Hvorslev Parameters

by

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Introduction

THE determination of fundamental parameters of shear strength of saturated cohesive soil was found to be a very illusive problem because of the large number of variables affecting the shear strength of a soil.

Hvorslev (1937) proposed the equation of shear strength as—

$$\tau_{ff}=C_e+\sigma'_{ff}\tan\phi'_e \qquad ...(1)$$

Where, C_e is a cohesion component independent of a normal stress acting on a plane of failure at failure and the stress history of the soil, but is dependent on the water content of the soil at failure and is in inverse proportion to it. $\sigma'_{ff}\tan\phi'_e$ is a friction component dependent on the magnitude of effective normal stress on the plane of failure at failure (σ_{ff}') and is independent of the stress history and water content of failure.

Hvorslev parameters were supposed to be of fundamental nature. An extensive work was done to determine them for different soils (Gibson 1953, Gilbert 1954, Plant 1955, Parry 1956, Rowe 1957, Schmertman and Osterberg 1960). On the basis of this study Hvorslev (1960) had finally proposed the expression of shear strength of soil as—

$$S=\tau_{\phi}+C_e+\tau_d \qquad ...(2)$$

Where, τ_{ϕ} is a friction component, τ_d is a surface energy component and C_e which consisted of two components C_v and C_u (transient and intransient components respectively, with reference to time) is a cohesion component.

Hvorslev (1960) further delineated the limitations of these parameters. Angle ϕ'_e and true cohesion C_e depend upon (*a*) orientation of clay particles with respect to failure surface and (*b*) time of failure. True cohesion, C_e, *may further depend upon (c) ions in the pore water or adsorbed on the* surface of clay particles and (*d*) temperature. He had, however, not given any experimental data, except for the effect of anisotropy, to give either quantitatively or qualitatively the manner in which these factors affect the parameters, true cohesion and the angle of true internal friction.

2. Present Study

The intention of the present study was to find at least qualitatively the direction in which various factors influence Hvorslev parameters. The effect of following factors on these parameters was studied :

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(1) Structure of soil which depends upon (*a*) initial moulding water content; (*b*) mode of consolidation and (*c*) orientation of failure plane with reference to the orientation of clay particles at the end of consolidation and at failure.

(2) Strain history.

(3) Nature of pore fluid, *i.e.*, difference in salt concentration of the pore fluid.

3. Time of Failure

SOILS USED FOR STUDY

A typical marine clay and a commercially available kaoline clay were taken for the study. The predominant clay mineral in Bombay marine clay was illite and kaolinite was the clay mineral in kaoline clay. The classification properties of these two clay soils are given in Table I.

TABLE I

	Bombay salt	Marine clay leached	Kaoline clay
Liquid limit %	115	109	47·5
Plastic limit %	45	46·2	32·4
Plasticity index	70	62·8	15·1
% clay ($<2\mu$)	48	48	54
Activity (PI/clay %)	1·46	1·31	0·28
Salt content (gm/litre)	26·0	0·0	nil
Specific gravity	2·70	—	2·69

4. Preparation of Samples with Different Structures

BOMBAY MARINE CLAY

Soil specimens with different fabrics (*i e.*, clay particle arrangements) were prepared by forming samples of different initial moulding water contents. One series of soil samples was prepared with initial water content equal to twice the liquidity index value and are called *slurry samples*. Another series of soil samples was prepared with a very high initial moulding water content (liquidity index was more than 14). These are called *flocculated samples*. These soil water mixtures were consolidated one-dimensionally so as to bring the water content to 80 percent. It was found that at this water content the clay was strong enough to enable a sample 3·81 cm in diameter and 7·62 cm in height to be handled satisfactorily.

KAOLINE CLAY

Only slurry samples of kaoline clay were used for this study.

5. Test Procedure

All tests (except a few which were tested by direct shear apparatus) were conducted in triaxial shear test apparatus.

TABLE II

Description of the preparation of soil samples and mode of their shearing : Bombay Marine Clay.

Test Series	Initial w.c.	Preparation of soil sample	Type of consolidation iao./K_o	N.C./O.C.	Type of shearing
1	80%	Remoulded	Isotropic	N.C. & O.C.	Controlled strain rate
2	166%	Slurry samples	K_o	,,	Controlled strain rate
3	166%	Slurry samples	K_o	,,	Controlled stress rate
3D	166%	Slurry samples	K_o	,, Drained	Controlled stress rate
4	166%	Slurry samples	K_o in direct	N.C. & O.C. Drained	Controlled strain rate
5	166%	Slurry samples	shear	,,	Controlled strain rate to determine residual strength
6	1000%	Flocculated samples	K_o	N.C. & O.C.	Controlled stress rate
7	1000%	Flocculated leached samples	K_o	,,	Controlled stress rate

Note :—Test series Nos. 1, 2, 3, 6 and 7 were conducted in triaxial shear apparatus under undrained condition with pore water pressure measurements.

Test series Nos. 4 and 5 were conducted in direct shear apparatus under drained condition.

Table II gives briefly the method of preparation of samples, type of consolidation, *i.e.*, isotropic or K_o, normal or overconsolidation, and the type of shearing procedure used.

CHANGING THE SALT CONCENTRATION IN TRIAXIAL SAMPLES

Those samples in which the salt content was to be changed after consolidation but prior to shear were set up with top drainage line, attached to the loading cap. After consolidation, water was forced under a small head (usually 2 to 3 m). Salt would then diffuse out of the sample. These soil samples which were consolidated from Sodium Chloride slurry and then had salt diffused out of the pores are called *leached samples.* Flocculated samples only were subjected to post-consolidation leaching.

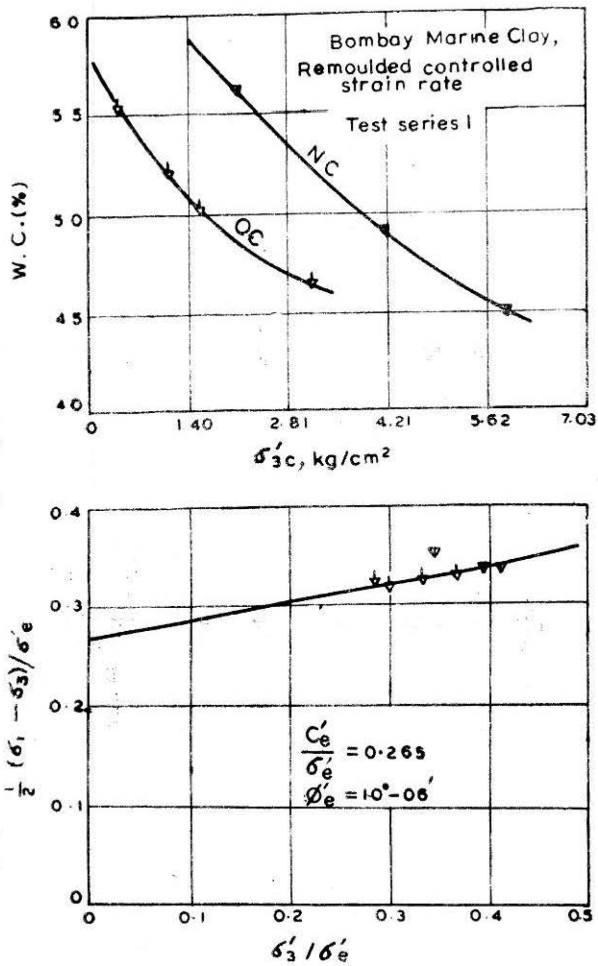


FIGURE 1 : Hvorslev parameters : Bombay Marine Clay—Remoulded—Controlled strain rate test series 1.

