An Experimental Evaluation of Stress Strain Behaviour of Soils

by

B.V. Ranganatham*

and G. Zacharius**

Introduction

The triaxial compression test with constant cell pressure is being widely used to determine shear strength parametres of soils for design purposes. However, stress changes in most practical problems do not always correspond to these conditions (Vaid et al 1974). In many field situations, the deformation conditions are more close to plane strain. Deformation conditions also vary from axial compression to axial extension. The subject of influence of stress path on the strength behaviour of soils has received considerable attention in the past. However, the reported results are conflicting (Henkel 1960, Shibata et al 1965, Ladd 1967, Hambly et al 1969, Dickey et al 1970). Hence, the accuracy of using unique shear strength parameters, obtained from conventional triaxial compression test, for varied deformation conditions observed in the field is worth examining. The investigations reported in this paper are concerned with the deformation behaviour of saturated remoulded clays, of low to high plasticity at triaxial stress conditions, with shear stress paths.

Test Procedure

The index properties of the soils used in the investigation are summarised in Table 1. The soils kaolinite, silty kaolinite and bentonite are commercially available. Black cotton soil and lateritic soil are local deposits. Saturated clay samples were obtained by consolidating the clay water slurry one-dimensionally with water content of twice the liquid limit in 305 x 305 x 305 mm perforated brass mould. The final consolidation pressure reached was about 0.4 kg/cm². Isotropically consolidated triaxal compression and extension tests (drained and undrained) were conducted on samples of 38.1 mm diameter and 76.2 mm height, trimmed from the soil cakes extracted from the bass mould. All testing procedures correspond to the methods detailed by Bishop and Henkel (Bishop et al 1962). The strain rate adopted corresponds to less than that required for 95 per cent dissipation or equalisation of pore pressure (assuming the filter parpers to be ineffective). Longitudinal filter paper strips were used for compression tests and spiral ones for extension tests. Degree of saturation of the specimens tested before shearing the specimens was found to be always one hundred per cent. End friction was reduced using the method suggested by Rowe and Barden (Rowe et al, 1964).

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* Professor, Department of Civil Engineering, Indian Institute of science, Bangalore, India.

** Materials Engineer, KM-ACEC Joint Venture, Riyadh, Saudi Arabia

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TABLE 1

Index properties of soils

Soil	Liquid limit	Plastic limit	Plasticity index
Kaolinite	63.1	33.2	29.9
Lateritic soil	28.0	23.2	4.8
Silty kaolinite	58.2	31.7	26.5
Black cotton soil	73.3	39.9	33.4
Bentonite	862.0	151.0	711.0

Drainage was allowed from both ends of the specimen for drained tests. Descriptions of the stress changes applied to the samples are given below together with abbreviations used to describe the tests.

1 Drained compression tests

$DC\sigma'_3$ Radial stress constant, axial stress increased

DCp Radial stress decreased and axial stress increased so that octahedral effective normal stress remains constant

2 Drained extension tests

$DE\sigma'_1$ Radial stress constant, axial stress decreased

$DE\sigma'_3$ Radial stress increased, axial stress constant

3 Undrained compression tests

$UC\sigma_3$ Radial total stress constant, axial total stress increased

$UC\sigma_m$ Radial total stress decreased and axial total stress increased so that octahedral total normal stress remains constant

4 Undrained extension tests

$UE\sigma_1$ Radial total stress constant, axial total stress decreased.

Tests were conducted on normally consolidated and over-consolidated soils. Over-consolidation of the soils was achieved by consolidating the trimmed sample in the triaxial cell to a pressure of 4.62 kg/cm^2 and then allowing to rebound to a pressure of 0.28 kg/cm^2 resulting in an initial over-consolidation ratio of 16.5 (= 4.62/0.28). [This over-consolidation ratio is designated as initial over-consolidation ratio, $(O.C.R.)_i$, in order to avoid confusion with the standard accepted term of $O.C.R.$ which is the ratio of the maximum overburden pressure in the past to the present overburden pressure.] The samples were then recompressed to different cell pressures less than 4.62 kg/cm^2 before shearing. Tests were repeated with $(O.C.R.)_l$ of 6 and 3 with the same maximum consolidation pressure of 4.62 kg/cm^2.

Undrained Test Results

The hypothesis of a unique undrained strength for samples consolidated under the same stress system, irrespective of the stress path, has been put forward on a number or occasions (Newill 1961, Wu et al 1963, Bishop 1971). However, some of the results of recent research are not in agreement with it. The influence of intermediate principal stress on the effective angle of shearing resistance ϕ', has been brought out by Wade (1963) Shibata and Karube (1965), Broms and Casbarian (1965), Mitchell and Wong (1973) and Vaid and Campanella (1974). The effective angle of shearing resistance obtainad from triaxial extension tests is reported to be different from that obtained from triaxial compression tests (Bjerrum et al 1966). Variation in ϕ' between triaxial extension and compression tests on anisotropically consolidated samples is reported by Parry and Nadarajah (1973). Duncan and Dunlop (1968) have reported an increase in ϕ' in tests in which the field vertical stress was increased to failure from tests in which the field horizontal stress was increased to failure. From the experiments of Parry (1956, 1960), Roscoe and Poorooshasb (1963) suggested that the effective stress path of specimens sheared under extension conditions with major principal stress, σ_1, constant and minor principal stress, σ_3, decreasing could be different from those of the specimens sheared under constant octahedral effective normal stress, p. However, a unique effective stress path for undrained shear tests is more widely accepted.

