

GEOCRES No. 3165-98DIST. 9 REGION W.P. No. CONT. No. REVIEW OF TECH REPORTW. O. No. 72-11087STR. SITE No. N/AHWY. No. N/ALOCATION SUBDIVISION - BEACONWOODNo of PAGES -=====OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT. REMARKS:

MINTO CONSTRUCTION LIMITED  
SOIL INVESTIGATION  
BEACONWOOD APARTMENT SITE  
GLOUCESTER TOWNSHIP                      ONTARIO

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Ottawa, Ontario.

September, 1972

72828



**Golder Associates**  
CONSULTING GEOTECHNICAL ENGINEERS

September 29, 1972.

Minto Construction Limited,  
P.O. Box 5152,  
Station F,  
Ottawa, Ontario.  
K2C 3H8

Attention: Mr. L.M. Erion, P.Eng.

RE: SOIL INVESTIGATION  
BEACONWOOD APARTMENT SITE  
GLOUCESTER TOWNSHIP, ONTARIO

Dear Sirs:

This letter reports the results of a soil investigation carried out on a parcel of land located on the north side of Montreal Road about 1000 feet west of Ogilvie Road in Gloucester Township, Ontario. Development of this property is to consist of two high-rise residential structures. The purpose of this subsurface investigation was to determine the soil and groundwater conditions at the site and, based on this information, to make recommendations for the design and construction of foundations for the proposed structures from a geotechnical viewpoint.

PROCEDURE

The field work for this investigation was carried out on September 14 and 15, 1972. Five boreholes, numbered 1 to 5 inclusive, were put down using a mobile power auger machine which was supplied and operated by the F.E. Johnston Drilling Company Ltd., of Ottawa. Standard drive open and thin walled Shelby tube samples were taken to a depth of about 30 ft. in the clay subsoil present at the site. In situ vane tests were performed at each borehole to determine the shear strength profile of this clay deposit. Standpipes were installed in boreholes 2 and 5 and water levels were observed in the open boreholes and standpipes at the termination of the field work to determine the groundwater conditions at the site. The field work was supervised throughout by a member of our engineering staff.

A detailed log of each boring is given on the Record of Borehole sheets following the text of the report. The locations of the borings together with a section of the inferred soil stratigraphy across the site are shown on Fig. 1.

The soil samples obtained during this investigation were brought back to our laboratory for detailed examination and testing. The results of the laboratory testing are shown on the Record of Borehole sheets.

The locations of the boreholes and the ground surface elevations given in this report were supplied to us by Minto Construction Limited survey personnel. These elevations are referred to Geodetic datum. The locations of a borehole previously put down by Butts, Magwood, Hall & Associates Ltd. and a borehole previously put down by us at the site are also shown on Fig. 1.

#### SITE AND GEOLOGY

The site is located on an upper river terrace about 1.4 miles east of the limits of the City of Ottawa on a parcel of land fronting on Montreal Road, from 700 ft. to 1500 ft. west of Ogilvie Road. The site slopes downward about 2 percent towards the southeast.

From available geological information it is known that bedrock in this area consists of limestone of the Ottawa formation. In general, bedrock is overlain a mantle of glacial till followed by an extensive deposit of sensitive silty clay of marine origin which extends to the ground surface. These clays were laid down in the Champlain Sea which occupied the area following the retreat of the glaciers.

#### SOIL CONDITIONS

The detailed soil stratigraphy encountered in each borehole is given on the Record of Borehole sheets and is illustrated on the stratigraphic section shown on Fig. 1. Following is a summarized account of the soil conditions at the site.

##### Sensitive Silty Clay

The principal subsoil stratum at this site is the firm to stiff grey sensitive silty clay which was found to be at least 105 ft. thick at borehole 3. The upper portion of this clay, some 12.5 to 14 ft. thick has been weathered to a very stiff crust of fissured grey brown silty clay.

The water content of the weathered crust increases with depth from about 30 percent near the ground surface to about 50 percent near the base of this desiccated stratum.

Below a depth of from 12.5 to 14 ft. the colour of the silty clay changes from grey brown to grey and the consistency decreases to firm to stiff. The in situ shear strength values below the very stiff crust are plotted on the Record of Borehole sheets; these values ranged from 1100 lb./sq. ft. in boreholes 1 and 3 to 1300 lb./sq. ft. in borehole 4, and 1400 lb./sq. ft. in borehole 5. The shear strength below the grey brown crust in borehole 2 was measured as about 720 lb./sq. ft. A plot of shear strength versus depth is shown on Fig. 2. Atterberg limit tests indicate that the grey silty clay is highly plastic (liquid limits from 60 to 80 and plasticity indices of from 34 to 48). The moisture content of the grey clay ranged from 51 to 79 percent, and was generally above the liquid limit value, typical of the sensitive clays of the Ottawa area.

#### GROUNDWATER CONDITIONS

A standpipe was installed at depth in boreholes 2 and 5 at the time of the investigation. Details of these installations are shown on the Record of Borehole sheets. Water levels taken in these standpipes and in the open boreholes on September 21, 1972 ranged from 3 ft. depth in boreholes 1, 3, and 4, to 8 ft. depth in boreholes 2 and 5. It is expected that the groundwater levels during drier periods would be lower, possibly within a few feet of the change in colour of the clay from grey brown to grey.

#### PROPOSED STRUCTURES

##### a) General

It is understood that development of this site will consist of two 13 storey residential buildings in the form of Y-towers with each wing approximately 150 ft. by 50 ft. in plan. These structures would be founded within the weathered clay crust which exists above the deep grey clay stratum at the site.

##### b) Foundations

The significant stratum for foundation design at this site is the very stiff to stiff surface crust of the silty clay stratum which extends to about 13 ft. depth. The founding of these structures near the ground surface maximizes the use of this crust and the allowable bearing value. All exterior footings and those in any unheated portions of the structures

should be provided with at least 5 ft. of earth cover for frost protection purposes. Based on the shear strength of the clay which varies from greater than 2,000 lb./sq. ft. in the upper crust to about 1,000 lb./sq. ft. in the grey clay below the weathered clay (see Fig. 2), the allowable bearing pressure of isolated spread footings and for strip footings will be dependent on footing size as well as on foundation level. The allowable bearing pressure for various combinations of founding depth and square footings size are shown on Fig. 3; Fig. 4 shows allowable bearing pressure for various combinations of founding depth and strip footing widths.

The site is underlain by a deep compressible clay stratum. The increase in load on this clay, both from footing loads and from earth fill around the structures will contribute to settlement of the footings. The results from previous consolidation tests on samples from this area indicate that the clay has been preconsolidated by 1 to 2 tons/sq. ft. in excess of present overburden pressure. It is expected that the settlement of the footings will be within tolerable limits. When the footing layout and loading is finalized and when the exterior grading design is known, a detailed settlement analysis will be carried out on the structure.

These high-rise structures are located beyond about 600 ft. from the crest of the slope along Rothwell Drive. As such, the loading from these high-rise structures would be outside the zone of influence for slope stability considerations.

An alternative foundation solution for these apartment structures would be the use of a raft or mat foundation placed at basement level in order to obtain the full buoyancy effect due to removal of the overburden weight. If a raft is employed at this site it is recommended that the net stress increase (contact pressure less weight of overburden removed) be limited to 1,000 lb./sq. ft. The use of piles driven to refusal on the surface of the underlying bedrock is also an alternative foundation solution.

#### c) Floor Slabs

In preparation for slab on grade construction at this site, the topsoil should be stripped from the site. The site may then be raised to the underside of the floor slab using compacted pit run sand and gravel material which meets M.T.C. Granular "C" specifications and topped by at least 6 ins. of crushed stone or crushed gravel of 3/4 in. nominal size.

d) Construction Procedures

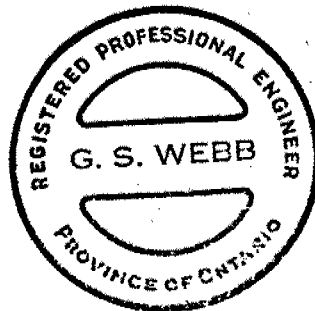
In order to avoid disturbance of the clay at and below foundation level due to construction operations or ponded water, it is recommended that a 3 in. mud mat of lean concrete be poured as soon as footing excavations are down to grade. It is recommended that the footing excavations be inspected by a Soils Engineer to verify that suitable bearing material has been reached.

No major dewatering problems are anticipated in excavations at this site. However, some water inflow is expected in excavations in the fissured clay below the ground-water level, such as at elevator pits. This water inflow should be relatively minor and should be handled by pumping from sumps.

We trust that this report contains sufficient soil information for your present design purposes. As your design proceeds, we should be given sufficient loading details to carry out a settlement analysis. Should you have any questions concerning this report or if we can be of further assistance to you on this project, please call us.

Yours very truly,

H.Q. GOLDER & ASSOCIATES LTD.



GSW/cn  
72828  
September, 1972.

*G. S. Webb*  
G.S. Webb, P.Eng.

## 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<sub>u</sub>, 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 hydrometer<sup>1</sup>  
*Q* undrained triaxial<sup>2</sup>  
*R* consolidated undrained triaxial<sup>2</sup>  
*S* drained triaxial  
*U* unconfined compression  
*V* field vane test

#### NOTES:

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

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

## LIST OF SYMBOLS

### I. GENERAL

$\tau$	= 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
$\sigma$	normal stress
$\sigma'$	normal effective stress ( $\bar{\sigma}$ is also used)
$\tau$	shear stress
$\epsilon$	linear strain
$\epsilon_{xy}$	shear strain
$\nu$	Poisson's ratio ( $\mu$ is also used)
$E$	modulus of linear deformation (Young's modulus)
$G$	modulus of shear deformation
$K$	modulus of compressibility
$\eta$	coefficient of viscosity

### III. SOIL PROPERTIES

(a) <i>Unit weight</i>	
$\gamma$	unit weight of soil (bulk density)
$\gamma_s$	unit weight of solid particles
$\gamma_w$	unit weight of water
$\gamma_d$	unit dry weight of soil (dry density)
$\gamma'$	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) *Consistency*

$w_L$	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) *Permeability*

$h$	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) *Consolidation (one-dimensional)*

$m_v$	coefficient of volume change = $-\Delta e / (1+e) \Delta \sigma'$
$C_c$	compression index = $-\Delta e / \Delta \log_{10} \sigma'$
$c_s$	coefficient of consolidation
$T_v$	time factor = $c_s t / d^2$ ( $d$ , drainage path)
$U$	degree of consolidation

### (e) *Shear strength*

$\tau_f$	shear strength
$c'$	effective cohesion
$\phi'$	effective angle of shearing resistance, or friction
$c_u$	apparent cohesion*
$\phi_u$	apparent angle of shearing resistance, or friction
$\mu$	coefficient of friction
$S_f$	sensitivity

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

## RECORD OF BOREHOLE 1

LOCATION See Figure 1

BORING DATE SEPT 15, 1972

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, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		SHEAR STRENGTH Cu, LB./SQ. FT.				WATER CONTENT, PERCENT					
								20	40	60	80	1x10	1x10	1x10	1x10		
OVER AUGER 4.5" DIAM (UNCORRECTED)	268.5	GROUND SURFACE															
	0.8	CROWN TOP SOIL		1	2"	7											
		VERY STIFF TO STIFF GRAY BROWN SILTY CLAY (WEATHERED CRUST)		2	"	26											
				3	"	33											
				4	"	6											
	254.5																
	140	STIFF GRAY SILTY CLAY		5	"	2											
			6	2"	PM												
			7	2"	2												
238.5	30.0	END OF HOLE															

0  
5  
10  
Percent axial strain at failure

 VERTICAL SCALE  
1 IN. TO 5 FT.

Golder Associates

 DRAWN G.F.  
 CHECKED B.W.

 W.L. IN  
 OPEN HOLE  
 AT ELEVATION  
 265.5  
 SEPT 21, 1972

RECORD OF BOREHOLE 2

LOCATION See Figure 1

BORING DATE SEPT. 15, 1972

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, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20	40	60	80	1x10	1x10	1x10	1x10		
								SHEAR STRENGTH Cu., LB./SQ.FT.		NAT. V. - + Q. - ● REM. V. - ● U. - ○		WATER CONTENT, PERCENT					
								500	1000	1500	2000						
POCKET PENETROMETER	272.4	GROUND SURFACE					275										
	0.5	BROWN TOPSOIL		1	2"	15	270										
		VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)		2	"	30	265										
				3	"	24	260										
				4	"	7	255										
	259.4	FIRM TO STIFF GREY SILTY CLAY		5	"	2	250	⊕	+								
	13.0						250	⊕		+							
				6	2"	TO PM	245	⊕		+							
				7	3"	TO PM	245	⊕		+							
	242.4						240										
	30.0	END OF HOLE														W.L. IN STANDPIPE AT ELEVATION 264.4 SEPT. 21, 1972	

Percent axial strain at failure

6  
15 5 Percent axial strain at failure  
10

# RECORD OF BOREHOLE 3

LOCATION See Figure

BORING DATE SEPT 15, 1972

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, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		20 40 60 80				1x10 1x10 1x10 1x10					
								SHEAR STRENGTH Cu, LB./SQ.FT.				WATER CONTENT, PERCENT					
								500 1000 1500 2000				wp — w — ws					
POWELL AUGER ELEV. 270.0 265.3 256.3 245.0 240.0 170.0 165.0 160.0							270										
	268.4	GROUND SURFACE															
	0.5	BROWN TOPSOIL		1	2"	DO 8											
				2	"	31	265										
		VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)		3	"	21	260										
	256.3			4	"	7											
	12.5			5	"	4	255	⊕	+								
		STIFF GREY SILTY CLAY		6	3" TO PM		250	⊕	+								
				7	2"	DO 4	245	⊕	+								
							240	⊕	+								
						170											
		PROBABLY STIFF GREY SILTY CLAY					165										
	163.3	END OF HOLE					160										

W.L. IN OPEN HOLE AT ELEVATION 265.8 SEPT 21, 1972

0  
15  
10  
5 Percent axial strain at failure

W.L. IN OPEN HOLE AT ELEVATION 265.8 SEPT 21, 1972

6  
15 5 Percent axial strain at failure  
10

VERTICAL SCALE  
1 IN. TO 5 FT.

Golder Associates

DRAWN GF  
CHECKED SW

## RECORD OF BOREHOLE 4

LOCATION See Figure 1

BORING DATE SEPT. 14, 1972

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., CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.											
								20	40	60	80	1x10	1x10	1x10	1x10		
POWER AUGER 4 5/8" DIAM UNCASED	265.2	GROUND SURFACE															
	0.2			1	2"	009											
		VERY STIFF TO STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)		2	"	60											
				3	"	26											
				4	"	7											
	254.2																
	14.0			5	"	2											
		STIFF GREY SILTY CLAY		6	3" TO	PM 245	⊕		+								
							⊕		+								
	240.7						⊕		+								
27.5	END OF HOLE																

W.L. IN OPEN HOLE AT ELEVATION 265.2 SEPT 21, 1972

5 Percent axial strain at failure

VERTICAL SCALE  
1 IN. TO 5 FT.

Golder Associates

DRAWN RF  
CHECKED RF

## RECORD OF BOREHOLE 5

LOCATION See Figure 1

BORING DATE SEPT 4, 1972

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, CM./SEC.				ADDITIONAL LAB. TESTING	PIEZOMETER OR STANDPIPE INSTALLATION
	ELEV'N. DEPTH	DESCRIPTION	STRAT. PLOT	NUMBER	TYPE	BLOWS/FT.		SHEAR STRENGTH Cu., LB./SQ.FT.				WATER CONTENT, PERCENT					
								20	40	60	80	1x10	1x10	1x10	1x10		
POWER AUGER SEE FIGURE 10 (UNCLASSIFIED)	268.2	GROUND SURFACE															
	0.2	268.2 TO 265.7		1	2"	23											
		VERY STIFF GREY BROWN SILTY CLAY (WEATHERED CRUST)		2	"	38											
			3	"	28												
			4	"	17												
	255.7	265.7 TO 255.7		5	"	4											
	12.5	STIFF GREY SILTY CLAY		6	3"	15											
	7		2"	10													
239.2	255.7 TO 239.2																
	239.2 TO 235.0																
	235.0 TO 230.0																
	230.0 TO 225.0																
	225.0 TO 220.0																
	220.0 TO 215.0																
	215.0 TO 210.0																
	210.0 TO 205.0																
	205.0 TO 200.0																
	200.0 TO 195.0																
	195.0 TO 190.0																
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	100.0 TO 95.0																
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	85.0 TO 80.0																
	80.0 TO 75.0																
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	35.0 TO 30.0																
	30.0 TO 25.0																
	25.0 TO 20.0																
	20.0 TO 15.0																
	15.0 TO 10.0																
	10.0 TO 5.0																
	5.0 TO 0.0																

VERTICAL SCALE  
1 IN. TO 5 FT.

Golder Associates

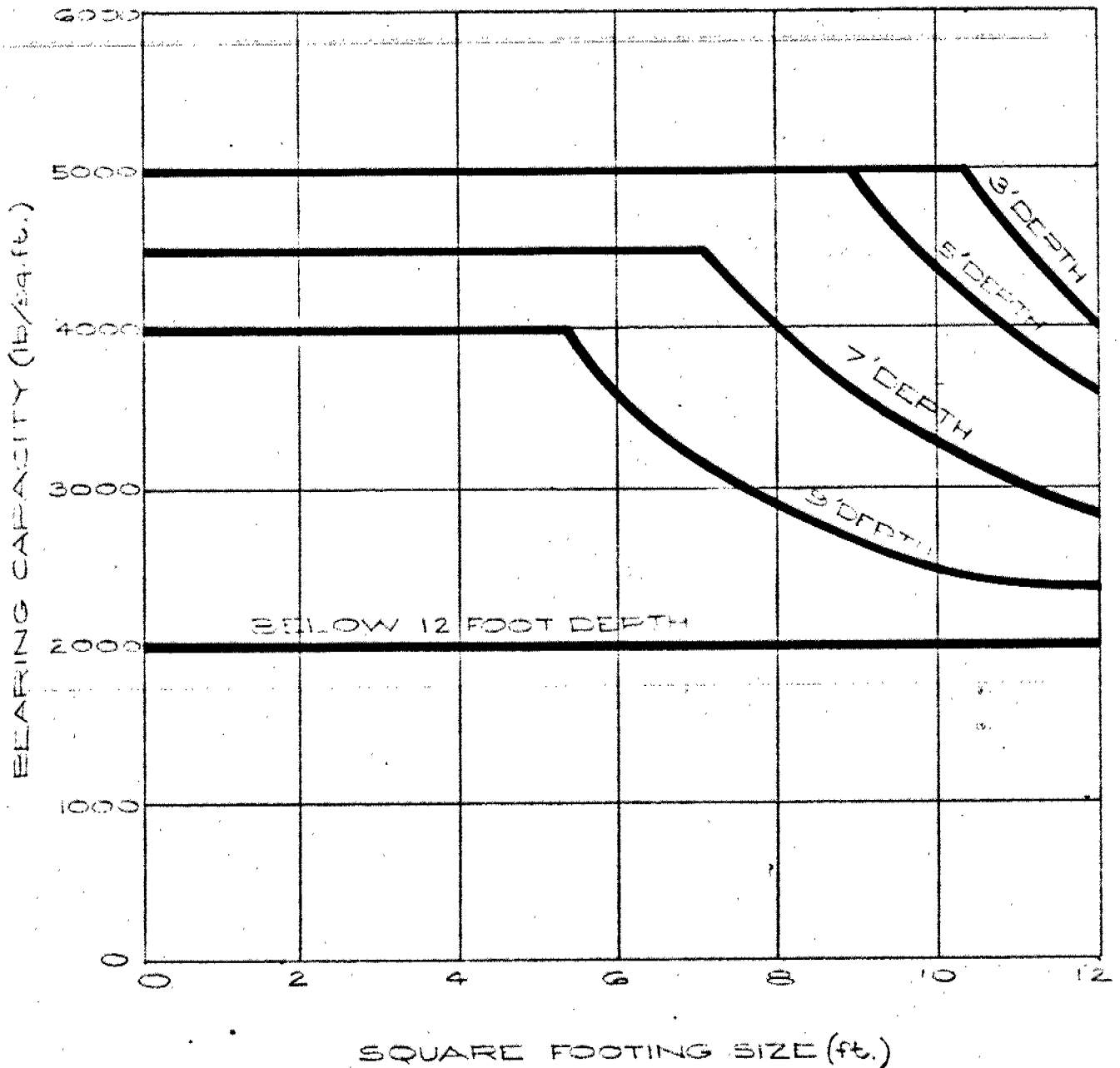
DRAWN SE  
CHECKED SW

# OVERSIZE DRAWING



# BEARING CAPACITY FOR SQUARE FOOTINGS

FIGURE 3.



Date SEPT. 22, 1972

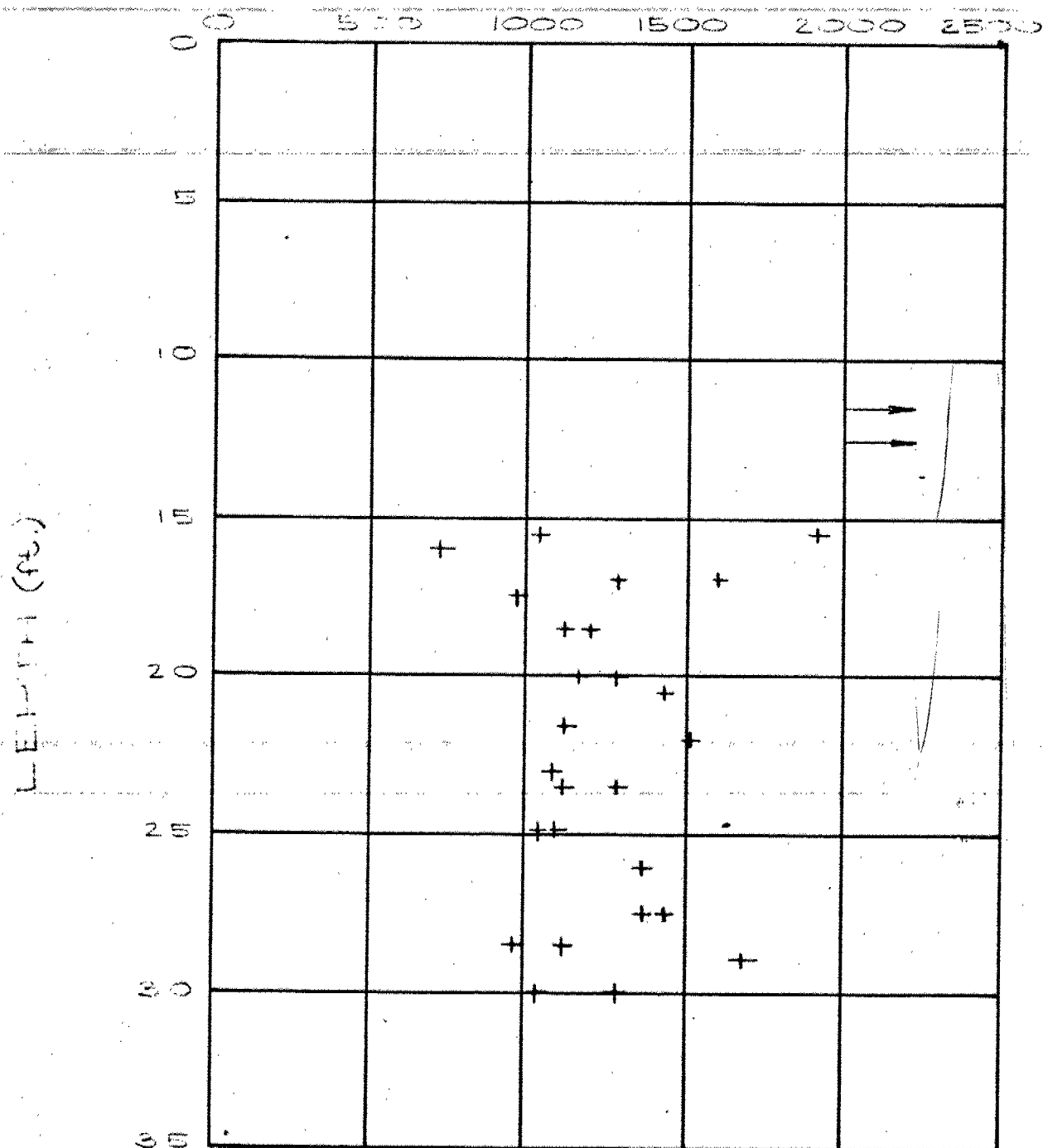
**Golder Associates**

Drawn G.F.  
Chkd. 22/5  
Appd. 3/7

# SHEAR STRENGTH VERSUS DEPTH

FIGURE 2

SHEAR STRENGTH,  $c_v$  (lb./sq. ft.)



