For Fig A $\underline{III}^2$

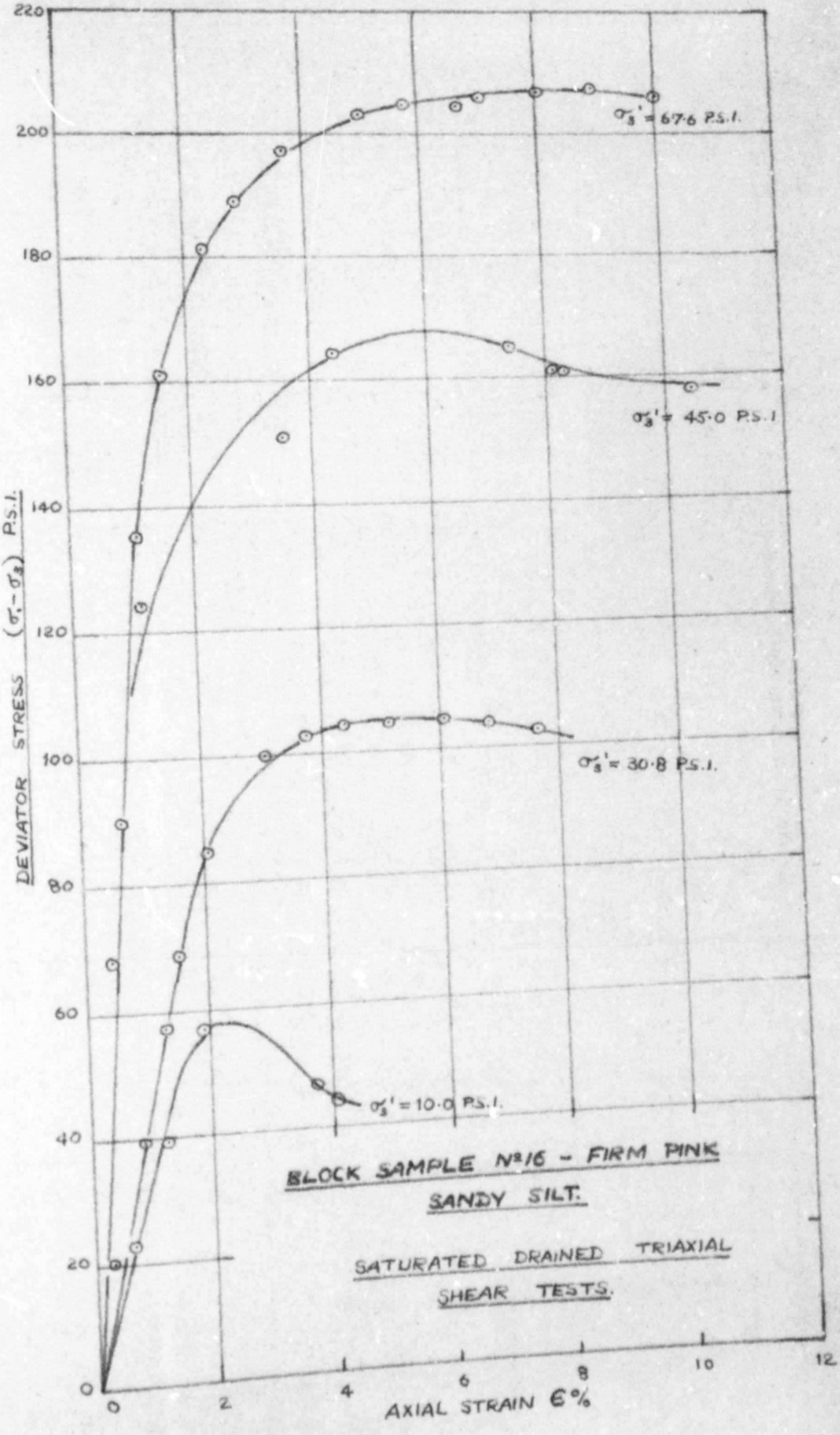
See last fiche.

APPENDIX IV

TABLE No. A IV 1

SAMPLE No. 16. SATURATED DRAINED TRIAXIAL SHEAR TEST. TABULATED RESULTS.

BEFORE TEST			σ_3'	STRAIN	VOLUME CHANGE	LOAD	CORRECTED LOAD	$\sigma_1 - \sigma_3$
γ LB.FT^{-3}	ω %	γ_d LB.FT^{-3}	P.S.I.	ϵ %	%	LBS.	LBS.	P.S.I.
115.2	17.33	98.1	10.0	0	0	0	0	0
				0.62	−0.5	41	41	23.4
				1.21	0.8	70	70	39.6
				1.87	0.8	102	101	57.1
				3.72	0.4	85	85	46.7
				4.08	0.2	81	78	44.1
116.2	17.02	99.6	30.8	0	0	0	0	0
				0.24	−0.1	18	18	10.2
				0.87	.6	70	70	39.7
				1.26	.7	102	101	57.6
				1.54	.8	122	121	69.1
				2.04	.9	152	150	85.1
				3.03	1.0	179	176	100.0
				3.73	.9	188	182	103.7
				4.37	.8	191	184	104.4
				5.16	.6	193	184	104.4
				6.03	.5	196	185	105.0
				6.80	.4	196	184	104.3
				7.66	.4	196	182	103.3
121.1	18.32	102.2	45.0	0	0	0	0	0
				1.11	−0.4	222	220	124.8
				3.53	.6	276	266	151.2
				4.40	.4	303	290	164.6
				7.40	.1	314	291	165.2
				8.10	.1	309	284	161.1
				8.23	0	309	284	161.1
				10.43	0	310	278	158.0
117.0	17.48	99.6	67.6	0	0	0	0	0
				0.63	−0.2	160	159	90.5
				1.56	.6	287	284	161.2
				2.87	.9	339	332	189.0
				3.70	.9	358	348	197.2
				5.76	.9	380	361	205.0
				4.97	.9	373	358	203.0
				6.63	.9	383	360	204.4
				7.47	.8	392	364	206.4
				8.93	.7	394	365	207.6



220
200
180
160
140
120
100
80
60
40
20
0
DEVIATOR STRESS $(\sigma_1 - \sigma_3)$ P.S.I.
$\sigma_3' = 67.6$ P.S.I.
$\sigma_3' = 45.0$ P.S.I.
$\sigma_3' = 30.8$ P.S.I.
$\sigma_3' = 10.0$ P.S.I.
BLOCK SAMPLE Nº 16 - FIRM PINK SANDY SILT.
SATURATED DRAINED TRIAXIAL SHEAR TESTS.
0
2
4
6
8
10
12
AXIAL STRAIN ϵ%

