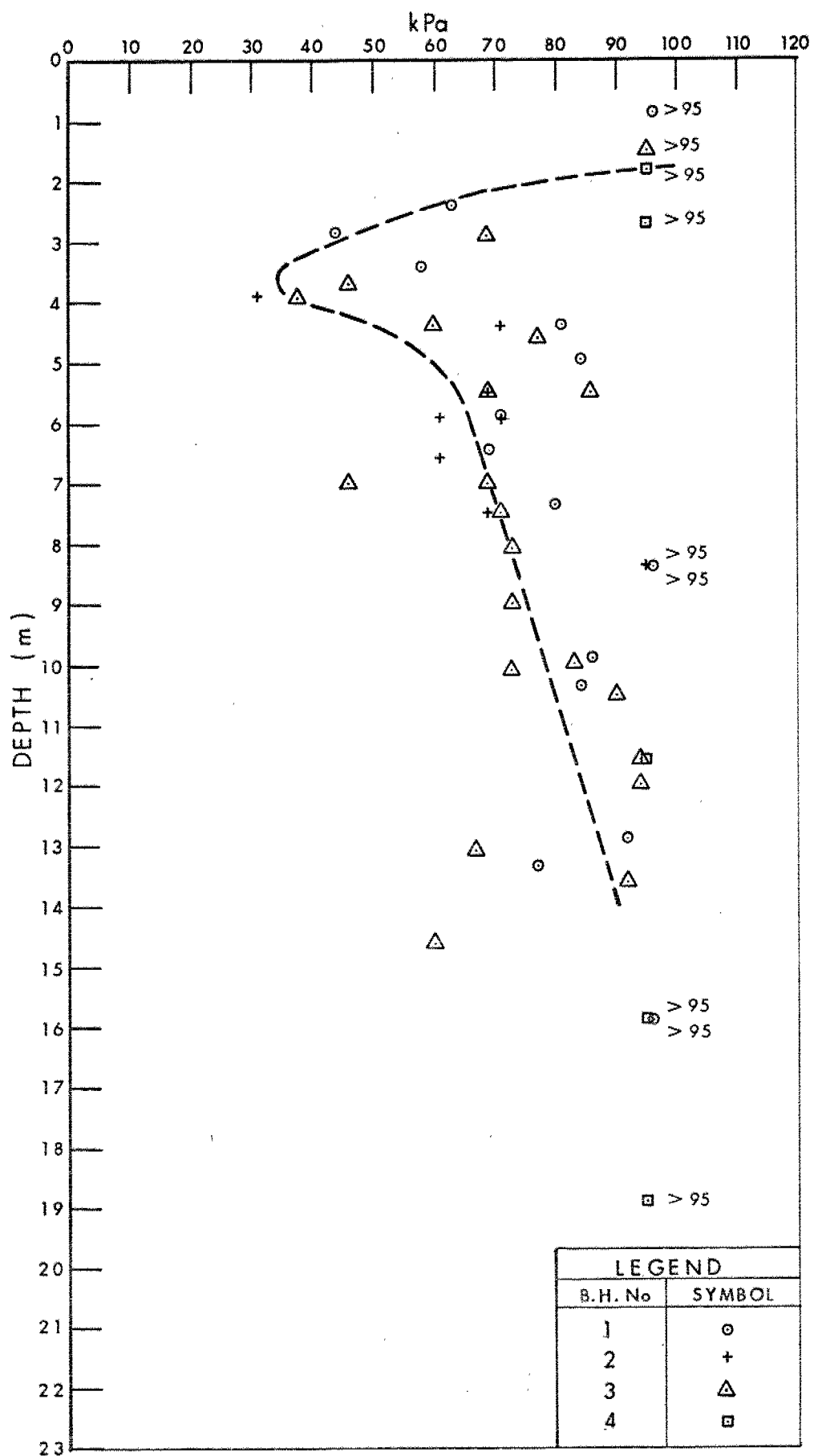


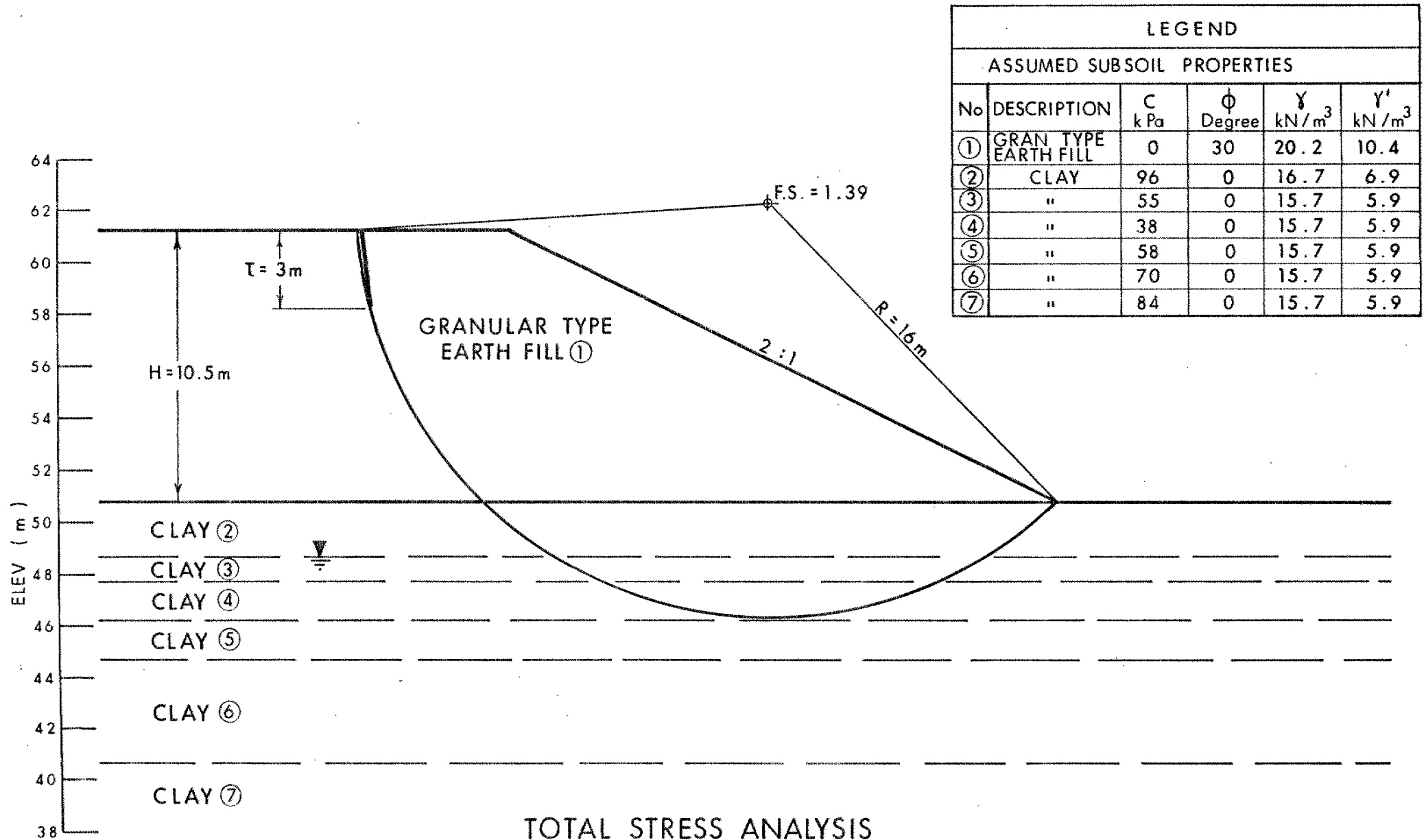
FIG. 3



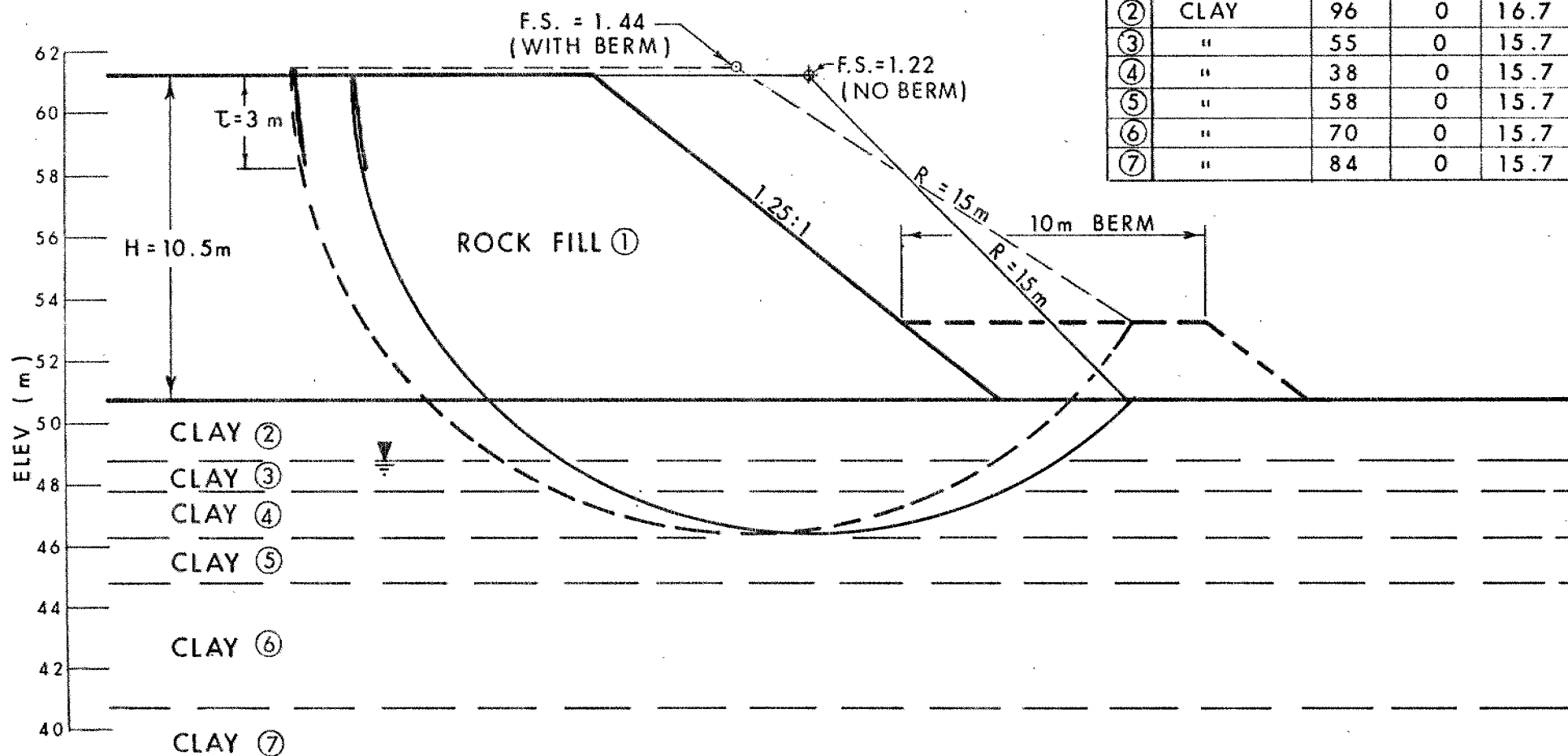
SHEAR STRENGTH vs DEPTH SUMMARY

WP 46 - 78 - 02

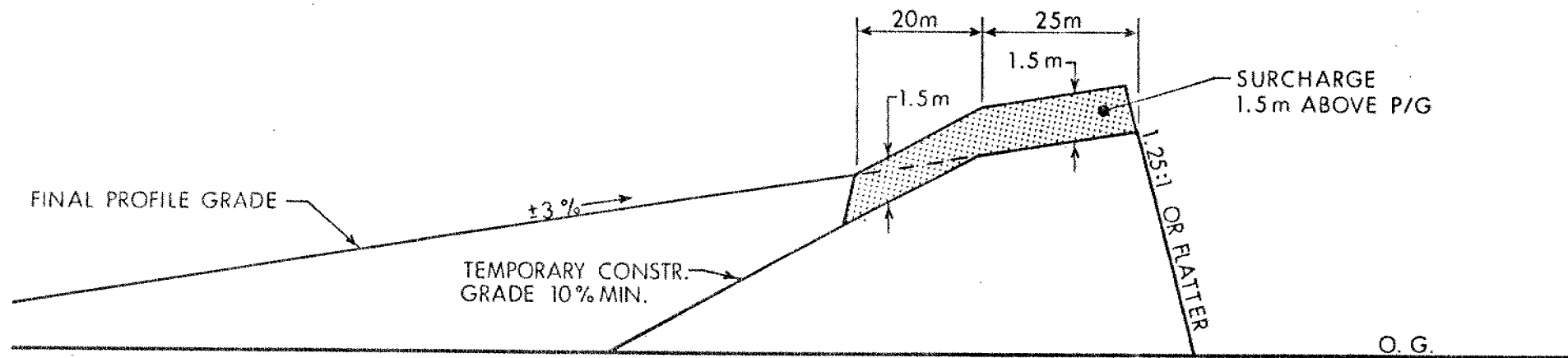
FIG 4



TOTAL STRESS ANALYSIS
