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Shear Behaviour of Partially Saturated Soils

by

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Introduction

In arid and semi-arid areas, fine grained soils are often encountered in a partly saturated state to a considerable depth. Compacted soils with or without additives are also used extensively in Civil Engineering works as construction materials and they are usually unsaturated. Atmospheric and hydrological conditions have a controlling influence on soil-moisture conditions in partly saturated soil and hence on its strength and volume change behaviour.

The strength of a compacted clay is mainly controlled by the void ratio (or dry density), water content (or degree of saturation), the soil fabric and the electrical forces at particle level (Lambe 1958, Seed *et al.* 1960, Sridharan 1968, Venkatappa Rao 1975).

In many investigations, the degree of saturation has been varied by compacting the soil to the same dry density at different moulding water contents. The compactive effort that is required to produce the same dry density would vary with the moulding water content as also the soil fabric. Variation in strength in such investigations can not wholly be attributed to the variation in the degree of saturation, since the changes in fabric may also affect the strength. Moreover, the available experimental data on the compacted specimens with the changing degree of saturation but having constant fabric pertained mostly to the unconfined compression tests (Sridharan 1968, Venkatappa Rao 1975, Venkatappa Rao and Sridharan, 1978). The experimental data from triaxial compression tests is either negligible or nil.

This paper presents the results of drained triaxial compression tests on compacted specimens investigating the variation of strength with changing

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degree of saturation keeping the other factors (i.e. soil fabric, void ratio) constant. Test results are analysed in terms of drained shear strength and the Mohr-Coulomb strength parameters as a function of the degree of saturation. On the basis of certain valid assumptions, an attempt has been made to estimate the effective negative pore water pressure at failure by relating it to the cohesion intercept.

Experimental work

Experimental investigation has been carried out on two soils (*i*) a commercially available kaoline ($G_s = 2.60$, $W_L = 49$ percent, $I_p = 20$ percent) and (*ii*) a natural soil called red earth ($G_s = 2.67$, $W_L = 38$ per cent, $I_p = 20$ percent). Proctor's optimum dry densities of the two soils are 1.4 and 1.52 g/cm^3 at water contents of 27.4 percent and 21.5 percent respectively. The triaxial specimens (3.8 cm in dia and 7.6 cm in length) were statically compacted to the dry densities of 1.62 g/cm^3 and 1.70 g/cm^3 with moulding water contents of 23 percent and 16.5 percent respectively for kaolinite and red earth. A special remoulding sampling device (Nagaraj, 1964) was used for compacting the specimens.

The compaction procedure briefly was to sieve the air dried soil through IS 425-micro sieve (BS 36), mix it with the desired quantity of distilled water and to store the same inside double plastic bags for one week. Just before compaction, the loose cured soil was again sieved through a IS 2.36 mm sieve (BS 7) to break up any lumps that were formed during mixing. All the specimens were compacted to constant dry density using one moulding water content so that the soil fabric and the void ratio essentially remained constant. Since the specimens were prepared using the same procedure, it is presumed that the fabric remained constant for the individual specimens. They were kept later in desiccators maintained at different relative humidities and desorbed to different equilibrium water contents. Since the moulding water contents used for compacting the specimens were less than the shrinkage limits of the respective soils, it is presumed that the shrinkage if any, in the specimens is minimal when the degree of saturation reduces. By this procedure, it was thus possible to obtain triaxial specimens having constant fabric and void ratio but with varying degree of saturation.

Use of aquous solutions of sulphuric acid, as per ASTM E 104-51 (1966), provided a simple means to maintain the different relative humidities. The bottom portion of the desiccators contained the sulphuric acid solution and the specimens were kept at the top half over a perforated sheet. The required relative humidity was obtained by using corresponding density of aqueous sulphuric acid maintained at a constant temperature of 25° ± 1°C in a chamber. The specimens were kept in the desiccators until a constant weight equilibrium was established. Generally, periods from 12-15 weeks were needed to attain the equilibrium. Before the specimen was tested, the bulk weight and volume were measured. The volume was measured by using mercury displacement method (ASTM, 1964) and the dry density was calculated from the known dry weight. Later the specimens were tested in triaxial compression under fully drained conditions (i.e. keeping the drainage lines open to atmosphere without any water in the burette). The specimens were tested at a controlled rate of 0.0475 per cent/minute which was sufficiently slow for the equilibrium to be reached.

Resaturation Procedure

The degree of saturation for the specimens kept in the desiccator maintained at 100 percent relative humidity (keeping distilled water in the bottom chamber of the desiccator) was about 96 percent. In order to obtain 100 percent degree of saturation the sample was placed on the triaxial cell base, enclosed in a thin rubber membrane and then subected to a confining pressure of 0.5 kg/cm^2. The bottom end was connected to a constant mercury control system and the top end to burette open to atmosphere. Distilled water was allowed to drain from bottom to top at a constant mercury head of 0.4 kg/cm^2. Slow drainage through the sample over a period of one week raised the degree of saturation to about 98-99 per cent. After a week or ten days, the back pressure was shut off and the bottom end was also connected to the burette. Latter, the specimen was consolidated by raising the cell pressure to the required level and then sheared under fully drained conditions. When the above procedure, although the degree of saturation of the specimen slightly increased, the initial void ratio was also found to have increased due to swelling. In order to check the initial void ratio, volume-weight measurements were taken for few specimens after resaturation by stripping off the rubber membrane. The void ratios were found to have increased from 0.61 to 0.70 for kaolinite and 0.57 to 0.65 for red earth due to swelling.

