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SHEAR STRENGTH BEHAVIOUR OF OVERCONSOLIDATED CLAYS

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ABSTRACT

It is generally understood that there exists a hysteresis loop between the normally consolidated and overconsolidated effective stress Mohr-Coulomb-failure envelopes, the size of hysteresis loop being a function of the past maximum consolidation pressure. Research findings of different investigators in this regard are at variance. This paper presents the results of an experimental investigation on two remoulded clays in $\overline{\mathrm{CIU}}$ compression tests, varying both maximum past consolidation pressure and the overconsolidation ratio. Test data is analysed in terms of Mohr-Coulomb as well as Hvorslev failure envelopes.

Test results show that the effective stress Mohr-Coulomb failure envelopes are uniquely defined independent of overconsolidation ratio, past maximum pressure and the failure condition such as the peak deviator stress, peak effective principal stress ratio and the peak porewater pressure. When the failure is denfined at a strain level lower than that of the peak stress condition, unique effective stress parameters (c' and ϕ') equal to the peak stress values are observed at pre-peak strains ($\epsilon_a \simeq 6$ percent) even bofore the peak stress conditions are reached ($\epsilon_f \simeq 15$ percent). From the view point of Hvorslev criterion, the shear strength of the two remoulded soils is essentially through ϕ_e' and it is independent of maximum past pressure.

Key words: clay, overconsolidation, rupture envelope, shear strength, soil mechanics, triaxial compression test
IGC: D 6

INTRODUCTION

Overconsolidated clays are very often encountered in Geotechnical Engineering practice. Many natural deposits of clay are heavily overconsolidated in their geological history due to glaciation. Other factors that may give rise to some degree of overconsolidation (slight to medium) in the soil are changes in ground water level, secondary or delayed consolidation, weathering, desiccation due to evaporation of moisture etc. Even certain construction procedures such as compaction in an earthdam, massive excavation for a foundation and the method of preloading in soft clays, all tend to introduce overconsolidation effects in the soil. Stability analysis of embankments, cuttings and foundations in these soils require an accurate assessment of strength response of such soil systems.

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LITERATURE REVIEW AND SCOPE

Investigations by several research workers (Henkel, 1956; Parry, 1960; Simons, 1960 a; Olson, 1962) have provided some understanding of the shear characteristics of overconsolidated clays. So far as the strength parameters are concerned, findings of different investigators are at variance. Henkel (1956) observed that the Mohr-Coulomb effective stress envelope for normally consolidated clays was a straight line passing through the origin but found difficult to assign a single value of c and ϕ to cover the wide range of overconsolidation in weald and London clays. However, Parry (1960) found practically no difference between the normally consolidated and overconsolidated envelopes for the same clays in remoulded state. Olson (1962) observed a slight hysteresis loop between the two envelopes for a sedimented calcium illite but they were identical for the remoulded material. However, for sodium illite, Olson (1963) found that the effective stress envelope of overconsolidated samples rebounded from 60 psi practically coincided with the normally consolidated envelope, but the samples rebounded from 120 psi apparently diverged from it. He also observed that the size of the hysteresis loop between the two envelopes was markedly affected by different failure criteria. Mesri and Olson (1970) also found the hysteresis loop between the two envelopes for a calcium montmorillonite. According to Olson (1974), both normally consolidated and overconsolidated samples yielded the same envelope. Few research workers (Simons, 1960 a; Simons, 1960 b; Lo, 1962) have reported that the overconsolidated envelope was displaced parallel to the normally consolidated envelope.

It can be seen from the literature that most of these findings are based on one maximum preconsolidation pressure. It is quite possible that the maximum preconsolidation pressure induced in the field varied somewhat from one location to another. This paper examines, in detail, the effect of previous stress history as influenced by the magnitude of past maximum pressure on the Mohr-Coulomb envelopes.

Further, most of the investigators have adopted the maximum deviator stress to define the failure of the specimen in a triaxial test. For many normally consolidated clays of low sensitivity, the point of maximum deviator stress coincides with that of the maximum effective principal stress ratio while for sensitive clays the maximum effective principal stress ratio occurs after the deviator stress reaches a maximum (Bjerrum and Simons, 1960). However, for heavily overconsolidated clays, the point of maximum principal effective stress ratio occurs before the deviator stress reaches a maximum value. In view of these considerations, an attempt has been made in this paper to study the effect of various failure conditions such as the peak deviator stress, peak effective principal stress ratio and the peak pore water pressure on the strength parameters.