## LEGEND

+ IN SITU FIELD VANE TEST

Date SEPT. 22, 1972

**Golder Associates**

Drawn SEA  
Chkd. SEA  
Appd. SEA

72-11087

**BUTTS, MAGWOOD & HALL LTD.**  
CONSULTING CIVIL ENGINEERS

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Tel: (613) 224-1414

GEOC 3105-98

SUBSURFACE INVESTIGATION  
FOR  
MINTO CONSTRUCTION LIMITED  
AT  
BLOCKS "N" AND "S", BEACONWOOD,  
TOWNSHIP OF GLOUCESTER  
ONTARIO

JOB REF. 12-208-L

OCT. 1972

## TABLE OF CONTENTS

<u>SECTION</u>	<u>DESCRIPTION</u>	<u>PAGE NO.</u>
1.	INTRODUCTION	1
2.	TOPOGRAPHY AND GEOLOGY	2
3.	SOIL PROFILE	2 & 3
4.	COMPUTER ANALYSES - SLOPE STABILITY	3
5.	DISCUSSION	3 & 4
6.	RECOMMENDATIONS	4, 5, & 6

### APPENDIX

Site and Borehole Location Plan  
Soil Profiles  
Scarp Cross Sections  
Borehole Logs  
Allowable Bearing Value Chart  
Letter Report Re Computer Analyses

## 1. INTRODUCTION

A soils investigation was made in Blocks "N" and "S" Beaconwood, Township of Gloucester. This investigation was authorized by Work Order No. 37567 issued by Mr. L. Erion, P.Eng., of Minto Construction Limited. This investigation was made to establish the soil types in the area, the depth of the soil, the slope stability of the scarp that runs through the area and using this data to define two lines in the area of Block "N" as follows:

- a) A line to delineate the most southerly and westerly limit for excavating structural foundations, utility and drainage trenches.
- b) A line to delineate the most southerly and westerly limit for earth grading works to be performed.

Additionally Butts, Magwood & Hall Limited were requested to consider the possibility of unusual trench and other construction conditions and to make recommendations for any special construction methods that may be required.

The soils investigation consisted of seven boreholes, four of which were advanced to refusal. Undisturbed samples were recovered from all boreholes in accordance with A.S.T.M. Procedure 1587. These samples were used for the purpose of classification and to establish their strength properties.

A borehole location plan and the borehole logs are appended to this report. The locations of these boreholes and their elevations were established by Minto Construction Limited. All elevations shown are related to Geodetic Datum.

Previous reports written by H.Q. Golder and Associates and J.D. Paterson and Associates and papers authored by Crawford, Eden and Mitchell and published by the National Research Council of Canada were reviewed prior to writing this report.

The soils stability analyses were made by R.J. Mitchell, Ph.D., using the computer facilities at Queen's University, Kingston.

## 2. TOPOGRAPHY AND GEOLOGY

The area of the proposed development is located partially in the basin of a former mud flow, Block "N", and partially adjacent to and above the former slide, Block "S". The two blocks are divided by the backscarp of the mud flow. The Geological Survey of Canada have established the age of the mud flow as approximately 1,000 years.

Within the bowl, the topography is somewhat humpy and drainage is poorly developed. Several swampy areas exist. Above the scarp, the area is well drained by field tile and has been farmed in the past.

The soils located during the subsurface investigation are known to be unusually sensitive. That is, they lose a considerable portion of their strength when disturbed. They are thought to have been deposited in brackish water some ten thousand years ago at the end of the last glacial invasion.

Profiles "C" through "Q" appended to this report, were surveyed to establish the shape of the scarp face at various locations. This data was used in the slope stability analyses discussed later in this report.

## 3. SOIL PROFILE

Throughout the area investigated, greyish brown stiff clay that tended to become more greyish with depth, was located in all boreholes. Four boreholes, located above the scarp, were advanced to a depth of 30 feet and undisturbed samples were recovered. Cones were then driven in these boreholes until refusal was encountered. This was encountered in boreholes 1, 3 and 4, at an approximate elevation of 170 feet. In borehole 2, refusal was encountered at elevation 206 feet.

It was noted that the unconfined compression strength in the clay tended to decline with depth. Above the scarp the average unconfined strength was found to be:

0 - 10 ft.	3.3 T.S.F.
10 - 20 ft.	1.6 T.S.F.
20 - 30 ft.	1.3 T.S.F.

Below the scarp, three boreholes were advanced to depths of 14 feet. The average unconfined compression values in this area were found to be:

0 - 7 ft.	1.9 T.S.F.
7 -14 ft.	1.0 T.S.F.

Profiles A-A and B-B depicting these soil conditions are appended to this report.

#### 4. COMPUTER ANALYSES - SLOPE STABILITY

The following parameters were used to establish the critical arcs by computer:

- (a) Bulk Unit Weight 100 P.C.F.
- (b) Pore Water Pressure Ratio 0.62
- (c) Apparent Cohesion, 420 P.S.F.
- (d) Apparent Friction Angle, 32°.

The pore water conditions in this analyses are assumed to be the worst possible. The strength parameters were selected by Dr. Mitchell with reference to his past research and experience in this area. His letter report is appended to this report.

#### 5. DISCUSSION

From inspection of the deduced safety factors, based on critical conditions, it can be seen that the area under study is stable at this time (see sections, dwg. no. 3). However, any change in the existing soil moisture regime could mobilize the soil in the area of the scarp face.

Therefore, the following recommendations should be made a matter of record for both long and short term consideration. With the enactment of these recommendations, all proposed construction operations in the proximity of the scarp, will be restricted to areas where the factor of safety to prevent soil movement is generally considered acceptable.

The lines defined to restrict both grading and excavation were established by the following method. The defined critical arcs were assigned a safety factor of unity and the restricting boundaries were arbitrarily established at 100 and 200 feet from the critical arc lines. The distances to the restricting boundaries were then reduced in inverse proportion as the critical arc safety factor was raised to the calculated values. In this way, boundaries related to the slope safety factor were established. Furthermore, the recommended construction procedures will ensure that long term conditions, such as leakage of pipes and ponding, will not occur and lower the design factor of safety at some future time.

6. RECOMMENDATIONS

a) Scarp Face

It is recommended that no machinery or construction equipment be permitted beyond the northern limit of the Rothwell Drive road allowance and south of the "Limit of Grading Line" shown on the site plan. Further no temporary or permanent access for vehicles should be permitted between Rothwell Drive and Block "N".

Construction of utility trenches should be kept to a minimum in this area and should only be permitted when dug at right angles to the scarp contours. Adequate backfilling and compaction in these trenches should be considered of prime importance.

b) Grading, Excavation and Trenching Operations Block "N"

Grading operations should not be permitted south of the "Limit of Grading" shown on the site plan. Where grading and landscaping is necessary, no net decrease in the overburden weight should be permitted. The line should be established by survey and delineated by snow fence to ensure that machinery does not inadvertently work in the restricted area.

Excavation and trenching in Block "N" should not be permitted south of the "Limit of Excavation" line also shown on the site plan. North of the limit of excavation line, proposed locations and depths of excavations in Block "N" should be reviewed and approved by a Geotechnical Engineer.

6. c) Grading, Excavation and Trenching Operations Block "S"  
No grading operation should be permitted north of the Rothwell Drive road allowance. It is recommended that snow fence be erected along the northern boundary of the road allowance to prevent machinery from approaching the top of slope.

- \* For construction operations in the Rothwell Drive road allowance, it is recommended that no net increase in overburden weight be permitted.
- \* No watermains should be constructed along Rothwell Drive and it is recommended that the storm sewer located between Ogilvie Road and Marquis Avenue be located as close to the southern boundary of the road allowance as practical, and at minimum permissible depths. It should be made a matter of record that this storm water sewer should be reserved solely for the drainage of Rothwell Drive.

No temporary storage of construction materials should be permitted north of a line that extends parallel to the Rothwell Drive road allowance and 25 feet south of it.

All water service lines within 170 feet and south of the southern boundary of Rothwell Drive road allowance should be provided with perforated pipe drainage lines to ensure that any future leakage that may occur will be removed from the area. The perforated pipe should be constructed to drain in a southerly direction.

No structures should be located within 25 feet of the southern boundary of the Rothwell Drive road allowance. Proposed locations should be reviewed and approved by a Geotechnical Engineer. Under no circumstances should the basement excavations be permitted to stand open and fill with surface water. Within 70 feet of the road allowance, construction should be restricted to four storey buildings and any planned highrise structures should be located south of this line. It is recommended that above ground swimming pools be located not closer than 100 feet from the southern boundary of the road allowance.

Note \* Rothwell Drive and services are at the present time under construction in accordance with registered plan no. 878. These recommendations have been complied with.

d) Preliminary Allowable Bearing Values

Allowable bearing values, for both Blocks "N" and "S", shown as a function of depth, are appended to this report. These values are intended to be used for planning purposes only and further detailed investigation is recommended when the locations of the proposed major structures are ascertained.


e) Trench Excavation

Trench excavation in this area should present no unusual problems. Intrusion of ground water into the trench excavation will be minimal and can be handled by pumping from within the trench. A box or close sheeting will be required in all trenches greater than 4 feet in depth unless the sides are sloped at a 1 to 1 slope.

f) Geotechnical Engineering Services

It is recommended that a qualified Geotechnical Engineer be engaged to supervise all aspects of the work discussed in this report and other grading and trenching operations that may be required in the area of this development. The Geotechnical Engineer should be required to provide written certification that the recommendations of this report have been carried out satisfactorily and in accordance with the approved plans.

BUTTS, MAGWOOD & HALL LTD.

A handwritten signature in dark ink, appearing to read 'G.R. Hall', is written over a horizontal line.

G.R. Hall, P.Eng.

GRH/amcp

A P P E N D I X

**BUTTS, MAWWOOD & HALL LTD.**  
**CONSULTING CIVIL ENGINEERS**

CLIENT Minto Construction Limited.  
 LOCATION Beacon Wood.

REMARKS \_\_\_\_\_

BOREHOLE ELEVATION 246.6

**BOREHOLE ANALYSIS**  
 BOREHOLE NO. 1  
 DRILLING DATE Apr. 7/72  
 TESTING DATE Apr 10 & 13/72

PENETRATION DATA	HAMMER	DROP
CASING		
CONE		
SAMPLER		

**DEPTHS MEASURED FROM GROUND LEVEL**

Depth	Cone Penetration					Description and Remarks	Sample					Vane					Water Table		
	Blows/foot						Type No.	Blows/ft	MC	LL	PL	PI	U.C.	und.	rem.	sen.		U.W.	Date
6"						Topsoil.													
						Very stiff fissured gray brown clay, some (-) silt.	T01						2.76						
10'						Very stiff brown-gray clay, some silt.	T02						2.37						
						Stiff gray clay, some silt.	T03						1.39						
													1.22						
20'						Stiff gray clay, some (-) silt.	T04						1.42						
						Firm to stiff gray clay, some (-) silt.	T05						0.97						
30'						Firm gray clay, some silt, slightly organic	T06						0.76						

**Symbols**

MC = Moisture content  
 LL = Liquid limit  
 PL = Plastic limit  
 PI = Plasticity index  
 U.C. = Unconfined compressive strength - tons sq/ft  
 U.W. = Unit weight  
 und. = Undisturbed shear strength Tons/sq ft  
 rem. = Remoulded " " "  
 sen. = Sensitivity - und

Cone probe driven to refusal at 78'9"

Plate No.

# BUTTS, MAGWOOD & HALL LTD. CONSULTING CIVIL ENGINEERS

BOREHOLE ANALYSIS  
BOREHOLE NO. 2  
DRILLING DATE Apr. 10/72  
TESTING DATE Apr 13/72

CLIENT Minto Construction Limited.  
LOCATION Beacon Wood.

REMARKS \_\_\_\_\_

BOREHOLE ELEVATION 263.3

PENETRATION DATA	HAMMER	DROP
CASING		
CONE	140lbs.	30 ins.
SAMPLER		

## DEPTHS MEASURED FROM GROUND LEVEL

Depth	Cone Penetration		Description and Remarks	Sample		Vane										Water Table	
	Blows/m	Blows/foot		Type No.	Blows/m	MC.	L.L.	P.L.	P.I.	U.C.	und.	rem.	sen.	U.W.		Date	Time
5"			Topsoil														
			Hard to very stiff gray brown clay, some silt.	T01						4.02							
10'			Very stiff gray brown clay, some silt.	T02						2.88							
			Very stiff gray clay, some (-) silt.	T03						2.20							
20			Very stiff gray clay, some (-) silt.	T04						2.21							
			Firm to stiff gray clay, some (-) silt.	T05						0.96							
30			Stiff gray clay, some (-) silt.	T06						1.73							

### Symbols

MC. = Moisture content  
L.L. = Liquid limit  
P.L. = Plastic limit  
P.I. = Plasticity index  
U/C = Unconfined compressive strength tons sq/ft  
U.W. = Unit weight  
und. = Undisturbed shear strength Tons/sq ft  
rem. = Remoulded " " "  
sen. = Sensitivity - und rem

Cone probe driven to refusal at 57'0"

Plate No.

**BUTTS, MAGWOOD & HALL LTD.**  
**CONSULTING CIVIL ENGINEERS**

CLIENT Hinto Construction Limited.

LOCATION Beacon Wood.

REMARKS \_\_\_\_\_

BOREHOLE ELEVATION 272.4

## BOREHOLE ANALYSIS

BOREHOLE NO. 3

DRILLING DATE Apr. 11/72

TESTING DATE APR 13/72

PENETRATION DATA	HAMMER	DROP
CASING		
CONE	140lbs.	30 ins.
SAMPLER		

DEPTHS MEASURED FROM GROUND LEVEL

Depth	Cone Penetration		Description and Remarks	Sample		Vane								Water Table					
	Blows/m	Blows/Foot		Type No.	Blows/m	M.C.	L.L.	P.L.	P.I.	U.C.	und. rem. sen.				U.W.				
6"			Topsoil																
			Hard gray brown clay some silt.	T01						4.76									
10'			Badly fissured gray brown clay, some silt.	T02						1.72									
			Stiff fissured gray clay, some(-) silt.	T03						1.61									
20'			Stiff to firm gray clay, some(-) silt.	T04						1.03									
			Stiff gray clay, some (-) silt.	T05						1.40									
30'			Firm to stiff gray clay some (-) silt.	T06						0.99									

## Symbols

**M.C. = Moisture content**

L.L = Liquid limit

P.L. = Plastic limit

P.I. = Plasticity Index

U/C = Unconfined compressive strength tons sq/ft

U.W. = Unit weight

und. = Undisturbed shear strength Tons/sq ft

rem. = Removed

sen. = Sensitivity -  $\frac{y_{11}}{y_{11} + y_{12}}$

Cone probe driven to refusal at 98'6".

Plate No.

**BUTTS, MAGWOOD & HALL LTD.**  
**CONSULTING CIVIL ENGINEERS**

**BOREHOLE ANALYSIS**  
 BOREHOLE NO. 4  
 DRILLING DATE Apr. 14/72  
 TESTING DATE Apr 25/72

CLIENT Minto Construction Limited.  
 LOCATION Beacon Wood.

REMARKS \_\_\_\_\_

BOREHOLE ELEVATION 259.2

PENETRATION DATA	HAMMER	DROP
CASING		
CONE	140 lbs.	30 ins.
SAMPLER		

**DEPTHS MEASURED FROM GROUND LEVEL**

Depth	Cone Penetration				Sample		Vane										Water Table	
	Blows/foot				Type No.	Blows/ft	M.C.	L.L.	P.L.	P.I.	U.C.	und.	rem.	sen.	U.W.		Date	Time
10'																		
20'																		
30'																		

**Symbols**

M.C. = Moisture content  
 L.L. = Liquid limit  
 P.L. = Plastic limit  
 P.I. = Plasticity index  
 U.C. = Unconfined compressive strength tons sq/ft  
 U.W. = Unit weight  
 und. = Undisturbed shear strength Tons/sq ft  
 rem. = Remoulded " " "  
 sen. = Sensitivity - und rem

Cone probe driven to refusal at 93'6".

Plate No.

**BUTTS, MAGWOOD & HALL LTD.**  
**CONSULTING CIVIL ENGINEERS**

**BOREHOLE ANALYSIS**  
 BOREHOLE NO. 5  
 DRILLING DATE Apr. 14/72  
 TESTING DATE Apr. 25/72

CLIENT Minto Construction Limited.  
 LOCATION Beacon Wood.

REMARKS \_\_\_\_\_

BOREHOLE ELEVATION 227.4

PENETRATION DATA	HAMMER	DROP
CASING		
CONE		
SAMPLER		

**DEPTHS MEASURED FROM GROUND LEVEL**

Depth	Blows/ft	Description and Remarks	Sample		M.C.	L.L.	P.L.	P.I.	U.C.	Vane			U.W.	Water Table	
			Type No.	Blows/ft						und.	rem.	sen.		Date	Time
3"		Topsoil													
		Very stiff gray brown clay, some (-) silt.	T01						2.13						
		Firm gray brown clay, some (-) silt.	T02						0.80						
		Firm gray clay, some (-) silt.	T03						0.72						
14'		Stiff to firm gray clay, some (-) silt.	T04						1.02						

**Symbols**

M.C. = Moisture content  
 L.L. = Liquid limit  
 P.L. = Plastic limit  
 P.I. = Plasticity index  
 U.C. = Unconfined compressive strength tons sq/ft  
 U.W. = Unit weight  
 und. = Undisturbed shear strength Tons/sq ft  
 rem. = Remoulded " " "  
 sen. = Sensitivity - und rem

Plate No.



BOREHOLE ANALYSIS  
BOREHOLE NO. 7  
DRILLING DATE Apr. 17/72  
TESTING DATE May 1/72

PENETRATION DATA	HAMMER	DROP
CASING		
CONE		
SAMPLER		

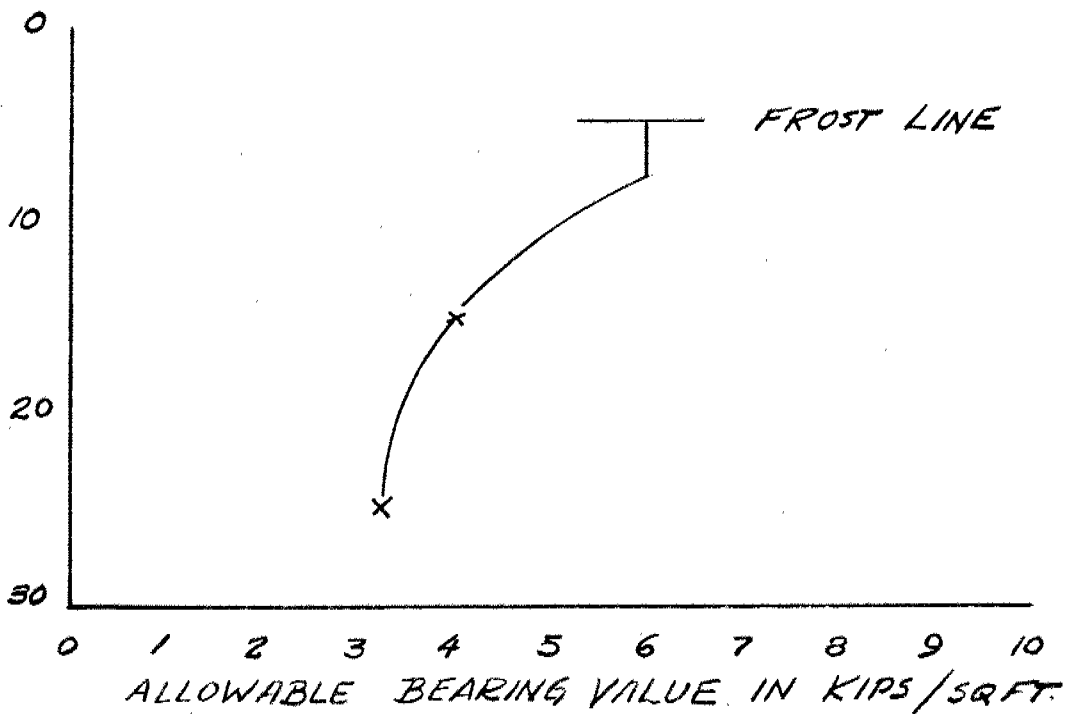
BOREHOLE ELEVATION 226.3

Depth	Blows/m	Blows/foot	Description and Remarks	Sample		M.C.	L.L.	P.L.	P.I.	U.C.	Vane			U.W.	Water Table	
				Type No.	Blows/m						und.	rem.	sen.		Date	Time
3"			Topsoil													
			Stiff brown clay, some(-) silt.	T01						1.83						
			Fissured brown clay, some(-) silt.	T02						1.53						
										0.85						
			Firm to stiff brown clay, some(-) silt.	T03						0.96						
14'			Soft to firm gray clay, some(-) silt.	T04						0.48						

M.C. = Moisture content  
 L.L. = Liquid limit  
 P.L. = Plastic limit  
 P.I. = Plasticity index  
 U.C. = Unconfined compressive strength tons sq/ft  
 U.W. = Unit weight  
 und. = Undisturbed shear strength Tons/sq ft  
 rem. = Remoulded " " "  
 sen. = Sensitivity - und rem

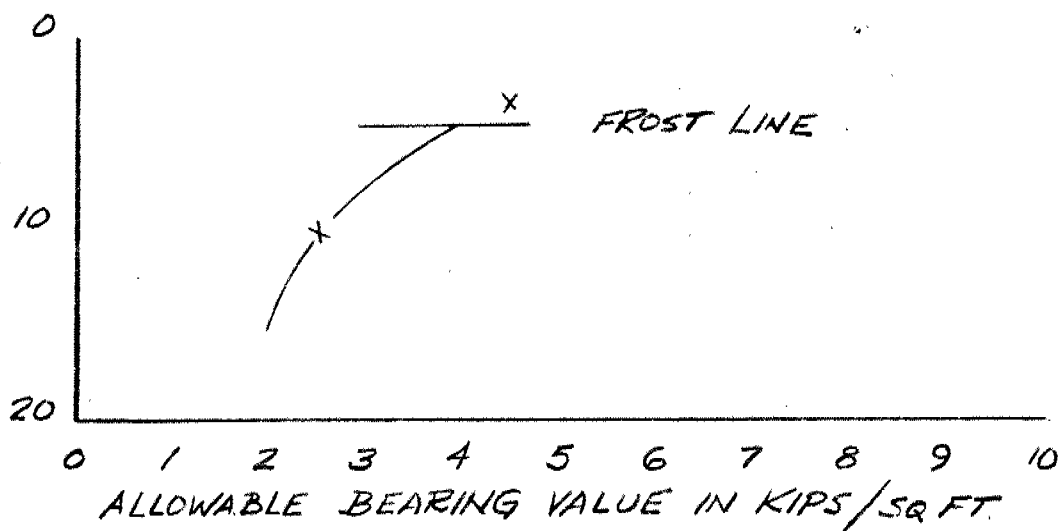
7

DEPTH TO BOTTOM OF FOOTING  
FROM EXISTING GRADE - FEET



RECOMMENDED PRELIMINARY BEARING  
VALUES - BLOCK S - BEACONWOOD

DEPTH TO BOTTOM  
OF FOOTING FROM  
EXIST. GRADE - FEET



RECOMMENDED PRELIMINARY BEARING  
VALUES - BLOCK N - BEACONWOOD

Robert J Mitchell, Geotechnical Consulting  
Woodbine Road, R.R. #3, Kingston, Ontario

14 June 1972

Mr. G.R. Hall, P.Eng.,  
Butts, Magwood & Hall Ltd.,  
1489D Merivale Rd., Ottawa 5, Ont.