The diffusion leaching process was carried on for one to three months. The salt content of the quantity of water coming out of the soil sample was determined at different time intervals and when it was found to have decreased to the desired value (*i.e.*, 1·5 to 2 gm/litre), the leaching process was stopped. Prior to testing, the flow was stopped for one week to allow equalization of the salt concentration as well as that of the pore pressure throughout the sample. After shearing the average salt content of the soil sample was determined. This concentration was generally within 0·2 to 0·5 gm/litre more or less than that of the drained out fluid. The salt content of the fluid was determined by flame photometer (Gallenkamp).

Shear Test Procedure

(*a*) *Shear Test by Step-by-Step Increase of Equal Amount of Vertical Stress at Constant Time Interval.** After the soil specimen was consolidated, the drainage valve was closed and the pore water pressure measuring system was brought into operation. The N.G.I. null system (Wykeham Farrance make) was used to measure pore pressure during shear. A pore pressure response exceeding 98 percent in 5 minutes was considered satisfactory. As almost every sample was backpressured there was no difficulty in obtaining good response in a very short time.

A vertical stress 1/6th to 1/8th of the failure stress was added on the hanger. The vertical deformation of the sample under this stress was observed at different time intervals (6 sec, 30 sec ; 1, 2, 4, 8, 15, 30, 60 minutes, then 2, 4, 8 hours and afterwards every 24 hours interval) Pore water pressure was simultaneously measured. With the vertical deformation the cross-sectional area of the soil sample increases and the stress level no longer remains constant. In order to avoid this, lead shots were added as required. The increase in the cross-sectional area of the sample was computed assuming that volume remains constant (in case of undrained test) and that the increase in cross-sectional area is same throughout the length of the soil cylinder. As the strain at failure was very small, of the order of 4 to 6 percent, no great error was committed by these assumptions. The next load was added when either the vertical deformation has stopped or the specific time interval (generally 6 to 8 days) between successive stress applications was over ; whichever was smaller.

(*b*) *Controlled Strain Rate Tests.* Soil samples which were tested by *controlled strain rate method using Wykeham Farrance load frame* have strain rates as obtained from the calculations given in Bishop and Henkel (1962). The standard rate applied for normally consolidated as well as for overconsolidated samples was 0·0020 cm per minute.**

6. Test Results

Different values of C_e and ϕ'_e as obtained by various test methods and by different graphical procedures are compiled in Tables III and IV. Hvorslev parameters were determined by the method given by Noorany and Seed (1965) for soil semples consolidated under K_0-condition in triaxial apparatus.

*This test, here-to-after is termed as "controlled stress rate test".

**The rate af strain for normally consolidated and overconsolidated kaoline clay samples was 0·0040 cm/min.

TABLE III

Hvorslev Parameters as obtained by different test methods.

BOMBAY MARINE CLAY

S . No.	Test Series	Type of soil	Type of test	C_e gm/sq cm	
1.	1	Remoulded	$\overline{CIU}$, controlled strain rate	977·3	10°-06′
2.	2	*Slurry*	$\overline{CAU}$, *controlled* strain rate	597·6	16°-54′
3.	3	Slurry	$\overline{CAU}$, controlled stress rate	703·1	14°-12′
.	3 D	Slurry	CAD, controlled stress rate	632·8	15°- 0′
5.	6	Flocculated	$\overline{CAU}$, controlled stress rate	527·3	14°- 0′
6	3 & 6	Slurry and Flocculated	$\overline{CAU}$, controlled stress rate	815·6	7°-30′
7.	1 & 2	Remoulded and Slurry	$\overline{CIU}$ & $\overline{CAU}$ controlled strain rate	745·3	13°-36′
8.	1 & 6	Remoulded and Flocculated	$\overline{CIU}$ & $\overline{CAU}$ controlled strainrate and controlled stress rate respectively	745·3	8°-30′
9.	1 & 3	Remoulded and Slurry	$\overline{CIU}$ & $\overline{CAU}$ controlled strain rate and controlled stress rate respectively	511·1	15°-40′

7. Discussion

INFLUENCE OF STRUCTURE (AS DEVELOPED BECAUSE OF DIFFERENT INITIAL MOULDING WATER CONTENTS AND DIFFERENT MODES OF CONSOLIDATION) ON HVORSLEV PARAMETERS

(i) Bombay Marine Clay

Test series 1 and 2 (Figures 1 and 2) : Soil samples of test series 1 were remoulded.* Their initial moulding water content was 80 percent. They were isotropically consolidated and sheared by controlled strain rate method. Soil samples of series 2 were slurry samples. Initial moulding water content was 166 percent. They were consolidated under K_o-condition and sheared by controlled strain rate method. The cohesion

* Remoulded soil samples were prepared by packing a clay soil with a thumb pressure into a polished brass tube 3·81 cm diameter and 8·2 cm long against an acrylic disc supported by a wooden dolly, (Henkel, 1958).

component, at water content of 52·5 percent†, of the isotropically consolidated soil was found to be higher (977·3 gm/sq cm) than that K_o-consolidated soil (597·6 gm/sq cm). The magnitude of the true angle of internal friction was higher (16° – 54′) for K_o-consolidated soil in comparision to that of isotropically consolidated soil (10°). It, therefore, appears that the difference in the initial moulding water content and in the mode of consolidation have a marked influence on the magnitude of Hvorslev parameters.