Fig AIV 1

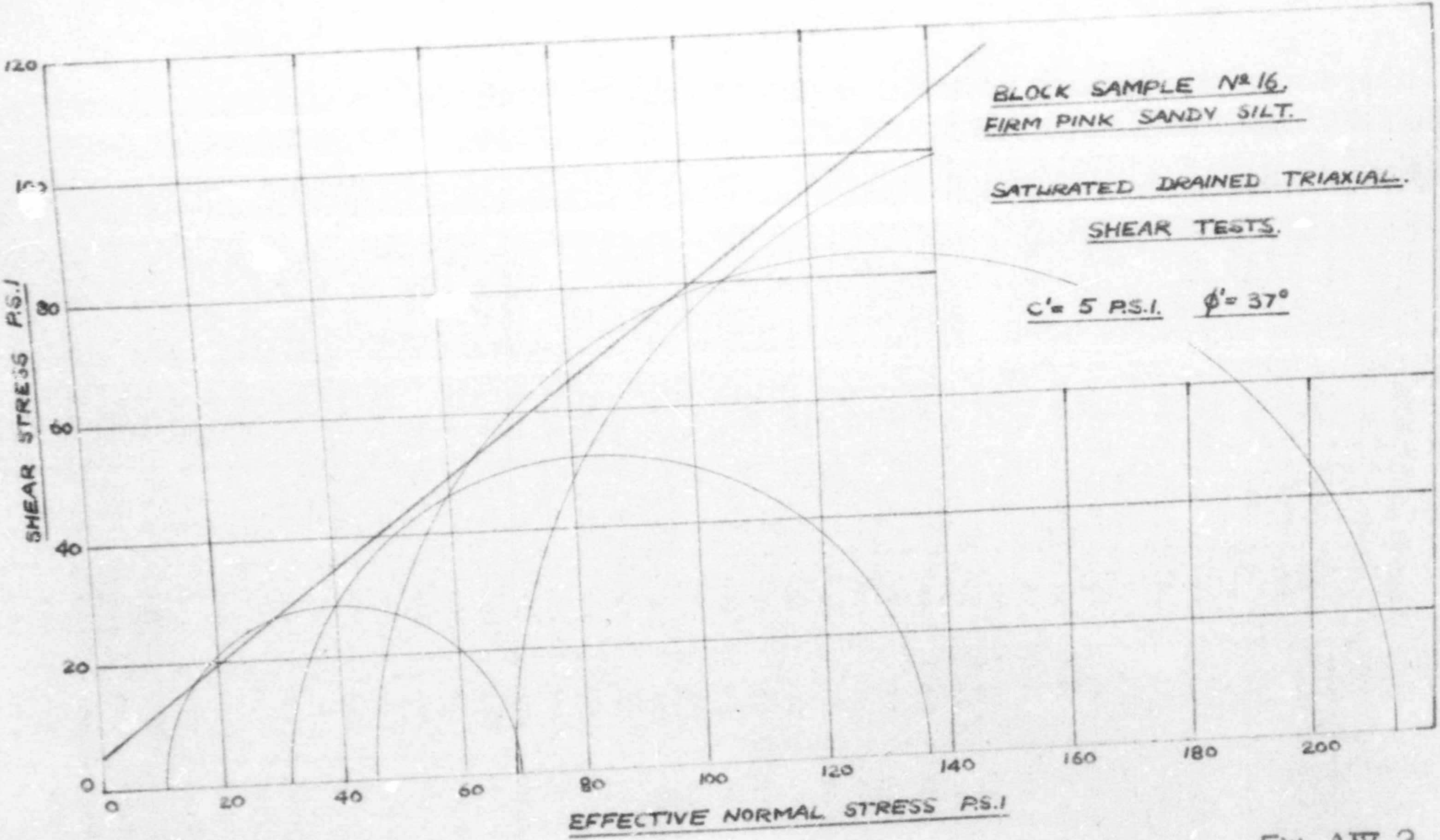
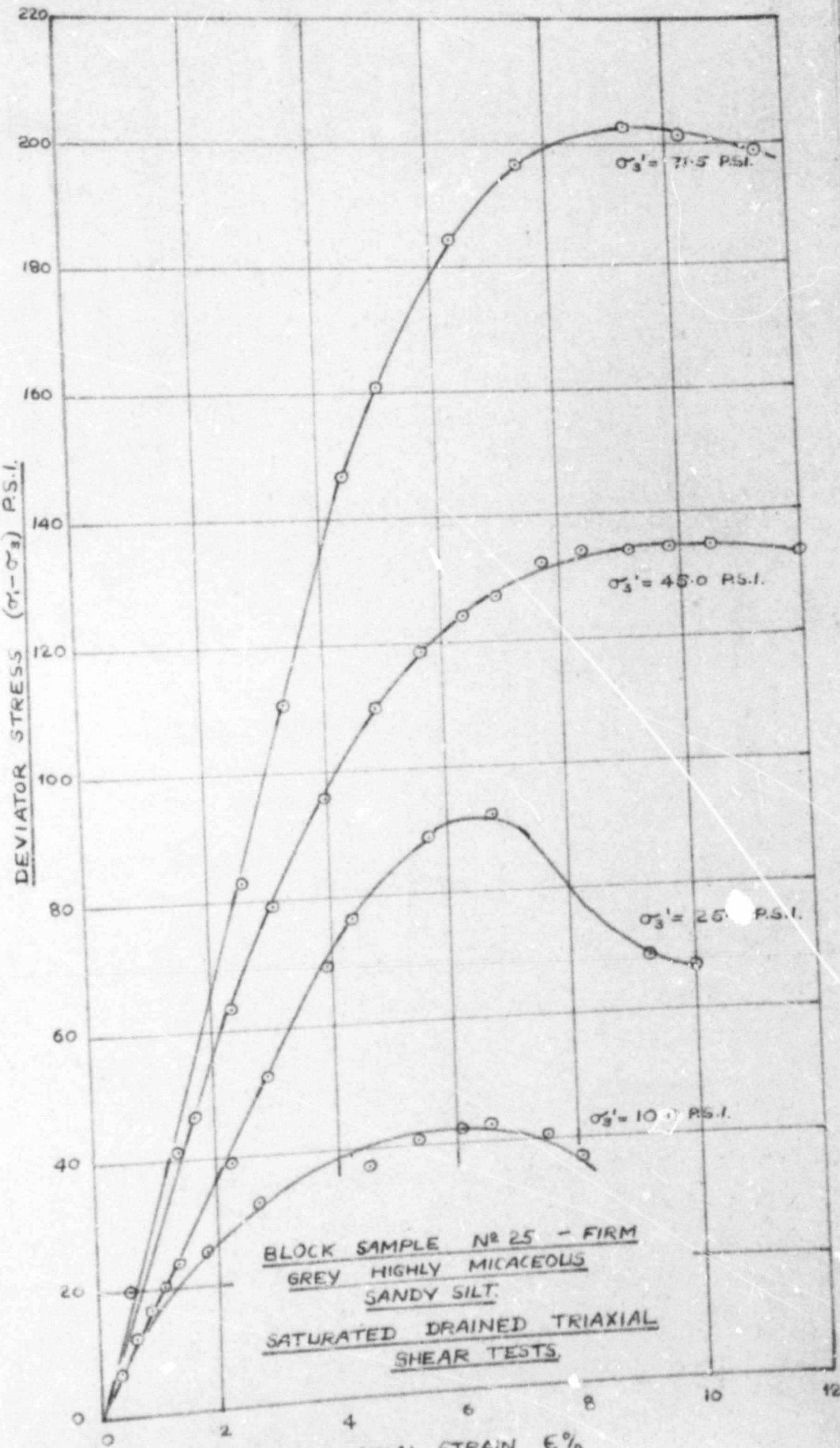

BLOCK SAMPLE № 16.
FIRM PINK SANDY SILT.
SATURATED DRAINED TRIAXIAL.
SHEAR TESTS.
C' = 5 P.S.I. ϕ' = 37°
SHEAR STRESS P.S.I
EFFECTIVE NORMAL STRESS P.S.I
0
20
40
60
80
100
120
140
160
180
200

Fig AIV 2

TABLE Nº A IV 2.

SAMPLE Nº 25. SATURATED DRAINED TRIAXIAL SHEAR TESTS. TABULATED RESULTS.