 ASSUMED SUBSOIL STRATIGRAPHY, CRITICAL CIRCLE & FACTOR OF SAFETY

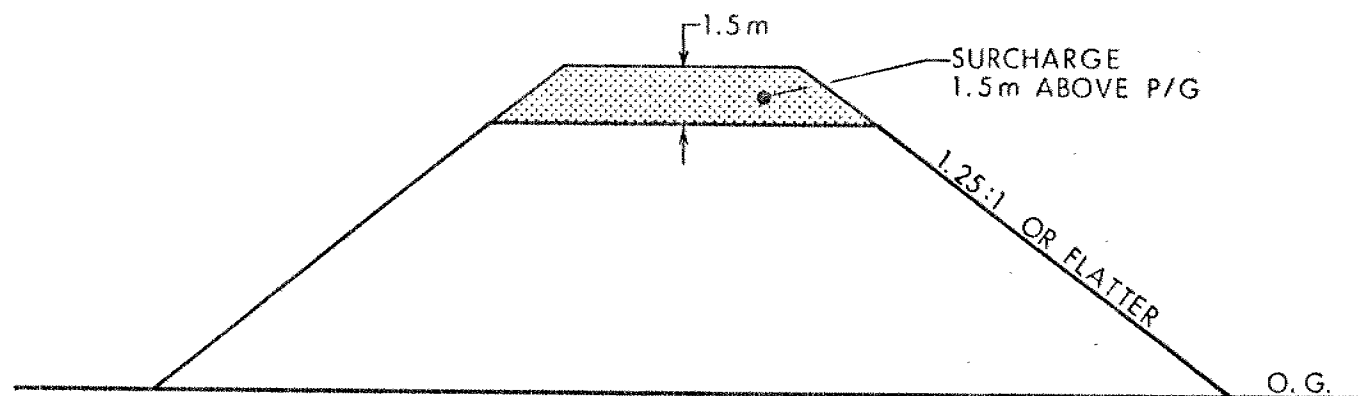


TOTAL STRESS ANALYSIS
 ASSUMED SUBSOIL STRATIGRAPHY, CRITICAL CIRCLE & FACTOR OF SAFETY



LONGITUDINAL PROFILE

APPROX. SCALE: HOR 1:1000