Volume Change Measurements

Volume changes in a saturated specimen during drained shear were measured through a burette connected to the ends of the specimen. However, this is not possible for an unsaturated specimen since it may absorb water contained in the burette due to capillary suction. Therefore, a standared volume gauge supplied by AIMIL was connected between the self-compensating mercury system and the triaxial cell. The principle of flow in or flow out from the triaxial cell was used for measuring volume changes. The volume of water flowing from the cell was considered as volume expansion and the water flowing into the cell was considered as volume decrease. Volume gauge readings were appropriately corrected for volume displacement caused by the plunger. No leakage of fluid between piston and bushing occured because of the 'O' ring in the bush.

Test Results and Analysis

Stress-Strain Behaviour

Figures 1 and 2 show the typical stress-strain-volume change relationships at various degrees of saturation for kaolinite and red earth respectively. Sample stiffness decreased and volume contraction and strain at failure increased with the increase in the degree of saturation. The specimens at low degree of saturation exhibited the brittleness with well defined peaks at the maximum deviator stress. In general, the specimens failed along a single dominant shear plane, thus marking an abrupt change in the stress-strain curves. The specimens particularly at low cell pressures, after undergoing maximum volume contraction, showed a tendency of dilation mainly due to slippage taking place along a single shear plane.

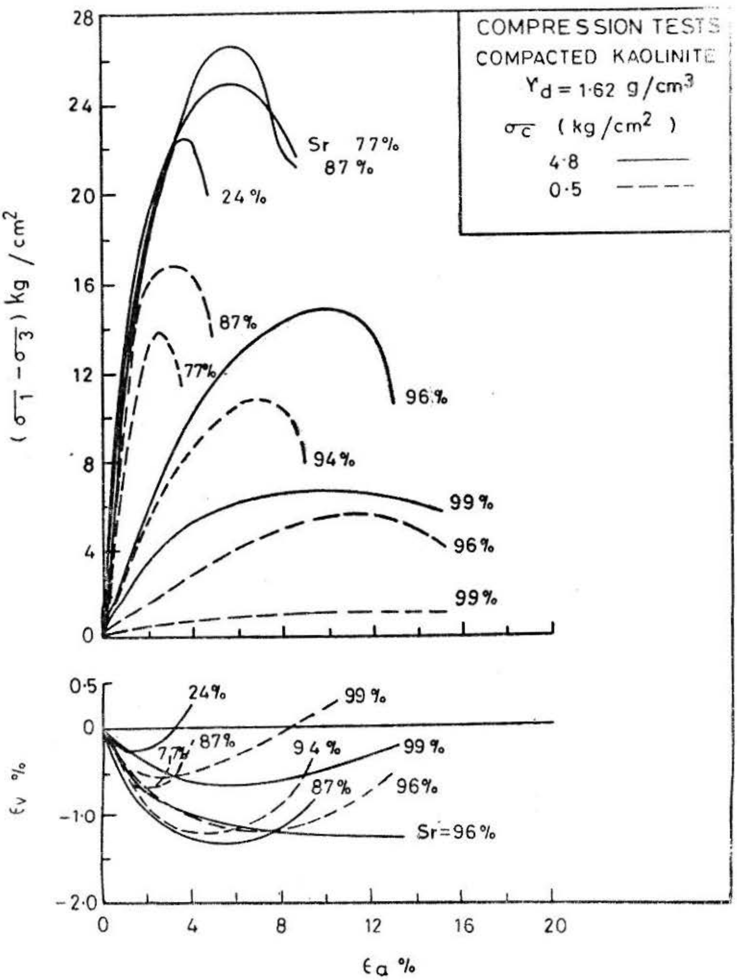

FIGURE 1 Stress-Strain-Volume Change Curves in Triaxial Compression for Kaolinite