Literature also reveals that all most all the investigators have analysed the test data of overconsolidated soils in terms of effective stresses, despite the fact that the total stress concept still has its merits for some soil engineering problems (Whitman, 1960). From the practical point of view, study of failure envelopes in terms of total stresses cannot be ignored. Further, many of the research workers have analysed the test data at the peak stress condition. Small strain behaviour, so far as the strength parameters are concerned has received very little attention. This paper thus presents the results of undrained triaxial tests investigating several of these factors in detail.

EXPERIMENTAL WORK

$\overline{\text{CIU}}$ compression tests were carried out on two commercially available clays (i) Kaolinite and (ii) Silty clay, the index properties of which are presented in Table 1.

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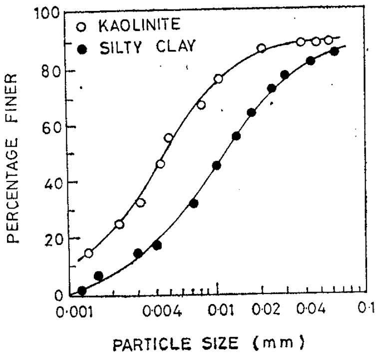

Fig. 1. Grain size distribution curves

Table 1.

Soils	W_L (%)	W_P (%)	I_P (%)	% finer than 0.002mm	Sp. Gr. (G)
Kaolinite	44	35	9	21	2.62
Silty clay	82	60	22	7	2.35

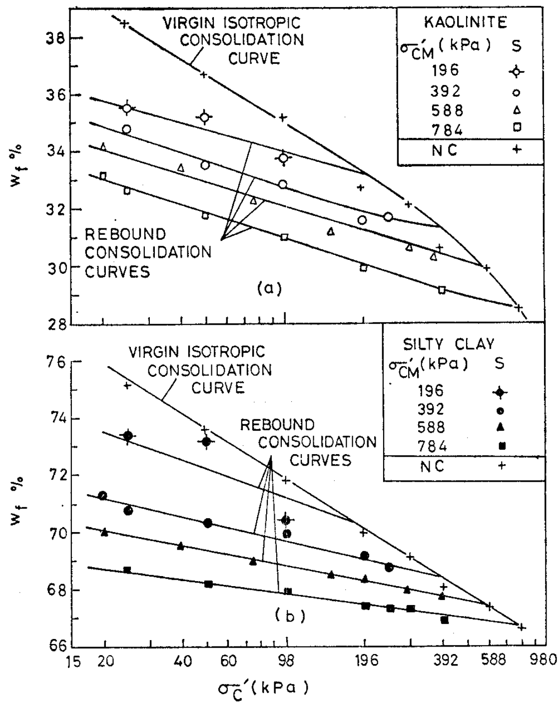

Fig. 2. Relationships between the water content and consolidation pressure

Fig. 1 shows their grain size distribution curves. Silty clay has a low specific gravity of 2.35. This was further verified by using carbon tetro-chloride (instead of distilled water) as the pore fluid in the pyknometer. Even then the specific gravity of the soil was found to be 2.35. X-ray diffraction indicated that it is a polymineral soil containing illite, quartz, chlorite etc.

Air dried clay, enough for 4 or 5 triaxial specimens was thoroughly mixed with distilled water to form a uniform mass. The initial water contents of the two clays i. e., kaolinite and silty clay were fixed at 40% and 80% respectively from the workability point of view after a number of trials. At these water contents, the triaxial specimens were not only fully saturated but also sufficiently stiff for handling. The method used for preparing a remoulded triaxial specimen closely followed the procedure outlined by Bishop and Henkel (1962).

Four series of tests, each series having the same maximum past consolidation pressure (σ_{cm}) with overconsolidation ratio varying from 1 to 32 were conducted for each soil. Maximum past consolidation pressures of 196, 392, 588 and 784 kPa (2, 4, 6 and 8 kgf/cm^2) were used. The specimens were allowed to consolidate under a maximum past consolidation pressure for two days during which period the primary consolidation was completed and later they were rebounded to lower cell pressure to obtain the required overconsolidation ratio. Back pressure technique with a back pressure of 196 kPa (2 kgf/cm^2) was used to ensure full saturation of the specimen and also to facilitate measurement of negative pore water pressure during shear. Fig. 2 shows the relationships between the water content and the consolidation pressure for all the series.