(ii) Test Series 3 and 6 (Figures 3 and 4)

Soil samples of these series were consolidated anisotropically (K_o-condition) in triaxial cell and were sheared by controlled stress rate method. The initial moulding water content of soil samples of series 3 was 166 percent and that for series 6 was more than 1000 percent. The magnitude of true cohesion decreased from 703·1 to 527·3 gm/sq cm because of the increase in the initial moulding water content. And there was no perceptible change in the magnitude of the true angle of internal

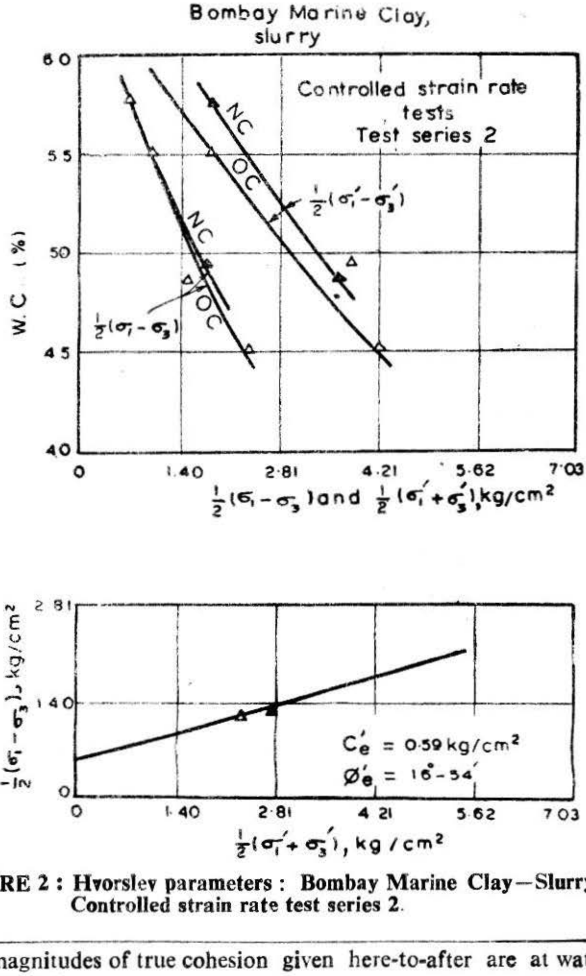


FIGURE 2 : Hvorslev parameters : Bombay Marine Clay—Slurry—Controlled strain rate test series 2.

† The magnitudes of true cohesion given here-to-after are at water content of 52.5 percent.

friction ($\phi'_e = 14°$). The difference in structure as developed because of different initial moulding water content, therefore, affects Hvorslev parameters. The difference in values of C_e and ϕ'_e of soil samples from these two test series is, however, not much. The probable reason may be that soil samples from both series were K_o-consolidated and time of failure was quite large (more than 20 days). Clay particles, therefore, may have become parallel to each other and got oriented parallel to the failure surface. The difference in structure at failure, therefore, may not be much.

TABLE IV

Hvorslev Parameters obtained by different test methods.

KAOLINE CLAY

Sl. No.	Test Series	Type of soil	Type of test	C_e gm/sq cm	
1.	1 A	Remoulded	$\overline{\text{CIU}}$, controlled strain rate	0·0	25°-36′
2	1	Slurry	$\overline{\text{CAU}}$, controlled strain rate	0·0	25°-0′
3.	2	Slurry	$\overline{\text{CAU}}$, controlled stress rate	70·31	21°-42′
4.	2 D	Slurry	CAD, controlled stress rate	105·5	24°-12′
5.	1 & 2	Remoulded and Slurry	$\overline{\text{CIU}}$, controlled strain rate and $\overline{\text{CAU}}$, controlled stress rate	224·9	18°-42′

Note :—Sl. No. 5—Bjerrum's method *C*.

(iii) Test Series 3, 3D and 6

It is expected that orientation of clay particles would be more parallel to the plane of failure at failure if soil samples were tested under drained instead of undrained condition. Soil samples of series 3*D* were slurry samples, anistropically consolidated and sheared by controlled stress rate method under drained condition. Values of C_e and ϕ'_e obtained from results of this test series are 632·8 gm/sq cm and 15° respectively. The corresponding values for soil samples of test series 3 are 703·1 gm/sq cm and 14° respectively. The difference in the magnitude of Hvorslev parameters for all these three test series is not much, probably because of the reason given above.

(iv) Test Series 3 and 6 (Figure 5)

Hvorslev parameters were determined by using test results of normally consolidated soil samples from series 3 and 6 by method ***C*** proposed by Bjerrum (1954 *b*)*. The magnitude of C_e and ϕ'_e obtained

* Drained or undrained shear tests with normally consolidated clay samples having different initial moulding water content.

were 815·6 gm/sq cm and 7°-30′ respectively. The magnitude of C_e is somewhat higher and that of ϕ'_e much smaller then those obtained from test results of series 3 and 6 by method *B*.

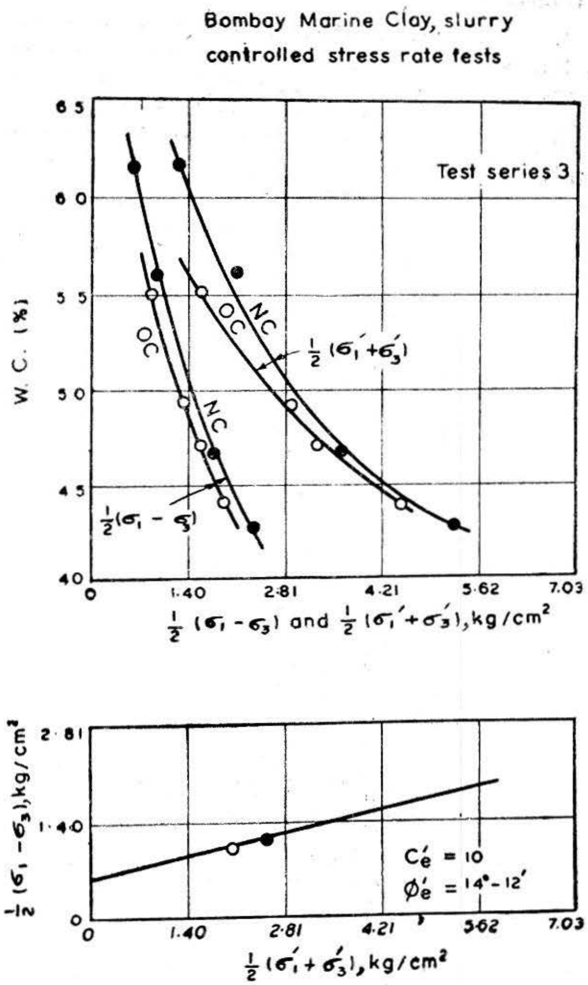


FIGURE 3 : Hvorslev parameters : Bombay Marine Clay—Slurry—Controlled stress rate test series 3.

(*v*) *Test Series 1 and 2, i.e., Remoulded and Slurry Samples both tested by Controlled Strain Rate Method* (*Figure 6*)

Values of C_e and ϕ'_e as obtained by method *C* were 745·3 gm/sq cm and 13°-36′ respectively. Soil samples of these two test series were not only of different initial moulding water contents but the method of consolidation was also different (isotropic for soil samples of series 1 and anisotropic for soil samples of series 2).