Stress-Strain Response

The influence of stress path on the stress-strain behaviour of normally consolidated and over-consolidated kaolinite is brought respectively in Figure 1 (*a*) and 1 (*b*) in which the values of deviator stress, $q\left[=(\sigma'_1 - \sigma'_3)\right]$, normalised by the consolidation pressure, p_o, is plotted against the

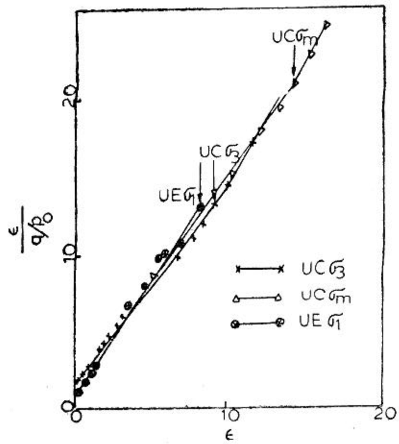

FIGURE 1 (*a*) **Stress-strain behaviour, Normally consolidated Kaolinite p_o = 1.78 kg/cm^2**

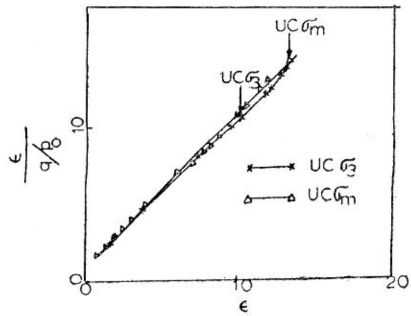

FIGURE 1(*b*) **Stress-strain behaviour, overconsolidated Kaolinite, $O.C.R. = 6, p = 2.24$ kg/cm²**

deviatoric strain, $[\varepsilon-(\varepsilon_a-\triangle V/V)$, ε_a = axial strain, $\frac{\triangle V}{V}$ = volumetric strain, $\varepsilon = \varepsilon_a$ for undrained tests on isotropically consolidated saturated clay]. A modified plot of $\frac{\epsilon}{q/p_o}$ versus ϵ, suggested by Kondner (1963) is used for this purpose. The arrows shown in the figures indicate the failure of the sample at the maximum deviator stress. It may be seen from the figure that the stress-strain relationship is not significantly influenced by the shear stress path.

Pore Pressure

The influence of the stress path on pore-pressure mobilisation is brought out in Figures. 2 (*a*) and 2 (*b*) respectively for normally consolidated and over-consolidated kaolinite. The measured value of increase in pore pressure during shear, $\triangle u$, normalised by the consolidation pressure is plotted against the normalised deviator stress. Also shown in the figures is the pore pressure corrected for the change in cell pressure, made to keep the octahedral total normal stress constant ($UC\sigma_m$-test), using Skempton's (1954) pore- pressure equation (the value of B is taken as unity). It may be seen from Figure 2 (*a*) that the pore-pressure response of normally consolidated kaolinite during undrained shear in

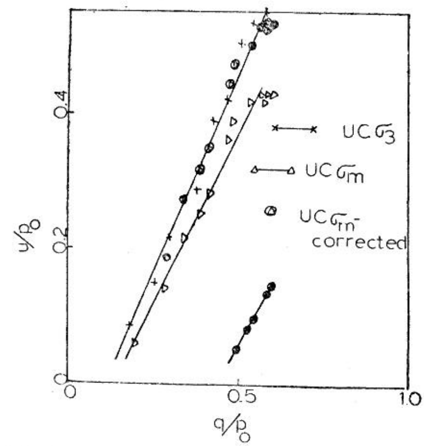

FIGURE 2 (*a*) **Pore pressure mobilisation during shear, Normally consolidated Kaolinite, p_o = 2.60 kg/cm²**

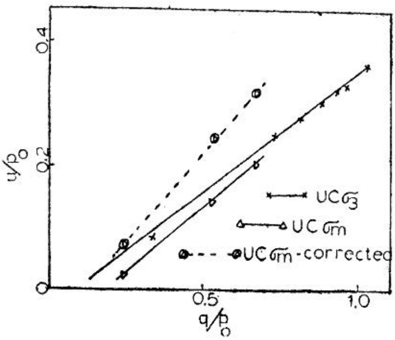

FIGURE 2 (*b*) Pore pressure mobilisation during shear, overconsolidated Kaolinita, p_o = 1.82 kg/cm²

$UC\sigma_3$-test and $UC\sigma_m$-test are more or less identical. But the corrected pore-pressure response for over-consolidated kaolinite in $UC\sigma_m$-test is found to be higher than that in $UC\sigma_3$-test, comparison being made for the same deviator stress [Figure 2 (*b*)]: Induced pore pressure in $UE\sigma_1$-test is found to be always very much smaller than that in compression tests. Incidentally, Figures 2 (*a*) and (*b*) suggest that Skempton's pore-pressure parameter *A* is a constant for a wide range of stresses. Figure 1 and 2 taken together are in agreement with the widely stated concept (Henkel 1960, Parry 1960) that the undrained shear response in terms of stress and strain of a saturated normally consolidated clay is independent of the stress path to failure, provided compression and extension modes of deformation are considered separately.

Effective Angle of Shearing Resistance

Figure 3 (*a*) presents modified Mohr-Coulomb plot for normally

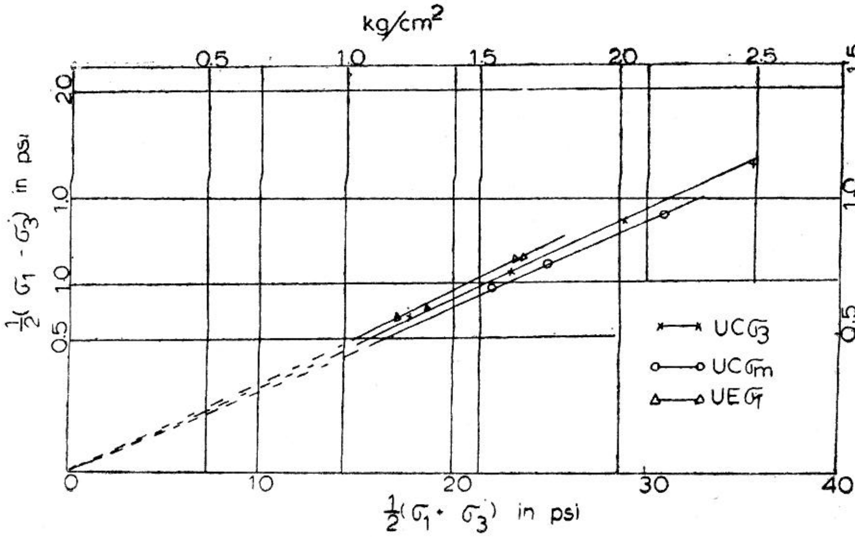

FIGURE 3 (*a*) Modified Mohr-coulomb plot for normally consolidated Kaolinite, undrained tests