Triaxial tests were conducted on specimens 7.6 cm in length and 3.8 cm in diameter. In all the tests, the cell pressure was held constant during shear and the axial load was applied under controlled rate of strain. A rate of strain of 0.0475 percent per minute which was sufficiently slow for 95 percent equalisation of pore water pressure, was adopted

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for all tests (Blight, 1963)

TEST RESULTS AND ANALYSES

Stress-Strain-Pore Pressure Relationships

Typical stress-strain-pore pressure relationships for the specimens having a past maximum pressure equal to 588 kPa are shown in Figs. 3, and 4 for Kaolinite and silty clay. While highest positive change in pore pressure occurred in normally consolidated state (OCR=1), the peak pore pressure decreased and became even negative as the OCR increased. Same trend is observed in all other series having different maximum past pressures (Krishna Murthy, 1979).

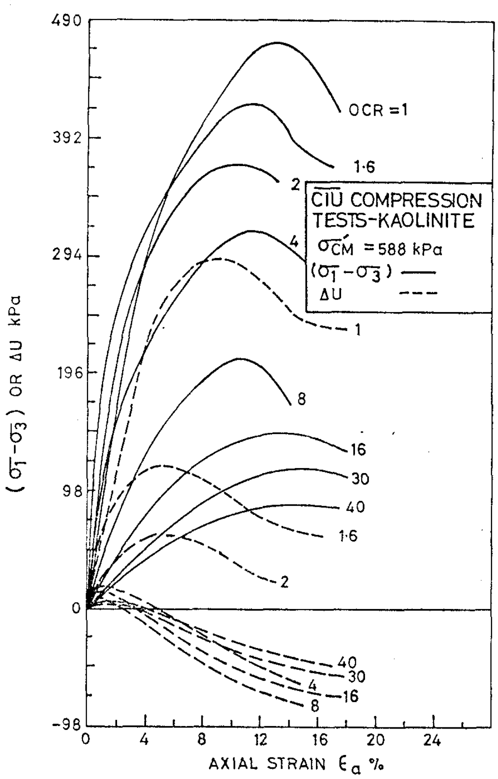

Fig. 3. Stress-strain-pore pressure relationships for Kaolinite (σ_{CM}'=588 kPa)

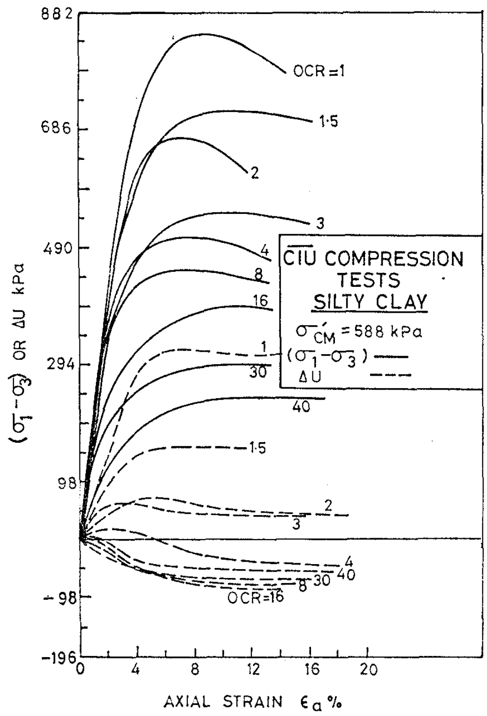

Fig. 4. Stress-strain-pore pressure relationships for silty clay (σ_{CM}'=588 kPa)

Effective Stress Paths

In Fig. 5 are presented the typical effective stress paths on a modified Mohr-Coulomb diagram for the specimens consolidated to a maximum past pressure equal to 588 kPa. For the purpose of clarity, stress paths are shown only for the specimens having OCR equal to 1, 2 and 8 which represent the normally consolidated, lightly overconsolidated and heavily overconsolidated states respectively. Eventhough there is no geometric similarity in all the three effective stress paths (obviously due to different stress histories of the specimens) the strength envelopes on effective stress basis are apparently independent of the stress path followed during the test. The figure also indicates clearly the points of maximum deviator stress, maximum effective principal stress ratio and the maximum pore water pressure for each test. This aspect will be further discussed in relation to effective stress envelopes.