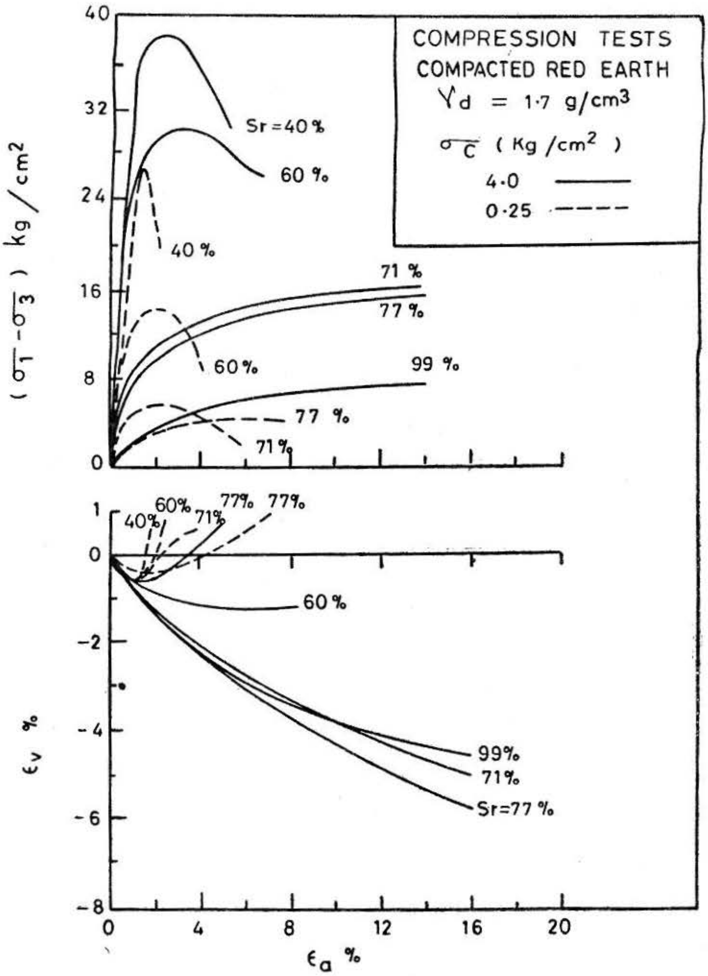

FIGURE 2 Stress-Strain-Volume Change Curves in Triaxial Compression for Red Earth

The formation of a slip plane prior to peak stress and also the wedging effect of the two halves of the specimen as a result of movement along the shear plane, led to the dilation of the specimen.

Secant Modulus

For the comparative study of small strain behaviour, the secant modulus (E^2) defined at a stress level equal to one-half of the maximum principal stress difference (i.e. $FS = \frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)} f = 2$) has been used. Figures 3 and 4 show the log-log plots between the normalised secant modulus, E_2/σ_c and the consolidation pressure, σ_c. Despite some scatter in the data points, the normalised secant modulus E_2/σ_c can be considered as linearly

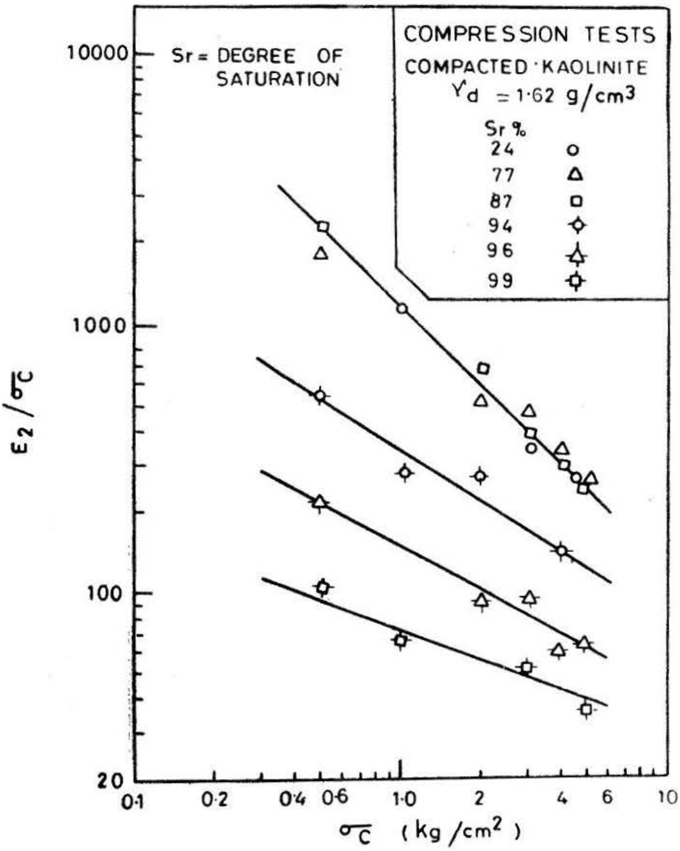

FIGURE 3 Effect of Degree of Saturation on Secant Modulus for Kaolinite

related to the consolidation pressure on a log-log plot. Interestingly at higher degrees of saturation, the relationships for kaolinite are several and distinctly different but at lower degree of saturation (24, 77 and 87 percent) an unique relationship has been obtained. In the case of red earth, the relationships are not only linear but also approximately parallel at various degrees of saturation. Thus, for a given degree of saturation the normallized modulus decreases with the consolidation pressure. It is seen that in both kaolinite and red earth, the effect of degree of saturation, S_r on normalised modulus, E_2/σ_c is significant. In red earth, there is a ten fold increase in E_2/σ_c with decrease in S_r whereas in kaolinite depending on the confining pressure, it increases about 10 to 30 times with the decrease in S_r.