Dear Mr. Hall:                      Your ref 12-208-L

I enclose the results of stability analyses as per your correspondence received 12 June, 1972. The following soil parameters were used throughout the analysis:

Bulk unit weight = 100 p.c.f. (Crawford and Eden, 1966 NRC 8763)  
Pore water pressure parameter,  $r_u = 0.62$  (hydrostatic sat.)  
Strength parameters  $c' = 420$  p.s.f. (Golder Associates data)  
 $\phi' = 32^\circ$

The strength parameters noted above fall into the range published by Eden and Mitchell, 1970 for these soils and are considered the most appropriate values to use in stability analyses. Factors of safety for the low stress range ( $c' = 200$  p.s.f.,  $\phi' = 37.5^\circ$ ) are included, however, for your information. Co-ordinates (with ref to your profiles) of the critical centres and the radius of the critical circular failure arcs are given in the attached table. The calculated factors of safety (F.S.) are also tabulated. The critical arc and other possible failure arcs are drawn on the profiles for your convenience. The dashed failure arcs sketched on the profiles are approximations to the shallower failures applicable to the low stress range strength parameters noted above.

Since toe erosion, artesian pressures, and progressive strength deterioration are all considered absent in this location, the calculated factors of safety may be considered to represent the minimum long term conditions and the slopes are considered to be

stable providing:

- (1) Material is not removed from the area of the toe of a slope
- (2) Fills, above ground pools, buildings etc. are not placed within the areas defined by the possible failure arcs shown on the profiles.

The calculated factor of safety, based on the low stress parameters  $c' = 200$  p.s.f.,  $\phi' = 37.5^\circ$ , for slope profile C (F.S. = 1.2) is considered minimal but adequate. It is recommended, however, that major permanent structures and services be located, if possible, a minimum distance of 150 ft. from the top of this slope (profile C) due to the likelihood of minor retrogression should an initial slip develop. Alternatively, slopes with profiles as steep as profile C may be flattened (from the existing 2.4:1 to, say, 2.75:1) to improve the safety factor.

I trust this information will be useful to you and include a typical computer output (profile A). Should additional data of this form be useful I would forward the remaining output. I also enclose my account in the sum of one hundred and eighty dollars for analyses provided.

Yours sincerely,  
ORIGINAL SIGNED BY.

Robert J. Mitchell, P.Eng.

PROFILE NO.	CRITICAL CENTRE CO-ORD		RADIUS FT.	F.S.	F.S. FOR $C'=200$ P.S.F. $\phi'=37.5^\circ$ B&M CHARTS $r_u=0.62$
	X	Y			
A	240.0	350.0	122.5	1.73	1.56
B	220.0	360.0	130.0	2.14	1.76
C	260.0	320.0	85.4	1.60	1.20 SEE
D	220.0	320.0	76.2	2.15	1.75 NOTE
E	220.0	320.0	89.4	1.79	1.38
F	1.90	320.0	97.8	1.75	1.35

Tabulated data from analysis of Beacon Hill  
slope profiles A,B,C,D,E,&F.

Note: The factors of safety noted in this column are considered to be lower than the operative F.S. for shallow failures since the  $r_u$  value is unlikely to be as high as 0.62 on the shallow failure arcs. In addition, the use of an average slope angle in the charts leads to a slightly conservative prediction in the profiles analyzed.

# OVERSIZE DRAWING

DOCUMENT MICROFILMING IDENTIFICATION

GEOCRES No. 316-98

DIST. 9 REGION

W.P. No. 128-87-09

CONT. No.

W. O. No.

STR. SITE No. 3-551

HWY. No. 416

LOCATION HWY 416 & CAMBRIAN RD.

U'PASS (STR #23)

No of PAGES -

=====

OVERSIZE DRAWINGS TO BE INCLUDED WITH THIS REPORT.

REMARKS:

G.I-30 SEPT. 1976



Ministry of  
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## **FOUNDATION DESIGN SECTION**

# **foundation investigation and design report**

ENGINEERING MATERIALS OFFICE  
FOUNDATION DESIGN SECTION

WP 128-87-09

DIST 9

HWY 416

STR SITE 3-551

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# FOUNDATION INVESTIGATION REPORT

For

Hwy. 416 and Cambrian Road Underpass

W.P. 128-87-09, Site No. 3-551

Hwy. 416, District 9, Ottawa

## INTRODUCTION

This report summarizes the results of a foundation investigation conducted at the aforementioned site. It is proposed to construct a two span structure which will carry Cambrian Road over the proposed Hwy. 416. This report contains factual information obtained from this investigation pertaining to structural foundations and related earth works within 100 m of the structure.

## SITE DESCRIPTION

The site is located approximately 1 km south of Jock River and 85 m west of Cedarview Road along Cambrian Road in the City of Nepean, Regional Municipality of Ottawa-Carleton.

The topography of the area consists of a flat plain occupied by farmers fields. East of Cedarview Road there is a swampy woodlot consisting mainly of cedars. Approximately 500 m west of the site there is a sand and gravel ridge which rises 22 m above the plain. To the north of Cambrian Road this ridge is being mined for aggregates and to the south lies the Trail Road Landfill site owned and operated by the Regional Municipality of Ottawa-Carleton.

At the time of the investigation the surrounding farmers fields and ditches were filled with water. The surface water drains north to Jock River through ditches along Cedarview Road and a ditch located 60 m west of the centreline of Hwy. 416.

Physiographically, the site lies in the area known as the Ottawa Valley clay plains founded in the lowlands of the St. Lawrence, which are characterized by clay plains interrupted by ridges of rock or sand and gravel. The bedrock in the area is of the Gull River formation of the

middle ordovician period. It consists of limestone with interbedded shale layers. The overburden is relatively thin and was deposited during and immediately following the wisconsinan glaciation at which time the area was depressed from the effect of the glaciation.

### INVESTIGATION PROCEDURES

Soil data and inherent properties were obtained by in situ and laboratory testing. The procedures employed are discussed below.

#### Field Investigation

The field work for the investigation was carried out between 89 05 18 to 89 05 30 and consisted of eight sampled boreholes which were advanced to a maximum depth of 27.8 m (elevation 64.3 m) below the ground surface. Boreholes were located along the relocated Cambrian Road south of the existing road with two boreholes placed at east abutment, two placed at the piers, two placed at the west abutment and two boreholes located within the east/west approach ramps. Dynamic cone penetration tests were advanced at all borehole locations with 2 additional tests at centre pier locations and one test approximately 50 m west of the proposed Hwy. 416 centreline. Dynamic cone tests were advanced to a maximum depth of 24.7 m (elevation 67.0 m). Bedrock cores were obtained at one location (BH 23-2). Artesian conditions overlying the bedrock made it difficult to core bedrock at the other borehole locations.

The elevations of the boreholes advanced at the site varied from 91.7 m to 92.3 m.

In general, the subsoil samples in the surficial cohesive overburden were retrieved at 0.8 to 1.5 m intervals using a shelly tube sampler in accordance with Standard Procedures (ASTM D 1587). The shelly tube samples were supplemented by retrieving disturbed subsoil samples using a split spoon sampler in accordance with Standard Practice (ASTM D 1587). The shelly tubes were used to provide relatively undisturbed material for laboratory evaluation and testing. In view of the extreme sensitivity of

the soil at this site, special care was required for sample and handling the samples. The sensitive clay is difficult to recover because it liquifies along the inside wall of the shelby tube when the tube is pushed into the clay.

In situ vane tests were also conducted between the aforementioned sampling intervals to determine the undisturbed and remolded undrained shear strengths of the cohesive deposits. The vane shear test was conducted employing the standard MTO 'N' value in accordance with ASTM D 2573 change in consistency due to remolding (sensitivity) was also found by measuring the ratio of the undrained, unconfined compression strengths before and after remolding. The clay is soft and sensitive so the vane had to be carefully lowered to the required depth. In areas of questionable shear strengths additional vane testing was conducted.

All subsoil samples were identified in the field and returned to the laboratory for further examination and applicable testing.

Water levels were monitored throughout the duration of the investigation in open boreholes, by installing two piezometers and in the case of artesian aquifer conditions water levels were monitored by stabilizing the head in the casing or augers by extending them to greater heights. All boreholes were backfilled upon completion of the field work.

Survey information related to the location and elevation of boreholes was provided by the Eastern Region, Surveys and Plans Section.

#### Laboratory Analysis

The following laboratory tests were carried out on select soil samples.

- 1) Atterberg Limit Test
- 2) Grain Size Distribution
- 3) Unit Weights
- 4) Natural Moisture Contents

Laboratory Test results are given in the following section of this report and are illustrated on Figures and Borehole Log sheets included in the Appendix.

#### SUBSURFACE CONDITIONS

The subsoil stratigraphy consisted of a surficial deposit of slightly desicated brown clayey silt ranging in thickness from 2.1 m to 2.7 m. This deposit is underlain by an extensive deposit of 14.6 m to 15 m of sensitive grey clayey silt to silty clay. Below the above deposits rests a 3.7 m to 6.4 m thick deposit of silty clay to clay. This deposit confines an aquifer contained in approximately 2 m of sand and gravel. The sand and gravel deposit overlies dolomite bedrock.

West of the site there is a sand and gravel ridge being mined for aggregates. The sand and gravel ridge is pervious and is believed to be the source for the confined augifer underlying the clay at the Cambrian Road site.

The plan and location of borings and the stratigraphical profile are shown on Dwg. No. 1288709-A in the attached appendix. The field and laboratory test results are plotted on the Record of Borehole sheets also included in the Appendix of this report. A brief description of the different soil types is given below.

#### Clayey Silt, some Sand, trace Organics

The surficial material consists of 2.1 m to 2.7 m of a desicated brown clayey silt, which contains a trace of organics and topsoil at or near the surface. Cambrian Road fill crosses this deposit and consists of 1 m and sand and gravel. This deposit extends down to an elevation of 90.0 m being relatively flat throughout the site with a distinct transition from the relatively strong brown clayey-silt to the underlying weaker grey clayey silt to silty clay.

Results of Grain Size Distribution tests carried out on select samples are shown on Figure 1 in the Appendix, in an envelope form. The results summarize Grain Size Distribution Tests carried out on this material throughout the site. The results indicate the material contains a large percentage of clay and silt with some sand. The deposit is comprised primarily of 0% gravel, 11-19% sand, 51-70% silt and 15-39% clay.

The results from the Atterberg Limit Tests performed on the fine fraction of this deposit is summarized as follows:

	<u>Range</u>	<u>No. of Tests</u>
Natural Moisture Content (w)	11-43	7
Liquid Limit ( $w_L$ )	25-32	7
Plastic Limit ( $w_p$ )	12-22	7
Plastic Index ( $I_p$ )	9-17	7
Sensitivity	5-10	5
Undrained Shear Strength $C_u$ (kPa) (Field)	27-54	5

From the Plasticity Chart (Figure 2), the layer can be classified as a clayey silt of low plasticity. Unit weight measurements carried out on samples from this stratum yielded dry unit weights of 17.9 kg/m<sup>3</sup> to 19 kg/m<sup>3</sup>.

In this stratum the shear strength (in situ) varied from 27-54 kpa indicating a firm to stiff consistency. The measured shear strengths are widely scattered due to the variable desiccation of this surficial deposit, however a clear decrease with depth can be seen (see Figure 2). In situ sensitivity ranged from 6 to 10 corresponding to a sensitive to very sensitive classification.

The Standard Penetration resistance 'N' values ranged from 2 to 9 blows/0.3 m.

Clayey Silt to Silty Clay with pockets of Organics, and occasional silty Fine Sand Seams

The brown clayey silt is underlain by a much softer and more sensitive grey clayey silt to silty clay deposit starting at an elevation of 90.0 m. This deposit is 14.6 m to 15 m thick with the bottom of the deposit relatively flat at elevation 75.0 m to 75.4 m. This marine clay increases in plasticity with depth such that at its lower boundary it is a silty clay. It contains charcoal black organics particles throughout distinguishing this deposit from the underlying silty clay to clay. Occasional seams of silty sand were encountered at the upper boundary of this deposit.

Results of Grain Size Distribution Tests carried out on select samples are shown on Figure 3 in the Appendix, in an envelope form. The results indicate the material contains a very large percentage of clay and silt with a trace of sand. The deposit is comprised primarily of 0-2% gravel, 2-17% sand, 34-56% silt and 32-64% clay.

The results from the Atterberg Limit Tests performed on the fine fraction of this deposit is summarized as follows:

	<u>Range</u>	<u>No. of Tests</u>
Natural Moisture Content (w)	34-62	25
Liquid Limit ( $w_L$ )	30-50	25
Plastic Limit ( $w_p$ )	13-24	25
Plastic Index ( $I_p$ )	11-28	25
Sensitivity	3-100	240
Undrained Shear Strength $C_u$ (kPa) (Field)	12-56	240
Unconfined Compression (U-kPa)	13-35	16
Unconsolidated Undrained (UU-kPa)	17-38	10

From the Plasticity Chart (Figure 4), the layer can be classified as a clayey silt to silty clay of low to intermediate plasticity. The average natural moisture content of this deposit exceeds the liquid limit by an average of 15%. When disturbed, this deposit will liquify because of the high moisture content.

The undrained shear strength ranges from 12 kPa to 56 kPa as indicated in Figure 5, which plots shear strength vs. elevation profile.

Observations reveal that the soil decreases in strength from an average of 32 kPa at the upper limit of this deposit to an average of 20 kPa between elevation 88 m to elevation 84 m. Below elevation 84 m the undrained shear strength gradually increases with depth to an average strength of 45 kPa at the lower boundary of this deposit. The shear strength has an average range of 8 to 10 kPa at any particular depth. The range could be caused by the random occurrence of silty fine sand seams. The sensitivity ranges from extra sensitive to quick throughout the deposit with higher sensitivities usually in the upper half. Unit weight measurements carried out on samples from this stratum yielded dry unit weights of 16-18.2 kN/m<sup>3</sup>.

Consolidation testing in the laboratory was also performed to determine the settlement characteristics of this deposit. Key inherent parameters measured from e vs. log curves are given below:

Table 1

	<u>Elevation (m)</u>	<u>Pc (kPa)</u>	<u>Cc</u>	<u>e<sub>o</sub></u>	<u>Cv (m<sup>2</sup>/year)</u>
BH 23-1 (TW6)	84.4	110	0.474	1.022	
BH 23-1 (TW8)	81.0	108	0.700	1.644	
BH 23-5 (TW3)	88.8	59	0.413	1.139	10.62
BH 23-5 (TW7)	82.6	145	0.856	1.394	
BH 23-6A (TW3)	85.8	102	0.716	1.249	4.14
BH 23-8 (TW7)	82.7	100	1.10	1.611	6.96
BH 23-11 (TW4)	87.3	82	0.584	1.181	

Cv - Coefficient of Consolidation

Pc - Preconsolidation Pressure - using casagrande

Cc - Coefficient Construction of Compressibility

e<sub>o</sub> - initial void ratio

\*Laboratory curves corrected using schmertmann method to account for the sampling and preparation disturbances.

The Figures indicate that the preconsolidation pressure ranged from 59 to 145 kPa and is on average 37 kPa greater than the existing effective overburden pressure.

Silty Clay to Clay, trace of Sand

Underlying the above material encountered at an elevation of 75.4 m to 75.0 m (3.7 m to 7 m thick) is a deposit of silty clay to clay, trace sand. The lower boundary of this material is not clearly defined because the deposit contains occasional sand and gravel pockets below elevation 74.0 m which increase in frequency with depth and as a result no clear demarcation can be defined between this layer and the underlying sand and gravel.

Categorization of the sand and gravel deposit has been based on the detection of artesian conditions. This deposit was distinguished from the overlying clayey silt to silty clay by a lighter grey colour and by the absence of black organic particles and silty fine sand seams.

Results of Grain Size Distribution Tests carried out on select samples are shown on Figure 6 in the Appendix, in an envelope form. The results indicate the material contains a very large percentage of clay and silt. The deposit is comprised of 0% gravel, 0-1% sand, 39-56% silt, and 44-60% clay.

The results from the Atterberg Limit Tests performed on the fine fraction of this deposit is summarized as follows:

	<u>Range</u>	<u>No. of Tests</u>
Natural Moisture Content (w)	52-62	5
Liquid Limit ( $w_L$ )	42-49	5
Plastic Limit ( $w_p$ )	20-22	5
Plastic Index ( $I_p$ )	22-28	5
Sensitivity	5-13	5
Undrained Shear Strength $C_u$ (kPa) (Field)	42-60	5
Unconfined Compression (U-kPa)	35-41	3
Unconfined Undrained (UU-kPa)	32-52	2

From the Plasticity Chart (Figure 7), the layer can be classified as having intermediate plasticity. The Natural Moisture Content is high in this deposit and exceeds the liquid limit. Bulk density ranged from 16-17 kN/m<sup>3</sup>.

The undrained shear strength ranged from 42 kPa to 60 kPa as indicated in Figure 5, which plots shear strength vs. elevation profile. The deposit thus has a firm to stiff consistence. The sensitivity ranges from sensitive to extra-sensitive, thus this deposit was not as sensitive as the overlying deposit.

Consolidation Tests were performed to determine the compressibility characteristics of this deposit. The key parameters taken from void ratio-pressure curves are given below.

	<u>Elevation (m)</u>	<u>Pc (kPa)</u>	<u>Cc</u>	<u>Co</u>
BH 23-1 (TW13)	73.7	205	1.18	1.517

\*See Previous deposit for parameter definitions.

#### Sand and Gravel, occasional Cobbles and Boulders

This permeable non-cohesive deposit underlies the silty clay to clay deposit. It is composed of sand and gravel with occasional boulders and cobbles and it overlies bedrock.

The upper boundary of this deposit ranges from elevation 71.7 m to 68.6 m and it is approximately 2.3 m thick. The exact boundaries of this deposit were only established in BH 23-2, because of the artesian conditions. The remaining boreholes were terminated above within this stratum. However, sampling and test wells done for the Region of Ottawa-Carleton Trail Road landfill site by Gartner Lee and Associates indicates that the sand and gravel deposit tends to thicken towards the west with the upper boundary rising and the thickness of the overlying confining clay decreasing. Due to the artesian conditions split spoon sampling was not done in this

deposit. Cone tests could not penetrate this deposit probably due to the presence of coarser particles within this deposit.

### Dolomite Bedrock

Bedrock was proven by obtaining BXL cores in one borehole (BH 23-2). In addition, Gartner Lee and Associates established bedrock elevations in a test well situated 65 m west of the proposed Hwy. 416 centreline. The bedrock is fairly flat at elevation 66.3 m west of Hwy. 416. Extrapolation of bedrock elevations in the area allows us to assume it also extends east of Hwy. 416 at an elevation of 66.3 m, however none of the boreholes penetrated to bedrock in this area.

Detailed descriptions of the rock are attached in the Appendix.

Core Recoveries and Rock Quality Designation (RQD) were determined in situ and also in the laboratory to evaluate the competence and integrity of the rock.

Based on these results, the bedrock can be described as fine grained containing interbeds of silt, strong to very strong, and unweathered. Rock Core Recovery and Rock Core Quality Designation values ranged from 91-100% and 0-77% respectively.

### GROUNDWATER

Groundwater was observed to be perched on the surface of the clayey silt layer in the surrounding farm fields. Within the surficial deposit groundwater is 0.3 m to 0.9 m below the ground surface. It rises from the west at elevation 91.2 m to the east at 91.9 m. There may be a seasonal fluctuation in the groundwater levels. There is also an aquifer confined below elevation 71.7 m to 68.6 m which causes an artesian flow to the surface when penetrated (BH 23-5, 23-11). The measured artesian heads ranged from elevation 94.5 m to 95.5 m or 2.4 m to 3.3 m above ground level. It should be noted that it was difficult to confine the flow from the aquifer and accurately measure the stabilized water level. The actual artesian head may be higher than recorded.

## DISCUSSION AND RECOMMENDATIONS

It is proposed to construct a 9 m wide two span underpass to carry Cambrian Road over the new Hwy. 416. The proposed construction would involve approach embankments of 7.4 m height above the existing grade.

A plan and profiles illustrating the proposed construction and shown on Dwg. 1288709-A included in the Appendix.

Recommendations from a geotechnical standpoint are given in the following sections regarding the design and construction of foundations and associated earthworks.

### Structure Foundations

The predominant soil strata at the site consists of a soft to firm clayey silt to silty clay and silty clay to clay. These strata are underlain by sand and gravel followed probably by bedrock. The weak consistency of the cohesive layers, extending to more than 18 m depth below existing grade, precludes then use of shallow spread footings located on or within the cohesive layers or a perched foundation on compacted granular fill placed above the cohesive layers. It is recommended that all foundations are supported on deep foundations.

### Steel H-Piles

From considerations of the high sensitivity of the cohesive deposits to disturbance and the presence of artesian groundwater conditions in the underlying sand and gravel stratum, it is recommended that a non-displacement type pile, such as steel H-piles would be the most suited foundation type for this site. For purposes of O.H.B.D.C., the following design axial bearing capacities are given for a 310x110 steel H-pile.

	<u>Pier</u>	<u>Abutment</u>
Factored Capacity at U.L.S. kN	1600	875*
Bearing Capacity at S.L.S. Type II kN	1100	620*

\*Lower capacities from consideration of downdrag forces acting on the piles.

Consideration should be given to advance fill placement to reduce post-construction settlement and associated downdrag forces. It is considered that if the fill could be placed, say, six to twelve months in advance of the pile installation, most of, if not all of the downdrag forces could be eliminated. However, if this option is favoured the embankment settlements should be monitored to ensure that the consolidation settlements have been fully realized before piling is commenced. The Foundation Design Section should be consulted regarding preloading, settlement monitoring and timing of pile installation. To prevent damage to the piles during installation, the steel H-piles should be provided with standard MTO tip reinforcement.

To prevent loss of fines along the wall of the piles resulting from the artesian groundwater pressures existing in the sand and gravel layer, it is recommended that an 'inverted' filter layer should be placed at the ground surface where the piles would be located. Typical details of such a scheme are shown on Figure 8.

It is recommended that all lateral loads are carried out by batter piles. However, from consideration of relatively large settlements that would result due to the construction of the approach embankments, the batter should be maintained at 6V:1H or steeper.

All pile caps should be provided with 1.8 m of equivalent earth cover for frost protection purposes.

### Caissons

Alternatively, a concrete filled caisson socketed into bedrock may also be used at the site. However, installation of the caisson should take into account, possible disturbance of the cohesive layers and the presence of excess hydro strata head in the underlying sand and gravel deposit. If this option is favoured, detailed comments would be provided by this section regarding the design and installation of caissons.