BEFORE TEST			σ_3' P.S.I.	STRAIN ϵ %	VOLUME CHANGE %	LOAD LB.	CORRECTED LOAD LB.	$\sigma_1 - \sigma_3$ P.S.I.
γ' LB.FT^{-3}	ω %	γ_d LB.FT^{-3}						
112·6	14·03	98·7	10·0	0	0	0	0	0
				1·21	0	24	24	13·3
				2·10	−0·1	36	36	20·2
				4·53	·6	69	66	37·5
				3·69	·5	59	57	32·6
				5·36	·5	76	72	41·0
				6·57	·4	81	76	43·3
				7·50	·4	77	71	40·4
115·2	14·03	101·1	25·0	0	0	0	0	0
				1·33		43	42	24·0
				2·25		71	69	39·4
				2·93		95	92	52·1
				3·90		128	123	64·8
				4·39		142	136	77·1
				5·66		167	157	89·5
				6·78		174	162	92·3
				9·23		133	121	68·7
109·8	153·2	95·0	45·0	1·42	−0·1	74	74	41·9
				2·31	·1	115	112	63·7
				3·95	·3	176	169	96·0
				4·85	·5	202	194	110·1
				5·67	·5	221	209	118·9
				6·90	·6	239	224	127·3
				7·73	·7	250	233	132·2
				9·18	·7	258	236	134·0
				10·53	·7	262	237	134·6
				11·97	·7	265	235	133·6
110·2	13·07	97·1	71·5	0	0	0	0	0
				1·67	−0·1	84	83	47·0
				2·60	·2	150	146	83·1
				3·35	·3	201	195	111·0
				4·44	·8	266	259	147·1
				5·03	·9	295	283	161·0
				6·37	1·1	343	325	184·8
				7·52	1·3	369	346	196·8
				9·38	1·4	388	356	202·4
				10·27	1·4	389	354	201·0



DEVIATOR STRESS $(\sigma_1 - \sigma_3)$ P.S.I.
AXIAL STRAIN ϵ%
$\sigma_3' = 71.5$ P.S.I.
$\sigma_3' = 45.0$ P.S.I.
$\sigma_3' = 25$ P.S.I.
$\sigma_3' = 10$ P.S.I.
BLOCK SAMPLE Nº 25 - FIRM GREY HIGHLY MICACEOUS SANDY SILT.
SATURATED DRAINED TRIAXIAL SHEAR TESTS.
0
20
40
60
80
100
120
140
160
180
200
220
2
4
6
8
10
12

Fig A IV 3

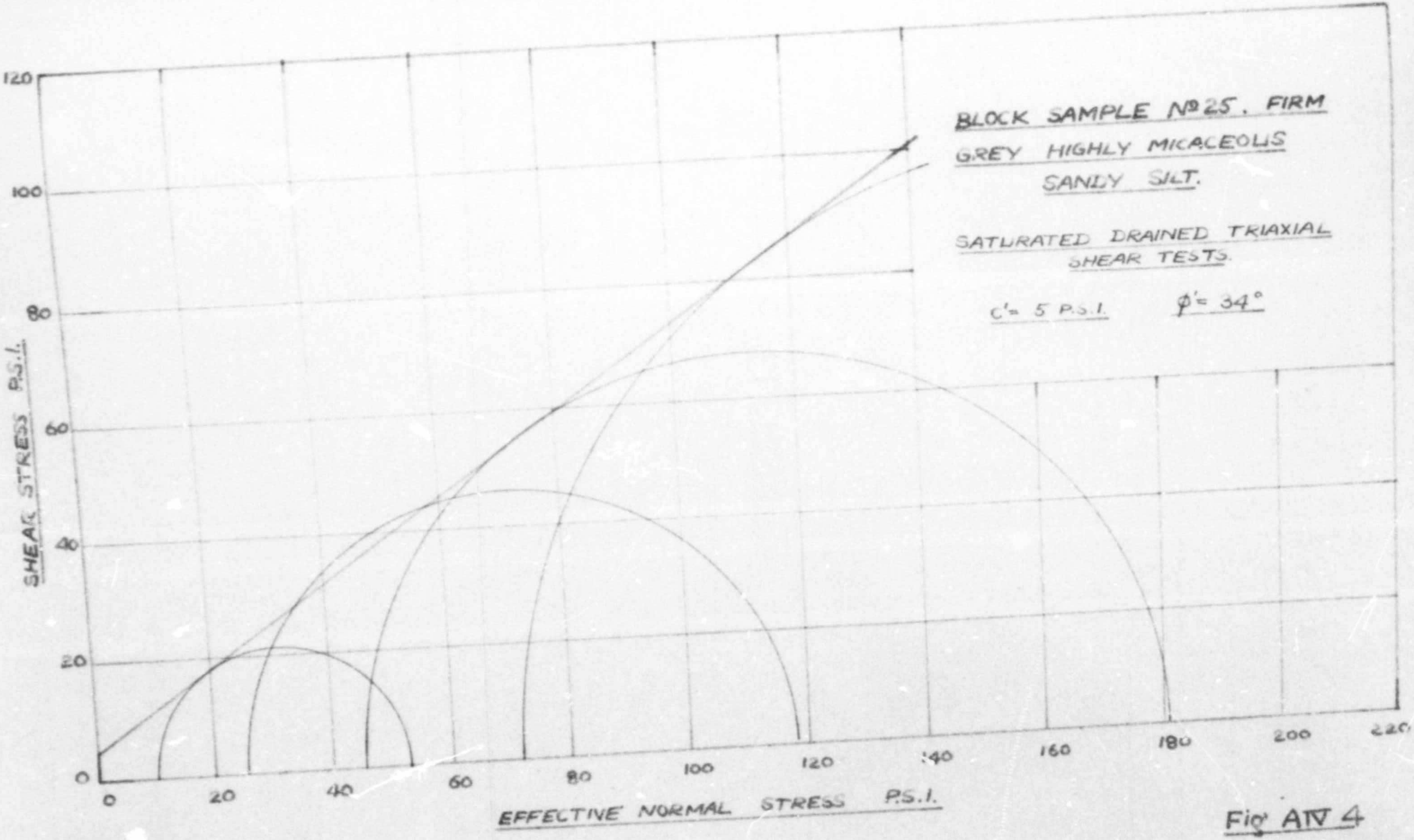
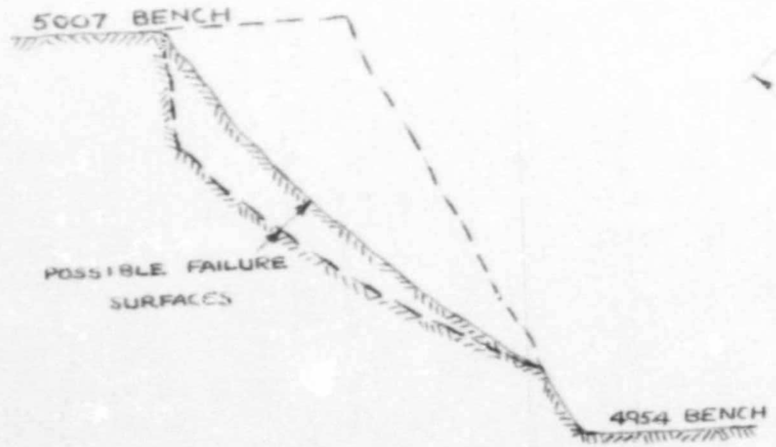

120
100
80
60
40
20
0
SHEAR STRESS P.S.I.
0
20
40
60
80
100
120
140
160
180
200
220
EFFECTIVE NORMAL STRESS P.S.I.
BLOCK SAMPLE No 25. FIRM
GREY HIGHLY MICACEOUS
SANDY SILT.
SATURATED DRAINED TRIAXIAL
SHEAR TESTS.
c' = 5 P.S.I.
φ' = 34°

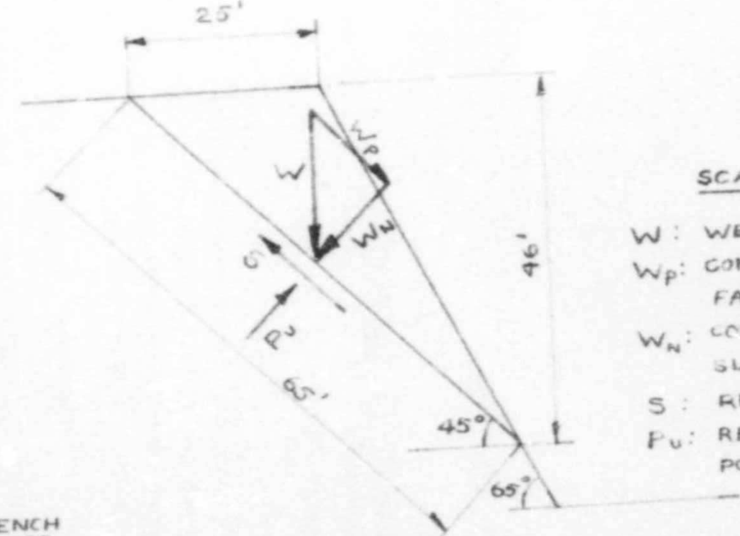
Fig AIV 4

APPENDIX V

ANALYSIS OF LOCAL BENCH FAILURE.