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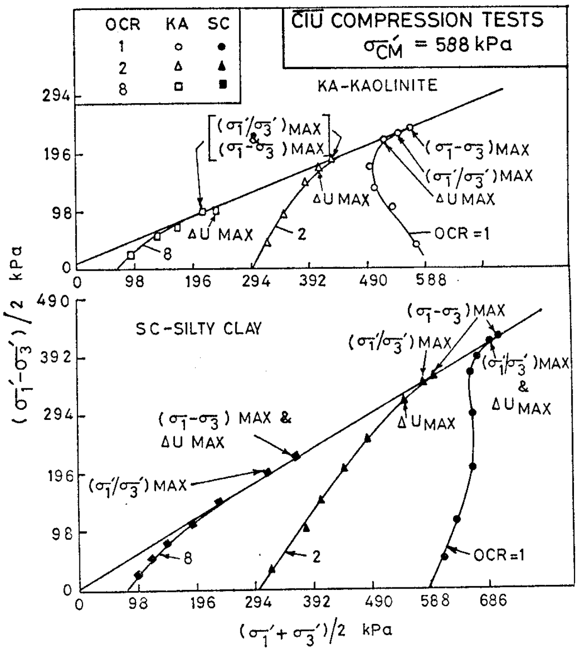

Fig. 5. Effective stress paths (σ_{CM}'=588 kPa)

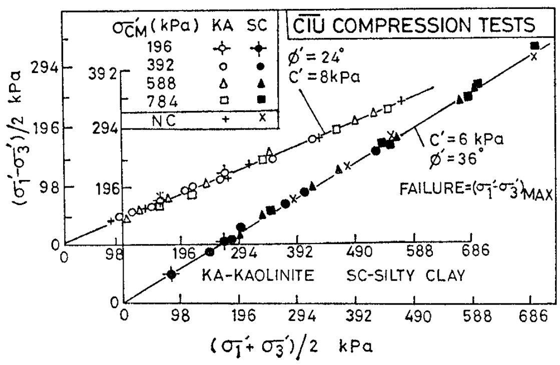

Fig. 6. Modified Mohr-Coulomb effective stress envelopes-failure at $(\sigma_1'-\sigma_3')_{\max}$

Strength Envelopes

Effect of failure condition

Figs. 6 to 8 show the plots of modified Mohr-Coulomb envelopes in terms of effective stresses for the three failure conditions which define the failure of a triaxial sample, namely, the maximum deviator stress, maximum effective principal stress ratio and the maximum pore water pressure. Referring to Fig. 6, all the test points fall on one common line which is apparently independent of the past maximum pressure and the overconsolidation ratio to which the samples were subjected before shear. Normally consolidated envelope is virtually identical with the envelope obtained for the samples overconsolidated from different past maximum pressures. In other terms, different overconsolidated envelopes (one for each past maximum pressure) have merged into a single envelope which is further identified with the normally consolidated envelope. So far as remoulded clays are concerned there is some evidence that the normally consolidated envelope and the envelope for the overconsolidated specimens rebounded from one and the same maximum pressure lie close to each other (Parry, 1960; Olson, 1962 and 1974). But there is no evidence that the normally consolidated envelope is identical with many rebounded envelopes. However, for some naturally sedimented clays, it was reported that the overconsolidated envelope, being displaced parallel to the normally consolidated envelope, exhibited slight cohesion without any change in the angle of shearing resistance (Simons, 1960 a, 1960 b; Lo, 1962). The slight cohesion intercept could be attributed to the development of interparticle bonds over a long period due to several environmental factors.