consolidated kaoinlite for the stresses at the failure of the samples. (Failure is assumed to occur at maximum deviator stress.) The values of the effective angle of shearing resistance in undrained shear, ϕ'_u, are computed from the figure for each shear stress path separately. It is evident that the parameter ϕ'_u differs, though slightly, with the type of shear stress path. Superficially, this may look contradictory to the unique effective stress-path theory. But a closer look in Figure 1 (*a*) reveals that the strains at failure are stress-path dependent. The strains to failure in $UE\sigma_1$ tests are of relatively smaller magnitude than those of $UC\sigma_m$ tests with intermediary values for $UC\sigma_3$, tests. Influence of stress path on peak value of strain has been mentioned by Ladanyi (1967) and Shibata and Karube (1967). It has been reported (Roserquest 1959, Bjerruns et al 1960, Crawterd 1961, olsen 1963,) that particles get re-arranged along a direction parallel to the failure plane or planes while shear stresses are applied. The amount of particle re-arrangement may depend upon the strain at failure. The particles of soil specimens with high peak strain may get arranged more parallel to the failure plane. This increases the repulsive forces between the soil particles at the failure plane. Hence, soil specimen failing at high peak strain will be having lower strength. Test results support this argument since stress paths, for which samples failed comparatively earlier, resulted in higher value of ϕ'_u and vice versa. Strength decrease and thereby reduction in ϕ'_u by this phenomenon is reported by Rao and Nagaraj (1973) by making the statement 'the difference in strength of clay from peak to residual is due to the development of preferred orientation of the particles in thin zones adjacent to the failure plane. Bjerrum (1961) has shown that reduction in ϕ may occur due to progressive failure. Ladanyi (1967) reports that the mode of failure depends on the peak strain. Samples failing at high values of strain may be undergoing an overall distortion, whereas those failing early may be undergoing a failure with slide plane formation. He also reports that the mode of failure depends upon the type of restraint imposed on the sample. In $UC\sigma_m$ tests continuous decrease of lateral pressure might have influenced the mode of failure. Hence, difference in the extent of particle re-orientation together with the mode of failure may be the reasons for the variations in ϕ'_u with stress path. From Figure 2 (b) it may be concluded that the principle of unique effective stress path during undrained shear is not valid for over-consolidated kaolinite. However, the influence of stress path on the effective angle of shearing resistance reported in Figure 3 (*b*) may also be attributed to the depeddency of failure strain on stress path and thereby difference in orientation of the particles at failure conditions of the samples.

Mitchell (1970) reports that the mode of failure in compression tests is dependent on the octahedral effective normal stress at failure. Hence, the values of effective octahedral normal stress at failure of the sample, p_f, have been plotted against the octahedral effective normal stress at consolidation conditions, p_o, in Figure 4 (*a*) for normally consolidated kaolinite. It may be seen that the ratio p_f/p_o is a constant for a particular stress path and the variation of this ratio with stress path is in the same order as that of ϕ' with stress path. Hence, tan ϕ'_u is plotted against the ratio p_f/p_o [designated as tan λ, λ being the slope of best lines fitted for each stress path in Figure 4 (*a*)] in Figure 4 (*b*). The variation of tan

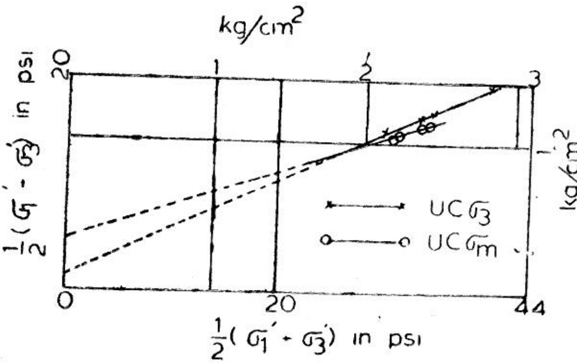

FIGURE 3 (*b*) Modified Mohr-coulomb plot for over-consolidated kaolinite, undrained tests, $(O.C.R.)_i = 6$

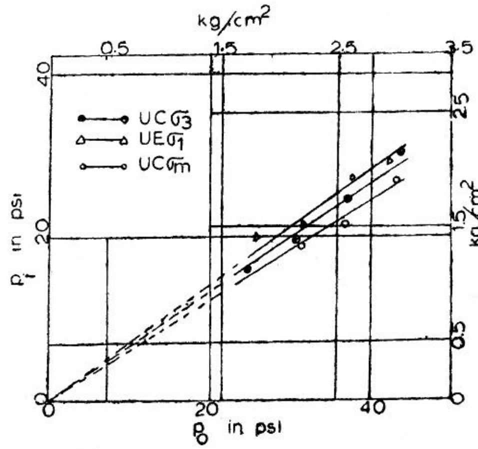

FIGURE 4 (*a*) p_f Verus p_o for normally consolidated kaolinite

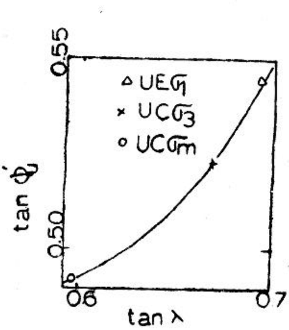

FIGURE 4 (*b*) Tan ϕ_u' versus tan λ for normally consolidated kaolinite

ϕ'_u with tan λ observed for bentonite and lateritic soil are similar and hence not reported. The results of Parry (1960) which he used to support the argument that the strength parameters of normally consolidated clays are stress-path independent have been analysed in the same way. Figure 5(*a*) reports the modified Mohr-Coulomb plot. In Figure 5(*b*) the values of p_f are presented against p_o and in Figure 5 (c) tanϕ'_u against tan λ. The results confirm the general validity of the approach that tan ϕ'_u decreases as p_f decreases relative to p_o.