### Lateral Earth Pressure on Structures

Free draining materials such as Granular 'A' or Granular 'B' shall be used within a wedge behind the abutments and retaining walls bounded by a planerising at 60° to the horizontal as per O.H.B.D.C. With the provision of weepholes or drains in the abutment walls, hydrostatic pressures behind the walls could be eliminated. Design parameters for a horizontal backfill are given below.

Table 2

	Granular 'A'	Granular 'B'
Angle of Internal Friction, $\phi$ (unfactored)	35	30
Unit Weight, $\text{kN/m}^3$	22.8	21.2
Coefficient of Active Earth Pressure, $K_a$		
- S.L.S.	0.27	0.33
- U.L.S.	0.33	0.40
Coefficient of Earth Pressure at Rest, $K_o$		
- S.L.S.	0.43	0.50
- U.L.S.	0.50	0.58

### Slope Stability

The stability of the slopes was analyzed using Bishop's total shear method assuming a fill density of  $21.1 \text{ kN/m}^3$  (Granular 'B') and static loading conditions. The parameters used in the design are shown on Figure 9 and the results of the analysis on Figures 10 and 11. The results of the stability analysis indicate that the maximum height of fill without a berm is only 3.5 m.

Alternatively, consideration may be given to the use of lightweight fills. The maximum height of the embankment without a berm would be about 5 to 5.5 m using a lightweight fill with a compacted unit weight of  $13.5 \text{ kN/m}^3$ .

### Settlement

Construction of an embankment as proposed would results in some immediate settlement due to elastic compression of the fill itself as well as some long term settlement due to the consolidation of the foundation silty clay layers. The calculated consolidation settlements are tabulated below.

Table 3

#### Computed long Term Settlements

Fill Height (m)	Settlement in mm	
	Granular Fill	Lightweight Fill
3	310	35
5	740	360
7	1100	630

Since the calculated long term consolidation settlements are large, several options may be considered to maintain the post construction differential settlement between the pile supported bridge structure and approach fill to, say, less than 0.3 m.

1. Increase the length of structure such that the height of fill would be less than 4.5 m, assuming that the embankment is constructed using a lightweight fill with a unit weight of  $13.5 \text{ kN/m}^3$ .
2. Preload the area (using granular fill). Use sand grains or circle drains in the foundation clay, to facilitate a faster rate of consolidation. Once sufficient consolidation settlement has been realized, the preload fill may be removed and replaced by lightweight fill.

Either of these two options would result in a satisfactory performance of the structure and approach fills. The choice between the above two options

or the selection of any other option would be based on technical and economical considerations.

### Construction

Existing ground is not considered traficable for heavy construction equipment, such as pile driving rigs, concrete trucks. Access and haul roads would be required to facilitate construction at this site.

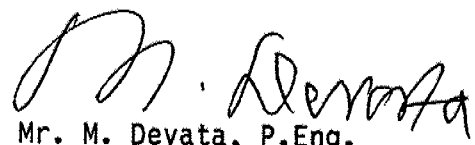
In the abutment area, the pile caps may be perched within the embankment fill. This would reduce the need to do excavation through the sensitive silty clay deposits at the abutment locations. However, at the pier locations, excavations for pile caps would be required below existing grade. Temporary excavations to depths of 2 m or less below grade may be carried out using 1V:1H. Slope, provided the excavated materials are not stockpiled near the crest of the slope. Under no circumstances should construction traffic be allowed on the excavated base, especially if the base is below or in close proximity to the groundwater level.

### MISCELLANEOUS

The field work for this project was supervised by Mr. S. Holmes, Foundation Engineer, and Mr. B. Sedgewick, Engineering Student. The equipment used was owned and operated by Marathon Drilling Co. Ltd. This report was prepared by Dr. B. Iyer, Senior Foundation Engineer, and reviewed by Mr. M. Devata, Chief Foundation Engineer.



Dr. B. Iyer, P.Eng.  
Senior Foundation Engineer



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

## APPENDIX

## 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

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

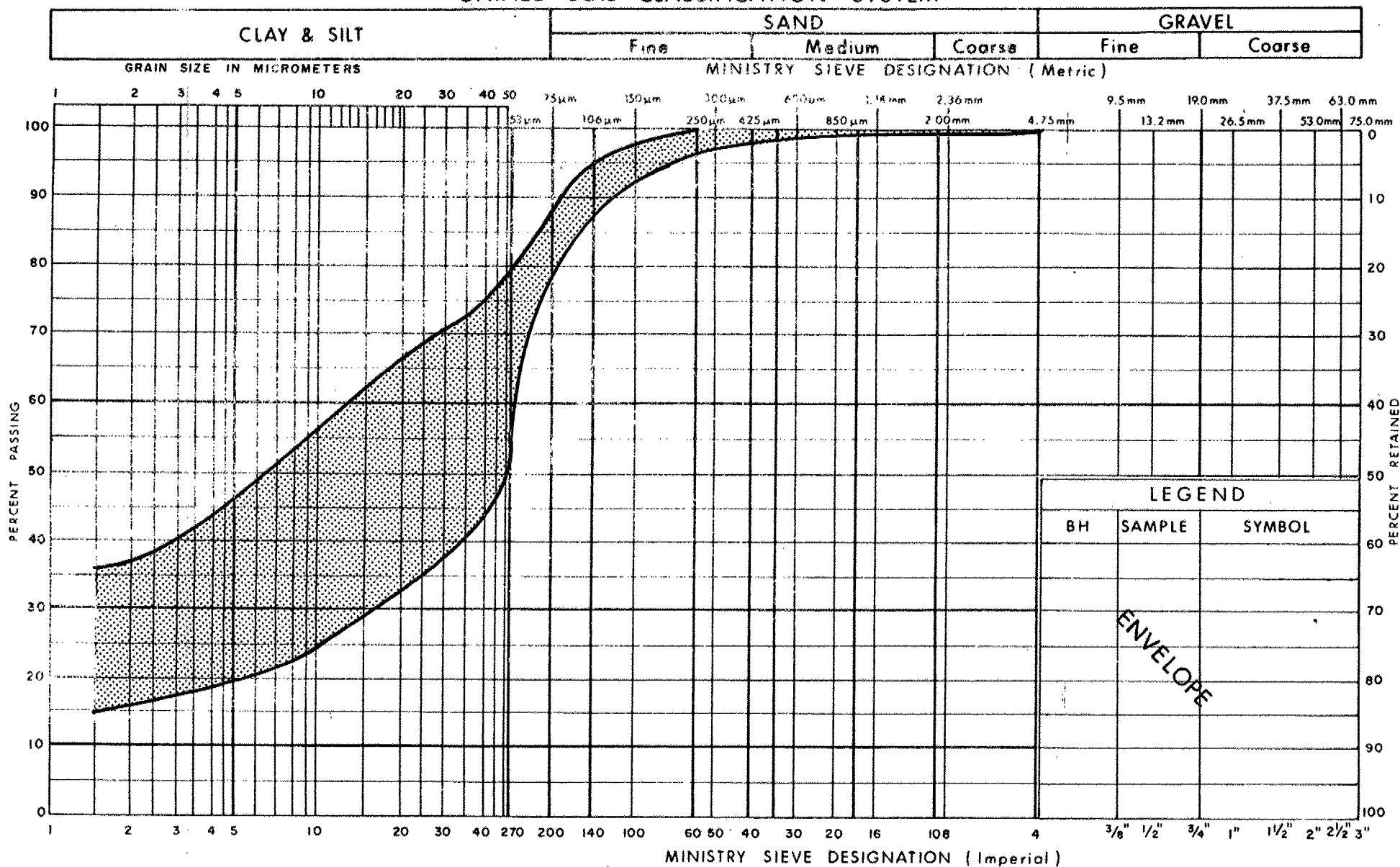
### STRESS AND STRAIN

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

### PHYSICAL PROPERTIES OF SOIL

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

## UNIFIED SOIL CLASSIFICATION SYSTEM

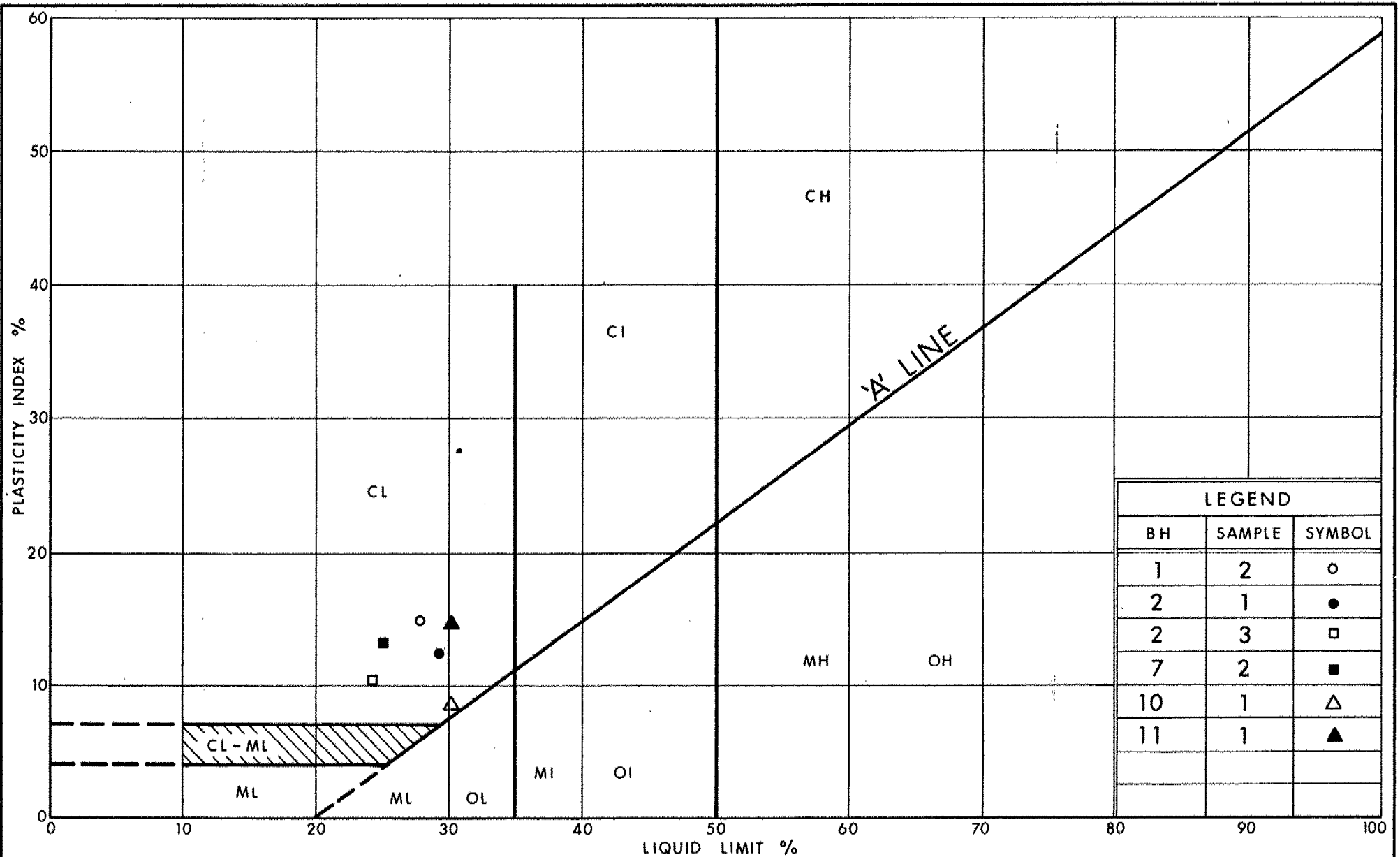


Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
**CLAYEY SILT**  
SOME SAND, TRACE ORGANICS

FIG No 1

W P 128-87-09



Ministry of  
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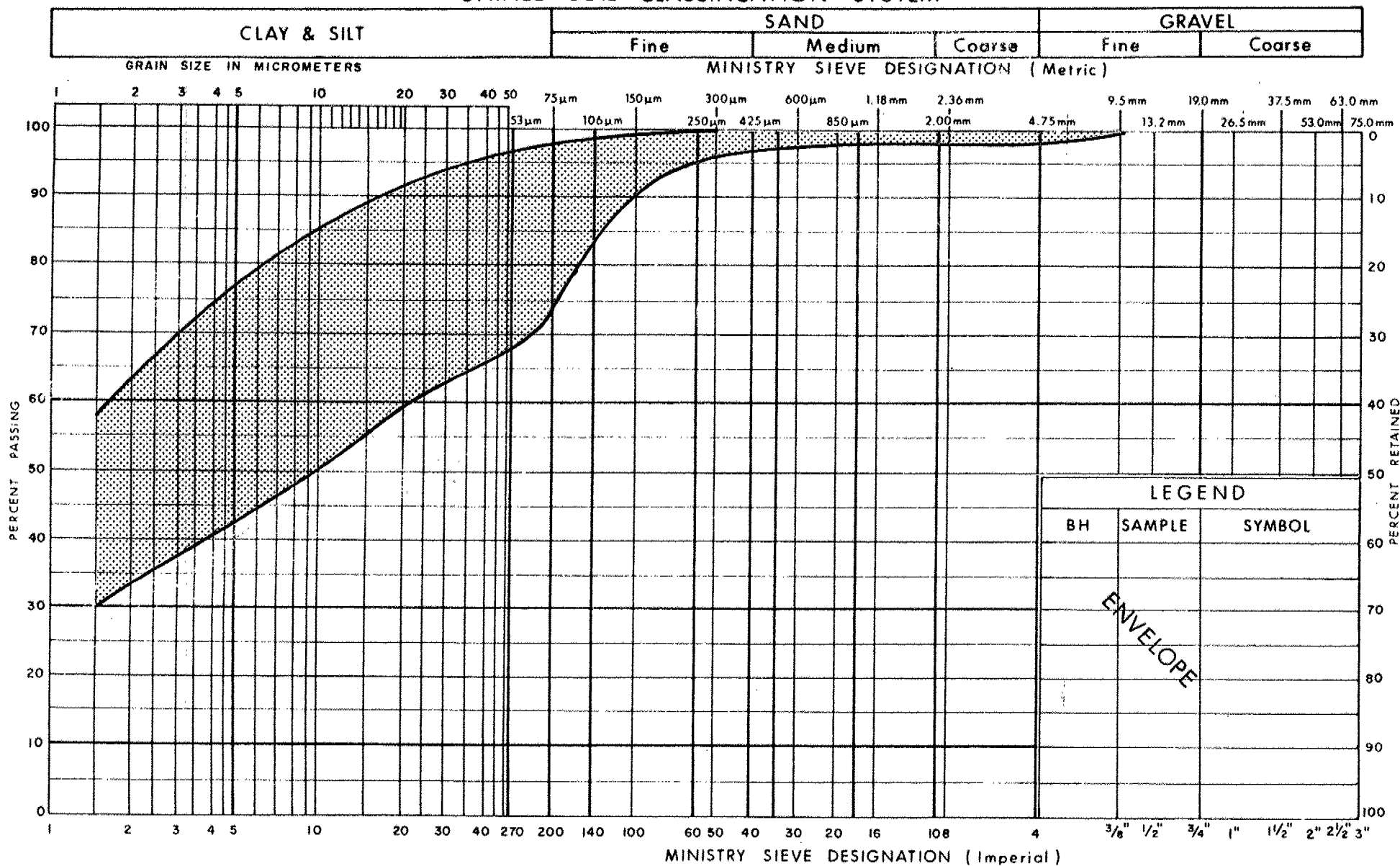
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PLASTICITY CHART  
CLAYEY SILT  
SOME SAND, TRACE ORGANICS

FIG No 2

W P 128-87-09

## UNIFIED SOIL CLASSIFICATION SYSTEM

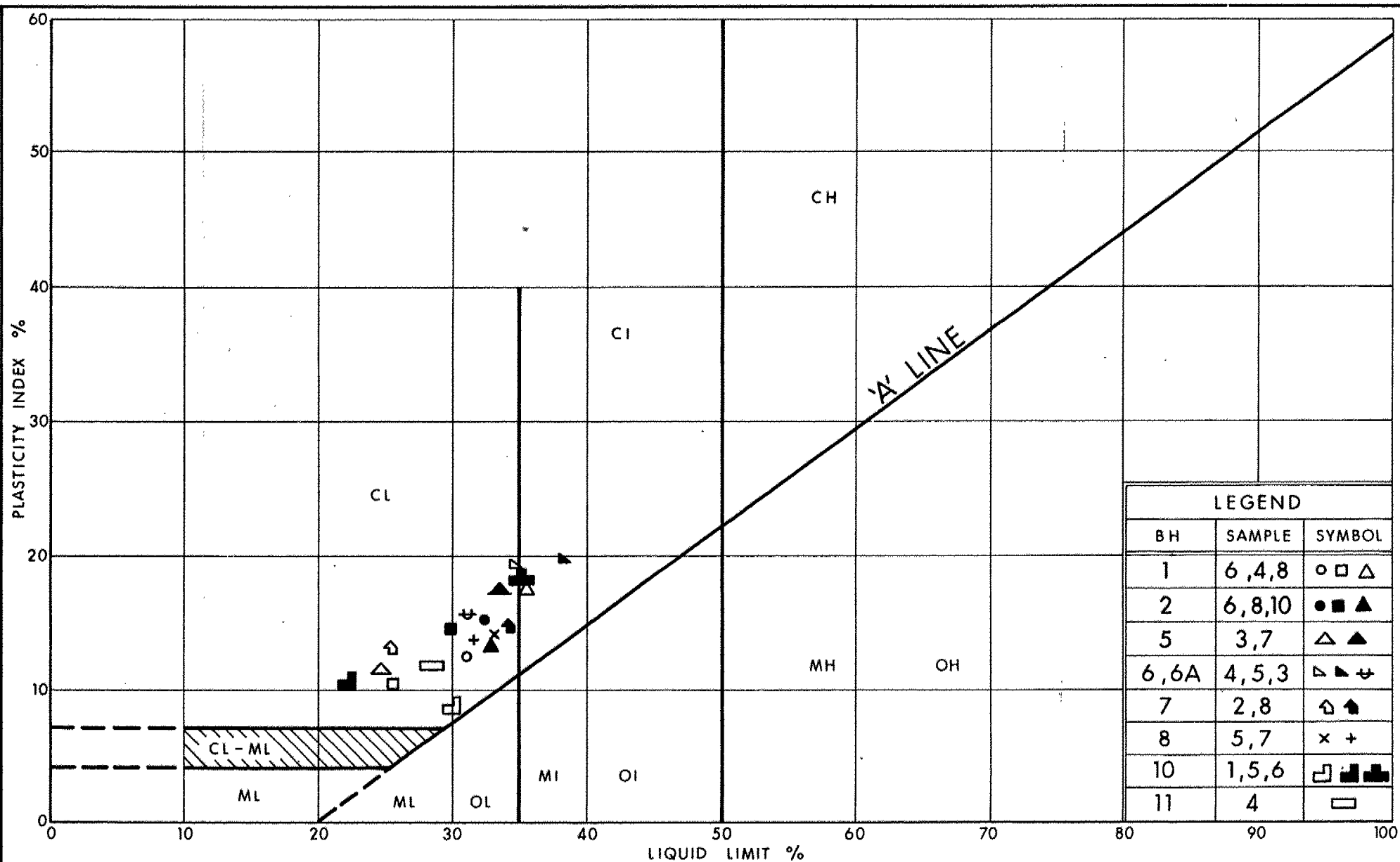


Ministry of  
Transportation

**GRAIN SIZE DISTRIBUTION**  
**CLAYEY SILT TO SILTY CLAY**  
 OCCASIONAL SILTY FINE SAND SEAMS, EXTRA SENSITIVE TO QUICK