Referring to Figs. 7 and 8, the modified Mohr-Coulomb envelopes plotted for the peak effective principal stress ratio and the peak pore water pressure are also independent of the past maximum pressure and the overconsolidation ratio. In order to further illustrate this, the points of maximum deviator stress, maximum effective principal stress ratio and the maximum pore water pressure are plotted on a single modified Mohr-Coulomb diagram (see Fig. 5) for the three specimens having different stress histories (OCR=1, 2 and 8). All the three points representing different failure stress conditions either lie on the strength envelope or very close to it regardless of the overconsolidation ratio. Table 2 provides a comparison of the strength parameters for different failure conditions. Values of c' and ϕ' are practically same for all the three stress conditions representing

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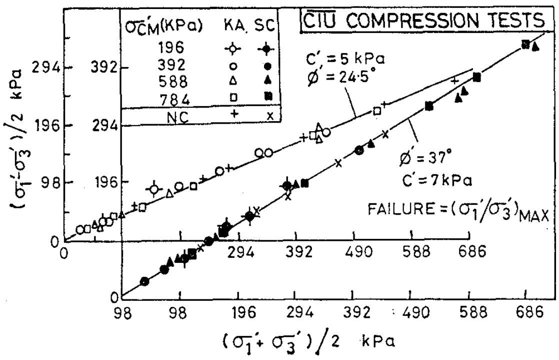

Fig. 7. Modified Mohr-Coulomb effective stress envelopes-failure at$(\sigma_1'/\sigma_3')_{max}$

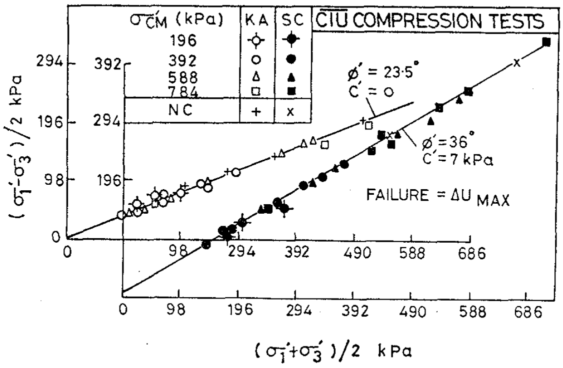

Fig. 8. Modified Mohr-Coulomb effective stress envelopes-failure at ΔU_{max}

Table 2.

Soil	Kaolinite		Silty Clay	
Failure condition	c' kPa	ϕ'	c' kPa	ϕ'
$(\sigma_1-\sigma_3)_{max}$	8	24°	6	36°
$(\sigma_1'/\sigma_3')_{max}$	5	24.5°	8	37°
ΔU_{max}	0	23.5°	8	36°

failure. From an engineering point of view, the slight differences, if any, in effective stress parameters arising from different failure conditions is not only insignificant but also indistinguishable in normal routine laboratory testing.

Simons (1960 b) attributed the small difference in effective stress parameters to the difference in strains at which the deviator stress and the effective principal stress ratio attained their maximum values. The greater the axial strain at which the deviator stress attained a maximum in the undrained tests, the smaller was the difference in the value of ϕ' obtained by the two criteria. Another reason that seems to influence the difference in the angle ϕ' obtained by the two criteria is the sensitivity of the natural sediments. Greater the sensitivity of the clay, greater is the difference between the values of ϕ' determined according to the two failure conditions (Bjerrum and Simons, 1960). However, as observed for some naturally sedimented clays, these differences between the failure envelopes for different failure conditions is fairly small (Simons, 1960 a and 1960 b).

For remoulded clays in this investigation, variation in the effective stress parameters is negligible since the strength parameters are measured at relatively high strains at which the two criteria are in close agreement. Another reason that could be attributed to the negligible effect of the failure condition is the low sensitivity of the remoulded clays. Therefore, so far as the clays of low sensitivity are concerned an important conclusion emerges from this. Inspite of highly stress history dependent stress-strain-porewater pressure behaviour of these soils (Figs. 3 and 4), the strength parameters on effective stress basis are found to be independent of failure condition and stress history (OCR and σ_{CM}) thus revealing the uniqueness of the conventional effective stress concept.

With the strength defined at maximum deviator stress, Fig. 9 shows the modified Mohr-Coulomb envelopes plotted with respect to the total stresses. The relationships are several and nonlinear, because the samples in each of the envelopes have different preconsolidation stress levels and stress histories (OCR). Moreover, the total stress representation if shear stress does not account for the changes in effective normal stress that occur during undrained shear in samples of different preconsolidation stress histories.