(vi) Test Series 1 and 3, (i.e., Remoulded and Slurry Samples tested by controlled Strain Rate and Controlled Stress Rate Methods Respectively) (Figure 5)

Soil samples of these two test series were of different initial moulding water content, were consolidated isotropically and anisotropically respectively and the time of failure was also different (10 to 12 hours in the former case and some days in latter case). Test results of normally consolidated soil samples of these two series were used to determine Hvorslev parameters (*i e.*, Method *C*). The magnitude of C_e and

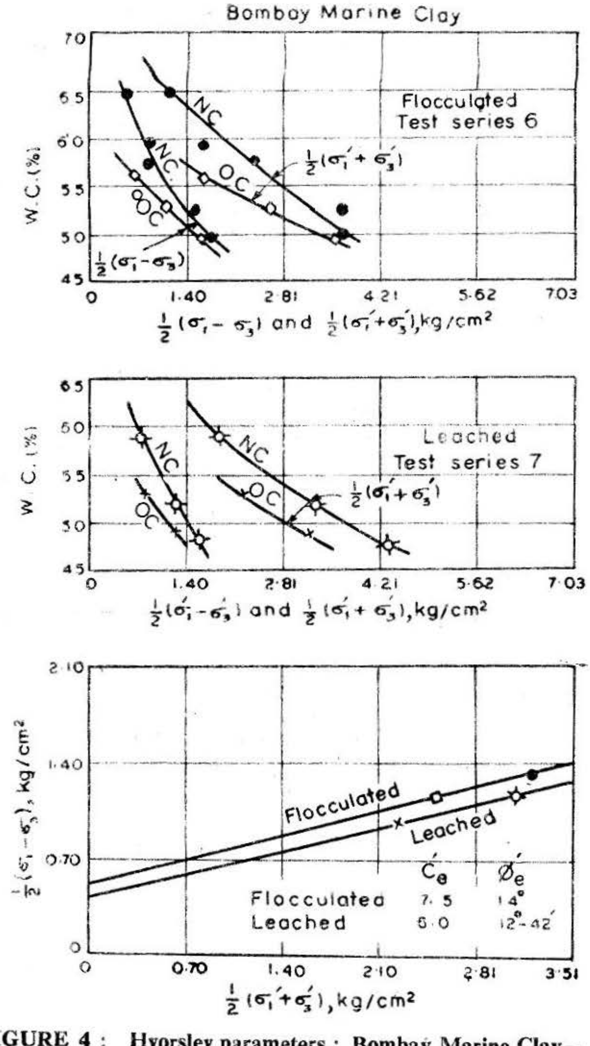
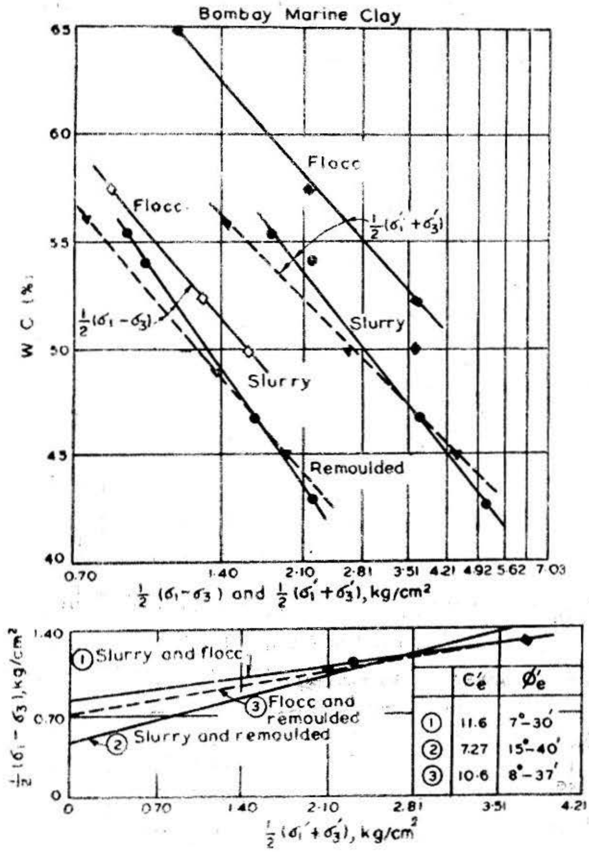


FIGURE 4 : Hvorslev parameters : Bombay Marine Clay—Flocculated test series 6.

ϕ'_e was 513·2 gm/sq cm and 15°-40′ respectively. The magnitudes of C_e and ϕ'_e for soil samples of test series 1 and 3 were 977·3 gm/sq cm and 703·1 gm/sq cm and 10° and 14° respectively (See Table III).

(vii) Test Series 1 and 6 (Remoulded and Flocculated Samples) (*Figure 5*)

Hvorslev parameters were obtained by method *C*. These soil samples have different initial moulding water content (80 percent and more than 1000 percent), different methods of sample preparation, different modes of consolidation (isotropic and anisotropic) and different periods of failure. Values of C_e and ϕ'_e obtained were 745·3 gm/sq cm and 8°-37′.



	C_e'	ϕ_e'
①	11.6	7°-30′
②	7.27	15°-40′
③	10.6	8°-37′

FIGURE 5 : Hvorslev parameters (Method C)—Bombay Marine Clay.

Hvorslev parameters as obtained by above test procedures and different graphical methods are compiled in Table IV. The magnitude of Hvorslev parameters in all nine cases would have been identical, if these parameters were not influenced by different structures of soil samples as developed because of different modes of consolidation and/or different initial moulding water contents.

Kaoline Clay

Hvorslev parameters as obtained by usual method *C* for slurry samples of kaoline clay are illustrated in Figure 7 and given in Table IV. These test results support the observations made for Bombay marine clay.

INFLUENCE OF ORIENTATION OF CLAY PARTICLES AT POST-CONSOLIDATION STAGE AND FAILURE WITH REFERENCE TO THAT OF PLANE OF FAILURE ON HVORSLEV PARAMETERS

Bombay Marine Clay

Test Series 4 and 2 (*Figure 8*) : Soil samples of test series 4 are slurry samples, were consolidated under K_o-condition in a direct shear box and sheared by controlled strain rate method. The normal stress applied is different in magnitude than horizontal stress at the end of consolidation stage. Clay particles were, therefore, oriented parallel to the major principal plane (Martin, 1966) which is horizontal. Clay particles were, therefore, more or less oriented at post-consolidation stage parallel to the plane of failure which is also horizontal.

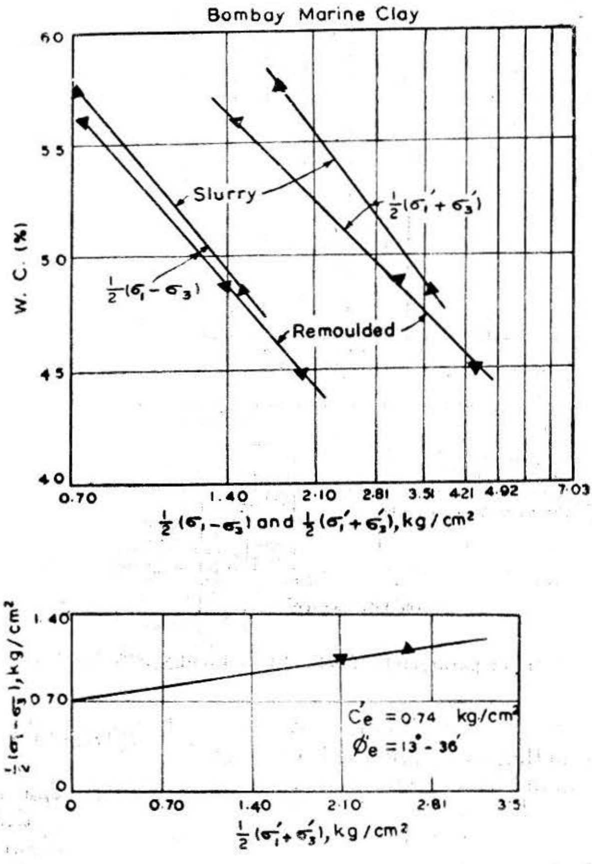


FIGURE 6 : Hvorslev parameters (Method C)—Bombay Marine Clay.