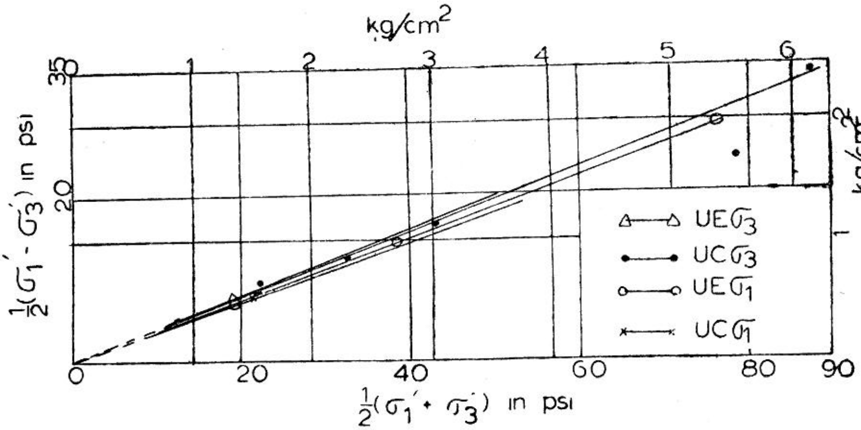

FIGURE 5 (*a*) **Modfied Mohr-Coulomb plot for weald clay, Undrained tests ($UC\sigma_1$- compression test with σ_1 kept constant, UE_1— extension test with σ_1 kept constant)**

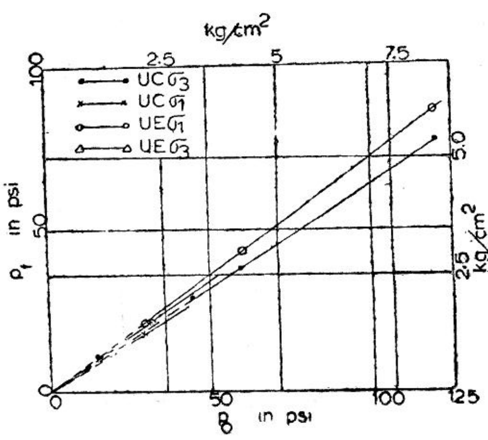

FIGURE 5(*b*) **p_f versus p_o for weald clay, undrained tests**

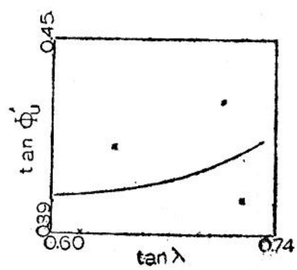

FIGURE 5(*c*) Tan ϕ_u versus tan λ for weald clay

Skempton's Pore-pressure Parameter A At Failure

The linear variation of p_f with p_o observed in the test results may be used to deduce an expression for A_f, Skempton's pore pressure parameter A at failure of the sample, in terms of deviator stress at failure, q_f, and the octahedral effective normal stress at cosnolidation conditions.T he value of octahedral effective normal stress at failure of the sample may be expressed in terms of increase in deviator stress from consolidation condition to failure, $(\Delta q)_f$, induced pore pressure $(\Delta u)_f$ and p_o. The expression for a linear total stress path may be stated as follows:

$$p_f = p_o + a(\Delta q)_f - (\Delta u)_f$$

where $a = \dfrac{\Delta p}{\Delta q}$ and is a constant for a linear stress path in $p-q$ plane. (The value a has been defined by Yudhbir and Varadarajan (1973) as the inverse slope of the stress path in a $p-q$ plan.)

For a normally consolidated clay $\dfrac{p_f}{p_o}$ = constant = tan λ

$$\frac{p_f}{p_o} = \tan\lambda = \frac{1}{p_o}\left[p_o + a\,(\Delta q)_f - (\Delta u)_f \right]$$

For isotropically consolidated sample $(\Delta q)_f = q_f$

$$A_f = \frac{(\Delta u)_f}{q_f} = \frac{p_o\,(1 - \tan\lambda)}{q_f} + a$$

(1) ie, $$A_f = \frac{b\,p_o}{q_f} + c$$

where b and c are constants.

Figures 6 (*a*) and 6 (*b*) report the values of A_f against the ratio $\dfrac{p_o}{q_f}$ which corroborate the linear relation deduced.

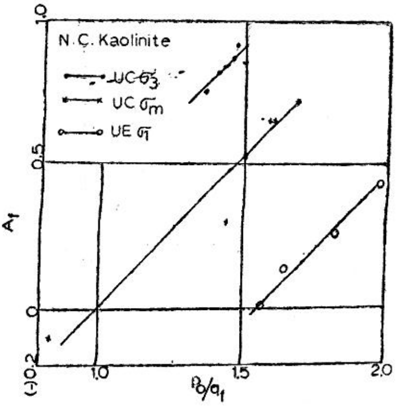

FIGURE 6(*a*) A_f versus p_o/q_f, normally consolidated kaolinite

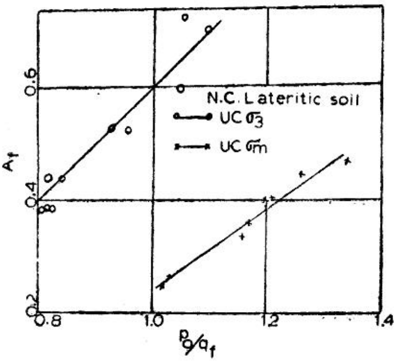

FIGURE 6 (*b*) A_f versus p_o/q_f normally Consolidated lateritic soil

Undrained Strength

The works of Skempton and Henkel (1953) and Skempton (1954) show that, in the case of normally consolidated undisturbed clays, the ratio of undrained shear strength, *Su*, to the effective overburden pressure, σ_o', can be correlated closely with plasticity index, *Ip*. Sridharan and Rao (1973) after making a critical study have concluded that a unique correlation between $\frac{S_u}{\sigma_o'}$ and I_p is not possible since undrained shear strength of a clay is also influenced by stress path, rate of loading, method of testing and temperature. They have also presented data of remoulded clays to prove the non-existence of a unique relation between undrained shear strength and plasticity index. Sridharan and Rao (1973) have expressed the term $\frac{S_u}{\sigma_o'}$ for normally consolidated clay (under isotropic stress conditions during consolidation) in terms of ϕ' and A_f as