FIG No 3

W P 128-87-09



Ministry of  
Transportation

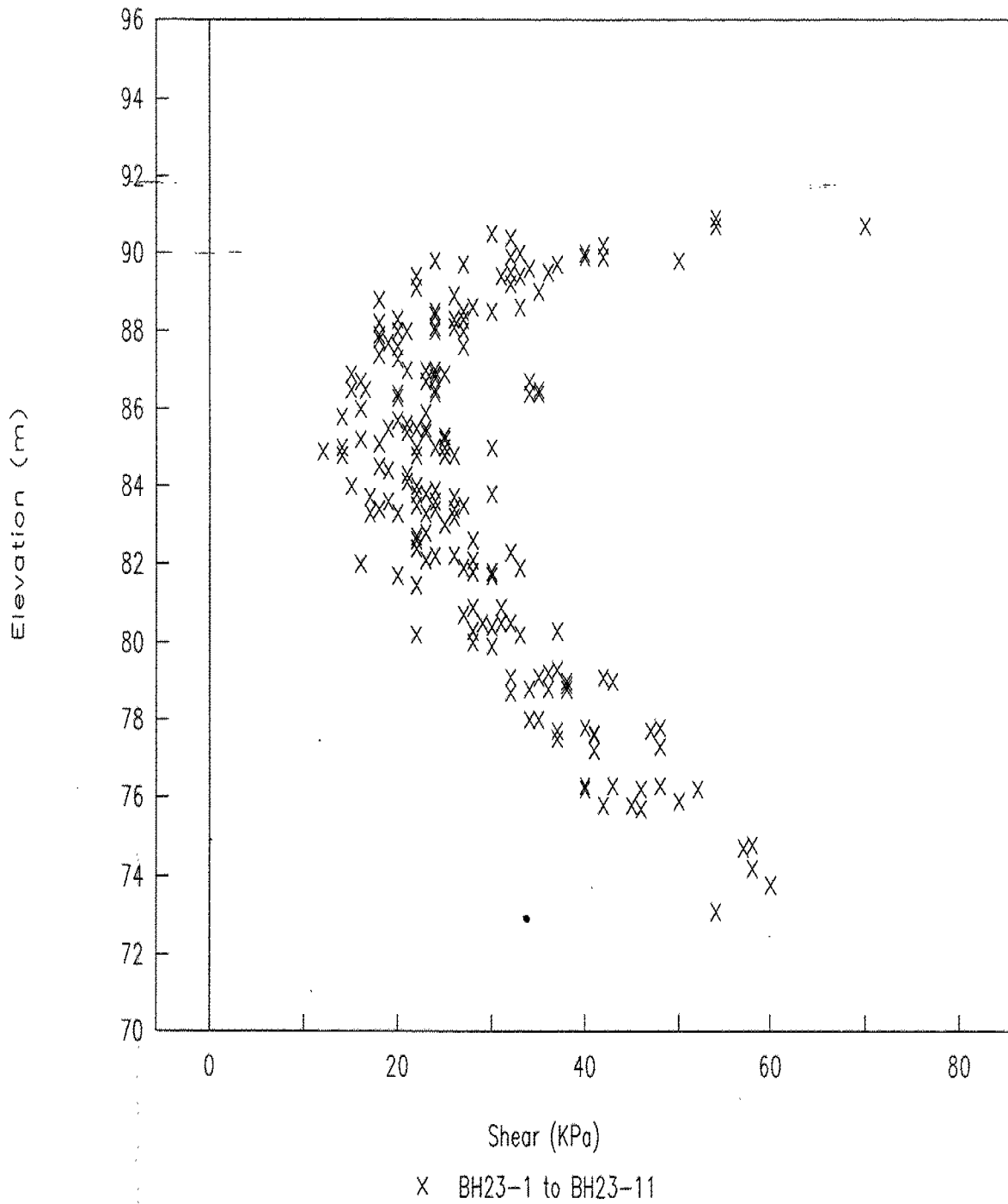
Ontario

**PLASTICITY CHART**  
**CLAYEY SILT TO SILTY CLAY**  
 OCCASIONAL SILTY FINE SAND SEAMS, EXTRA SENSITIVE TO QUICK

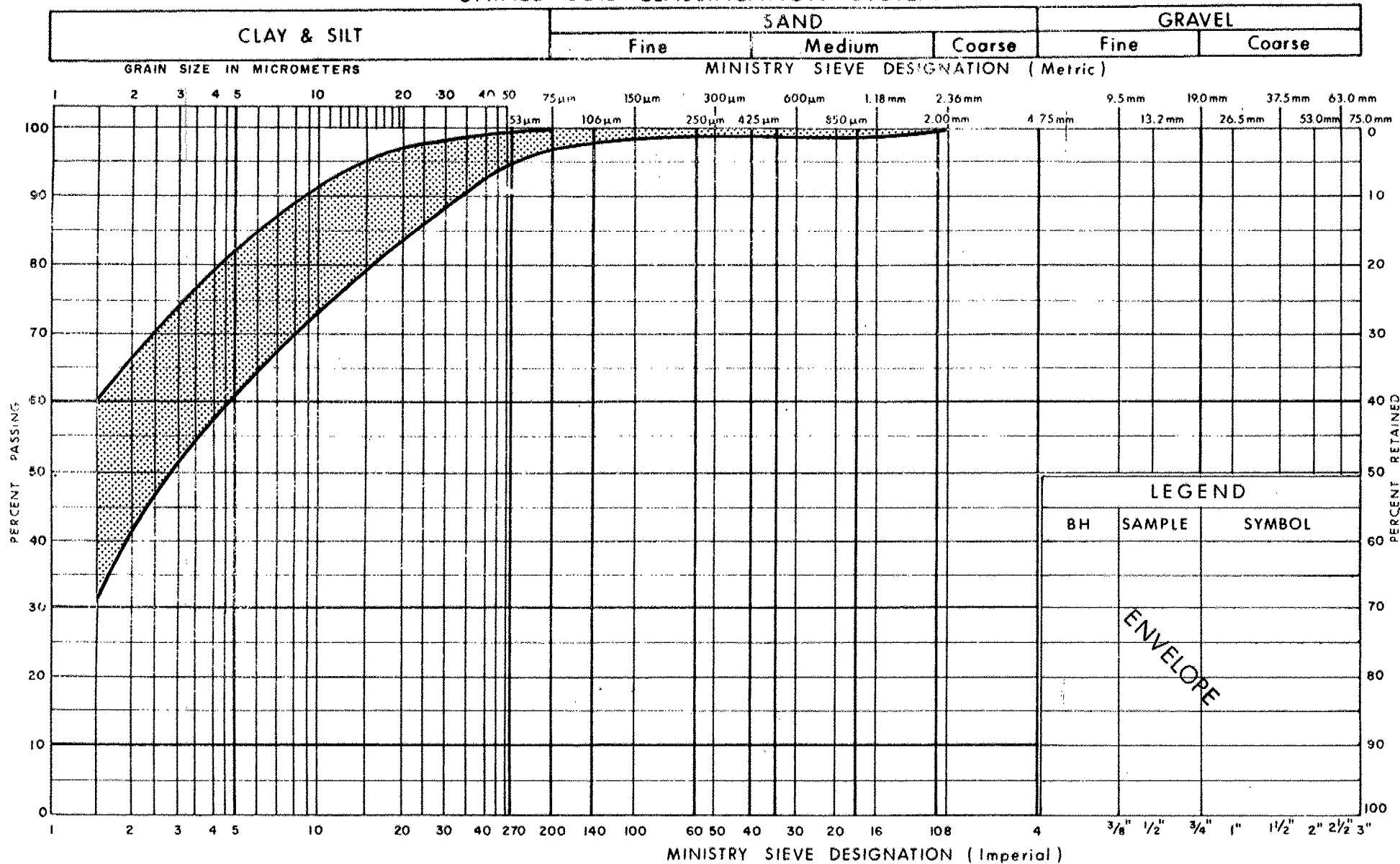
FIG No 4

W P 128-87-09

03-06-91



## UNIFIED SOIL CLASSIFICATION SYSTEM

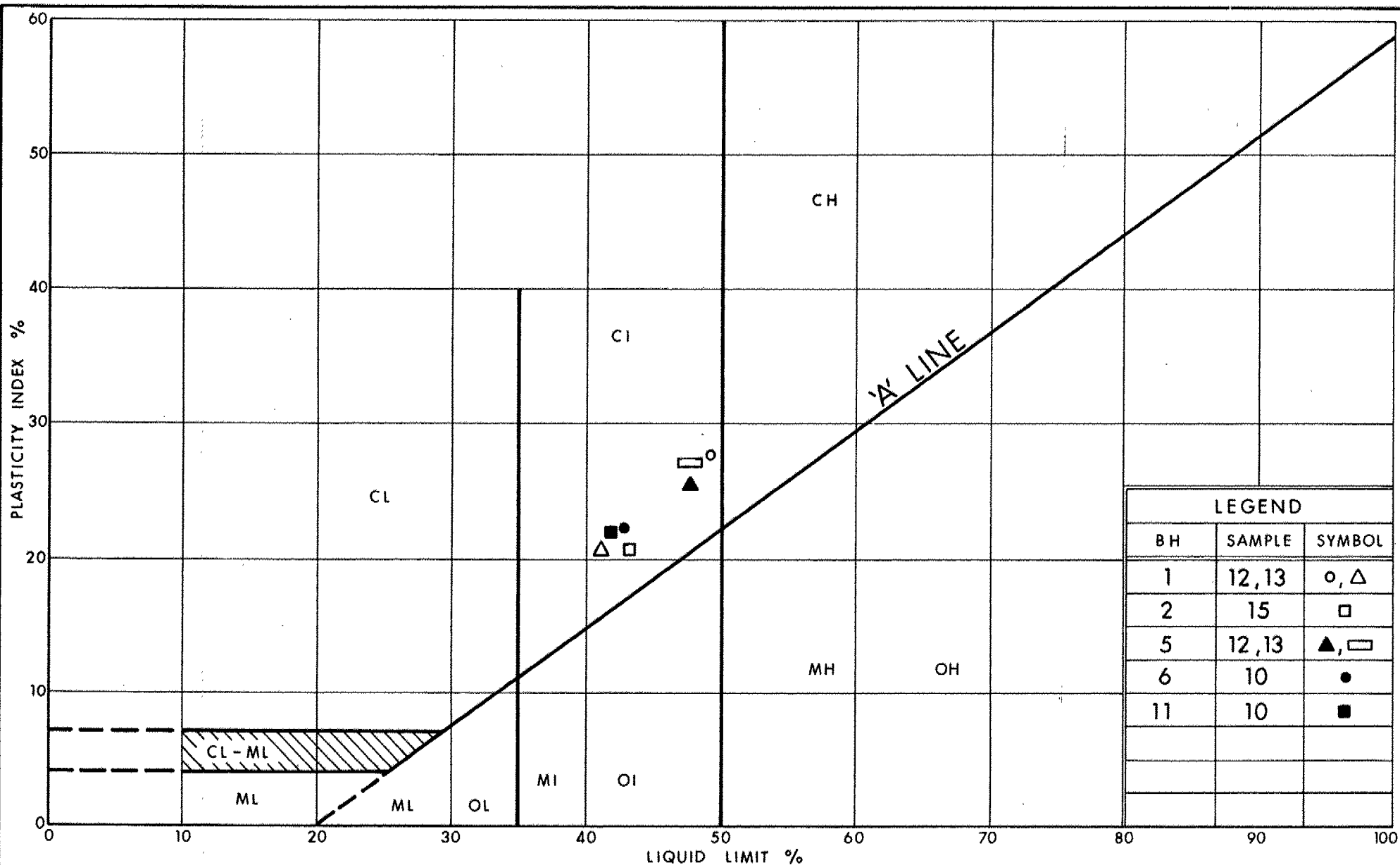


Ministry of  
Transportation

GRAIN SIZE DISTRIBUTION  
SILTY CLAY TO CLAY  
TRACE SAND

FIG No 6

W P 128-87-09



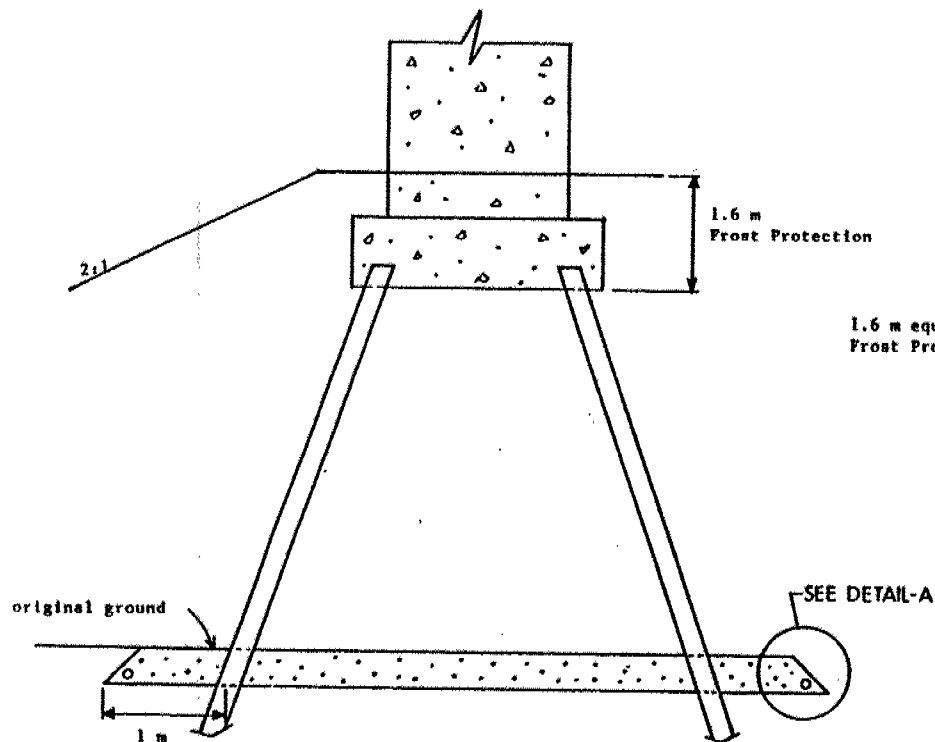
Ministry of  
Transportation

Ontario

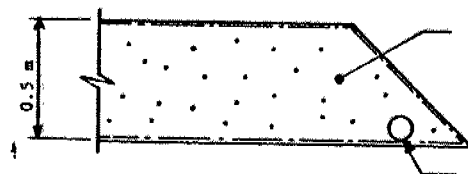
# PLASTICITY CHART SILTY CLAY TO CLAY TRACE SAND

FIG No 7.

W P 128-87-09



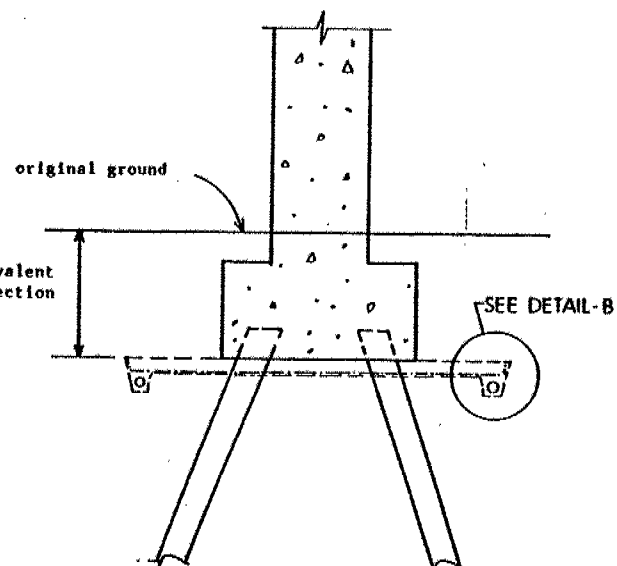
ABUTMENT SECTION (TYP)



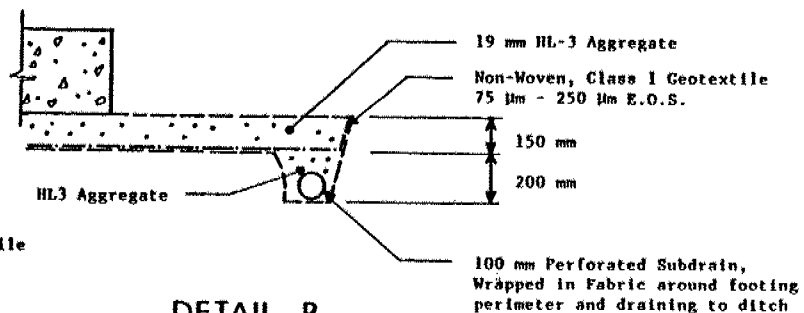
DETAIL A

19 mm HL-3 Aggregate  
Completely wrapped in  
non-woven, Class I Geotextile  
75  $\mu$ m - 250  $\mu$ m E.O.S.

100 mm Perforated  
Subdrain, wrapped in  
Fabric around perimeter  
of blanket and draining  
to ditch



PIER SECTION (TYP)



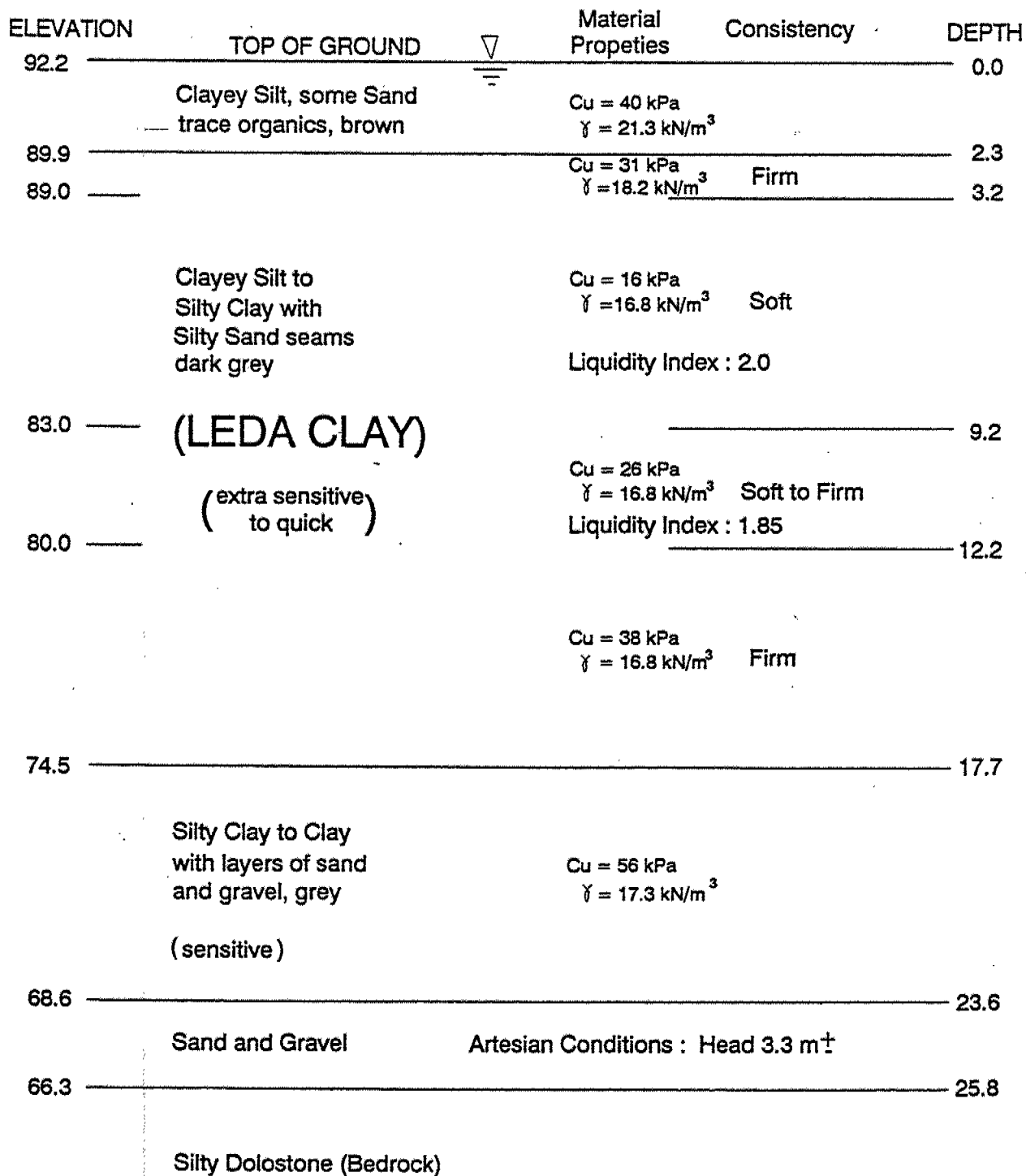
DETAIL B

NOTES:

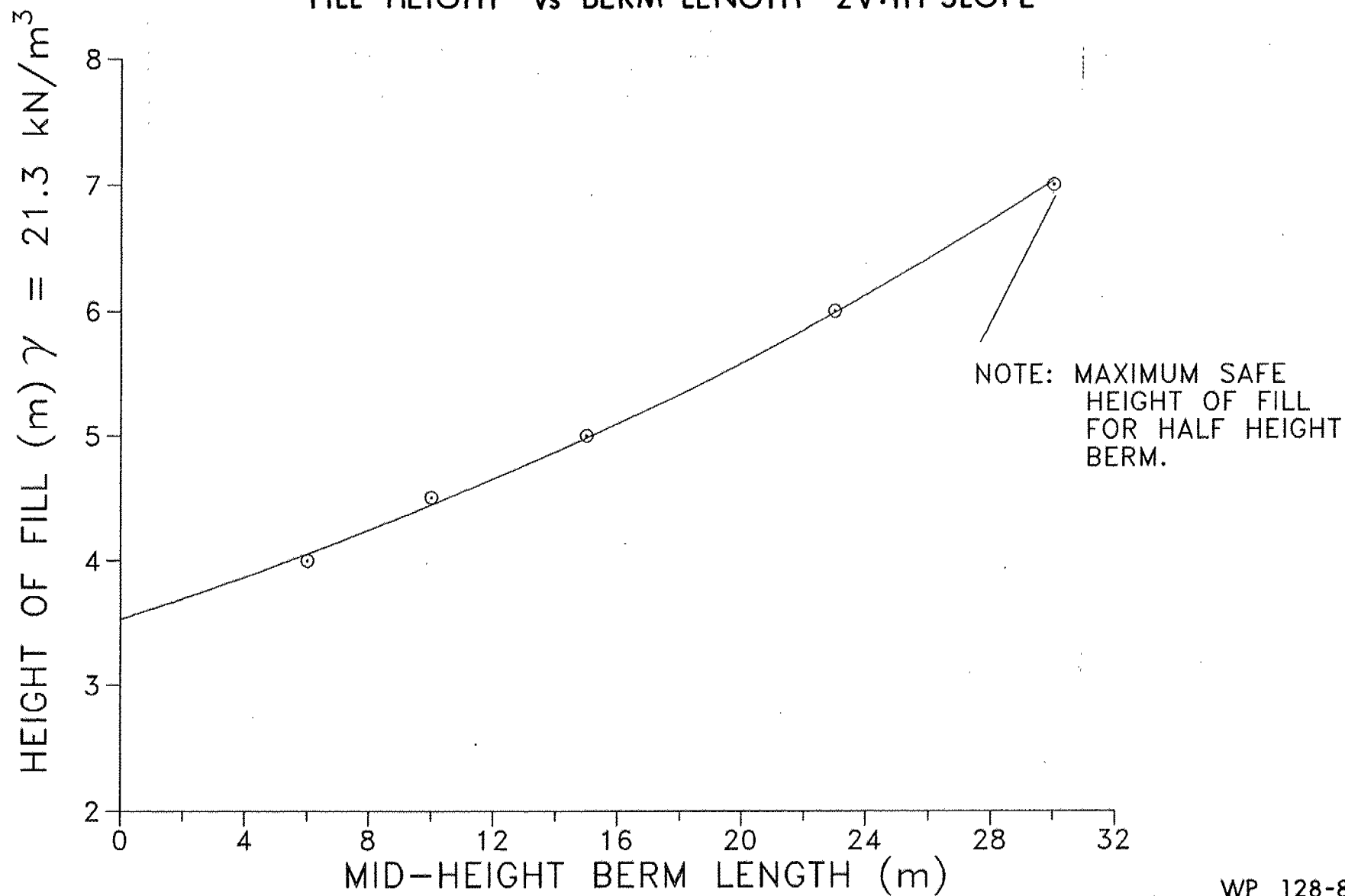
1. The drainage blankets should be in place prior to pile driving
2. The geotextile should be cut with a 300 mm x 300 mm "x" at locations where piles will penetrate. This is applicable only to the pier locations
3. If blanket at pier locations is disturbed during pile driving, the blanket should be restored to the details shown on this drawing after the completion of the pile driving

FIGURE 8 - DRAINAGE BLANKET DETAILS  
FOR ABUTMENTS & PIERS

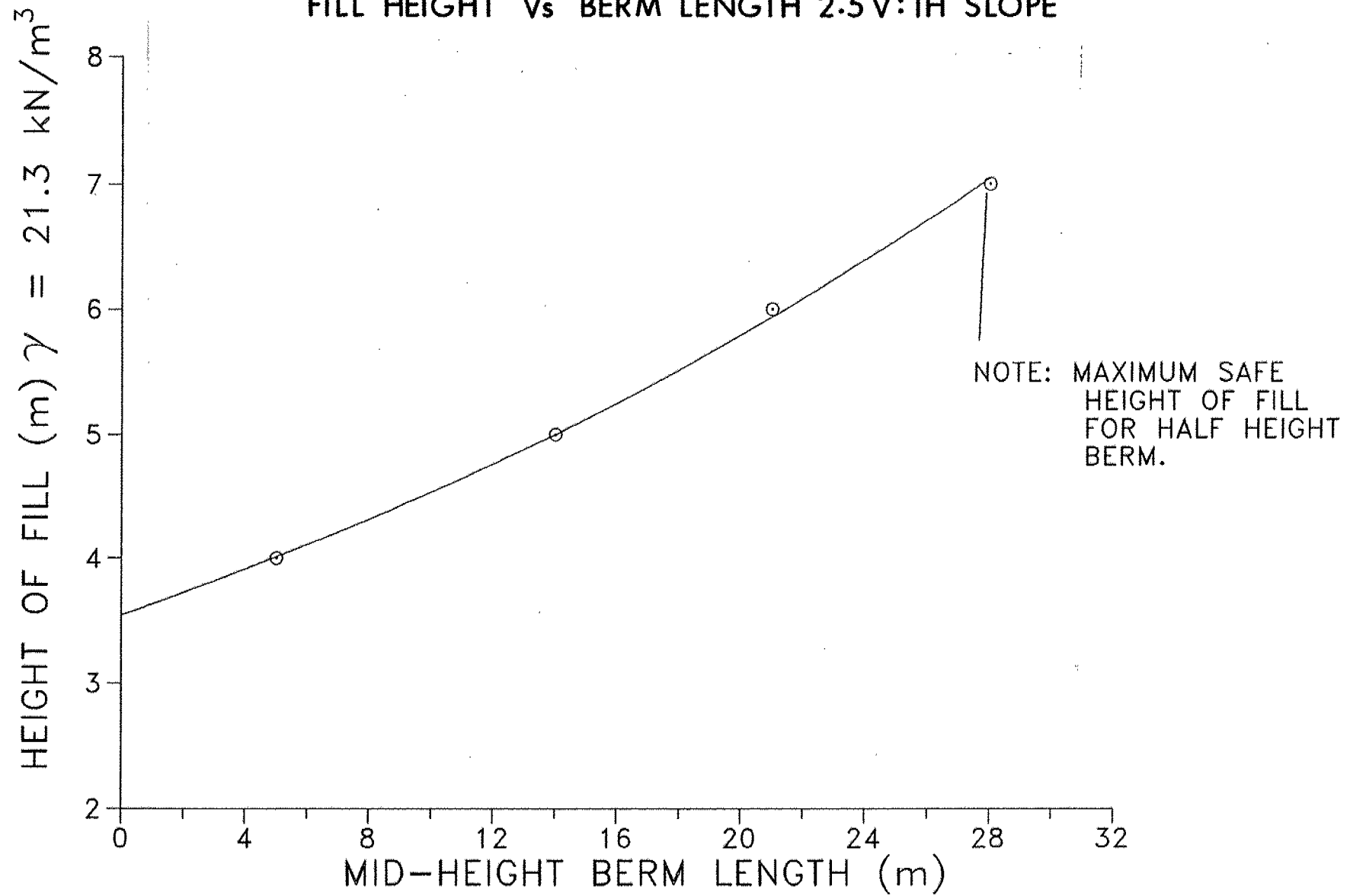
# STRATIGRAPHICAL PLOT - CAMBRIAN Rd.