Drained Shear Strength

Figures 5 and 6 show the test results of kaolinite and red earth with the maximum principal stress difference plotted against the consolidation pressure for various degrees of saturation. The compressive strength

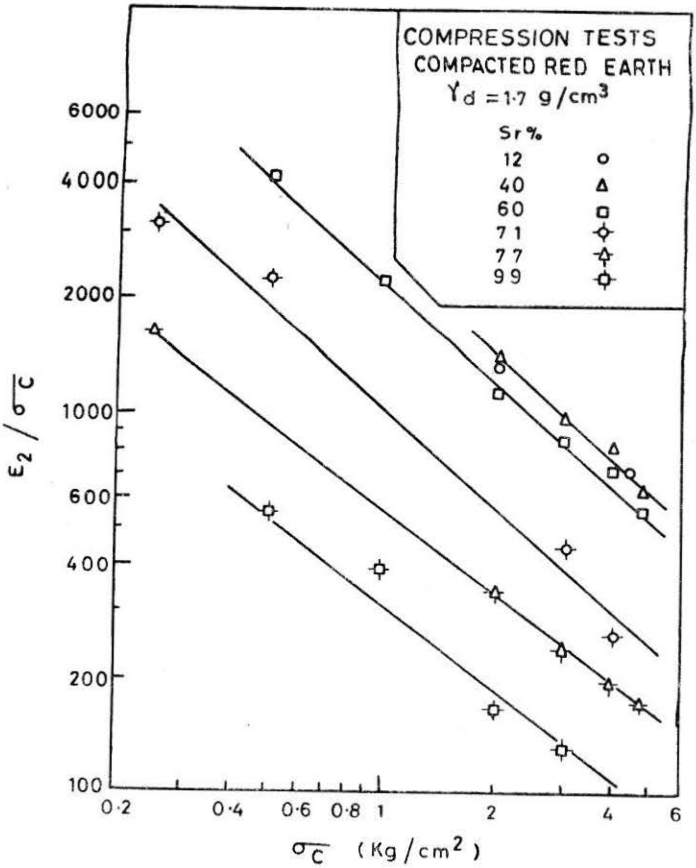

FIGURE 4 Effect of Degree of Saturation on Secant Modulus for Red Earth

increases linearly with the consolidation pressure, σ_c and the relationships are also parallel at all degrees of saturation. However, for $S_r = 99$ per cent, slightly low strengths have been observed obviously due to the increase in the intial void ratio of the specimens during resaturation.

The test points plotted in Figs. 5 and 6 are replotted in Figs. 7 and 8 in a different form to show the variation in strength with the degree of saturation. Starting from low degree of saturation, the compression strength increases with an increase in degree of saturation upto a certain optimum value beyond which the strength drops due to further increase in degree of saturation. So, for any consolidation pressure, there appears to be an optimum degree of saturation at which the strength is maximum. Despite the fact that there is lack of experimental data in the optimum range, the figures indicate a clear hump for all consolidation pressures.

With the present state of our knowledge in relation to unsaturated soils, this type of optimum strength behaviour can only be explained

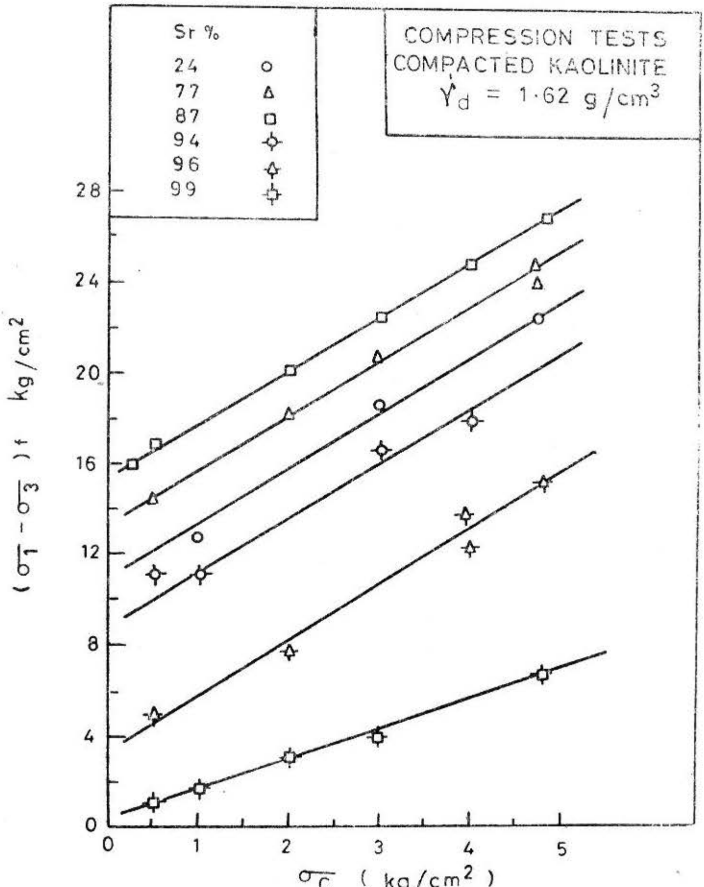

FIGURE 5 Maximum Deviator Stress vs Consolidation Pressure for Kaolinite

qualitively because of the difficulty in determining certain relevant parameters which define the effective stress. Apart from the confining stress, the parameters controlling the effective stress are the effective pore water pressure and the electrical attractive and repulsive pressures. All the three vary with the degree of saturation. Sridharan (1968) rewrote the effective stress equation of Lambe (1960) as follows :