$$\frac{S_u}{\sigma_o'} = \frac{\text{Sin } \phi'}{1 + (2A_f - 1)\sin \phi'}$$

Substituting for A_f from Equation.(1) with $p_o = \sigma_o'$ and $q_f = S_u$ and re-arranging

$$\frac{Su}{\sigma_o'} = \frac{\text{Sin}\,\phi'\,(1-2b)}{1+2c\,\text{Sin}\,\phi - \text{Sin}\,\phi}$$

Substituting for the constants b and c

(2) $$\frac{S_u}{\sigma_o'} = \frac{\text{Sin}\,\phi'\,(2\tan\lambda - 1)}{1+2\alpha\,\text{Sin}\,\phi' - \text{Sin}\,\phi'}$$

Since the ratio $\frac{S_u}{\sigma_o'}$ depends upon the values of ϕ', tan λ, and α, a unique relation between this ratio and the plasticity index may not be strictly existing. Test results have been analysed in order to verify this argument. It has been found that the value of $\frac{S_u}{p_o}$, $(\sigma_o' - p_o)$ is varying slightly with the consolidation pressure also. The average value of $\frac{S_u}{p^o}$ for each stress path is reported in Table 2 along with the values of the plasticity index.

TABLE 2

Average values of S_u/p_o for each stress path

Soil	Stress history	Type of test	$\frac{S_u}{\sigma_o'}$
Kaolinite	Normally sonsolidated	$UC\sigma_3$	0.72
Kaolinite	Normally consolidated	$UC\sigma_m$	0.64
Kaolinite	Normally consolidated	$UE\sigma_1$	0.58
Lateritic soil	Normally consolidated	$UC\sigma_3$	1.09
Lateritic soil	Normally consolidaied	$UC\sigma_m$	0.87
Bentonite	Normally consolidated	$UC\sigma_3$	0.34
Bentonite	Normally consolidated	$UC\sigma_m$	0.29
Black cotton soil	Normally consolidated	$UC\sigma_3$	0.79
Silty kaolinite	Normally consolidated	$UC\sigma_3$	0.84
Kaolinite	Over-consolidated $(O.C.R.)_i=6$	$UC\sigma_3$	1.13
Kaolinite	Over-consolidated $(O.C.R.)_i=6$	$UC\sigma_m$	0.94

Drained Test Results

Effective Angle of Shearing Resistance

The controversy with regard to the influence of stress path on the effective angle of shearing resistance during drained shear, ϕ'_d, is equally as strong as that during undrained shear. Figure 7(*a*) reports a typical variation of deviator stress with deviatoric strain for normally consolidated kaolinite. (The deviator stress-deviatoric strain curve is assumed to be hyperbolic and, hence, a modified plot of $\frac{\varepsilon}{q/p_o}$ versus ε is used for reporting the results.) The arrows in the figure indicate deviatoric strain at maximum deviator stress from which the dependency of peak strain on stress path is brought out. In drained tests the extent of particle re-orientation or re-arrangement during shear loading has to be assessed in terms of the amount of volume change and the magnitude of axial strain. Hence, it is assumed that greater the value of the ratio of volumetric strain to axial strain at failure of the sample, greater is the probability of particles getting oriented parallel to the failure plane. Hence, samples having high value of this ratio will be having less strength and thereby lesser effective angle of shearing resistance. In Table 3 the values of the ratio

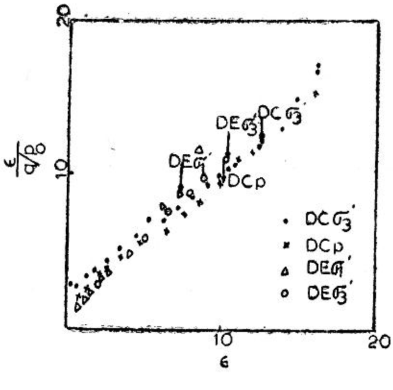

FIGURE 7 (*a*) Stress-strain response, normally cossolidated kaolinite

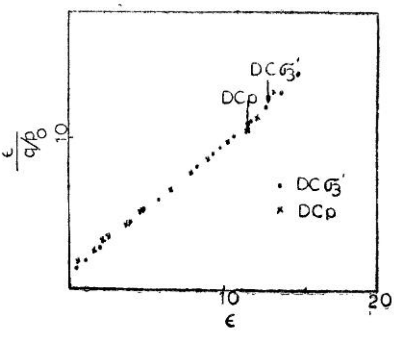

FIGURE 7 (*b*) Stress-strain response, over consolidated, kaolinite, $(O.C.R.)_i = 6$, $p_o = 1.82$ kg/cm²

TABLE 3

Ratios of the volumetric strain to axial strain at failure

Soil—Normally consolidated kaolinite

Type of test	$\left[\left(\frac{\Delta V}{V}\right)_f - \varepsilon_{if}\right]$	$\tan \phi'_d$
$DE\sigma'_3$	0.36	0.476
$DC\sigma'_3$	0.33	0.478
DC_p	0.22	0.496
$DE\sigma'_1$	0.14	0.527

of the volumetric strain to axial strain at failure conditions,

$$\left[\left(\frac{\Delta V}{V}\right)_f \div \varepsilon_{if}\right],$$

for all drained tests on normally consolidated kaolinite corresponding to a consolidation pressure of 2.8 kg/cm² are reported. If the strength of the soil samples is influenced by the extent of particle re-arrangement occuring during shear and thereby influenced by the ratio

$$\left(\frac{\Delta V}{V}\right)_f \div \varepsilon_{if},$$

the order in which ϕ'_d has to decrease with stress path is $DE\sigma'_1$, DCp, $DC\sigma'_2$ and $DE\sigma'_3$. Figure 8(*a*) in which the failure stresses are reported in a modified Mohr-Coulomb plot is in agreement with this.