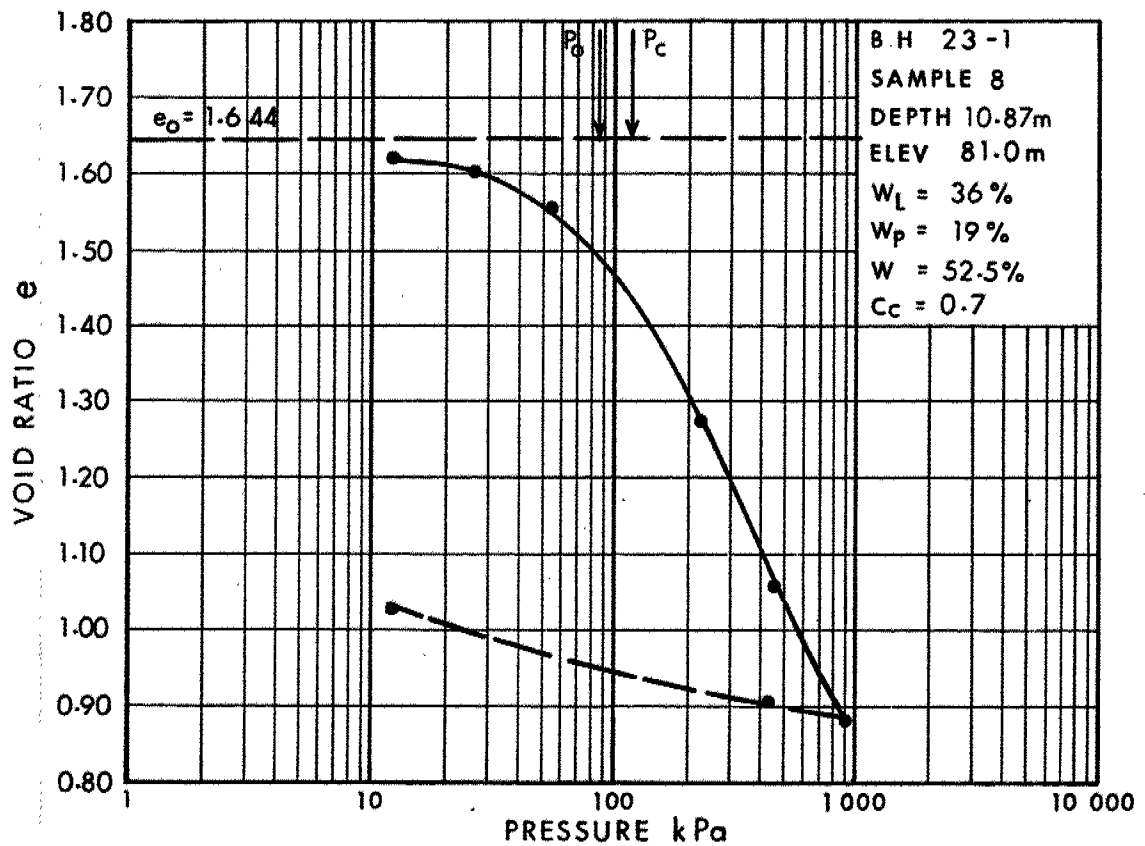
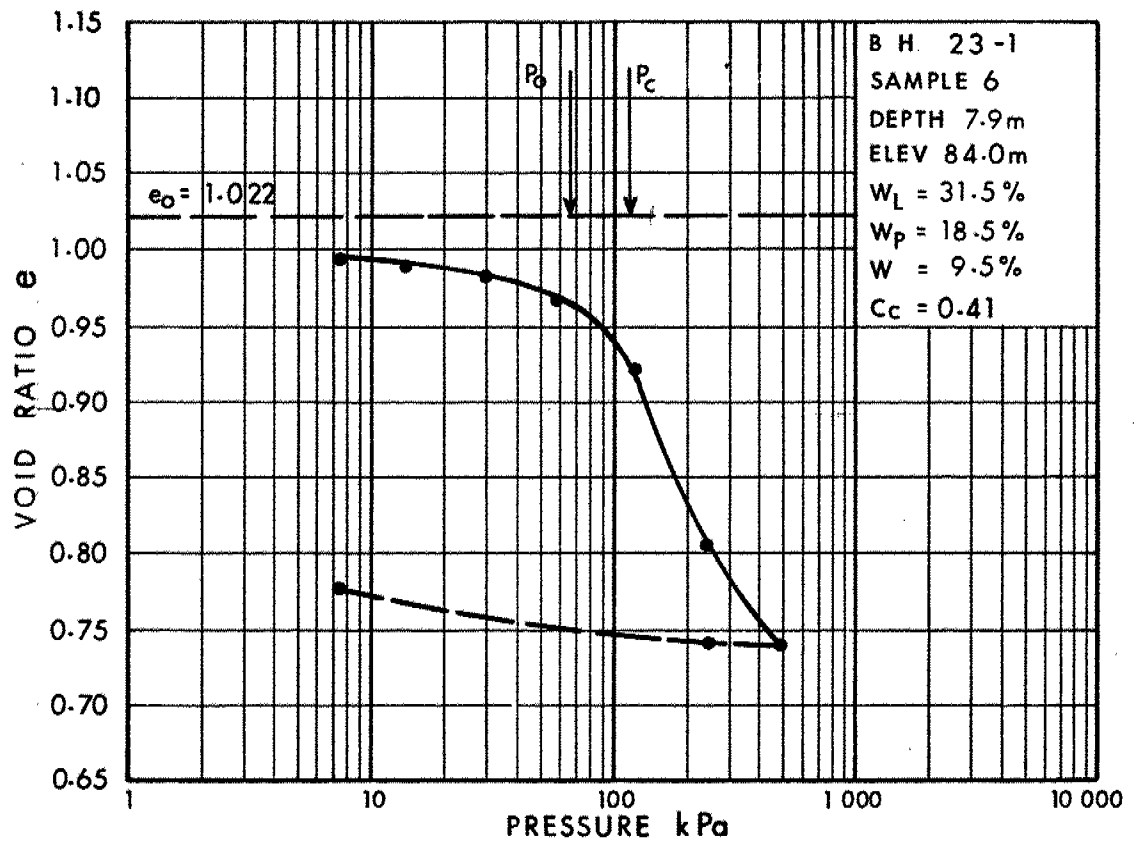
# FILL HEIGHT Vs BERM LENGTH 2V:1H SLOPE



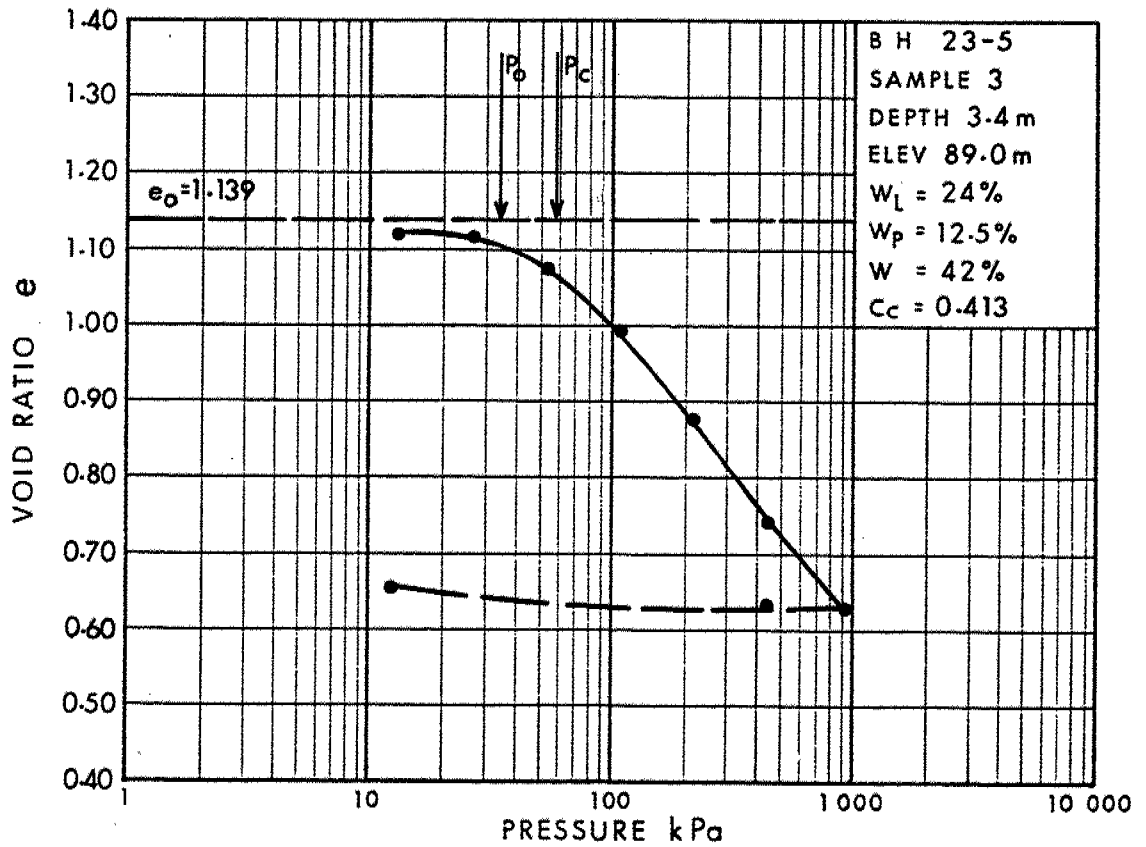
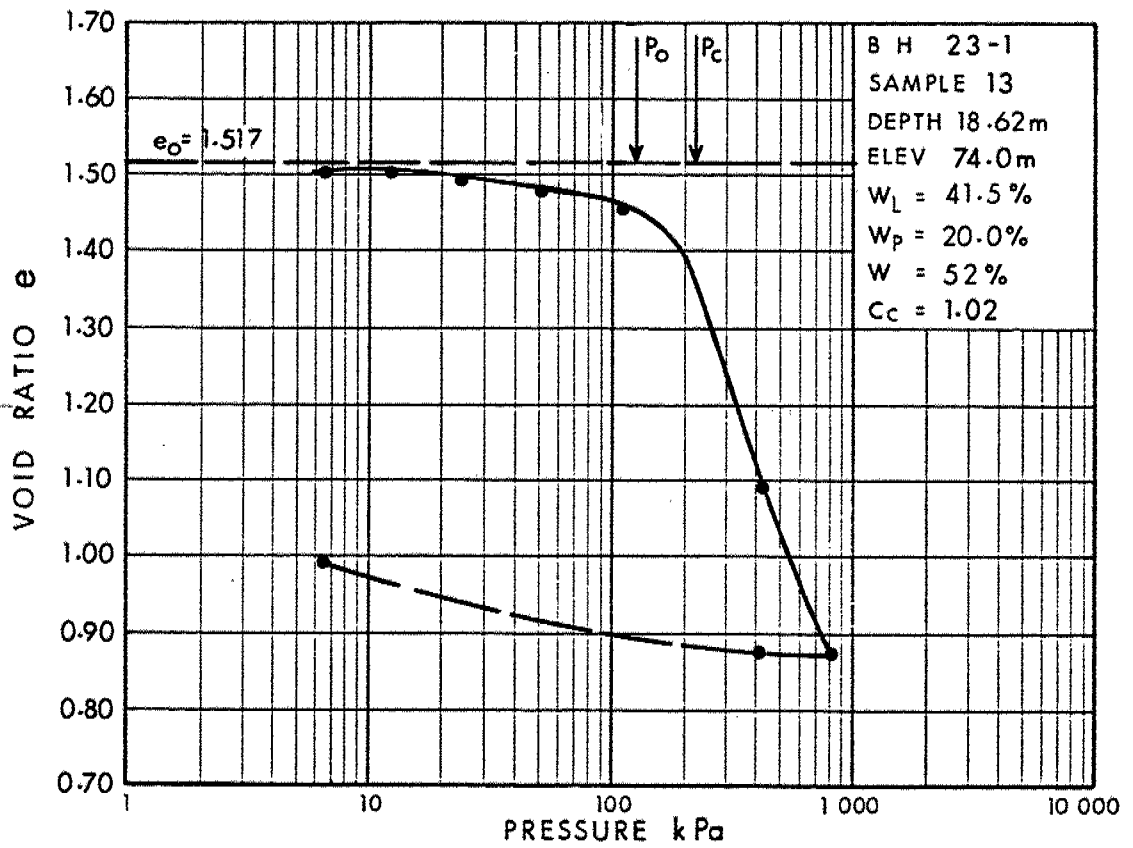
# FILL HEIGHT Vs BERM LENGTH 2.5 V:1H SLOPE



# VOID RATIO - PRESSURE CURVES



# VOID RATIO - PRESSURE CURVES



# VOID RATIO - PRESSURE CURVES

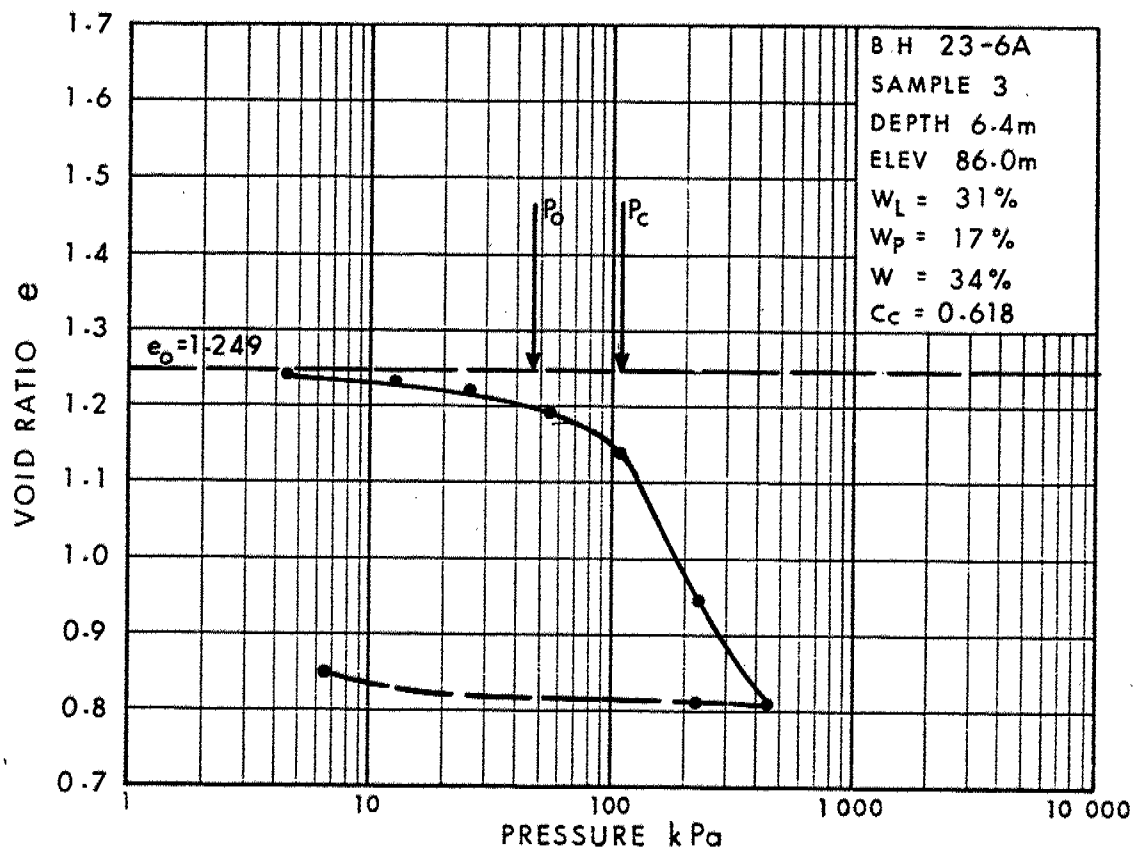
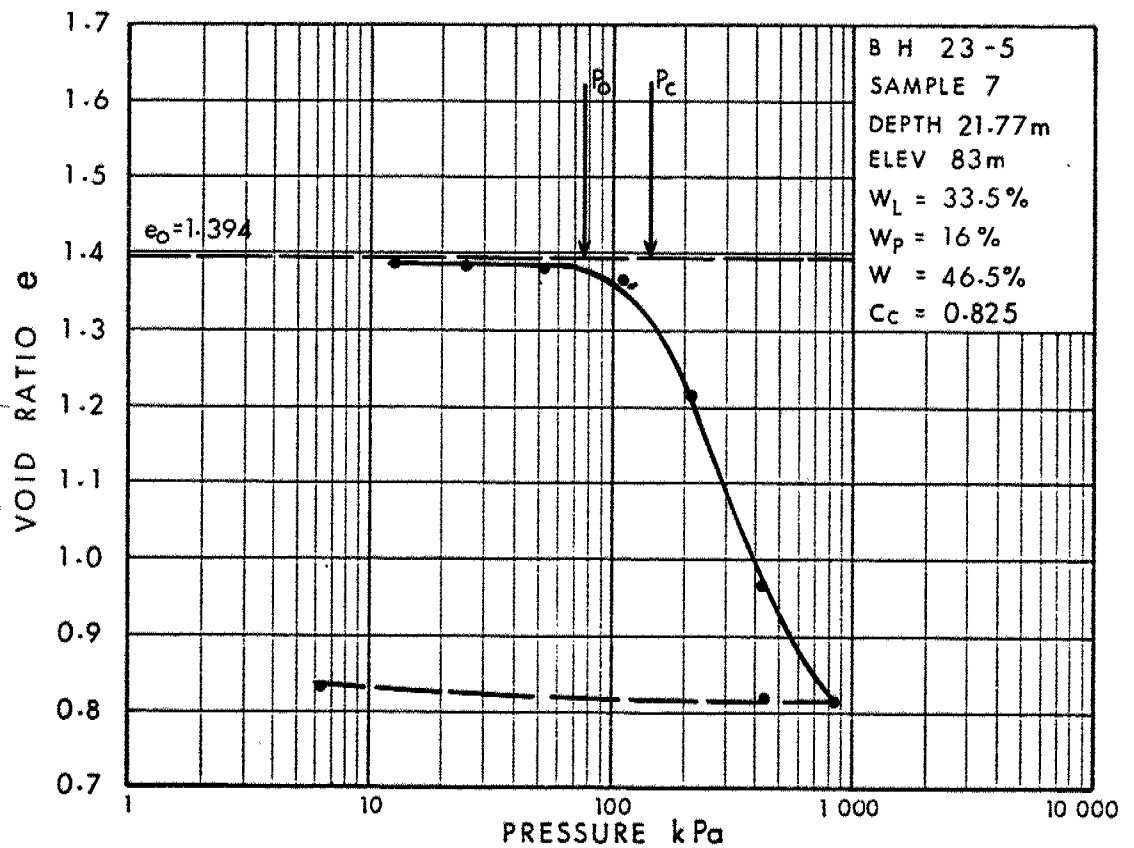


Fig 14

# VOID RATIO - PRESSURE CURVES

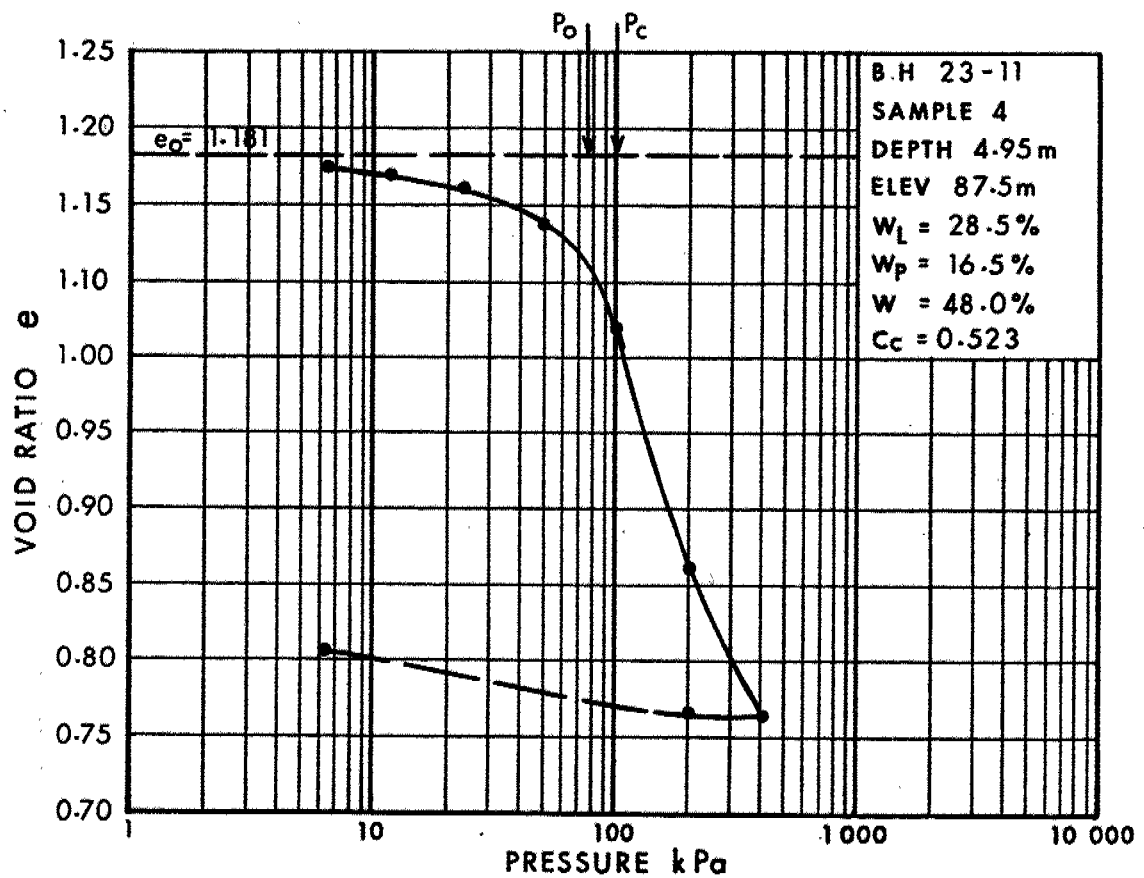
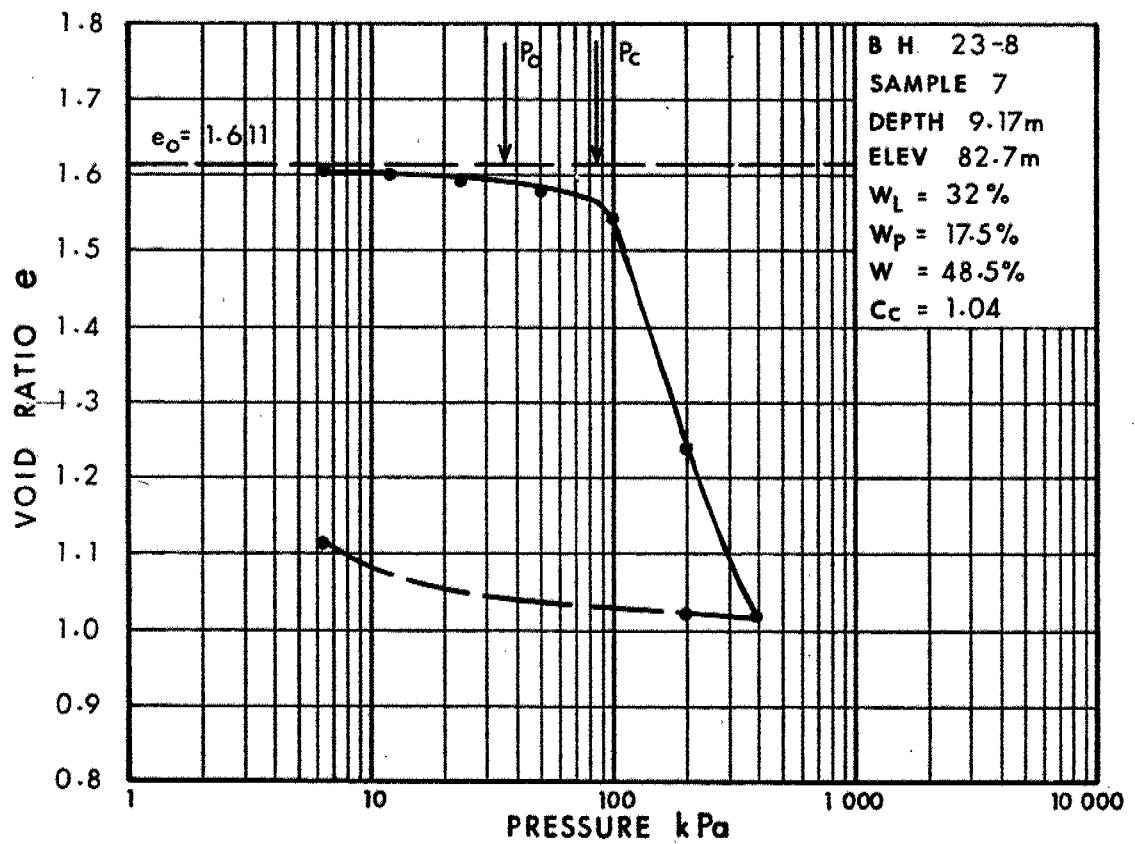


Fig 15

W P 128-87-09

# RECORD OF BOREHOLE No 23-1 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Coords: N 5 011 496.5, E 362 755.6 ORIGINATED BY S.M.H.  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger, Cone Test COMPILED BY M.M.  
 DATUM Geodetic DATE 89-05-25 CHECKED BY B.I.



SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL		
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			'N' VALUES	SHEAR STRENGTH kPa						WATER CONTENT (%)	
								○ UNCONFINED ● QUICK TRIAXIAL							+ FIELD VANE x LAB VANE
92.3	Ground Surface						20 40 60 80 100	10 20 30 40 50	20 40 60						
0.0	Clayey Silt, some Sand, trace Organics Firm  Clayey Silt to Silty Clay with pockets of Organics, Occasional Silty fine Sand seams Extra Sensitive to Quick Soft to Firm		1	SS	4								16.7	0 15 56 29	
89.9			2	SS	1										
2.4			3	TW	PM										
			4	TW	OW										
			5	TW	OW										
			6	TW	OW										
			7	TW	OW										
			8	TW	OW										
			9	SS	OW										
			10	SS	OW										
			11	TW	OW										
75.0			12	SS	OW										
17.3	Silty Clay to Clay, trace Sand Stiff		13	TW	OW										
73.1															
19.2	End of Borehole														
69.7															
22.6	Probable Sand and Gravel														
68.8															
23.5	End of Cone Test						40/18cm								

# RECORD OF BOREHOLE No 23-1A 1 of 1 METRIC

W.P. 128-87-09 LOCATION Coords: N 5 011 499.4, E 362 760.9 ORIGINATED BY BS

DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger COMPILED BY AL/MM

DATUM Geodetic DATE 89-05-30 CHECKED BY BI

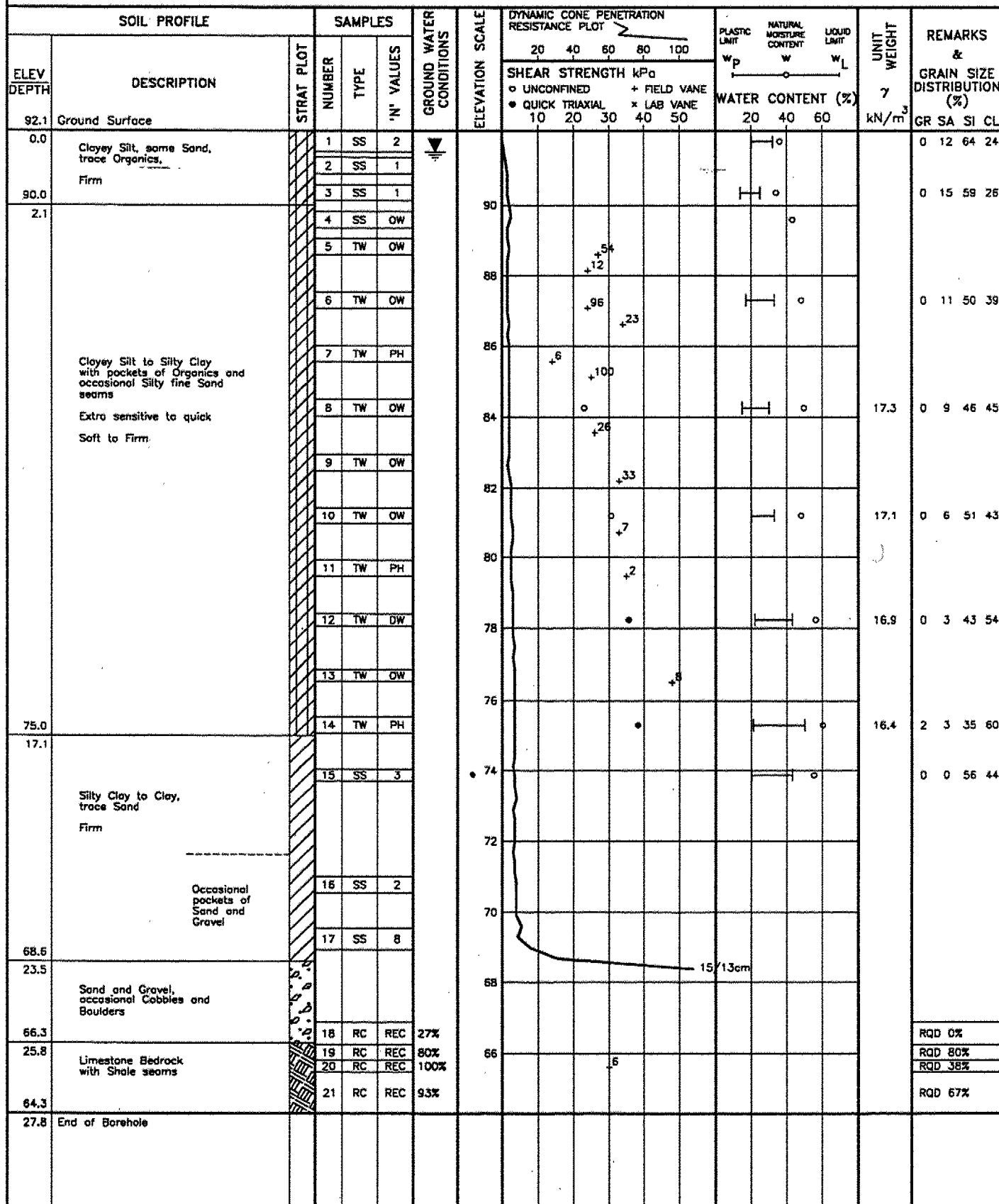
SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT			PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT  γ  kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL			
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa								WATER CONTENT (%)		
								20 40 60 80 100		10 20 30 40 50							20 40 60	
92.3	Ground Surface																	
0.0	Probable Clayey Silt, some Sand, trace Organics  Firm						92											
89.9								90										
2.4	Probable Clayey Silt to Silty Clay with pockets of Organics and occasional Silty fine Sand seams  Extra Sensitive to Quick  Soft to Firm							88										
82.4								86										
							84											
9.9	End of Borehole																	

# RECORD OF BOREHOLE No 23-2

1 OF 1

METRIC

W.P. 128-87-09 LOCATION Coords: N 5 011 499.7, E 362 772.0 ORIGINATED BY S.M.H.  
DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger, BW Casing, BXL Rack Core, Cone Test COMPILED BY M.M.  
DATUM Geodetic DATE 89-05-23 CHECKED BY B.I.



# RECORD OF BOREHOLE No 23-2B 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Coords: N 5 011 500.7, E 362 771.0 ORIGINATED BY SH  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger COMPILED BY AL/MM  
 DATUM Geodetic DATE 89-05-30 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100					
92.1	Ground Surface																
0.0	Probable Clayey Silt, some Sand, trace Organics Firm																
90.0																	
2.1																	
	Probable Clayey Silt to Silty Clay with pockets of Organics and occasional Silty fine Sand seams  Extra Sensitive to Quick  Soft to Firm																
82.2																	
9.9	End of Borehole																

# RECORD OF BOREHOLE No 23-3 1 OF 1 METRIC

W.P. 128-97-09 LOCATION Coords: N 5 011 519.2, E 362 743.6 ORIGINATED BY SH  
 DIST 9 HWY 416 BOREHOLE TYPE Cone Test COMPILED BY JW/MM  
 DATUM Geodetic DATE 89-05-24 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40					
91.7	Ground Surface													
0.0	Probable Clayey Silt, some Sand, trace Organics													
90.0														
1.7														
	Probable Clayey Silt to Silty Clay with pockets of Organics and occasional Silty fine Sand seams													
75.0														
16.7	Probable Silty Clay to Clay trace Sand													
68.0														
23.7	Probable Sand and Gravel, occasional Cobbles and Boulders													
67.0														
24.7	End of Cone Test													

# RECORD OF BOREHOLE No 23-4 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Coords: N 5 011 515.8; E 362 801.0 ORIGINATED BY SH  
 DIST 9 HWY 416 BOREHOLE TYPE Cone Test COMPILED BY JW/DT  
 DATUM Geodetic DATE 89-05-24 CHECKED BY BI

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100						
92.3	Ground Surface													
0.0	Probable Clayey Silt, some Sand, trace Organics													
90.1														
2.2	Probable Clayey Silt to Silty Clay with packets of Organics and occasional Silty fine Sand seams													
75.0														
17.3	Probable Silty Clay to Clay													
69.0														
23.3	Probable Sand and Gravel occasional Cobbles and Boulders													
67.6														
24.7	End of Cone Test													

# RECORD OF BOREHOLE No 23-5 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Co-ords: N 5 011 542.2; E 362 834.4 ORIGINATED BY B.S.  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger and Cone Test COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-24 to 25 CHECKED BY B.J.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>P</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	20 40 60						
92.2	Ground Surface														
0.0	Clayey Silt, some Sand, trace Organics Firm		1	SS	2										
90.1			2	TW	PM										
2.1	Clayey Silt to Silty Clay with pockets of Organics and occasional silty fine Sand seams  Extra Sensitive to Quick Soft to Firm		3	TW	PM								18.2	0 13 52 35	
			4	TW	OW										
			5	TW	OW										
			6	TW	OW										
			7	TW	OW								17.4	0 8 50 42	
			8	TW	OW										
			9	TW	OW										
			10	TW	OW								16.6	0 4 33 63	
			11	TW	OW										
75.4				12	TW	OW							16.3	0 1 41 58	
16.8	Silty Clay to Clay, trace Sand Firm to Stiff		13	TW	PM							16.4	0 1 39 60		
73.0															
19.2	End of Borehole														
71.0	Probable Silty Clay to Clay, trace Sand														
21.2	Probable Sand, Gravel occasional Cobbles and Boulders														
69.9															
22.3	End of Cone Test														
	** Estimated elevation of artesian flow encountered from cone hole  • To 95.2 ±														

# RECORD OF BOREHOLE No 23-6 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Coords: N 5 011 530.5; E 362 828.0 ORIGINATED BY B.S.  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger, BW Casing and Cone Test COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-18 to 23 CHECKED BY B.L.

SOIL PROFILE			SAMPLES			GROUND WATER * CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT				UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>		
92.2	Ground Surface												
0.0	Clayey Silt, some Sand, trace Organics Firm		1	SS	2	To 94.1	92						
90.1			2	TW	PH								
2.1			3	SS	OW		90						
			4A	SS	OW		88						
	Clayey Silt to Silty Clay with pockets of Organics and occasional Silty fine Sand seams extra sensitive to quick soft to firm		5A	SS	OW		86						
			4	TW	OW		84					17.4	0 5 46 48
			5	TW	OW		82					16.7	0 3 44 53
			6	TW	OW		80						
			7	TW	OW		78						
			8	TW	OW		76					16.2	
			9	TW	OW		74					17.0	0 0 50 50
75.4			10	TW	OW		72						
16.8	Silty Clay to Clay Firm		11	SS	2								
	Occasional pockets of Sand and Gravel		12	SS	2								
71.7													
20.5	Probable Sand and Gravel												
70.9	occasional Cobbles and Boulders												
21.3	End of Borehole												
	* Phreatic Water Surface not observed												

# RECORD OF BOREHOLE No 23-6A 1 OF 1 METRIC

W.P. 128-87-08 LOCATION Co-ords: N 5 011 527.2; E 362 829.6 ORIGINATED BY B.S.  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-23 CHECKED BY B.I.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					UNIT WEIGHT $\gamma$ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES		20	40	60	80	100		
92.2	Ground Surface												
0.0	Clayey Silt, some Sand, trace Organics					92							
90.0	Firm					90							
2.2	Clayey Silt to Silty Clay with pockets of Organics and occasional Silty fine Sand seams		1	TW	OW	88							
	Extra Sensitive to Quick Soft		2	TW	OW	86							
84.9			3	TW	OW	86						17.6	0 8 50 42
7.3	End of Borehole												
	• Water Level Not established												

# RECORD OF BOREHOLE No 23-7 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Co-ords: N 5 011 453.2, E 362 688.9 ORIGINATED BY B.S.  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger and Cone test COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-27 to 29 CHECKED BY B.J.

SOIL PROFILE		SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT W <sub>p</sub>	NATURAL MOISTURE CONTENT W	LIQUID LIMIT W <sub>L</sub>	UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE			20 40 60 80 100						
92.1													
0.0	Clayey Silt, some Sand, trace Organics Firm		1	SS	2							19.0	0 19 52 29
89.7			2	TW	PM								
2.4	Clayey Silt to Silty Clay with pockets of Organics and occasional Silty Fine Sand seams Extra Sensitive to Quick Soft to Firm		3	SS	PM								
			4	TW	OW								
			5	SS	OW								
			6	TW	PM								
			7	SS	OW								
			8	TW	OW							16.9	0 6 50 44
78.7			9	SS	OW								
13.4	End of Borehole												

# RECORD OF BOREHOLE No 23-8 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Co-ords: N 5 011 472.4; E 362 722.4 ORIGINATED BY S.M.H.  
 DIST 9 HWY 415 BOREHOLE TYPE H.S Auger and Cone Test COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-26 CHECKED BY B.I.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL	
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			SHEAR STRENGTH kPa							WATER CONTENT (%)
								○ UNCONFINED ● QUICK TRIAXIAL	+ FIELD VANE x LAB VANE						
92.1	Ground Surface							20 40 60 80 100	10 20 30 40 50						
0.0	Clayey Silt, some Sand, trace Organics Firm		1	SS	1										
89.8			2	TW	PH										
2.3	Clayey Silt to Silty Clay with pockets of organics and occasional Silty fine Sand seams  Extra Sensitive to Quick Soft to Firm		3	SS	OW										
			4	TW	PM										
			5	SS	OW										
			6	TW	OW										
			7	TW	OW										
			8	SS	OW										
			9	TW	OW										
			10	SS	2										
			11	TW	OW										
75.8															
16.3	End of Borehole														
	Probable Silty Clay to Clay, trace Sand														
70.8															
21.3	End of Cone Test														

## 1 OF 1

ORIGINATED BY S.H.

COMPILED BY P.T.

CHECKED BY B.I.

+3, x5: Numbers refer to Sensitivity

# RECORD OF BOREHOLE No 23-9 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Co-ords: N 5 011 557.8; E 362 869.9 ORIGINATED BY S.H.  
 DIST 9 HWY 416 BOREHOLE TYPE Cone Test COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-23 CHECKED BY B.I.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT w <sub>p</sub>	NATURAL MOISTURE CONTENT w	LIQUID LIMIT w <sub>L</sub>	UNIT WEIGHT γ	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	WATER CONTENT (%)					
92.1	Ground Surface													
0.0	Probable Clayey Silt, some Sand, trace Organics													
90.0														
2.1	Probable Clayey Silt to Silty Clay with pockets of Organics and Occasional Silty fine Sand seams													
75.5														
16.6	Probable Silty Clay to Clay, trace Sand													
68.8														
23.3	End of Cone Test													

# RECORD OF BOREHOLE No 23-10 1 OF 1 METRIC

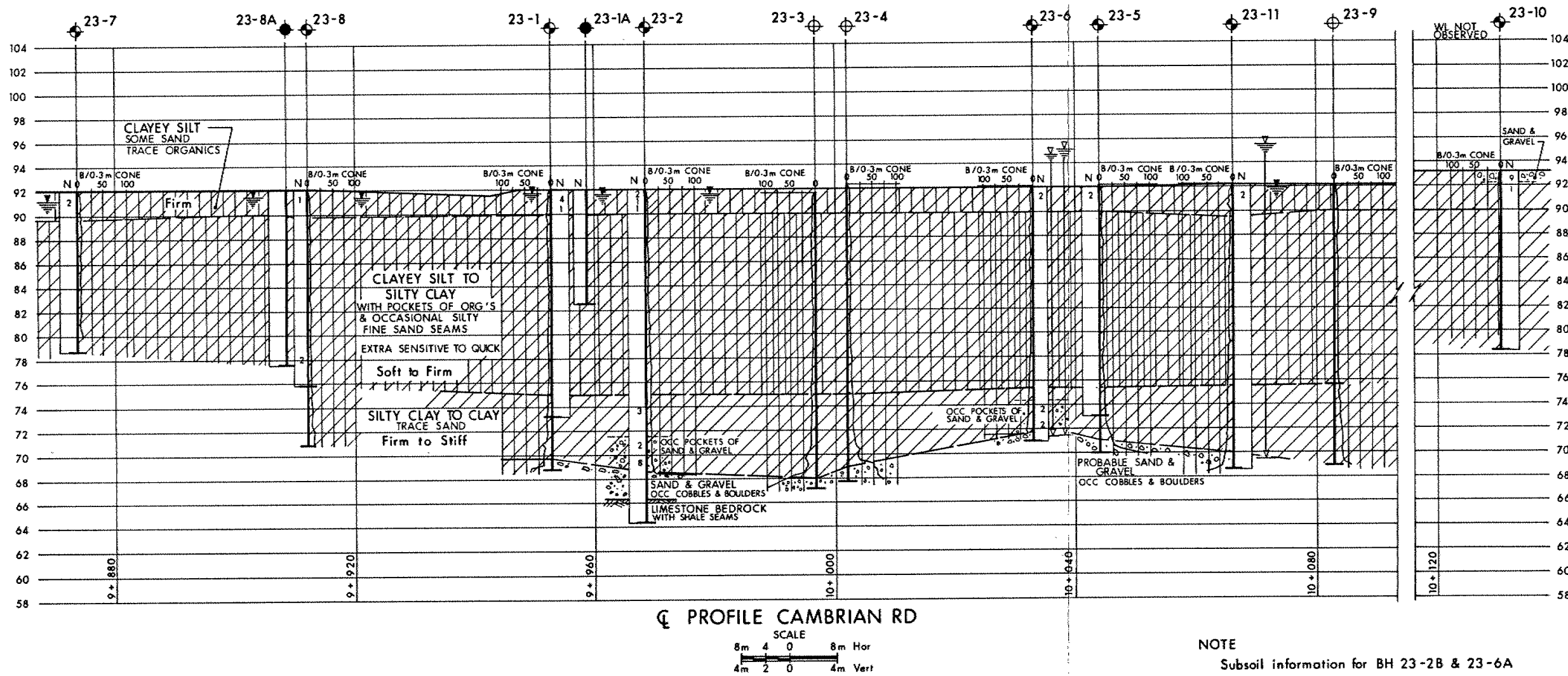
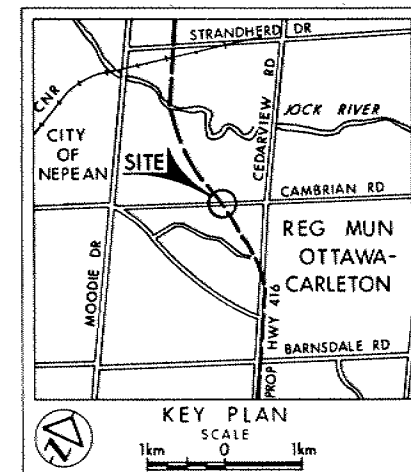
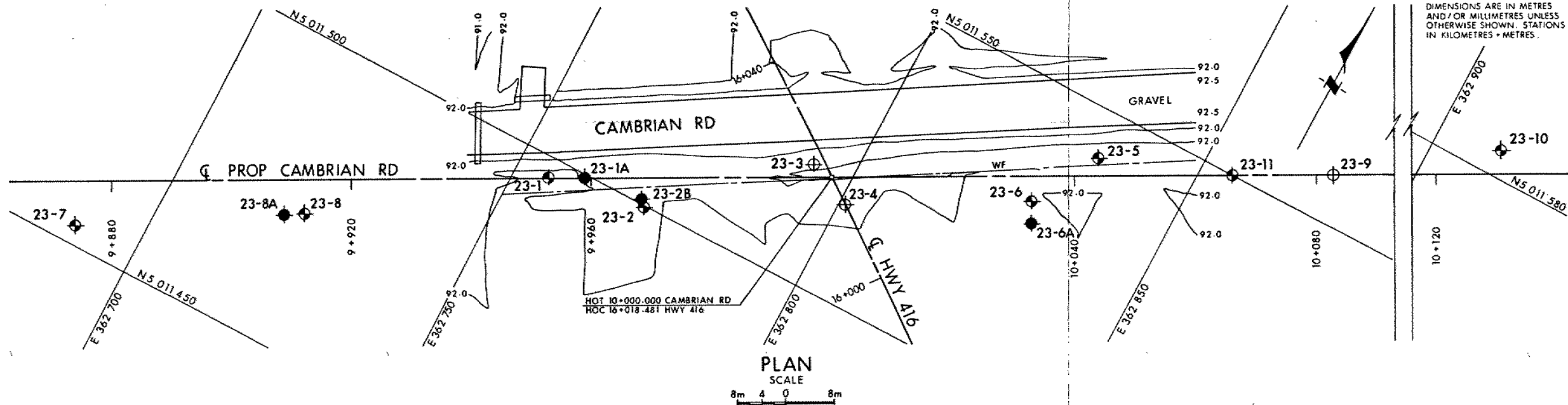
W.P. 128-87-09 LOCATION Co-ords: N 5 011 584.4; E 362 910.6 ORIGINATED BY B.S.  
 DIST 9 HWY 416 BOREHOLE TYPE H.S. Auger and Cone Test COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-29 CHECKED BY B.L.

SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT					NATURAL MOISTURE CONTENT	LIQUID LIMIT	UNIT WEIGHT	REMARKS & GRAIN SIZE DISTRIBUTION (%)
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20	40	60	80	100				
93.2	Ground Surface															
0.0	Sand and Gravel		1	SS	9	*	92									0 15 70 15
	Clayey Silt, some Sand, trace Organics, occasional pockets of Sand		2	SS	1											
90.0	Stiff															
3.2	Clayey Silt to Silty Clay with pockets of Organics and occasional Silty fine Sand seams		3	TW	PH		90									
	Extra Sensitive to Quick		4	SS	OW		88									
	Soft to Firm		5	TW	OW		86								17.3	0 14 49 37
			6	SS	OW		84									0 10 45 45
			7	TW	OW		82									
			8	SS	OW		80									
			9	TW	OW											
78.3			10	SS	OW											0 3 45 52
14.9	End of Borehole															
	* Water level not observed															

# RECORD OF BOREHOLE No 23-11 1 OF 1 METRIC

W.P. 128-87-09 LOCATION Co-ords: N 5 011 550.0; E 362 855.1 ORIGINATED BY B.S.  
 DIST 9 HWY 416 BOREHOLE TYPE H.S Auger and Cone Test COMPILED BY P.T.  
 DATUM Geodetic DATE 89-05-26-27 CHECKED BY B.I.






SOIL PROFILE			SAMPLES			GROUND WATER CONDITIONS	ELEVATION SCALE	DYNAMIC CONE PENETRATION RESISTANCE PLOT		PLASTIC LIMIT NATURAL MOISTURE CONTENT LIQUID LIMIT			UNIT WEIGHT 7 kN/m <sup>3</sup>	REMARKS & GRAIN SIZE DISTRIBUTION (%) GR SA SI CL
ELEV DEPTH	DESCRIPTION	STRAT PLOT	NUMBER	TYPE	'N' VALUES			20 40 60 80 100	W <sub>P</sub> W W <sub>L</sub>	WATER CONTENT (%) 20 40 60				
92.2														
0.0	Clayey Silt, some Sand, trace Organics  Firm		1	SS	2								0 11 50 39	
89.5			2	TW	-									
2.7	Clayey Silt to Silty Clay with packets of Organics and occasional Silty fine Sand seams  Extra Sensitive to Quick  Soft to Firm		3	SS	OW								17.3	0 12 45 43
			4	TW	OW									
			5	SS	OW									
			6	TW	-									
			7	SS	-									
			8	TW	OW									
			9	SS	OW									
			10	TW	PM									0 2 43 55
			11	SS	PM									
75.6														
16.6	End of Borehole													
	Probable Silty Clay to Clay, trace Sand													
70.0														
12.2	Probable Sand and Gravel, occasional Cobbles and Boulders													
68.5														
23.7	End of Cone Test  ** Estimated elevation of Artesian flow encountered from Cone hole  * To 95.5 ±													



**NOTE**

Subsoil information for BH 23-2B & 23-6A  
refer to Record of Borehole sheets

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}  
 WL at time of investigation 89 05  
 Head  
 ARTESIAN WATER  
 Encountered

No	ELEVATION	CO-ORDINATES	
		NORTH	EAST
23-1	92.3	5 011 496.5	362 755.6
23-1A	92.3	5 011 499.4	362 760.9
23-2	92.1	5 011 499.7	362 772.0
23-2B	92.1	5 011 500.7	362 771.0
23-3	91.7	5 011 519.2	362 743.6
23-4	92.3	5 011 515.8	362 801.6
23-5	92.2	5 011 542.2	362 834.4
23-6	92.2	5 011 530.5	362 828.0
23-6A	92.2	5 011 527.2	362 829.6
23-7	92.1	5 011 453.2	362 688.9
23-8	92.1	5 011 472.4	362 722.4
23-8A	92.1	5 011 470.7	362 719.4
23-9	92.1	5 011 557.8	362 869.9
23-10	93.2	5 011 584.4	362 910.4
23-11	92.2	5 011 550.0	362 855.6

NOTE

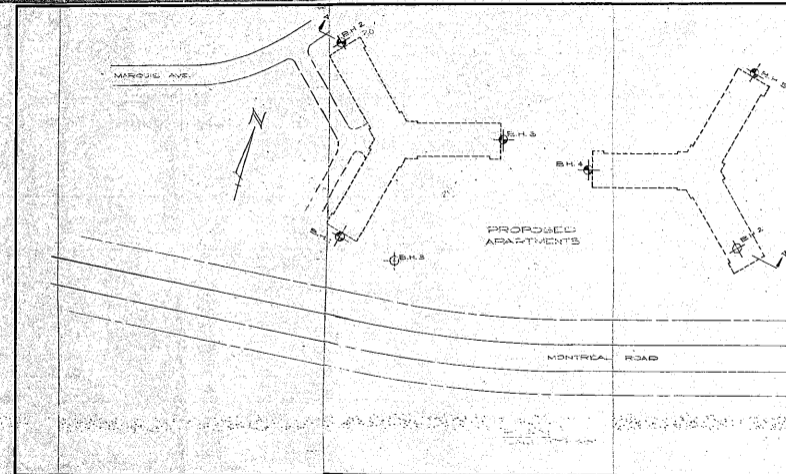
The boundaries between soil strata have been established only at Bore Hole locations. Between Bore Holes the boundaries are assumed from geological evidence.

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			

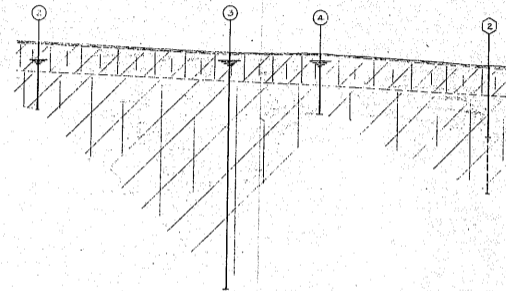
Geocres No 31G-198

HWY No 416		DIST 9	
SUBA'D MM	CHECKED	DATE 91 06 07	SITE 3-551
DRAWN DT	CHECKED	APPROVED	DWG 1288709-A



ELEVATION IN FEET

280  
260  
240  
220  
200  
180  
160



SECTION AA  
HORIZONTAL SCALE: 1" = 10' 0"  
VERTICAL SCALE: 1" = 10' 0"

GROUND PLAN AND  
SECTION CRADLE

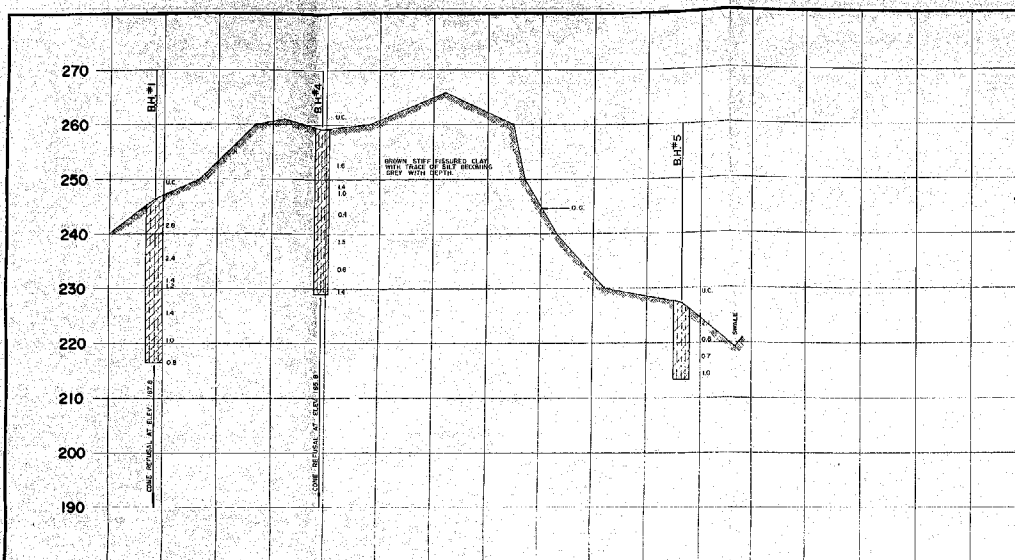
FIGURE

- LEGEND**
- BROWN TOPSOIL
  - VERY STIFF TO STIFF BROWN SILTY CLAY (VERY FINE GRAIN)
  - FIRM TO STIFF BROWN SILTY CLAY
  - BEDROCK
  - BOREHOLE IN PLAN PRESENT INVESTIGATION
  - BOREHOLE IN PLAN PREVIOUS GELLY REPORT 1962
  - BOREHOLE IN PLAN BY OTHERS APRIL, 1972
  - BOREHOLE IN ELEVATION PRESENT INVESTIGATION
  - BOREHOLE IN ELEVATION BY OTHERS APRIL, 1972
  - PENETRATION TEST IN ELEVATION
  - WATER LEVEL, APRIL 21, 1972
- REFERENCE: SITE PLAN SUPPLIED BY MINTO CONSTRUCTION, NO. 10000 000000

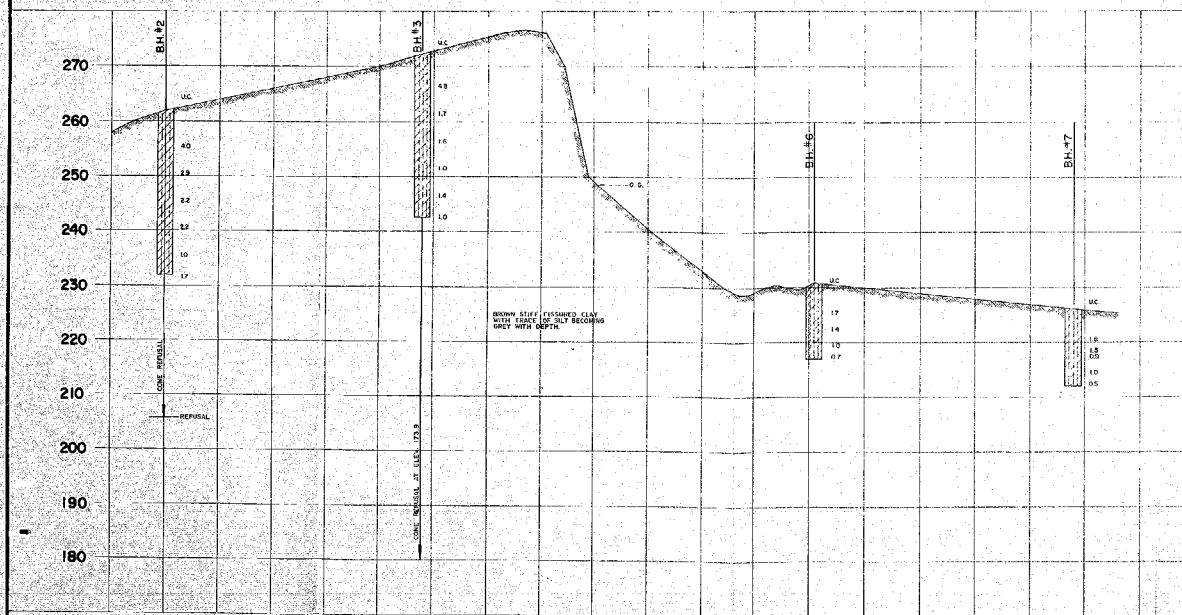
Date: 05/01/72

Golder Associates

Drawn: J.E.  
Checked: J.E.  
Approved: J.E.




SOIL PROFILE A-A

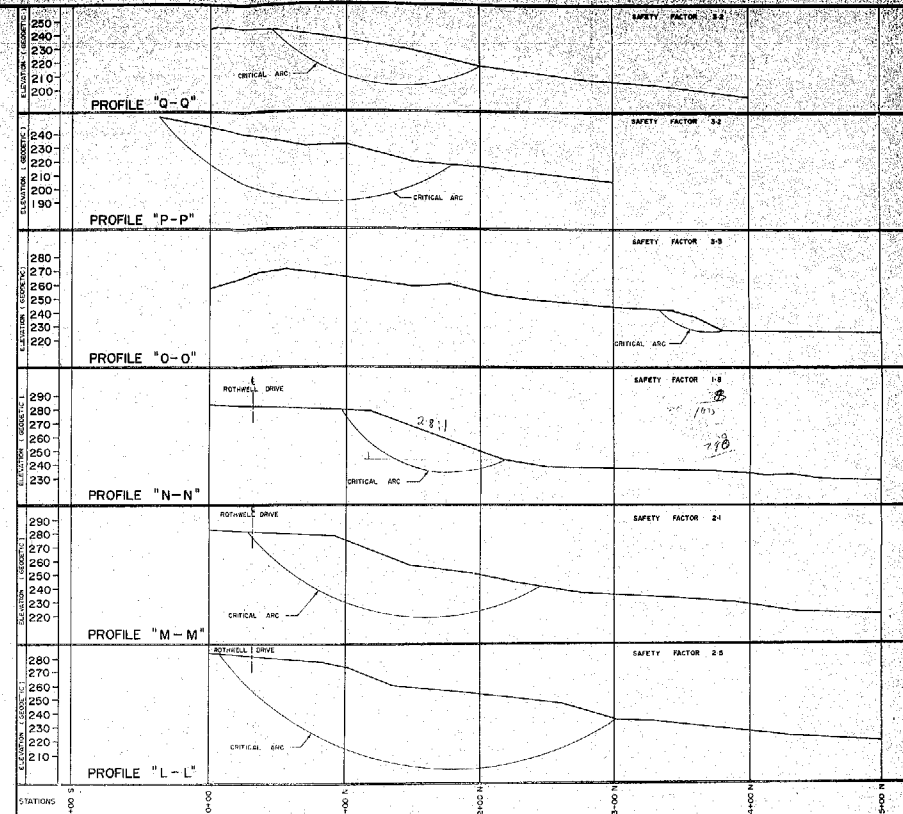
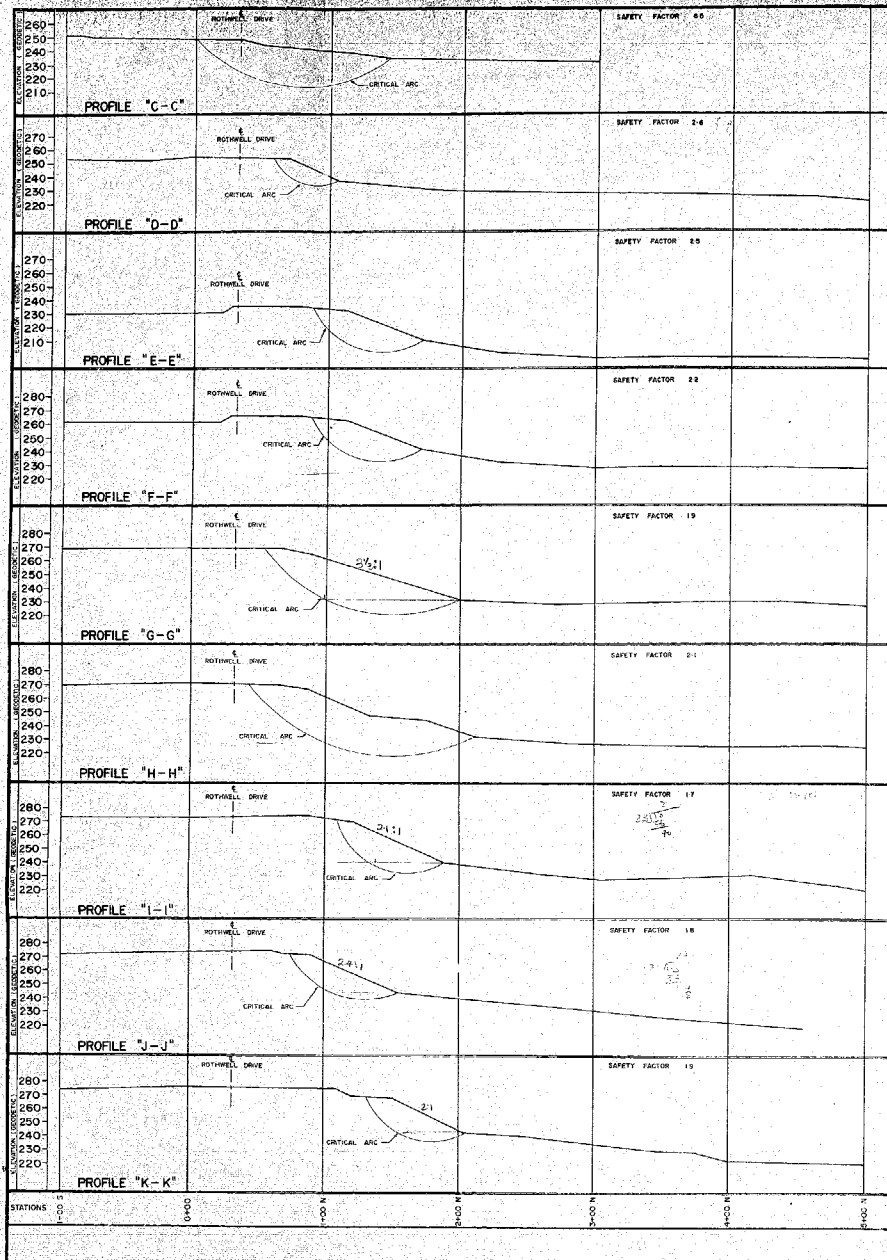


SOIL PROFILE B-B

# NOTES

1. ALL ELEVATIONS ARE GEODETIC.
2. SCALES: VERT. 1"=10' HORIZ. 1"=100'
3. U.C.—UNCONFIRMED COMPRESSION STRENGTH T.S.F.
4. SOIL TYPES ARE ACCURATE AT BOREHOLE LOCATIONS BETWEEN BOREHOLES SOIL TYPES HAVE BEEN INFERRED.
5. O.G.—ORIGINAL GROUND.

No.	DATE	DESCRIPTION	BY
			
<p>2000 No. <b>BEACONWOOD BLOCKS N. &amp; S.</b> MINTO CONSTRUCTION LTD., OTTAWA</p>			
<p><b>SOILS PROFILES</b></p>			
<p><b>BUTTS, MAGWOOD &amp; HALL LTD.</b> CONSULTING CIVIL ENGINEERS 1400 D. MERVILLE RD. OTTAWA TEL. 22-1414</p>			
DESIGNED BY:	C.E.M.	DATE:	JUNE, 1972
CHECKED BY:	C.E.M.	SCALE:	VERT. 1"=10' HORIZ. 1"=100'
APPROVED BY:	G. B. HALL, P. ENG.	CONT. No.	12-208-1
		DATE:	JULY, 1972



No.	DATE	DESCRIPTION	BY



ROOM No. \_\_\_\_\_

BEACONWOOD BLOCKS N. & S.  
MINTO CONSTRUCTION LTD. OTTAWA

PROFILES OF SCARP

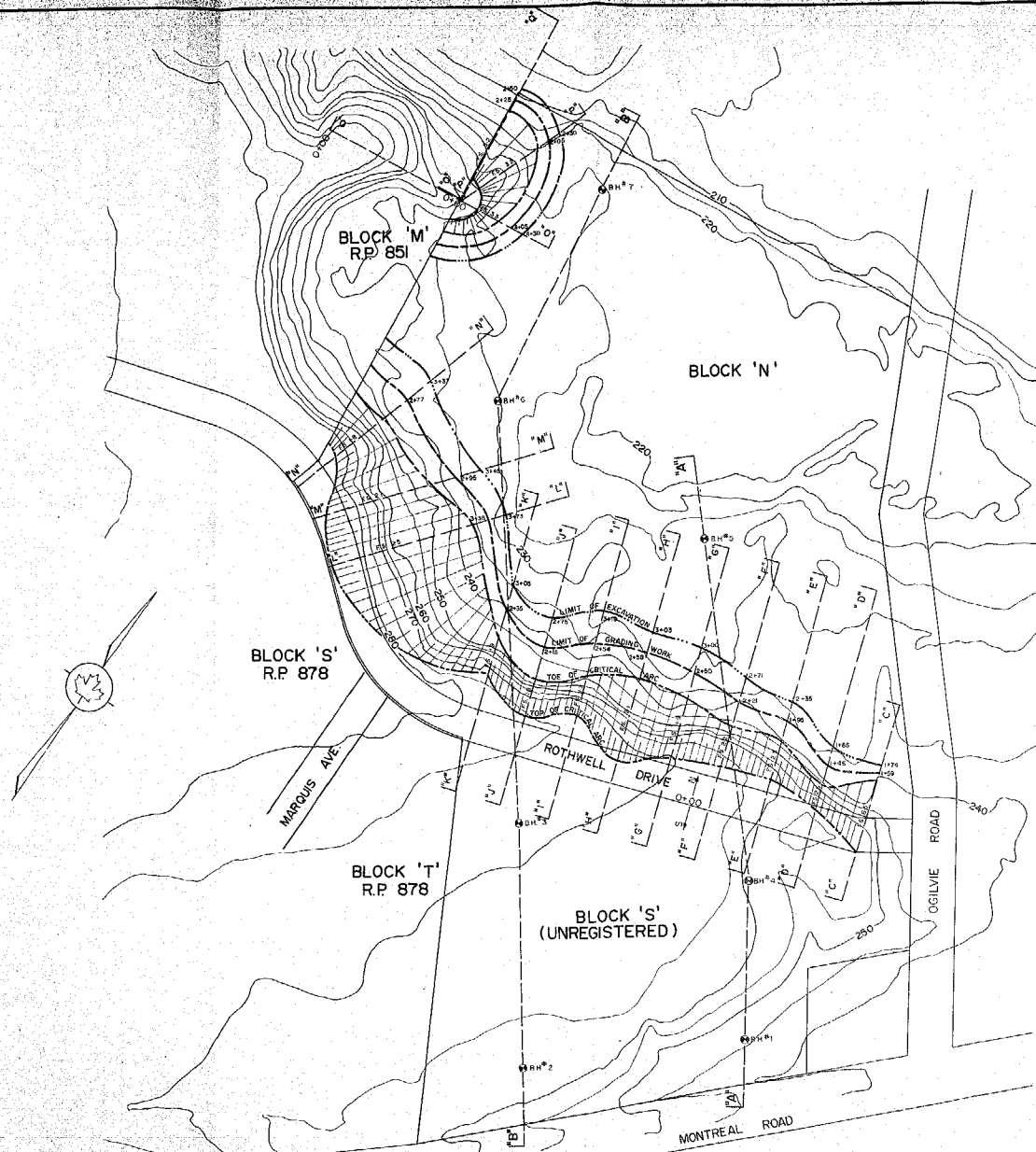
BUTTS, MAGWOOD & HALL LTD.

CONSULTING CIVIL ENGINEERS  
14800 MERIVALE RD. OTTAWA TEL. 224-1414

DRAWN BY: B.T. DATE: OCTOBER, 1972

CHECKED BY: G.R.H. SCALE: 1" = 40'

APPROVED BY: G.C. HALL, P.E. CONST. NO. 12-208-L DIV. NO. 1 FILE NO.



2	OCT '72	EXTERNAL REVISIONS	BY
1	JUNE '72	INTERNAL REVISIONS	BY
NO.	DATE	DESCRIPTION	BY
BOOK NO.			
<b>BEACONWOOD BLOCKS N. &amp; S.</b> MINTO CONSTRUCTION LTD. OTTAWA			
<b>SOILS STABILITY ASSESSMENT</b>			
<b>BUTTS, MAGWOOD &amp; HALL LTD.</b> CONSULTING CIVIL ENGINEERS 14850 MERIVALE RD. OTTAWA TEL. 224 1414			
DRAWN BY:	P. S. J. R.	DATE:	JUNE 1972
CHECKED BY:	G.B.H.	SCALE:	AS SHOWN
APPROVED BY:	G. B. HALL	DATE:	JUNE 1972