$$\overline{C} = \sigma - \overline{U_w} - \overline{U_a} - R + A \qquad \ldots(1)$$

where, $\overline{C}$ = Effective contact stress

σ = Externally applied pressure on unit area

$\overline{U_w}$ = Effective pore water pressure

$\overline{U_a}$ = Effective pore air pressure

R = Total interparticle electrical repulsion divided by total interparticle area

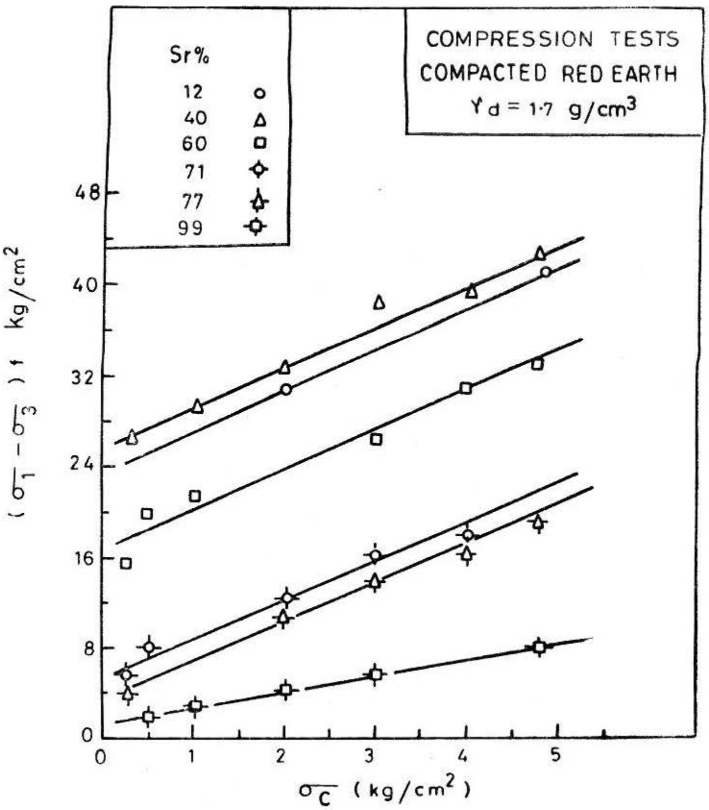

FIGURE 6 Maximum Deviator Stress vs Consolidation Pressure for Red Earth

A = Total interparticle electrical attraction divided by total interpartical area

Equation 1 has been used successfully in understanding the shear strength characteristics of saturated and unsaturated clays (Sridharan *et al.* 1971, Venkatappa Rao 1975, Sridharan and Venkatappa Rao, 1979). Reliable experimental determination of negative pore water pressures in soils involves many difficulties. Using pore size measurements and assuming the validity of capillary equation, Sridharan (1968) estimated the negative pore water pressures in unsaturated soils and showed that the effective negative pore water pressure increased to a peak value at an optimum degree of saturation beyond which it decreased. The increase in the effective negative pore water pressure with the degree of saturation upto an optimum level has been shown due to the increase in the number of pores that contribute to the effective negative pore water pressure. He further showed that the electrical attractive forces varied inversely and the electrical repulsive forces varied directly with an increase in degree of saturation and that both these forces decreased with an increase in particle spacing.