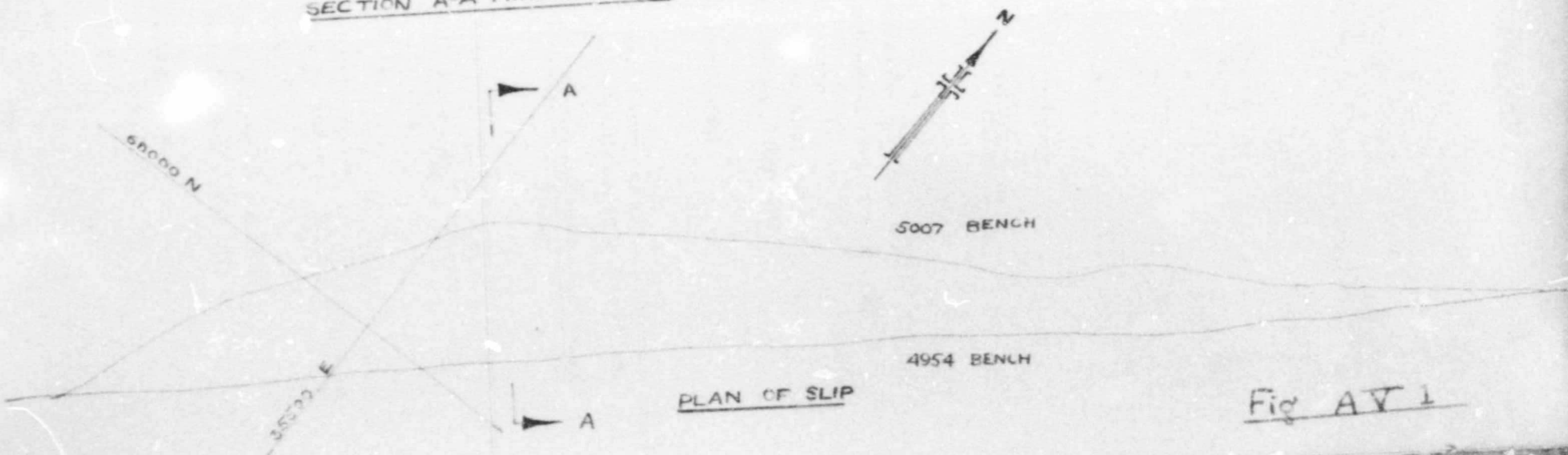
SECTION A-A THROUGH SLIP



SCALE : 1 : 250

W : WEIGHT OF SLIDING MASS
W_P: COMPONENT PARALLEL TO FAILURE SURFACE.
W_N: COMPONENT NORMAL TO FAILURE SURFACE.
S : RESISTING SHEAR
P_U: RESULTANT FORCE DUE TO PORE WATER PRESSURE.

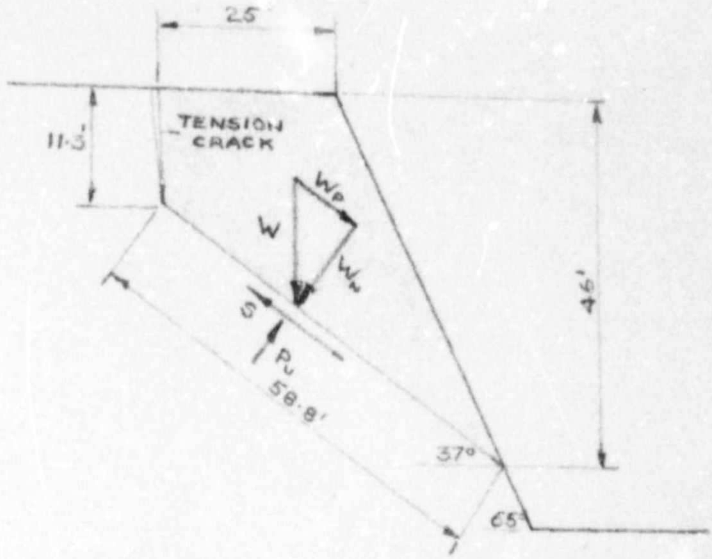
IDEALISED FAILURE (NO TENSION CRACK)



PLAN OF SLIP

Fig AV 1

ANALYSIS OF LOCAL BENCH FAILURE.



IDEALISED FAILURE (WITH TENSION CRACK)

SOIL PROPERTIES : DENSITY: 127 $LB.FT^{-3}$

C' APPARENT COHESION 720 $LB\,FT^{-2}$

ϕ' ANGLE OF SHEARING RESISTANCE: 35°

W = WEIGHT OF SLIDING MASS PER FOOT.
W_P = COMPONENT OF W PARALLEL TO FAILURE SURFACE
W_N = COMPONENT OF W NORMAL TO FAILURE SURFACE
S = RESISTING SHEAR FORCE
P_U = UPLIFT DUE TO PORE WATER PRESSURES

(i) NO TENSION CRACK

$W = \frac{25 \times 46 \times 127}{2} = 73100$ LB $\qquad W_P = 73100 \cos 45° = 51600$ LB

$\qquad W_N = 73100 \sin 45° = 51600$ LB

$$F.S. = \frac{\text{RESISTING FORCE}}{\text{SLIDING FORCE}} = \frac{720 \times 65 + (51600 - P_U) \tan 35°}{51600}$$

$$F.S. = \frac{83{,}000 - 0{\cdot}7 P_U}{51{,}600}$$

FULL HYDROSTATIC PRESSURE: $P_U' = \frac{62{\cdot}4 \times (46)^2}{2} = 66{,}000$ LB.

F.S. = 1·0 $\qquad P_U = 0{\cdot}68\ P_U'$

F.S. = 1·61 $\qquad P_U = 0$

(ii) WITH TENSION CRACK $\qquad$ MAX. DEPTH OF CRACK: $\frac{2 \times 720}{127} = 11{\cdot}3$ FT.

$W = 127 \left(25 \times 11{\cdot}5 + \frac{5}{2} \times 11{\cdot}3 + \frac{1}{2} \times 34{\cdot}7 \times 30\right)$

W = 105, 600 LB. $\qquad W_P = 105{,}600 \sin 37° = 63{,}500$ LB.

$\qquad W_N = 105{,}600 \cos 37° = 84{,}300$ LB.

$$F.S. = \frac{720 \times 58{\cdot}8 + (84{,}300 - P_U) \tan 35°}{63{,}500}$$

$$F.S. = \frac{101{,}300 - 0{\cdot}7 P_U}{63{,}500}$$

F.S. = 1·0 $\qquad P_U = 0{\cdot}82\ P_U'$

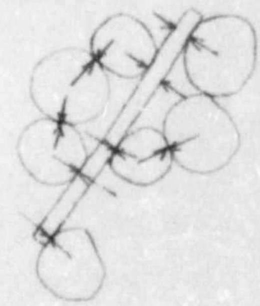
F.S. = 1·59 $\qquad P_U = 0$

Fig A V 2

APPENDIX VI

PRINCIPLES INVOLVED IN THE CONSOLIDATION OF A SOIL AS A RESULT OF LOWERING THE GROUND WATER.

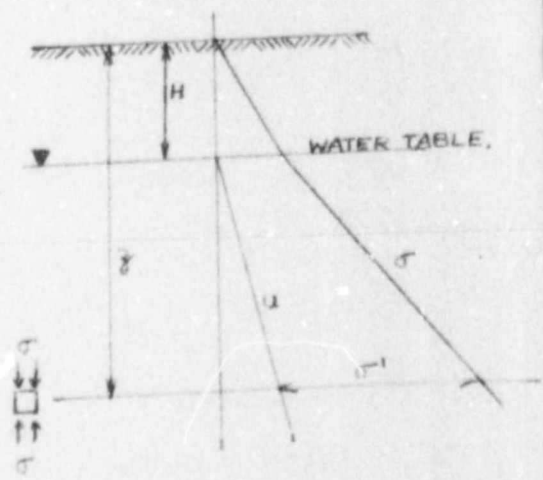
(1) Consolidation takes place due to increase of intergrain stress.
Consider grain structure shown: voids are filled with water under pressure "u"
Grain-to-grain contact areas are small.
Then change in "u" will not affect the volume of the soil skeleton shown.
However the volume will decrease when the intergrain forces are increased.
The intergrain forces may be considered statistically to act over a whole cross-sectional area through the soil giving rise to the intergranular pressure.