VERT 1:200



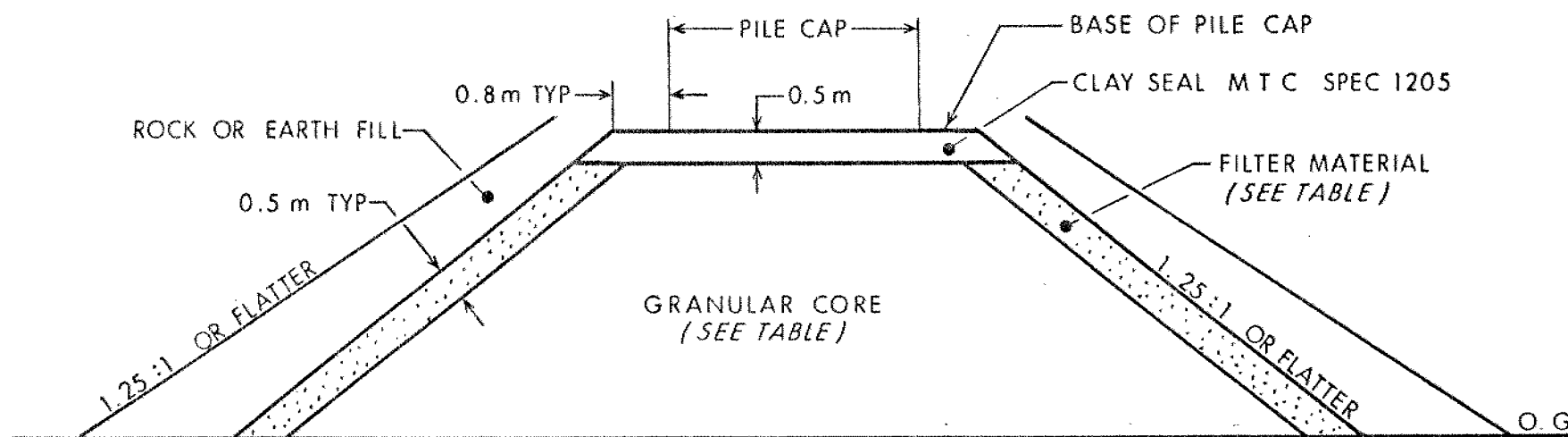
X - SECTION

APPROX. SCALE: HOR & VERT 1:200

SURCHARGE REQUIREMENTS

FIG 7

FILTER MATERIAL REQUIREMENTS	GRANULAR CORE COMPOSITION
NOT REQUIRED	GRAN 'A'
NOT REQUIRED	GRAN 'B'
GRAN 'A' or 'B'	GRAN 'C' MINUS STONES LARGER THAN 75 mm
DEPENDENT ON GRADATION OF CORE	OTHER GRANULAR MATERIAL



REQUIREMENTS FOR GRANULAR CORE

N T S

FIG 8

WP 46-78-02

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 (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