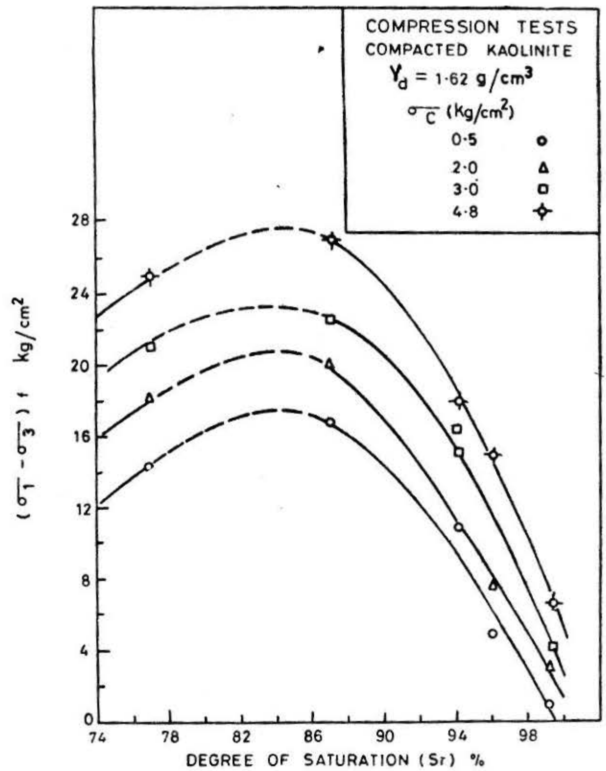

FIGURE 7 Maximum Deviator Stress vs Degree of Saturation for Kaolinite

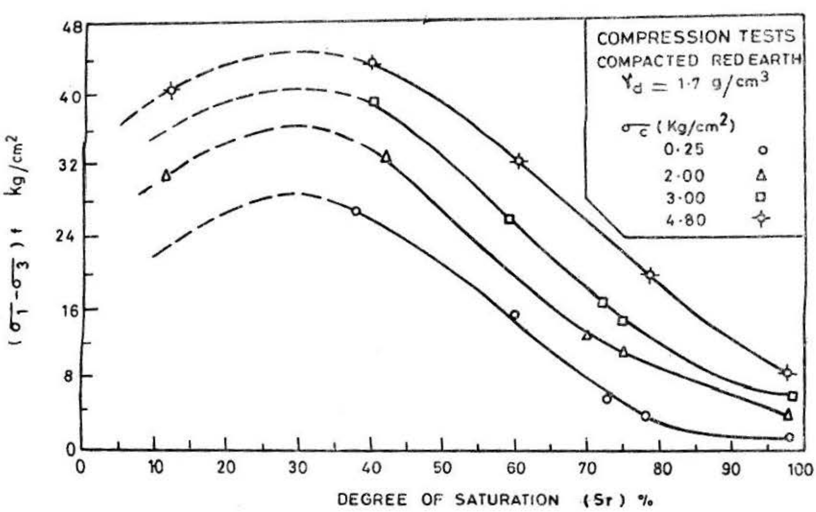

FIGURE 8 Maximum Deviator Stress vs Degree of Saturation for Red Earth

Starting from low degree of saturation, the effective negative pore water pressure $\overline{U_w}$ increased and the net (A—R) forces decreased with an increase in degree of saturation. Thus, at low degree of saturation the effects of negative pore water pressure and the net (A—R) forces oppose each other. Since the modified effective stress (Eq. 1) depends on these components, the strength variation should depend on the relative magnitude of $\overline{U_w}$ and (A—R) and their variation with the degree of saturation. Sridharan (1968) showed that except at low degree of saturation, the contribution from electrical forces to the strength might be negligible beyond about 40 percent degree of saturation for most soils. Hence, in the range of *degree of saturation used in this investigation*, the effective negative pore water pressure is the primary influence on the strength rather than the electrical forces. So, the increasing effective negative pore water pressure contributed to the strength in an increasing fashion with degree of saturation upto the optimum degree of saturation. In other words, as the degree of saturation increases (*i*) $\overline{U_w}$ decreases (i.e. it becomes less negative) and hence the effective contact stress, $\overline{C}$ decreases (*ii*) $\overline{U_w}$ acts over a larger area leading to increase in $\overline{C}$. This results in an optimum degree of saturation at which the shear strength is maximum. This is in agreement with the results of unconfined compression tests carried out by Venkatappa Rao (1975) and Venkatappa Rao and Sridhāran (1978). For both red earth and black cotton soil, they found that the degree of saturation at which the peak strength was observed practically corresponded to that at which the effective negative pore water pressure was estimated to be maximum. The effective negative pore water pressure rapidly decreased with a further increase in degree of saturation beyond the optimum degree of saturation. The electrical attractive force, A decreased and the repulsive force, R increased with the result the net (A—R) force, if any, also decreased. So, the net effective stress at failure decreased so also the strength upto 99 percent degree of saturation.

Strength Parameters

With the strength defined at maximum principal stress difference, Figs. 9 and 10 show the results of drained compression tests in the modified Mohr-Coulomb diagrames. The strength parameters, c and ϕ determined from the plotted envelopes are to be considered in the context of total stresses rather than the true effective stresses since the effective negative pore water pressure, $\overline{U_w}$ and R and A forces are not considered. The angle of shearing resistance ϕ remained approximately constant for a degrees of saturation except $S_r = 99$ per cent. Lower angle of ϕ for $S_r = 99$ may be attributed to the slight increase in the intial void ratio during resaturation. However, the cohesion intercept, c varied with the degree of saturation. This is as ought to be under the conditions in which the tests were carried out in this investigation. For the specimens