(2) The total pressure at any point in the soil = sum of intergranular pressure plus porewater pressure ie.

σ = total stress = $\sigma' + u$.

This is the effective stress law and depends on the experimental fact that grain-to-grain contact areas are small.



At any depth 'z' in an isotropic soil mass the total pressure must be due to the weight of everything lying above the point.
Say, soil above Water table has density γ wt/unit vol.
and soil below Water table is saturated and has density γ_{sat}.
Let γ_w = density of water.

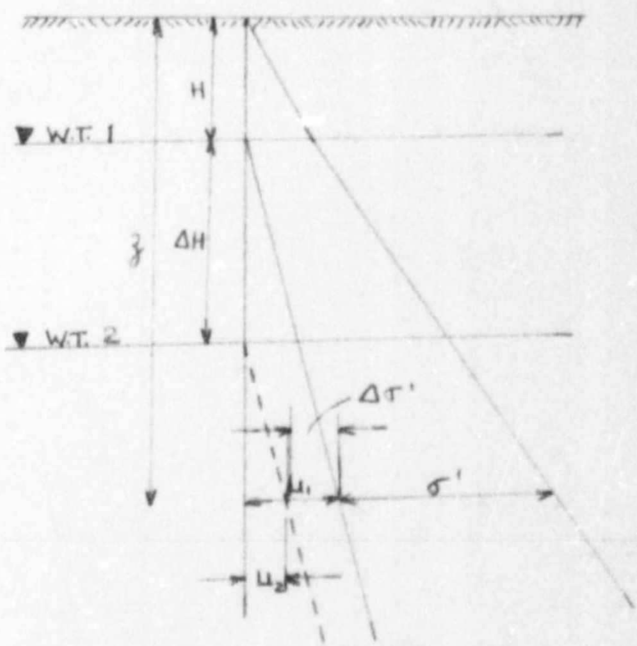
Fig AVI 1(a)

Then $\sigma = H\gamma + (z-H)\gamma_{sat.}$

and $u = (z-H)\gamma_w$

$$\therefore \sigma' = H\gamma + (z-H)\gamma_{sat.} - (z-H)\gamma_w$$
$$= H\gamma + (z-H)(\gamma_{sat.} - \gamma_w)$$
$$= H\gamma + (z-H)\gamma' \quad \text{where } \gamma' = \text{submerged density of the soil.}$$

(3) When a water table is lowered the effect is to increase the intergranular stress in the soil below the original water table.



At depth z initially with W.T. at H. $\sigma' = H\gamma + (z-H)\gamma'$

and $\sigma' = H\gamma + \Delta H.\gamma + (z-H-\Delta H)\gamma'$ for W.T. at $H+\Delta H$.

$$\therefore \Delta\sigma' = \Delta H.\gamma - \Delta H.\gamma'$$
$$= \Delta H(\gamma - \gamma')$$
$$= \Delta H.\gamma_w$$

= Wt. of column of water of height equal to lowering of W. Table.

Fig AVI 1(b)

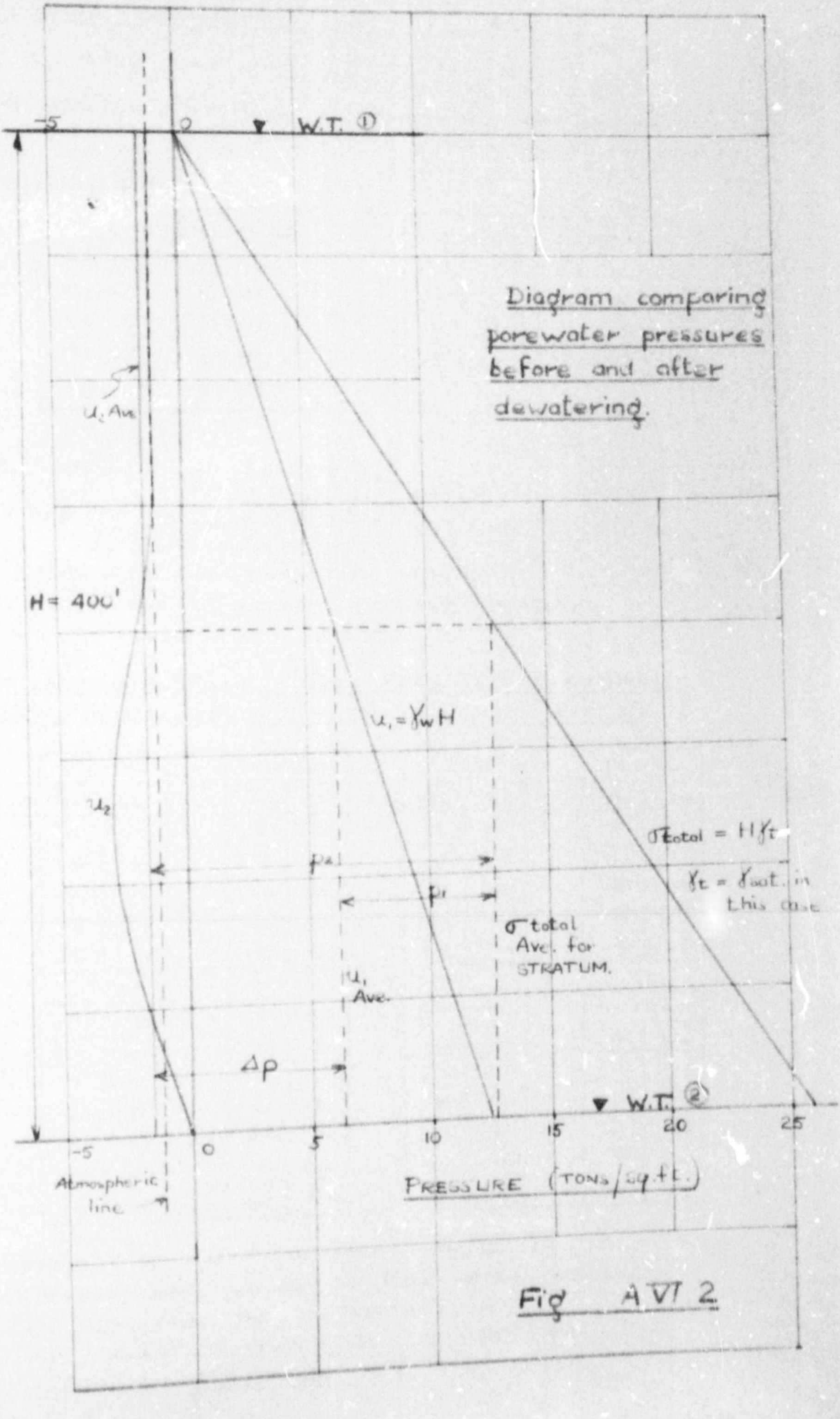


Diagram comparing porewater pressures before and after dewatering.
W.T. ①
W.T. ②
H = 400'
u_2 Ave
u_2
$u_1 = \gamma_w H$
$\sigma_{total} = H\gamma_t$
$\gamma_t = \gamma_{sat}$ in this case
p_2
p_1
σ total Ave. for STRATUM.
u_1 Ave.
Δp
Atmospheric line
-5
0
5
10
15
20
25
PRESSURE (TONS/sq.ft.)

Fig AVI 2

APPENDIX VI

DETERMINATION OF ISOCHRONES FOR 400 ft STRATUM UNDERGOING DEWATERING [SEE Fig AVI 2]

TAKE C_v IN THIS CASE AS 0·01 cm²/sec

TIME FACTOR $T = \frac{C_v . t}{H^2}$ where t = time in years
H = thickness of stratum.