It has been shown earlier that the ratio p_f/p_o influences the effective angle of shearing resistance in undrained shear. In the case of drained tests the value of the octahedral effective normal stress at failure of the sample for isotropically consolidated sample, $p_f = p_0 + \alpha q_f$. Hence, p_f will be varying linearly with p_o for a constant value of α. This has been verified in Figure 8 (*b*) for normally consolidated kaolinite. Using Mohr-Coulomb equation for normally consolidated clays

$$q_f = (\sigma'_1 + \sigma'_3)_f \text{ Sin } \phi'$$

and equation $\quad p_f = p_0 + \alpha q_f$

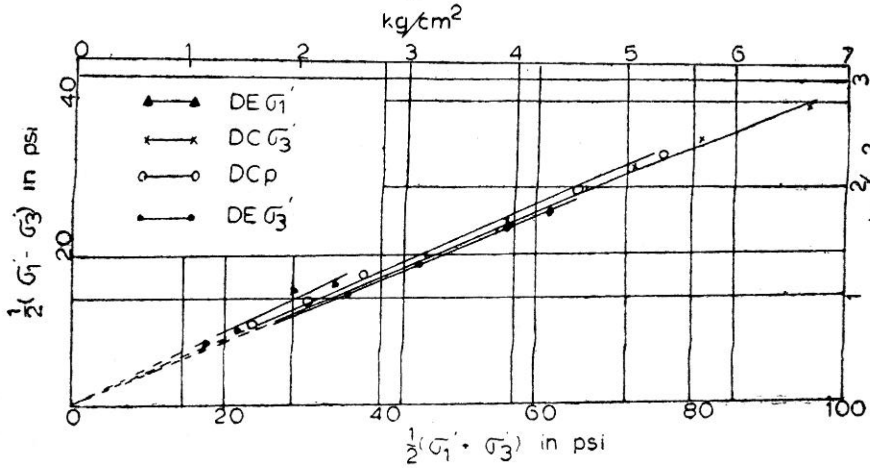

FIGURE 8 (*a*) **Modified Mohr-coulomb plot for normally consolidated kaolinte, Drained tests**

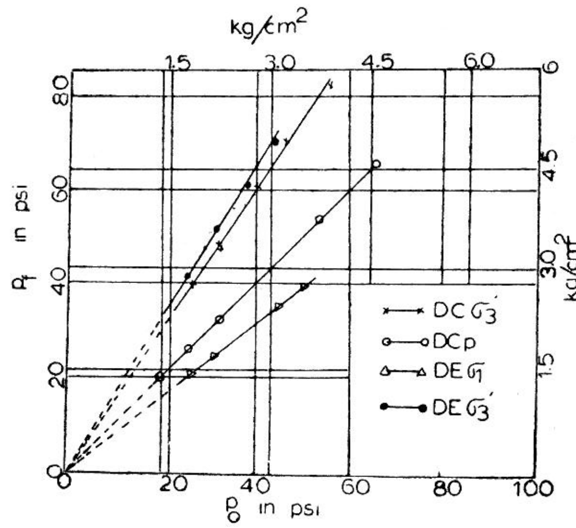

FIGURE 8(*b*) p_f **versus** p_o **for normally consolidated kaolinite**

it can be shown that the ratio p_{f/p_o} is related to ϕ'_d in the following form:

$$\frac{3-\text{Sin }\phi'}{3-\text{Sin }\phi'-6\ \alpha\ \text{Sin }\phi'}=\frac{p_f}{p_0}$$

However, for comparison of the test results with others, a plot of tan ϕ'_d versus tan $\lambda\left(=\frac{p_f}{p_0}\right)$ is used. Such a plot for normally consolidated kaolinite is shown in Figure 8(*c*). The tests conducted on normally consolidated lateritic soil are observed to give the same trend of variation and

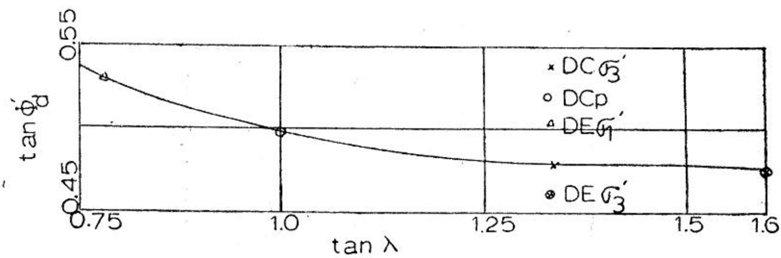

FIGURE 8 (*c*) Tan ϕ_d' versus tan λ for normally consolidated kaolinite

hence not reported herein. Drained triaxial test results reported by Parry (1960) for different shear stress paths are plotted in Figure 9(*a*) [Modified Mohr-Coulomb], 9(*b*) [p_f versus p_0] and 9(*c*) [tan ϕ_d' versus tan λ] which conform to the same trend.