Linear strength envelopes in terms of total stress can be obtained, if only the stress history (OCR) of the samples in each of the envelopes is kept the same. Fig. 10

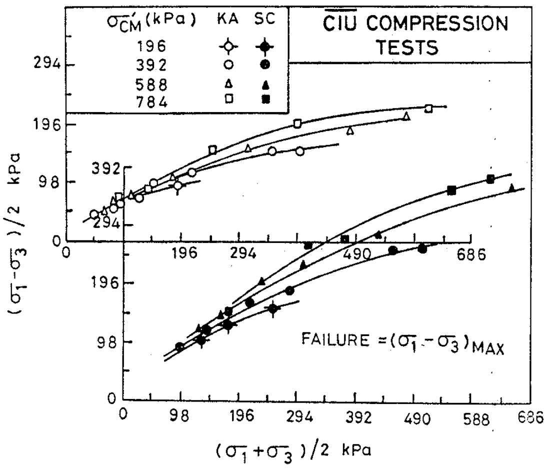

Fig. 9. Modified Mohr-Coulomb total stress envelopes

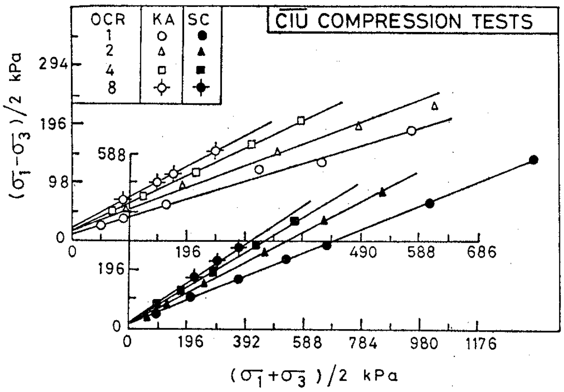

Fig. 10. Modified Mohr-Coulomb total stress envelopes

presents the test data of the Fig. 9 in such a form. Strength envelopes are plotted so that in each of the envelopes, the specimens have the same overconsolidation ratio. The results indicate that the envelopes are linear and distinct, with the inclination depending on the overconsolidation ratio. Obviously, the angle of shearing resistance increases with OCR. From normally consolidated condition to OCR equal to 8, there is approximately a two fold increase in the total stress angle of shearing resistance. As the overconsolidation ratio increases, the change in effective normal stress also increases from its pre-shear condition because of the induced negative pore water pressures during undrained shear. While the observed shear stresses are plotted, the consequential changes in the effective normal stress during undrained shear have not been accounted for, thus resulting in several Mohr-Coulomb envelopes, one for each OCR. Therefore, the test results presented in Figs. 6 to 10 show that while the strength envelopes on effective stress basis are uniquely defined, independent of stress history (OCR and σ_{CM}') and the failure condition, the total stress envelopes presented altogether a different picture resulting in several sets of envelopes depending on the manner in which they are plotted. So, it needs to be hardly emphasised that one has to be very cautious while choosing the total stress parameters. From the practical point of view, it is important to realise that the total stress parameters are highly dependent on OCR, while the effective stress parameters are not. The research findings which consider the failure envelopes (total stress) as a function of OCR and preconsolidation pressure are new in the field of shear strength parameter analysis. The test results could not be compared with those of previous research workers (Henkel, 1956; Olson, 1962; Simons, 1960 a) since they did not consider the total stress envelopes in their analysis. Furthermore, they did not have more than two specimens with the same OCR or preconsolidation pressure. Only Henkel (1956) used three maximum past pressures, 120, 60, 30 psi (206, 414, 827 kPa) to generate three series of overconsolidated specimens. However, his analysis was limited to the series with $\sigma_{CM}'=120$ psi (827 kPa).

Effect of Strain Level

In many practical problems, it is essential to limit the strains to much lower level than that would occur at the peak stress condition. So, it is of some practical significance to know how the strength parameters vary with failure defined at a strain level much lower than that would occur at the peak stress condition. Figs. 11 and 12 show the

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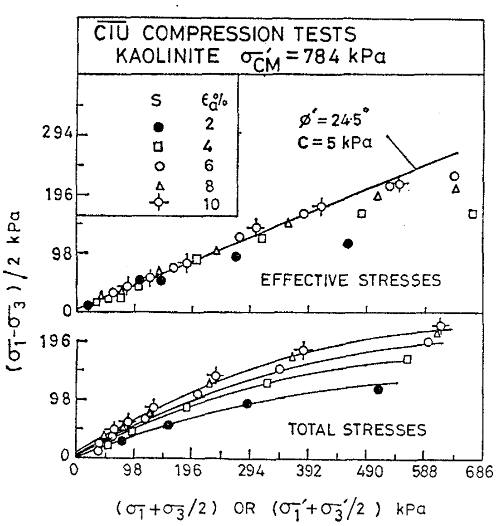