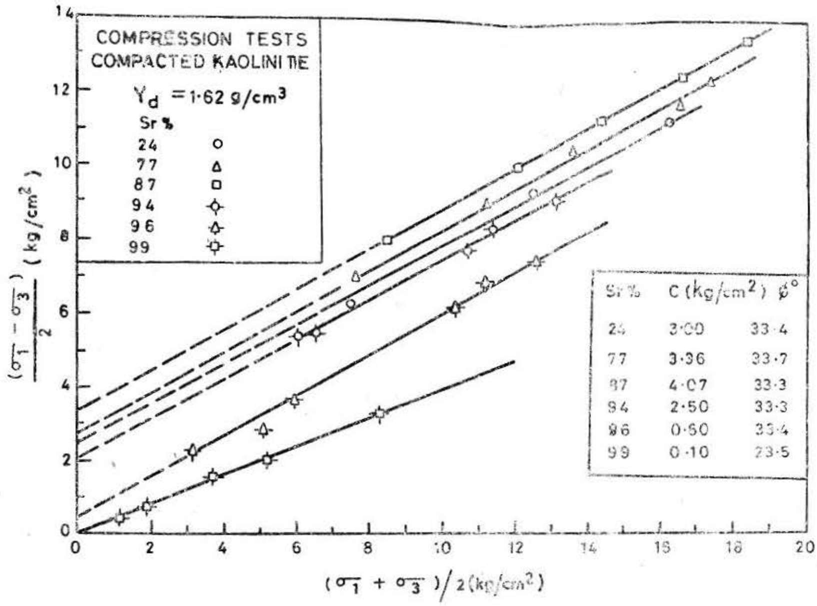

FIGURE 9 Modified Mohr-Coulomb Strength Envelopes for Kaolinite

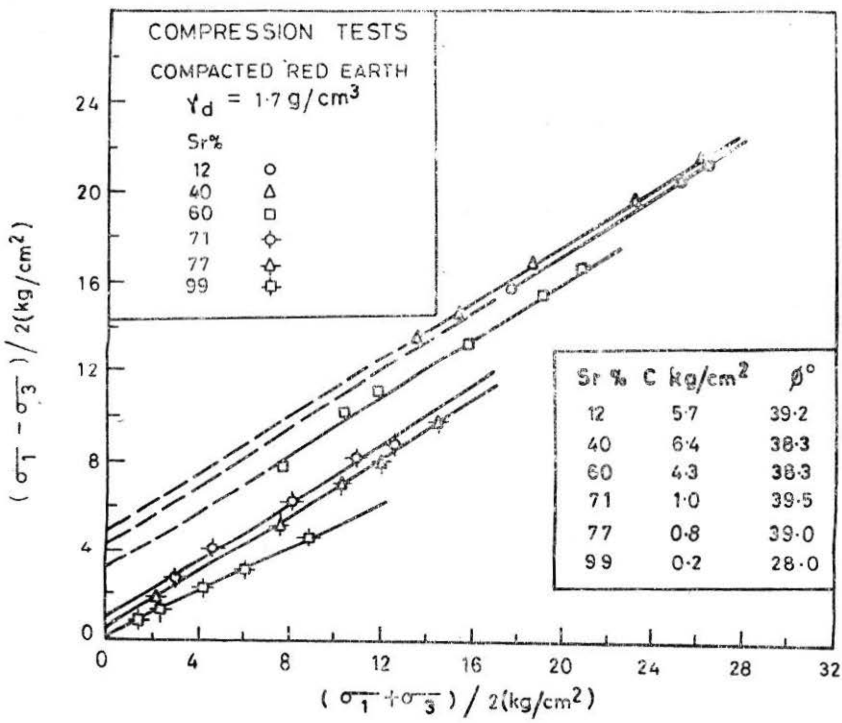

FIGURE 10 Modified Mohr-Coulomb Strength Envelopes for Red Earth

having constant dry density and fabric (in this investigation both have been maintained approximately constant), the angle of shearing resistance need not be affected by the changes in the degree of saturation. A change in the degree of saturation produces necessary change in the effective negative pore water pressure, $\overline{U_w}$ and R and A forces which have been reflected as change in the apparent cohesion. Thus, the apparent cohesion may vanish if the results are plotted truly on the basis of effective stress defined by the Eq. (1) From the tabulations inserted in the respective Figs. 9 and 10, it is further noticed that, starting from low degree of saturation, the cohesion intercept increased with the degree of saturation upto a certain optimum value and thereafter decreased rapidly with the degree of saturation.

According to Sridharan (1968), except under very low degree of saturation, variation in strength is primarily influenced by the variation in pore water tension rather than R and A forces. Further, the effective negative pore water pressure increased to a peak value at an optimum degree of saturation beyond which it decreased with the degree of saturation. Also from the consideration that the anagle of shearing resistance need not be affected by the degree of saturation, the variation in the cohesion intercept with the degree of saturation may be attributed to the variation in the effective negative pore water pressure at failure. This explains why the cohesion intercept with S_r upto a certain optimum value and thereafter decreased with S_r.

Making the following assumptions, one can relate the average effective negative pore water pressure at failure to the cohesion intercept.