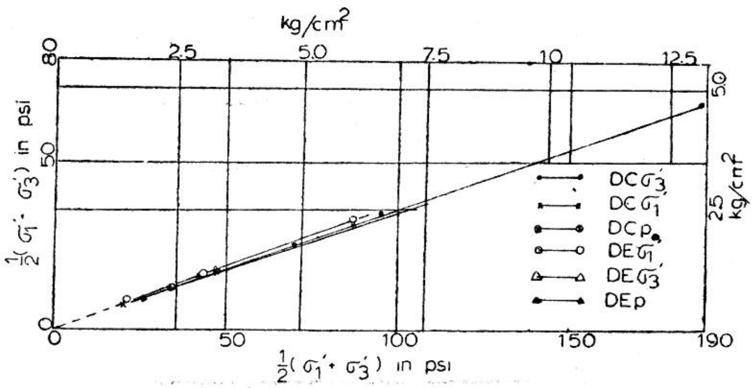

FIGURE 9(*a*) Modified Mohr-Coulomb plot for weald clay, Drained tests ($DC\sigma_1'$ — compression test with σ_1' kept constant, DE_p—extension test with p kept constant

In Figure 7(*b*) typical stress-strain behaviour of over-consolidated kaolinite with initial over-consolidation ratio of 6 is reported. The influence of stress path on the peak strain may be noticed in this case also. Figure 10 reports the modified Mohr-Coulomb plot for the same soil. The reasoning that stress path which mobilises lower value of the ratio $\left(\frac{\Delta V}{V}\right)_f$ $\div\ \varepsilon_{1f}$ should mobilise a higher value of ϕ_d' is no longer applicable for this over-consolidated clay. Table 4 reports the values of the ratio of volumetric strain to axial strain at failure conditions for tests on over-consolidated kaolinite.

In order to separate out the influence of stress path on frictional and dilatancy components of strength (Sridharan et al 1973, Poorooshարb et al 1961, Rowe 1962, Bishop 1954), the result from the tests on normally

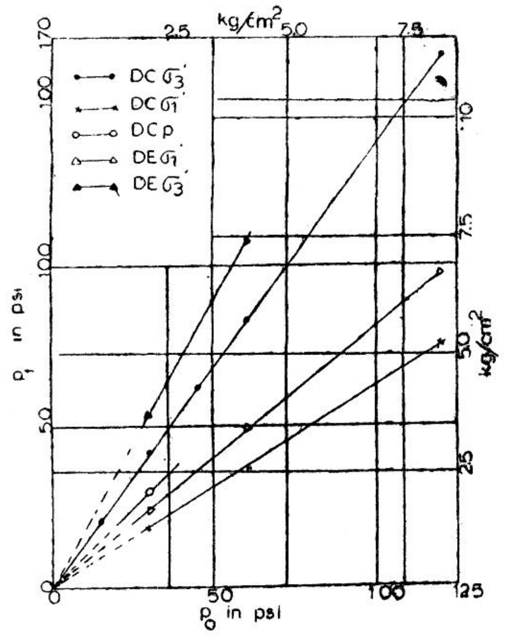

FIGURE 9(*b*) p_f versus p_o for weald clay

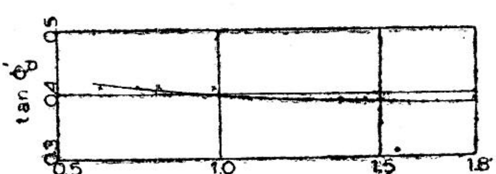

FIGURE 9(*c*) Tan ϕ_d' versus tan λ for weald clay

consolidated kaolinite are reported in an effective stress ratio, σ_1' / σ_3', versus $\left[1 - \frac{d\left(\frac{\Delta V}{V}\right)}{d\varepsilon_1} \right]$ plot in Figure 11. No specific conclusions can

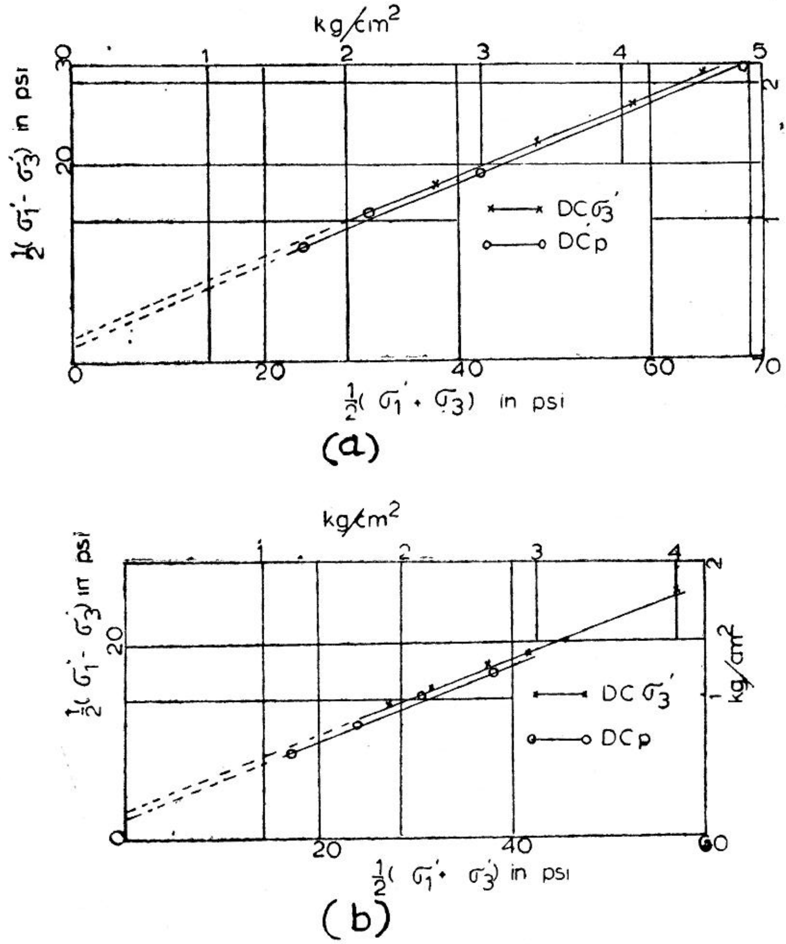

FIGURE 10 Modified Mohr-Coulomb plot for over consolidated koalinite, drained tests,
(a) $(O.C.R.)_i = 6$
(b) $(O.C.R.)_i = 16.5$