FROM THIS WE GET

WHEN

T	t
T = 0	t = 0 yrs
T = ·05	t = 23 "
T = ·10	t = 46 "
T = ·15	t = 69 "
T = ·20	t = 92 "
T = ·30	t = 138 "
T = ·40	t = 185 yrs
T = ·50	t = 230 "
T = ·60	t = 278 "
T = ·80	t = 370 "
T = 1·0	t = 465 "

FROM CONSOLIDATION THEORY.

$$U_z = 1 - \sum_{m=0}^{m=\infty} \frac{2}{M}\left(\sin \frac{Mz}{H}\right) e^{-M^2 T}$$

U_z = THE CONSOLIDATION RATIO.
Z = THE CO-ORDINATE DISTANCE MEASURED FROM THE SURFACE.

THE FOLLOWING VALUES WERE READ OFF STANDARD CONSOLIDATION CURVES. SEE PAGE 235 REF ②

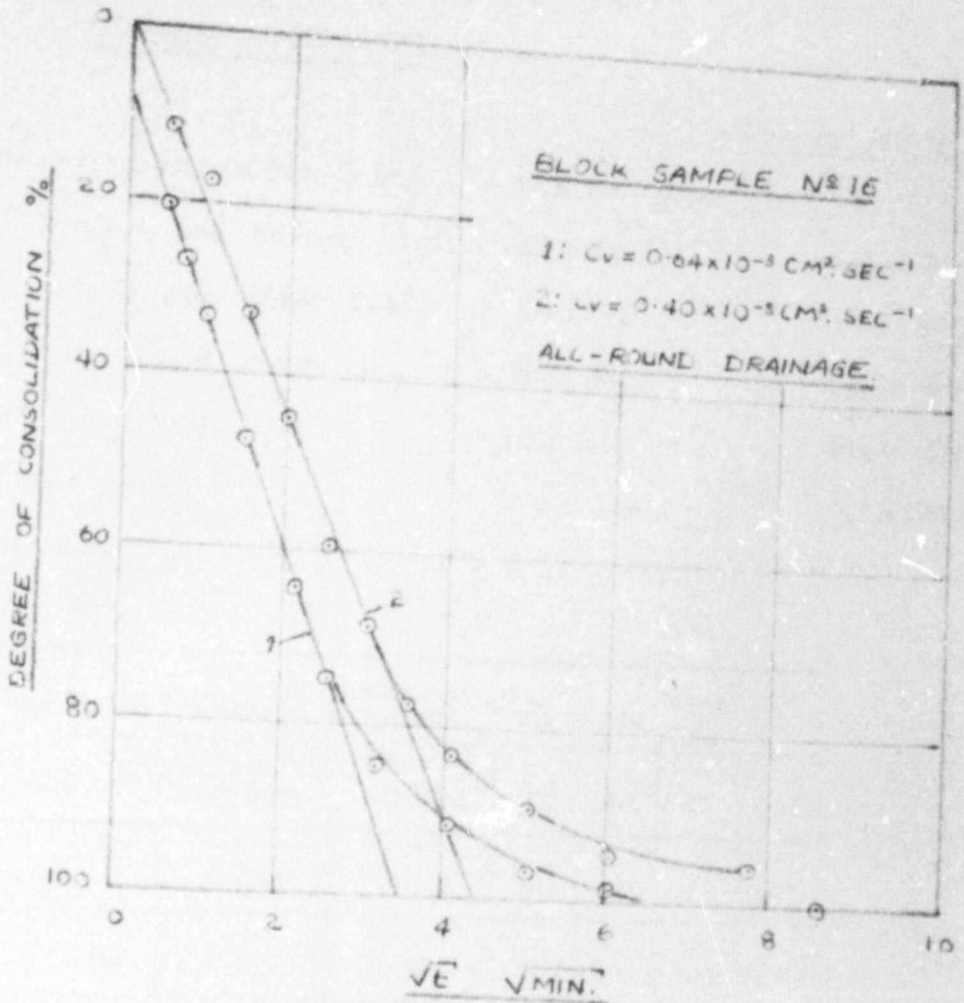
z/H	T = 0·05 U_z	T = 0·10 U_z	T = 0·15 U_z	T = 0·20 U_z
0	1·0	1·0	1·0	1·0
·25	·44	·58	·64	·70
·50	·11	·26	·36	·44
·75	·02	0·1	0·20	·29
1·0	·01	·05	·14	·23

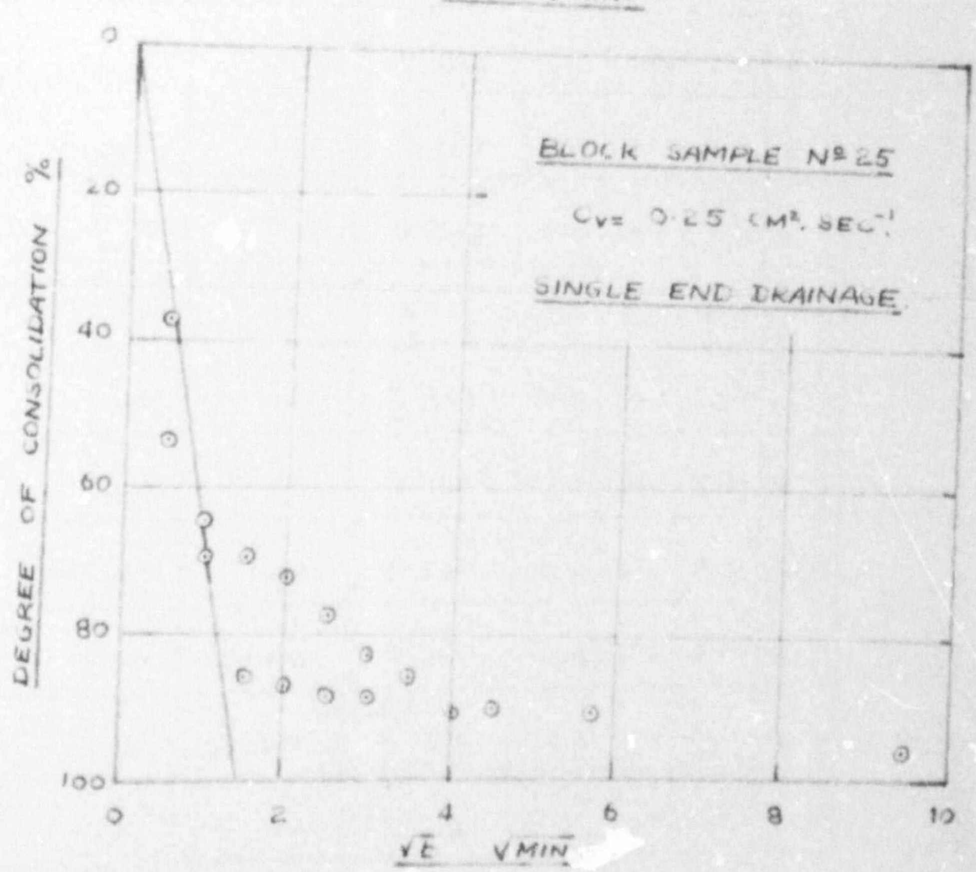
z/H	T = 0·30 U_z	T = 0·40 U_z	T = 0·50 U_z	T = 0·60 U_z
0	1·0	1·0	1·0	1·0
·25	·77	·82	·86	·90
·50	·57	·67	·74	·80
·75	·44	·56	·66	·74
1·0	·40	·53	·63	·71

VALUES OF U_z WERE THEN CHANGED TO THE CORRESPONDING VALUES OF PORE WATER PRESSURE BY ADJUSTING THE HORIZONTAL SCALE. THE FAMILY OF CURVES WAS PLOTTED AS SHOWN IN FIG. 16(a)

Fig AVI 3

APPENDIX VII





MEASUREMENTS OF COEFFICIENT OF CONSOLIDATION.

Fig's A VII 1(a) + (b)

TABLE Nº A VII 1

Sample Nº 16 L.H.

Weight of consolidometer ring = 123·17 gms

Weight of ring + soil before test = 337·52 ..

Weight of ring + soil after test = 345·70 ..

Weight of ring + soil oven dry = 306·40 ..

Weight of dry soil. = 183·23 ..