Fig. 11. Effect of strain level on modified Mohr-Coulomb strength envelopes-Kaolinite

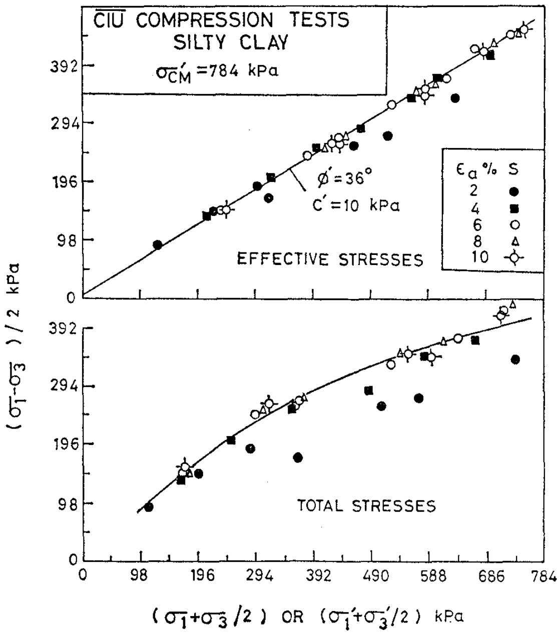

Fig. 12. Effect of strain level on modified Mohr-Coulomb strength envelopes-silty clay

modified Mohr-Coulomb diagrams plotted for the stress conditions at prepeak axial strains of 2 percent, 4 percent, 6 percent, 8 percent and 10 percent for the samples rebounded from the same maximum past pressure.

For both the remoulded clays, inclinations of effective stress envelopes gradually increase upto or about 4 percent strain and then became stable with further strain. Effective stress envelopes plotted at 6 percent strain or more, define unique strength parameters, equal to the peak stress values, suggesting that the effective peak parameters could be mobilised much before the peak stress conditions are reached. After the peak angle of shearing resistance was mobilised at or about 6 percent axial strain, the changes in pore pressure in the sample were such that both the strength and the effective normal stress increased but the angle of shearing resistance remained unaltered. In contrast, the relationships plotted on total stress basis are several and nonlinear (one for each strain level). However, for silty clay, the test points defining total stress conditions at or after 6 percent strain lie close to a single nonlinear envelope. Similar behaviour has been observed also for other past maximum pressures (Krishna Murthy, 1979).

Hvorslev Envelopes

Hvorslev (1937) proposed a strength criterion which is independent of the previous stress history (i. e. normally consolidated or overconsolidated) of the soil but only valid for a specific water content at failure, as follows:

$$\tau_f = C_e' + \sigma_f' \tan\phi_e' \qquad (1)$$

C_e' is considered primarily a function of water content as follows:

$$C_e' = k\sigma_e' \qquad (2)$$

in which σ_e' is the equivalent consolidation pressure and k is the coefficient of cohesion being constant for a given soil.

The Coulomb-Hvorslev failure criterion expressed in Eq. (1) may be rewritten in terms of principal stresses to match the triaxial shear conditions as follows:

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$$\frac{(\sigma_1-\sigma_3)}{2}=C_e'\cos\phi_e+\frac{(\sigma_1'+\sigma_3')}{2}\sin\phi_e' \tag{3}$$

By substituting Eq. (2) in Eq. (3) and rearranging,

$$\frac{(\sigma_1-\sigma_3)}{2\sigma_e'}=k\cos\phi_e'+\frac{(\sigma_1'+\sigma_3')}{2\sigma_e'}\sin\phi_e' \tag{4}$$

By plotting $\frac{(\sigma_1-\sigma_3)}{2\sigma_e'}$ against $\frac{(\sigma_1'+\sigma_3')}{2\sigma_e'}$ and drawing the best fitting straight line through the points, the Hvorslev failure envelope is obtained. The slope and the intercept of the envelope are $\sin\phi_e'$ and $K\cos\phi_e$. It is more convenient to plot the shear data in this modified Coulomb-Hvorslev diagram than the one suggested by Bishop and Henkel (1962). Olson (1963) also used this diagram to analyse the Hvorslev parameters.