TABLE 4

Ratios of volumetric strain to axial strain at failure

Over-consolidated kaolinite, $p_o = 1.82$ kg/cm²

Type of test	Initial over-consolidation ratio	$\left[\left(\frac{\Delta V}{V}\right)_f \div \varepsilon_{1f}\right]$
$DC\sigma_3'$	3	0.15
DC_p	3	−0.03
$DC\sigma_3'$	6	0.16
DC_p,	6	0.04
$DC\sigma_3$	16.05	0.20
DC_p	16.05	0.05

TABLE 5

Variation in ϕ' under different conditions

Variable causing the change in angle of shearing resistance	Change in the effective angle of shearing reristance	Reference
Replacement of water by ethylene glycol	2°50′	Yong et al, (1963).
Replacement of adsorbed ion of sodium by calcium in illite	4°42′	Olsen, (1963).
Oven-drying the wet soil and remoulding	1°00′	Kirwan et al, (1961).
Sample disturbance	3°20′	Kenny, (1969).

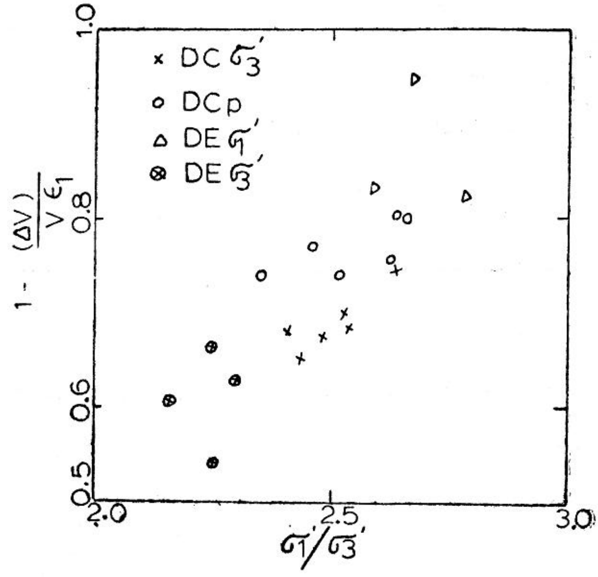

FIGURE 11 Stress dilatancy plot for normally consolidated kaolinite

be drawn from the figure because of the non-uniform scatter of the points. However, it may be tentatively concluded that stress path influences both frictional and dilatancy components.

It may be interesting to observe the variation of tan ϕ' with tan λ for normally consolidated soils of varying plasticity index for a constant stress path. Such a plot for undrained conditions is reported in Figure 12, and in Figure 13 that for drained conditions.

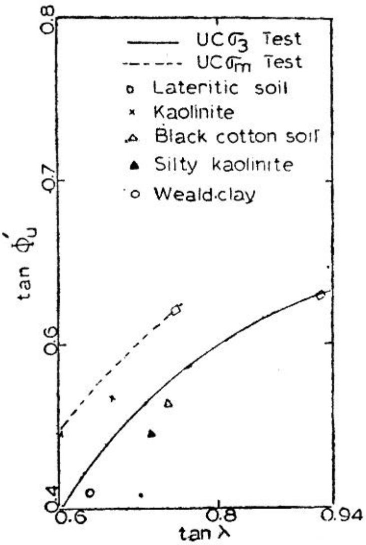

FIGURE 12 Tan ϕ'_d versus tan λ, identical stress paths

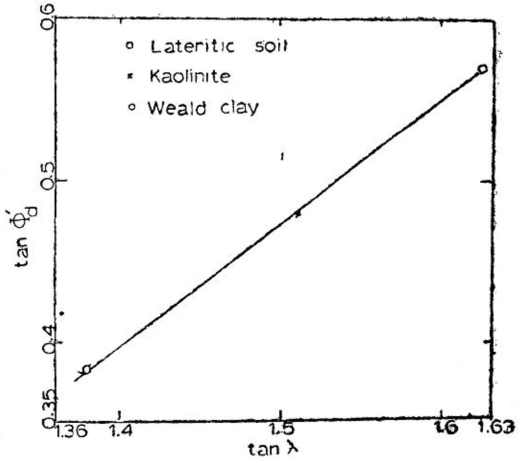

FIGURE 13 Tan ϕ'_d versus tan λ, identical stress paths

Concluding Remarks

The effective shear strength parameter, ϕ', is shown to be dependent on the shear stress path. The maximum difference in ϕ' for undrained test with stress path is found to be 2°21′ and for drained tests 2°20′ for kaolinite. The influence of stress path on ϕ', though the variation is small, is not to be neglected since the observed differences have been logically explained in this study. Further, the magnitude of this variation is seen to be quite similar to that due to many other variables such as adsorbed ions, nature of pore fluid, sample disturbance, etc. Table 5 reports variation in ϕ' observed due to some of these factors which have been collected from published literature.

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Notations

A	Skempton's pore-pressure parameter
b, c	Constants
p	Octahedral effective normal stress
p_o	Value of p at consolidation conditions
p_f	Value of p at failure of the sample
q	Principal stress difference, deviatoric stress
S_u	Undrained shear strength
u	Pore pressure
V	Volume
$(O.C.R.)_i$	Initial over-consolidation ratio
$\sigma_1\ \sigma_2\ \sigma_3$	Major, intermediate and minor principal stresses
$\sigma_1'\ \sigma_2'\ \sigma_3'$	Major, intermediate and minor effective principal stresses
ϕ'	Effective angle of shearing resistance
ϕ_u'	Effective angle of shearing resistance, undrained tests
ϕ_d'	Effective angle of shearing resistance, drained tests
ε_1	Axial strain
ε_{1f}	Axial strain at failure
Δ	Change in quantity
ε	Deviatoric strain
λ	Slope of the linear relation between p_f and p_o
α	Inverse slope of the stress path in a $p-q$ plane