Soil samples of test series 2 are slurry samples, K_o-consolidated in a triaxial cell. Clay particles are, therefore, oriented parallel to the major principal plane, *i.e.*, horizontal plane. These specimens were sheared by controlled strain rate method. The direction of plane of failure was different than horizontal. In this case, orientation of clay particles at post-consolidation stage and that of failure plane was different. The test duration was small (few hours).

The magnitude of C_e and ϕ_e' for soil samples tested in direct shear box is 703·1 gm/sq cm and 13°-30′ respectively. The corresponding figure for soil samples of test series 2 are 597·6 gm/sq cm and 16°-54′ respectively. It, therefore, appears that the difference in the orientation o clay particles at post-consolidation stage from that of failure plane also influences Hvorslev parameters, provided sufficient time is not given for clay particles to orient parallel to the plane of failure during shear stage.

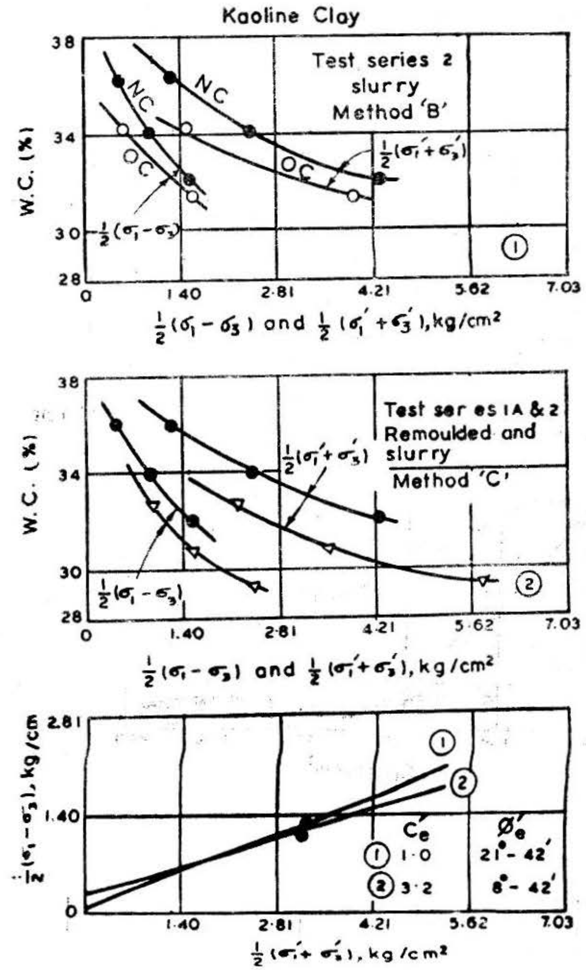


FIGURE 7 : Hvorslev parameters—Kaoline Clay.

Kaoline Clay

Test Series 2 and 4 (*Figure 9*) : Slurry samples of kaoline clay were tested in a direct shear box. The magnitude of C_e and ϕ_e' obtained were

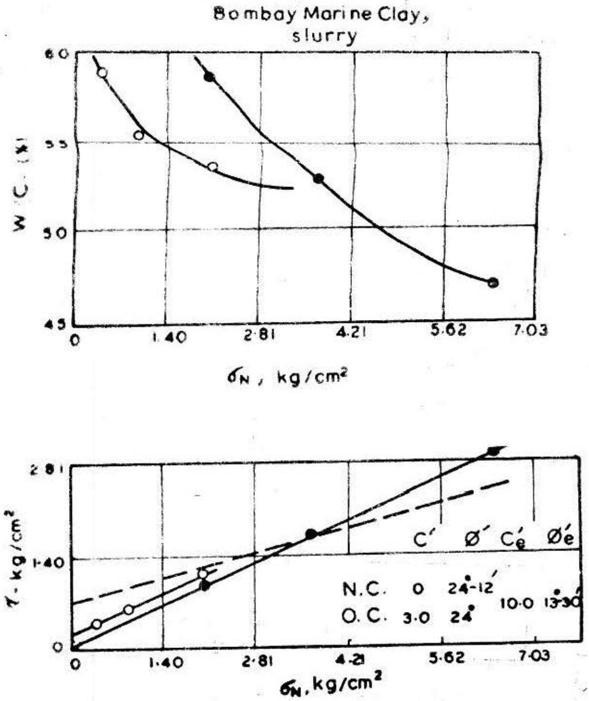


FIGURE 8 : Hvorslev parameters : Direct shear tests—Bombay Marine Clay—(Slurry).

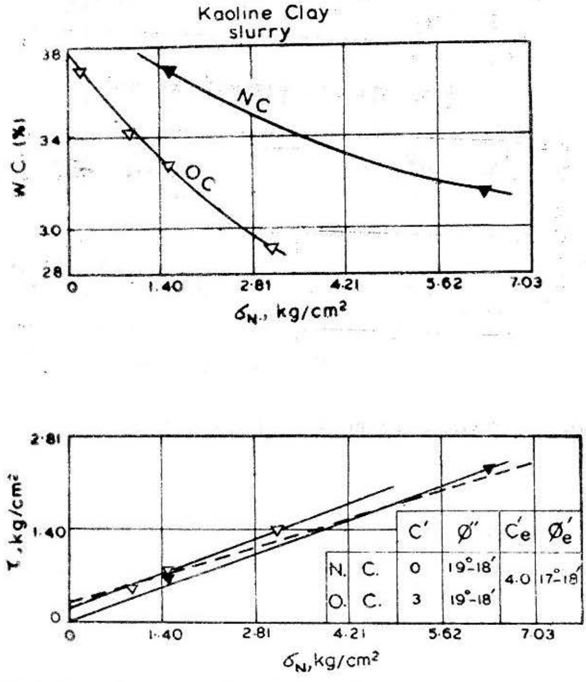


FIGURE 9 : Hvorslev parameters : Direct shear tests—Kaoline Clay (Slurry).

281·2 gm/sq cm and 17° respectively. The values of C_e and ϕ_e' for slurry samples, consolidated anisotropically and tested by controlled strain rate method in triaxial shear apparatus were 70·3 gm/sq cm and 21°-42′ respectively. These results support the above observation for Bombay marine clay.