MECHANICAL PROPERTIES OF SOIL

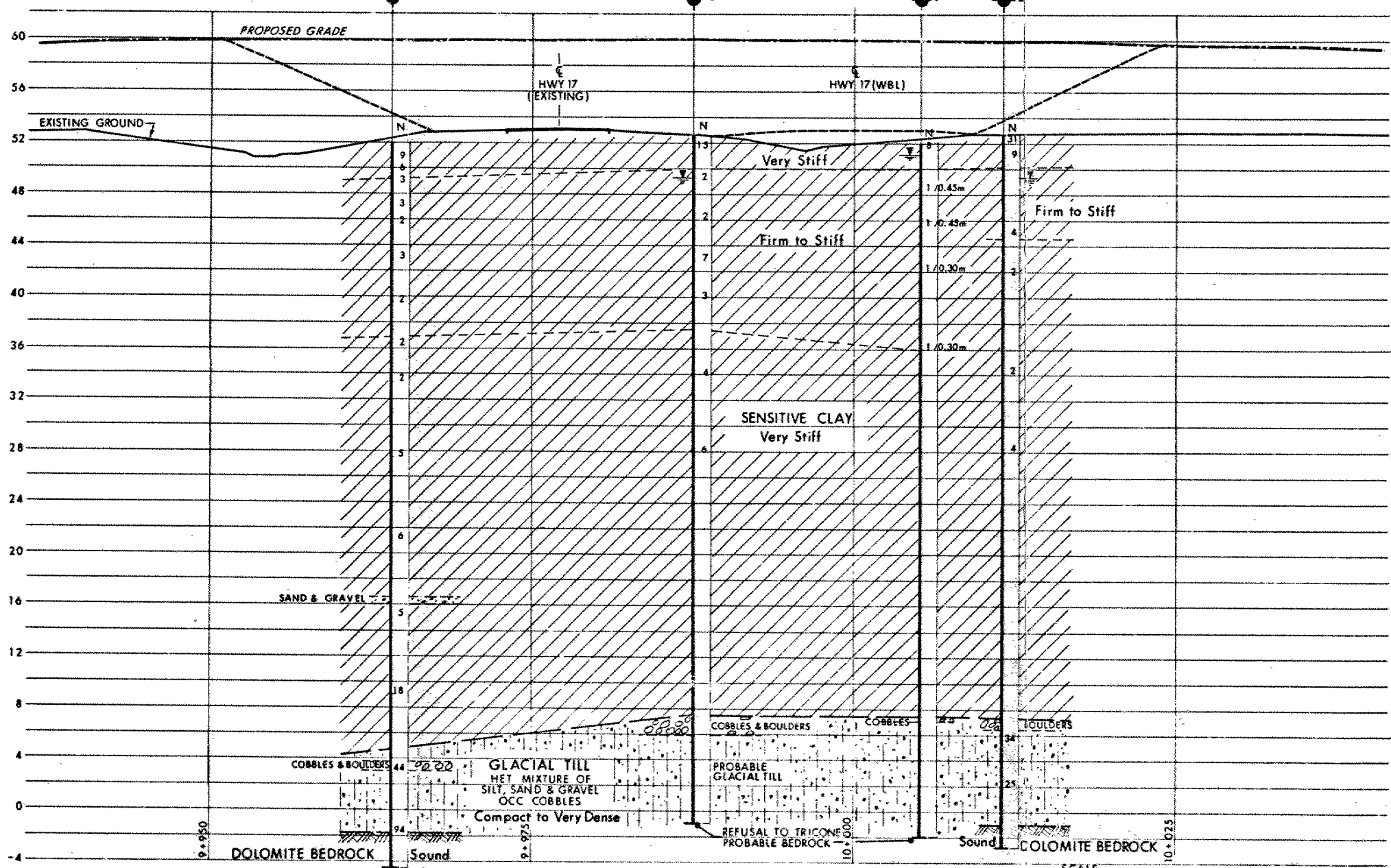
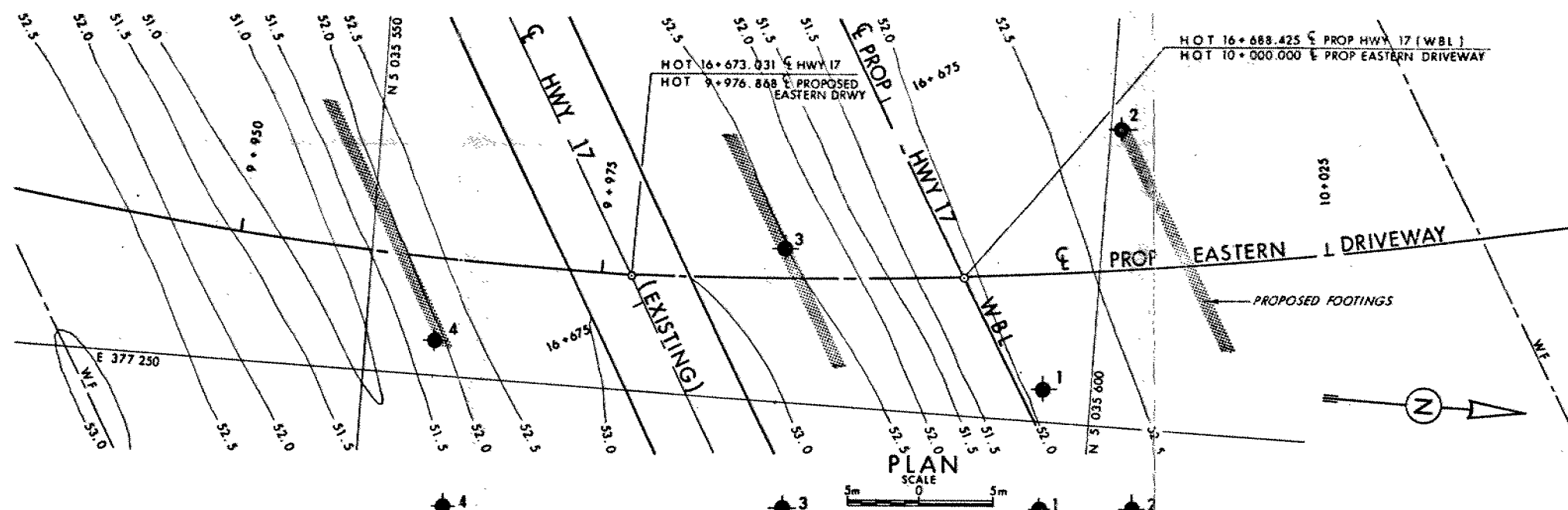
m_v	kPa^{-1}	COEFFICIENT OF VOLUME CHANGE
C_c	1	COMPRESSION INDEX
C_s	1	SWELLING INDEX
C_α	1	RATE OF SECONDARY CONSOLIDATION
c_v	m^2/s	COEFFICIENT OF CONSOLIDATION
H	m	DRAINAGE PATH
T_v	1	TIME FACTOR
U	%	DEGREE OF CONSOLIDATION
σ'_{vo}	kPa	EFFECTIVE OVERBURDEN PRESSURE
σ'_p	kPa	PRECONSOLIDATION PRESSURE
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 = $\frac{c_u}{\tau_r}$