Fig. 13 shows the modified Coulomb-Hvorslev envelopes for the two remoulded clays. Like Mohr-Coulomb failure envelopes, Hvorslev envelopes are also uniquely defined independent of OCR and maximum past pressure. Since the Hvorslev Cohesion is either negligible or nil, the shearing strength of the two remoulded soils is dominantly a friction phenomenon. Another interesting feature is that the Hvorslev envelopes have approximately the same slopes as those of Mohr-Coulomb envelopes. Thus, the two remoulded soils exhibit uniqueness not only with respect to the stress history (σ_{CM}' and OCR) but also with the strength criterion. Such unique Hvorslev envelopes can be expected due to the abscence of any hysteresis loops between the normally consolidated and overconsolidated Mohr-Coulomb envelopes. One of the drawbacks of the Hvorslev treatment of shear strength is that it does not include the effect of soil fabric, although it incorporates the all important void ratio effect. In this investigation, since all the specimens have been remoulded to the same water content, it is reasonable to assume that the initial fabric of the specimens may be same. Fabric changes during rebound from different maximum consolidation pressures, might have been very negligible thus resulting in unique values of ϕ_e' and C_e'/σ_e'. In literature, apart from Olson (1963), there appears to be no other reference in which the past maximum pressure was a variable in the analysis of Hvorslev parameters. Olson obtained unique Hvorslev envelope for sodium illite only when the failure was defined at the tangent point between the stress path curve and the failure envelope. Simons (1960 a) did report for an undisturbed Oslo clay but he used only one past maximum pressure for analysing Hvorslev parameters. Bjerrum and Simons (1960) showed that the effective angle of internal friction, ϕ_e', corresponding to a given water content was greater for the undisturbed state than for the remoulded state, the difference increasing with increasing plasticity index. This appears to be quite reasonable and may be attributed to the difference in fabric.

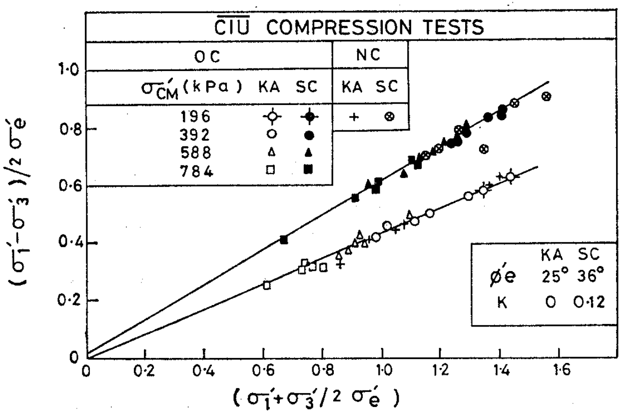

Fig. 13. Effect of past maximum consolidation pressure on Hvorslev failure envelopes

CONCLUSIONS

Stress-strain pore pressure behaviour is greatly influenced by the stress history (OCR and σ_{cm}') during undrained shear.

Mohr-Coulomb strength envelopes on effective stress basis are uniquely defined, independent of OCR and the past maximum pressure, while no such uniquenss is found with respect to total stress. Effect of failure condition, namely peak deviator stress, peak effective principal stress ratio and peak pore water pressure is either negligible or nil for the remoulded clays used.

Unique effective stress parameters (c' and ϕ') equal to the peak stress values are observed at pre-peak strains ($\epsilon_a \simeq 6$ percent) even before the failure stress conditions are reached ($\epsilon_f \simeq 15$ percent). In contrast, the relationships plotted on total stress basis are several and nonlinear (one for each strain level).

From the view point of Hvorslev criterion, the shear strength of the two remoulded soils is essentially through ϕ_e' and the Hvorslev envelopes are uniquely determined independent of past maximum pressure.

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NOTATION

c_u=cohesion, total stress
c'=cohesion, effective stresses
c_e'=effective cohesion-Hvorslev parameter
ϕ_u=angle of shearing resistance, total stresses
ϕ'=angle of shearing resistance, effective stresses
ϕ_e'=effective angle of internal friction-Hvorslev parameter
ϵ_a=axial strain
ϵ_f=strain at failure
NC=normally consolidated
OC=overconsolidated
σ_{CM}'=maximum past consolidation pressure
σ_e'=equivalent consolidation pressure
k=coefficient of cohesion

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