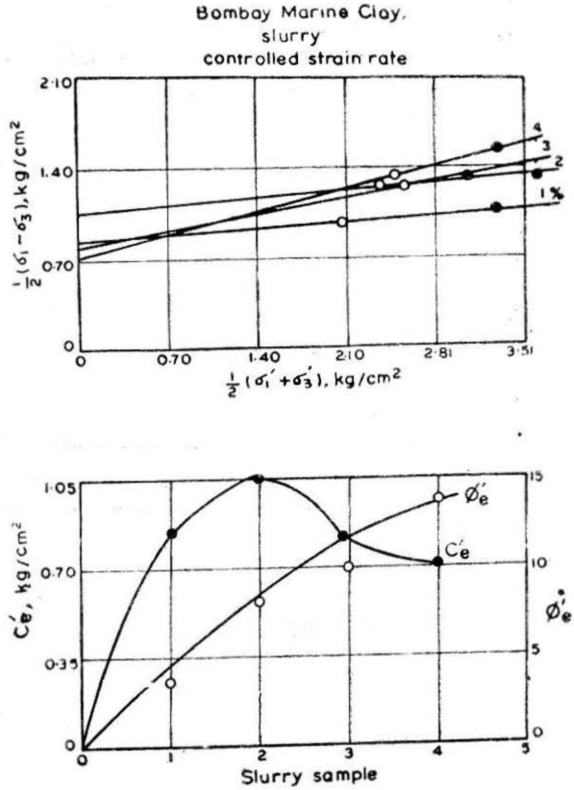


FIGURE 10 : Hvorslev parameters : Variation with strain—Bombay Marine Clay (Slurry).

EFFECT OF STRAIN HISTORY ON HVORSLEV PARAMETERS

Bombay Marine Clay

(*i*) *Test Series 2 and 3* : Soil samples of these test series are slurry samples and consolidated under K_o-condition. Soil samples of test series 2 were tested by controlled strain rate method and of series 3 were tested by controlled stress rate method. Variation of Hvorslev parameters with strain as obtained from test results of these two series is given in Figures 10 & 11.

For soil samples tested by controlled stress rate method, the magnitude of true cohesion was found to be decreasing with increase in strain from the first mobilization of shear stresses to a strain percent of 2. It remained constant at a value of 168·7 gm/sq cm with further increase in

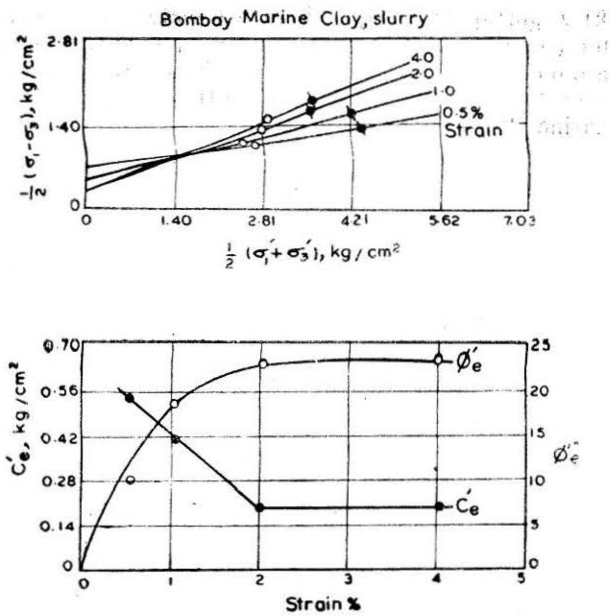


FIGURE 11 : Hvorslev parameters :Variation with strain—Bombay Marine Clay (Slurry).

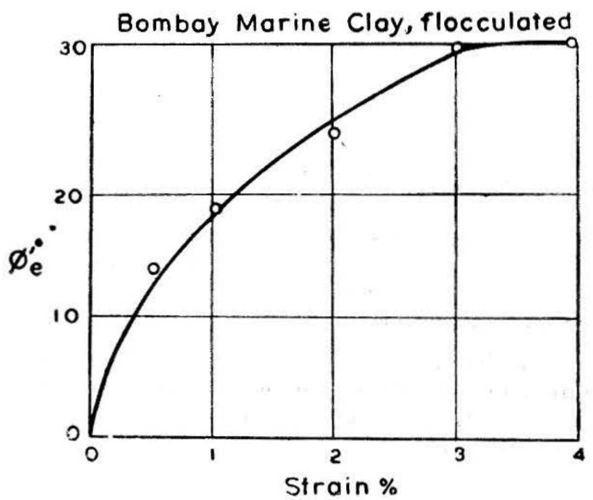


FIGURE 12 : Hvorslev parameters : Variation with strain—Bombay Marine Clay (Flocculated).

strain. For soil samples tested by controlled strain rate method, mobilised cohesion component was initially small and it increased with the increase in strain up to 2 percent. It then decreased gradually but remained as high as 703·1 gm¹ sq cm although the magnitude of strain was increased to 4 percent.

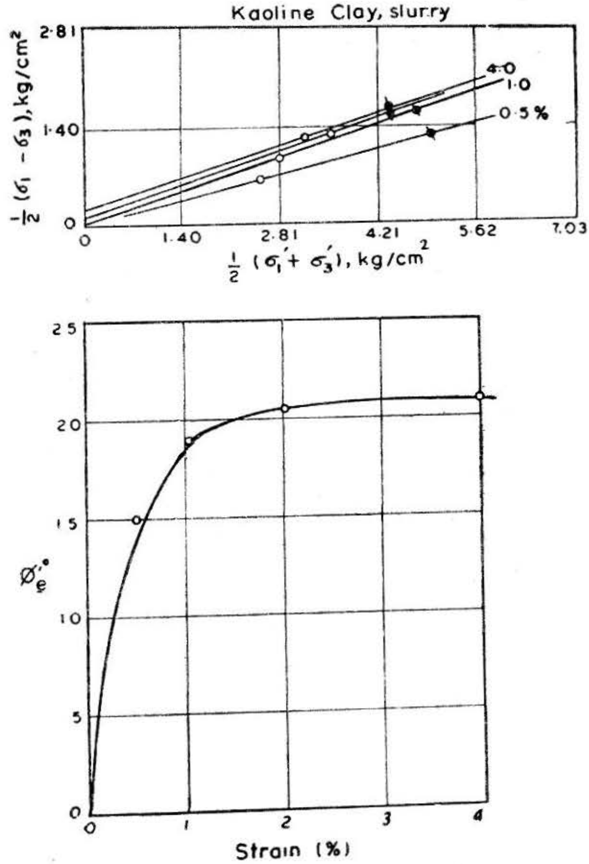


FIGURE 13 : Hvorslev parameters : Variation with strain—Kaoline Clay (Slurry).

The magnitude of true angle of internal friction for soil samples tested by controlled stress rate method increased with the increase in strain up to 2 percent and then remained constant with further increase in strain. The value of ϕ_e' increased continuously with the increase in strain up to 4 percent of strain for soil specimens tested by controlled strain rate method.