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

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						



PROFILE - PROPOSED EASTERN DRIVEWAY

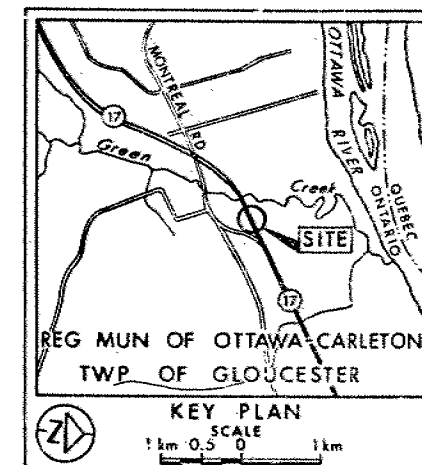
METRIC

CONT No
WP No 46-78-02

EASTERN DRIVEWAY U/PASS
(1.2 km East of Montreal Road)
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 1979 01
- WL for Bore Hole 1 1978 04

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
1	52.0	5 035 596.6	377 247.6
2	52.7	5 035 600.8	377 229.4
3	52.6	5 035 578.4	377 239.4
4	52.0	5 035 554.8	377 247.6

NOTE

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

REVISIONS	DATE	BY	DESCRIPTION

Geocres No 31 GS-133

HWY No 17	SUBMD M.M. CHECKED	DATE 1979 02 23	DIST 9
DRAWN R.S. CHECKED	APPROVED		SITE 3-335
			DWG 467802-A