Diameter of ring = 7·62 cms

Height of ring = 2·51 cms

$S.G = 2{\cdot}76$

$V_s = 67{\cdot}7$ cc.

$H_v = {\cdot}405$ ins.

$H_s = {\cdot}583$ ins

$\gamma_d = 174{\cdot}5$ lb/cu.ft.

DATE	TIME	(lb) LOAD	READING	CORRECTED READING	Δ	H_v	LOAD T/□'	e
19/4/61	11·00	0	0·303	0·303	0	·413	0	·695
	11·00	5	0·314	0·314	·011	·402	·5	·671
20/4/61		10	0·319	0·319	·005	·397	1·0	·667
		20	0·332	0·332	·013	·384	2·0	·655
		40	0·337	0·337	·005	·379	4·0	·640
		80	0·347	0·347	·010	·369	8·0	·626
		160	0·411	0·361	·014	·355	16·0	·604
		200	0·419	0·369	·008	·347	20·0	·597
		160	0·418	0·368	-·001	·348	16·0	·604
		80	0·414	0·364	-·004	·352	8·0	·604
		40	0·410	0·360	-·004	·356	4·0	·610
		20	0·406	0·356	-·004	·360	2·0	·619
		10	0·401	0·352	-·004	·364	1·0	·627

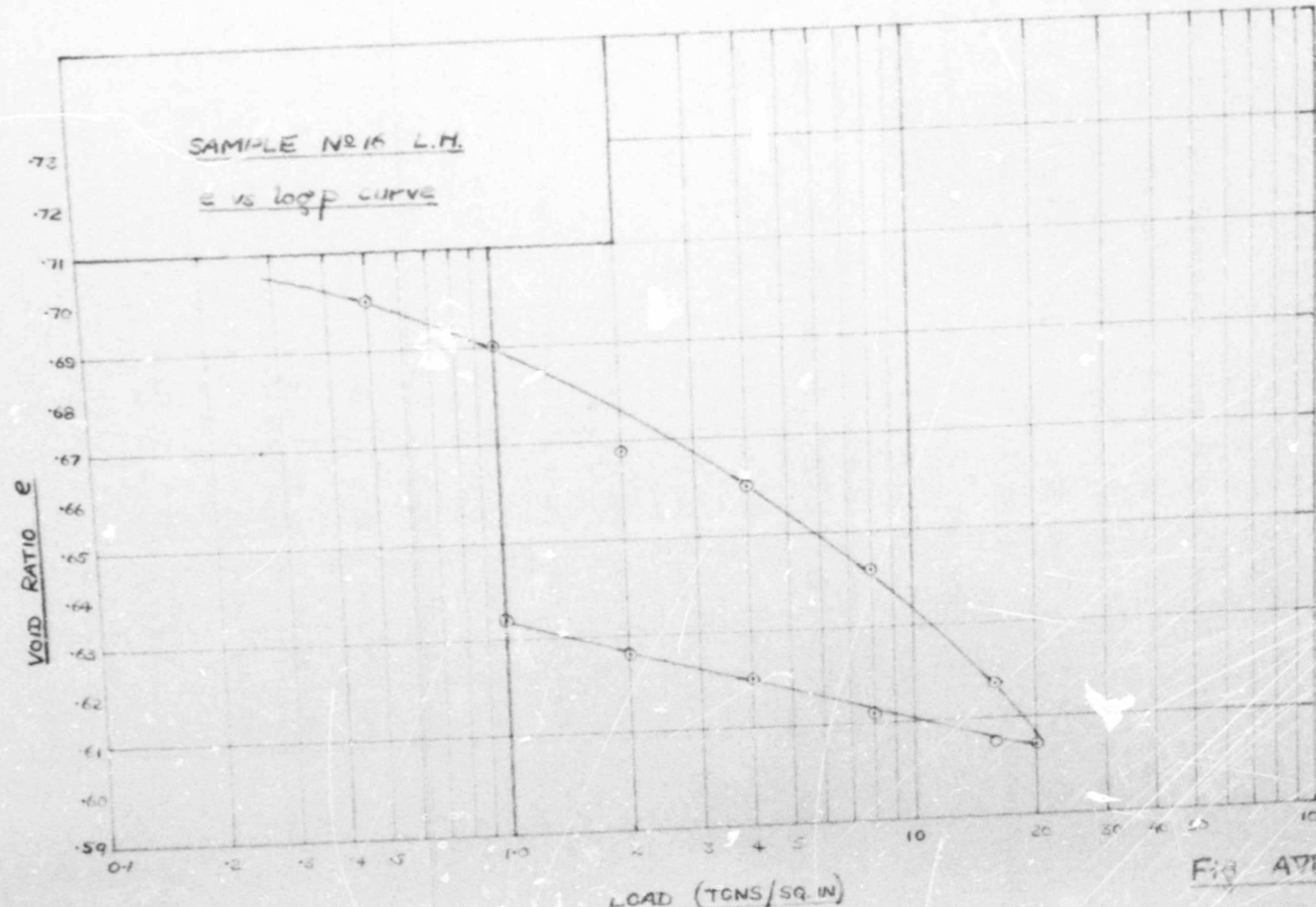

SAMPLE Nº 16 L.H.
e vs log p curve
VOID RATIO e
·73
·72
·71
·70
·69
·68
·67
·66
·65
·64
·63
·62
·61
·60
·59
0·1
·2
·3
·4
·5
1·0
2
3
4
5
10
20
30
40
50
100
LOAD (TONS/SQ. IN)

Fig AVII 2

TABLE Nº A VII 2

Sample Nº 16 R.H.

Weight of consolidometer ring	= 120·98 gms		S.G. = 2·76
Weight of ring + soil before test	= 339·30 "		V_s = 67·7cc.
Weight of ring + soil after test	= 346·20 "		H_v = ·405 ins
Weight of ring + soil oven dry	= 307·70 "		H_s = ·585 "
Weight of dry soil	= 186·72 "		γ_d = 174·5 lb/cu.ft.
Diameter of ring	= 7·62 cms		
Height of ring	= 2·51 "		

DATE	TIME	LOAD lb	LOAD T/□'	READING	CORRECTED READING	Δ	H_v	e
19/4/61	11 40	0	0	0·328	0·328	0	·405	·695
	11 40	5	·5	0·342	0·342	·014	·391	·671
[illegible]		10	1·0	0·344	0·344	·002	·389	·667
		20	2·0	0·351	0·351	·007	·382	·655
		40	4·0	0·360	0·360	·009	·373	·640
		80	8·0	0·368	0·368	·008	·365	·626
		160	16·0	0·381	0·381	·013	·352	·604
		200	20·0	0·385	0·385	·004	·348	·597
		160	16·0	0·385	0·385	·000	·348	·597
		80	8·0	0·381	0·381	–·004	·352	·604
		40	4·0	0·377	0·377	–·004	·356	·610
		20	2·0	0·372	0·372	–·005	·361	·619
		10	1·0	0·367	0·367	–·005	·366	·627