(*ii*) *Test Series 6* : These are flocculated soil samples, K_o-consolidated and tested by controlled stress rate method. The variation of true angle of internal friction with strain is shown in Figure 12. The cohesion component seems to have mobilized at a strain much smaller than 0·5 percent as its magnitude is negligible at this strain and for further deformation.

On the other hand, the magnitude of the true angle of internal friction increased with the increase in strain and it became constant after 3 percent strain.

It appears that the mobilization of cohesion and friction components with strain is different for soils of almost similar post-consolidation structure (series 2 and 3) but tested by different method, *i.e.*, controlled stress rate and controlled strain rate. Further, the mobilization of cohesion component to its full extent and its destruction takes place at a very small shear deformation if the soil is of flocculated structure at post-consolidation stage.

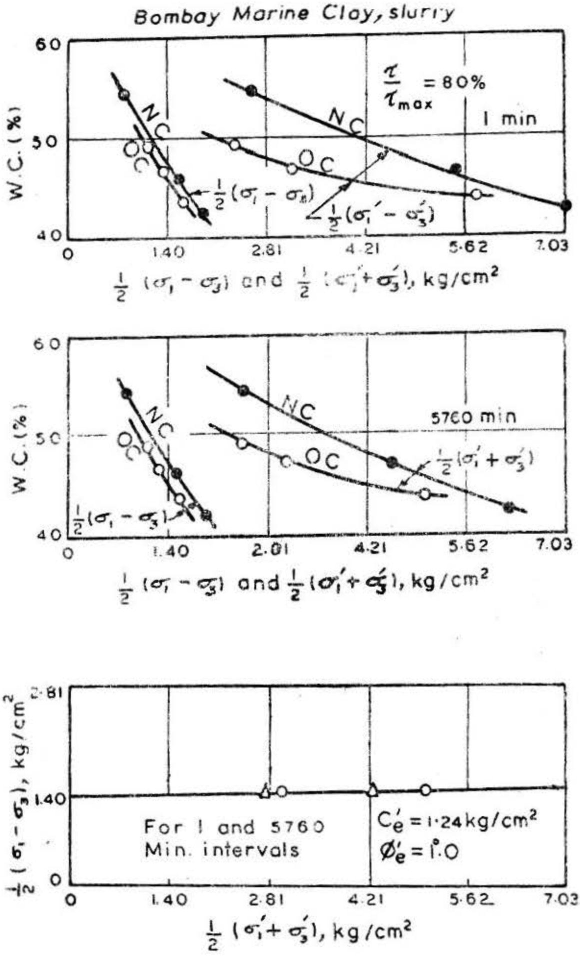


FIGURE 14 : Hvorslev parameters : Variation with time) τ/τ_{max}=80%) —Bombay Marine Clay (Slurry).

Kaoline Clay

Test Series 2 : Soil samples of these test series were slurry samples, K_o-consolidated and tested by controlled stress rate method. The variation of Hvorslev parameters with strain is given in Figure 13. Here again the magnitude of true angle of internal friction increased with the increase in strain and became almost constant after 2 percent strain. The magnitude of C_e is very small at all strains and hence its variation with strain is not shown in Figure 13.

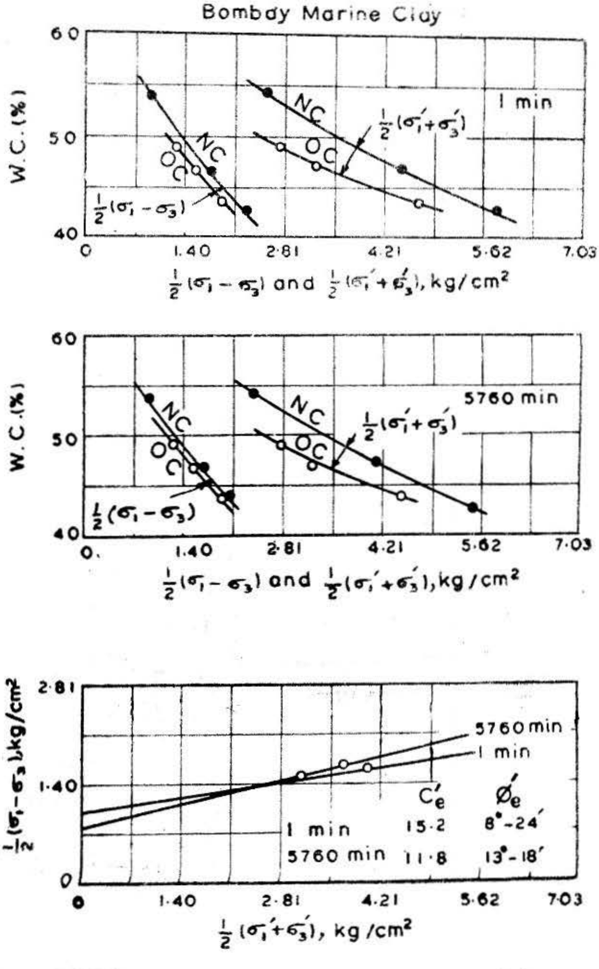


FIGURE 15 : Hvorslev parameters : Variation with time (τ/τ_{max}=95%) —Bombay Marine Clay.

INFLUENCE OF THE NATURE OF PORE FLUID (I.E., DIFFERENCE SALT CONCENTRATION OF PORE FLUID) ON HVORSLEV PARAMETERS

Test Series 6 and 7

Soil specimens of both test series were flocculated samples, consolidated under K_o-condition and tested by controlled stress rate method. The salt concentration of pore fluid of soil specimens of series 6 was 26 gm/litre. The salt concentration of pore fluid of soil samples of series 7 was decreased by water leaching, at post-consolidation stage, from 26 gm/litre to 1·5 to 2 gm/litre. The magnitude of Hvorslev parameters of soil samples of test series 6, which have high salt concentration, is 527·3 gm/sq cm and 14°; corresponding values for soil samples of test series 7. which have very low salt concentration, were 421·8 gm/sq cm and 12°-42′ respectively (Figure 4). These observations indicate that the magnitude of Hvorslev parameters decreased with the decrease in salt concentration.

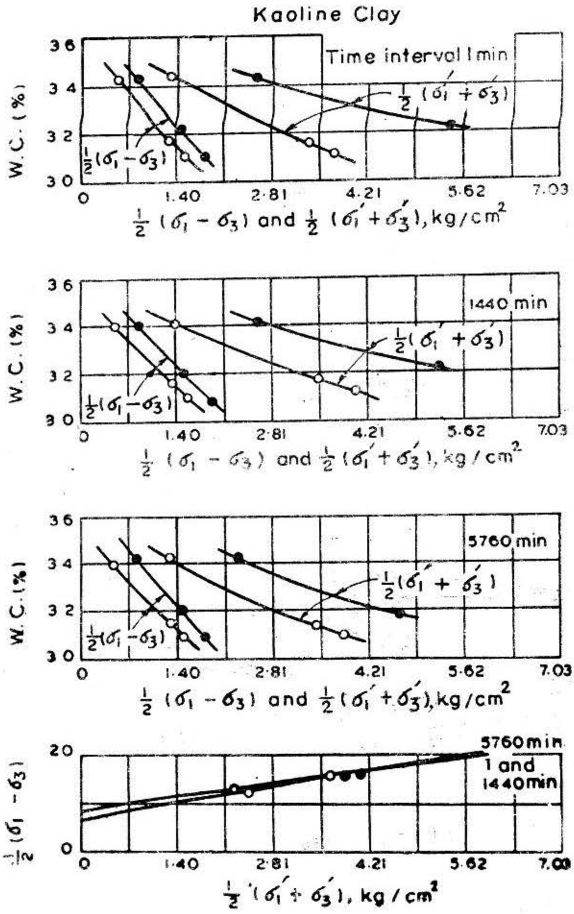


FIGURE 16 : Hvorslev parameters : Variation with time(τ/τ_{max}=85%)—Kaoline Clay.

Bjerrum and Rosenqvist (1956) attributed the reduction of shear strength of leached clays to the development of high pore pressure at failure and explained it on the basis that leached clays show an unstable structure which on application of shear stresses causes 'partial collapse' of the structure, resulting in high pore water pressure.

A comparsion of test results of leached and salt samples of Bombay marine clay show that the angle of shearing resistance (ϕ_f') of leached clays is smaller (18°) than that of salt clays (24°). The magnitude of A_f of leached sample was same or somewhat less than that of salt clays. It was further found that the sensitivity of soil, which indicates unstable structure, was not increased because of leaching. The value of sensitivity was almost unity after leaching of the sample. It may also be noted that reduction in activity of clay because of leaching was very small (1·46 to 1·31) for Bombay marine clay unlike that of Norwegian clays (0·6 to 0·2). It, therefore, appears that the decrease in shear strength of Bombay marine clay because of leaching was due to the decrease in the cohesion and the friction components rather than because of the development of unstable structure due to leaching.

INFLUENCE OF TIME OF FAILURE ON HVORSLEV PARAMETERS

Bombay Marine Clay

(*i*) *Test Series 3* : These are slurry samples, K_o-consolidated and tested by controlled stress rate method. The variation in Hvorslev parameters for different time intervals from 1 minute to 4 days, for which a sustained constant stress of a particular stress level (in comparison to the failure stress level) was acting, could be found by this method. Figure 14 gives, for a stress level of 80 percent (*i.e.*, $\tau/\tau_{max}=80$ percent), the variation in Hvorslev parameters for the time intervals of 1 minute and 5760 minutes (*i.e*, 4 days). It was found that the change in Hvorslev parameters for this time, interval when stress level is 80 percent was negligible. Figure 15 gives variation in Hvorslev parameters for the same time intervals but at a stress level of 95 percent, *i.e.*, when the stress level approaches failure stress. Here the magnitude of true cohesion decreased and that of the true angle of internal friction increased with the increase in time. Compiled information on variation of Hvorslev parameters because of these stress and time conditions is given in Table V. These observations indicated that the change in the shear resistance of soil with time, as represented by Hvorslev parameters, takes place when the applied stress level approaches failure stress level and that the magnitude of C_e was not zero but it has a finite value at failure condition although the constant stress was acting for a long time.

(*ii*) *Test Series 6* (*Table VI*) : These were flocculated samples, K_o-consolidated and tested by controlled stress rate method. Here again it was found that the variation in Hvorslev parameters with increase in time interval for which a sustained constant stress is acting, is negligible for stress level of 80 percent. On the other hand, at stress level of 95 percent, magnitude of the true cohesion decreased and that of ϕ_e' increased with the increase in time from 1 minute to 1.440 minutes. It may be noted that the magnitude of true cohesion at 1 minute interval is very small and is zero at 1,440 minutes interval for these flocculated samples unlike for slurry samples. These observations regarding the different mode of decrease in the magnitude of true cohesion, depending upon the post-consolidation structure of the soil are similar to those found by Wu et al (1962).

TABLE V

Influence of Time Interval on Hvorslev Parameters—
Bombay Marine Clay—Slurry samples.

	C_e (p.s.i.)					
Stress level	80%	90%	95%	80%	90%	95%
Time interval :						
1 minute	17·7		15·2	1°-0′	1°-0′	1°-0′
15 minutes	17·7	17·5	15·2	1°-12′	3°-18′	8°-24′
1,440 minutes	17·7	17·5	14·5	1°-12′	3°-24′	9°-06′
2,880 minutes	17·7	17·5	13·5	1°-18′	3°-24′	10°-0′
5,760 minutes	17·0		11·8	2°-06′		13°-18′

1 p. s. i.=70·31 gm/sq cm.

TABLE VI

Bombay Marine Clay—Flocculated Samples.

	Time interval	C_e p.s.i.	
80%	1 minute	2.0	16°-16″
	1,440 minutes	1.0	17°-28′
95%	1 minute	—	21°-42′
	1,440 minutes	—	23°-36′

1 p. s. i.=70·31 gm/sq cm.

TABLE VII

Kaoline Clay

	Time interval	C_e (32·5%)	
	1 minute	9·0	6°-54′
85%	1,440 minutes	8·0	8°-06′
	5,760 minutes	6·0	11°-32′

Kaoline Clay

Test Series 2 (*Table VII*) : Variation in Hvorslev parameters with time, from 1 minute to 4 days, for a stress level of 85 percent (τ/τ_{max}= 85 percent) is illustrated in Figure 16. The magnitude of true angle of internal friction increased like that of Bombay marine clay, with increase in time. The value of true cohesion also decreased with the increase in time.

8. The Direction of Influence of Different Factors on Hvorslev Parameters

The information about the direction of the change in the magnitude of Hvorslev parameters because of different factors, as discussed above, is compiled in Table VIII.

9. Summary

Above test results show conclusively that Hvorslev parameters are influenced by the structure of soil, orientation of clay particles at failure with reference to plane of failure, strain history, time of failure and the salt concentration of the pore fluid.

TABLE VIII

Influence of various Factors on Hvorslev Parameters.

Factors affecting Hvorslev Parameters	C_e	ϕ_e'
1. Mode of consolidation	Decreases if consolidated anisotropically	Increases if consolidated anisotropically
2. Initial moulding of water content	Increases with increase in initial moulding water content	Not influenced
3. Mutual orientation of failure plane and *that of clay particles*	Decreases if plane of failure and clay particles *are oriented parallel to* each other as in direct shear test	Decreases if plane of failure and clay parti- *cles are oriented para-* llel to each other as in direct shear test
4. Nature of pore fluid	Decreases with decrease in salt concentration	Decreases with decrease in salt concentration
5. Time of failure	Decreases with increase in time of failure	Increased for Bombay marine clay and for kaoline clay

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