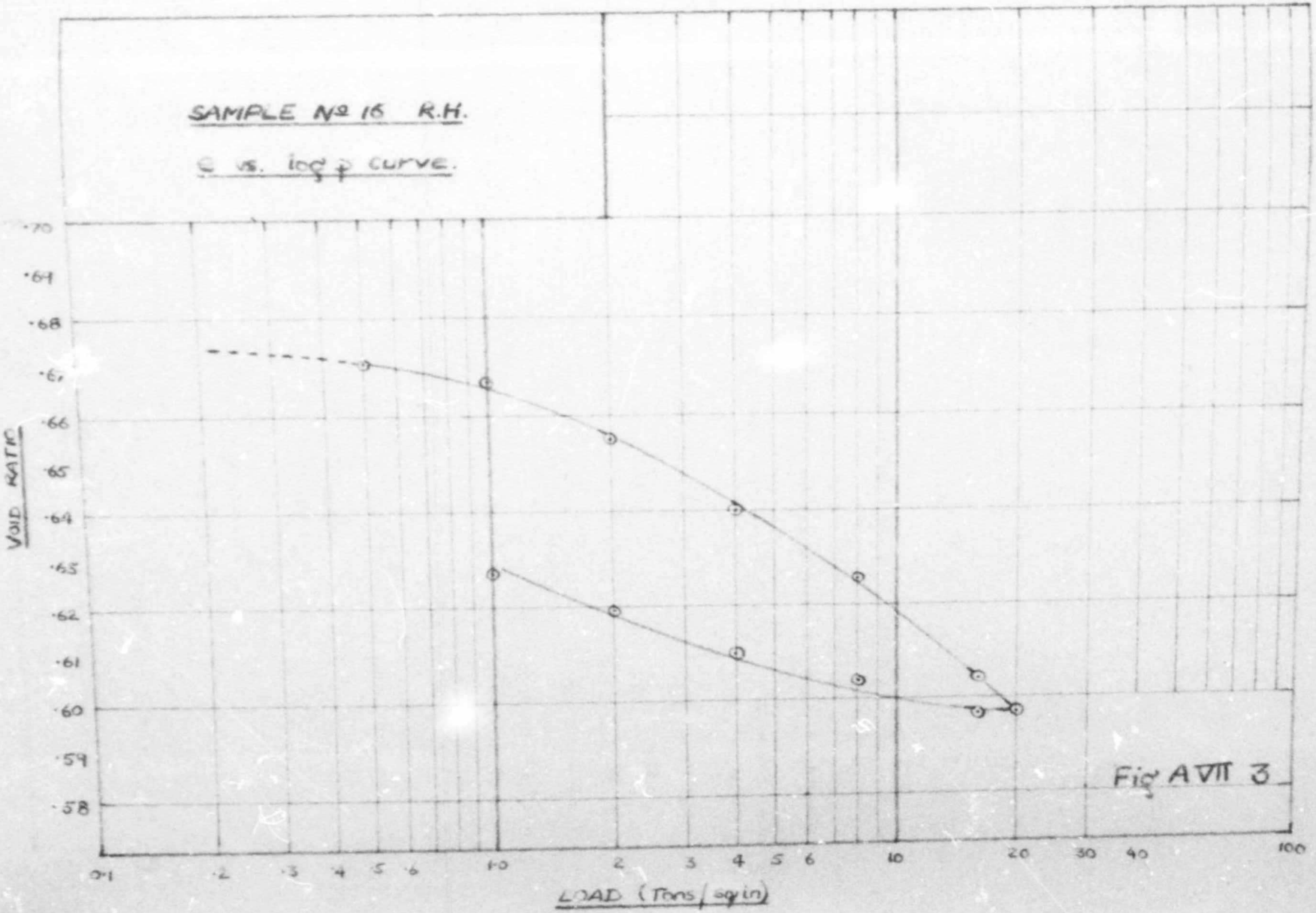

SAMPLE No 16 R.H.
e vs. log p curve.
VOID RATIO
·70
·69
·68
·67
·66
·65
·64
·63
·62
·61
·60
·59
·58
0·1
·2
·3
·4
·5
·6
1·0
2
3
4
5
6
10
20
30
40
100
LOAD (Tons/sq in)

Fig AVII 3

APPENDIX VII

THE DETERMINATION OF SETTLEMENT IN THE DEWATERED STRATUM.

SEE TABULATED RESULTS OF CONSOLIDOMETER TEST.

$\gamma_s = 174.5$ lb/cu.ft $W_s = 186.72$ gms
$\gamma_w = 62.4$ lb/cu.ft $V_s = 67.6$ cc's.
$\gamma_{sat} = 130.0$ lb/cu.ft.

CONSOLIDOMETER RING.

Dia. = 7.62 cm.
h = 2.51 cm (or) 0.988 ins.
CROSS SECTIONAL AREA = 45.7 cm² (or) 7.1 sq.in.

$$H_s = \frac{V_s}{A} = \frac{67.7}{45.7} = 1.48 \text{ cm (or) } \underline{0.583 \text{ ins}}$$

$$H_v = 0.988 - 0.583 = \underline{0.405 \text{ ins.}}$$

VOID RATIO $e = \dfrac{H_v}{H_s}$ (SEE TABULATED VALUES ABOVE).

FROM ISOCHRONES. (SEE Fig AVI 2 DEWATERING OF 400 ft STRATUM)

INITIAL POREWATER PRESSURE (AVE. OVER 400 ft STRATUM) $u_1 = 6.23$ tons/sq.ft.
FINAL POREWATER PRESSURE (" " " ") $u_2 = -1.44$ "

INITIAL TOTAL PRESSURE (" ") $\sigma_t = 13$ tons/sq.ft.
FINAL " " (" ") ditto.

INITIAL EFFECTIVE PRESSURE $p_1 = 6.77$ tons/sq.ft
FINAL " $p_2 = 14.44$ tons/sq.ft.

INCREASE IN EFFECTIVE PRESSURE $\Delta p = 7.67$ tons/sq.ft.

DRAINAGE PATH LENGTH $H = 400$ ft
FROM e vs log p CURVES, $e_1 = 0.628$
AND $C_c = 0.10$

$$\therefore \text{ TOTAL SETTLEMENT } \rho_u = \frac{2H_1}{1+e} \cdot \frac{p_2 - p_1}{\frac{1}{2}(p_2+p_1)} \cdot 0.435\, C_c$$

$$= \underline{15.45 \text{ ft}}$$

AFTER 34 years.

TIME FACTOR $T = 0.075$
% PERCENT CONSOL. $\%U = 30\%$ APPROX. READ OFF CURVE Fig 9

$\therefore$ SETTLEMENT OF 400 ft STRATUM AFTER 34 YEARS OF DEWATERING $= \frac{30}{100} \times 15.45 = \underline{4.6 \text{ ft.}}$

Fig A VII 4

CAVE CRACKS ON 5007 BENCH AS AT 21 FEB. 1961.

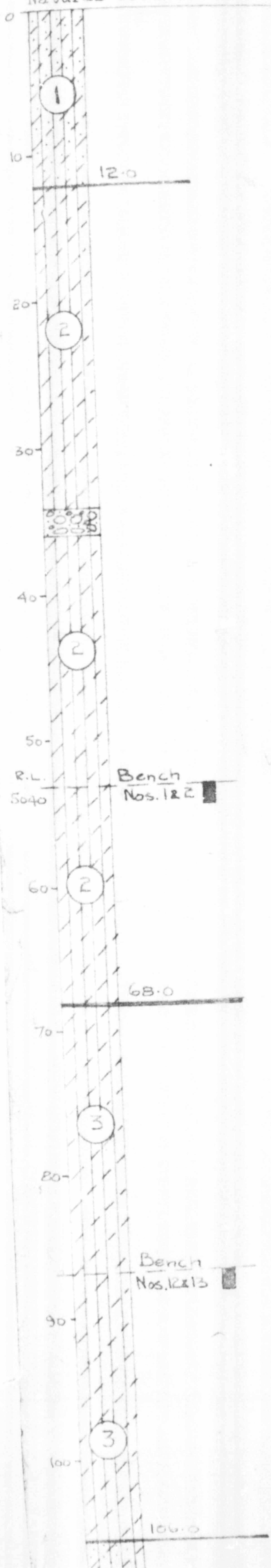
SCALE 1:250

Fig A III 2

INVESTIGATION OF SLOPE STABILITY - NCHANGA OPEN PIT

SOIL PROFILE ON NORTH FACE OF PIT - SECTION LINE NO. 3.

Natural Ground Level North Lip of Open Pit.



Red, soft to firm intact, clayey silt with small sand fraction; probably altered aeolean sand.

Yellow, firm, intact, clayey silt and sandy silt with some coarse sand; in profile mottled red yellow and white; evidence of boulders decomposed in situ. Gravel from 34 to 36 ft. $\frac{1}{8}$ in. to 6 in. variegated subangular and subrounded rocks, predominantly quartz; whole stratum was originally gravel with one more resistant layer.

Reddish grey, firm, jointed clayey silt; in profile mauve banded grey with many fissures and chert stringers which are much more profuse in upper 10 ft. Decomposed dolomitic schist.