(*i*) All shearing resistance in soils (except for cemented soils) is of a frictional nature (Crawford, 1963). In other words, if the true effective stress is known then the cohesion intercept should become zero.

(*ii*) The angle of shearing resistance is not affected by the degree of saturation. This could be considered reasonably a valid assumption in view of the fact that both fabric and void ratio (or dry density) have been kept constant. Moreover, the surface frictional characteristics are not affected by the degree of saturation.

(*iii*) For the range of degree of saturation used in this investigation, variation in strength is primarily influenced by variation in pore water tension and the area overwhich $\overline{U_w}$ acts rather than R and A forces (Sridharan, 1968).

With these assumptions, the cohesion intercept reflects nothing but the effect of negative pore water pressure at failure. For a fully saturated specimen, the Mohr-Coulomb envelope passes through the origin owing to the absence of any negative pore water pressure. But in an unsaturated condition, the negative pore water pressure is a function of the degree of saturation, thus producing a cohesion intercept in the Mohr-Coulomb diagram if the effective stress is not considered. So, the effective negative pore water pressure could be computed by shifting the shear stress axis laterally so that the cohesion intercept is zero. The distance, $c \cot \phi$ through which the axis is shifted provides an estimate of the effective pore water pressure at failure at any degree of saturation. Table 1 presents the computed values of $c \cot \phi$ ($= \overline{U} wf$) for different degrees of saturation. The effective negative pore water pressure as estimated by $c \cot \phi$ increased upto a certain optimum degree of saturation and thereafter decreased. The maximum estimated negative pore water pressures are 6.2 kg/cm² and 8.1 kg/cm² respectively in koalinite and red earth.

TABLE 1

Estimated Effective Negative Pore Water Pressure at Failure

Kaolinite		Red Earth	
S_r (%)	$c \cot \phi$ or $\overline{U}_{wf}$ kg/cm²	S_r (%)	$c \cot \phi$ or $\overline{U}_{wf}$, kg/cm²
24	4.55	12	7.0
77	5.05	40	8.1
87	6.2	60	5.4
94	3.8	71	1.2
96	0.9	77	1.0
99	0.23	99	0.38

Pore water pressures less than-1 atmosphere have rarely been measured and reported in engineering literature. The main difficulty involved in the measurement of negative pore water pressure is the cavitation of water in the measuring system at a pressure approximately equal to —1 atmosphere. Using the axis translation technique, Olson and Langfelder (1965) reported for as compacted condition the measured pore water pressures for five. different soils. Depending on the compaction procedure, water content, and the soils type of soil, the pore water pressures had a wide range from —0.35 kg/cm² to—17.6 kg/cm². Using capillary equation and pore size measurements, Sridharan (1968) estimated negative pore water pressures for compacted kaolin in the range 15.5-19.7 kg/cm² depending on the void ratio. Using capillary equation and pore size distributions, Venkatappa Rao and Sridharan (1978) estimated the negative pore water pressures for kaolinite and red earth. Depending on the void ratio and the degree of saturation, negative pore water pressures were in the range 8—28 kg/cm^2 and 3—220 kg/cm^2 respectively for kaolinite and red earth.

The high range of negative pore water pressure obtained by Sridharan (1968) and Venkatappa Rao and Sridharan (1978) is primarily due to the assumptions made in their calculations that the capillary equation is valid in the estimation of negative pore water pressure and for any particular S_r, the average pore size filled with water determines the magnitude of the negative pore water pressure. But in an experimental investigation like this, the effective negative pore water pressure at failure is primarily due to the pore water tension in the pores through which the failure plane passes. *(This implies that the effective negative pore water pressure need not be same throughout the specimen for the experimental conditions obtaining in this study).* Obviously the failure plane passes through the path of least resistance and so probably through the pores of large size. The corresponding effective negative pore water pressure is likely to be much less than that obtained by theoretical calculation. The theoretical calculation assumes that for any particular degree of sataration, the pores are filled with water uniformly to a particular level of pore size (average pore size) which is always less than the pore size that is actually filled in the test specimen. Therefore, the theoretical calculations have serious limitations in that they always estimate the negative pore water pressure on the higher side. The results obtained in this study cannot be compared directly with those of Sridharan (1968) or Venkatappa Rao and Sridharan (1978) as they were theoretically overestimated. However, the range of negative pore water pressures measured by Olson and Langfelder (1965) compre favourably with the results obtained in the present investigation.

Conclusions

The effect of degree of saturation, S_r on the normalized secant modulus, E_2/σ_c is significant. Depending on the type of soil and the consolidation pressure used, E_2/σ_c increased 10-30 times with the decrease in degree of saturation, thereby eliciting the importance of accurate determination of this parameter for use in any analytical procedure of design of earth structure.

At any degree of saturation, the drained compression strength increased linearly with the consolidation pressure, σ_c and these relationships are parallel at different degrees of saturation for $S_r = 99$ per cent. The test results also showed that for any consolidation pressure, σ_c the deviator stress at failure, $(\sigma_1-\sigma_3)_f$ increased to a peak value at an optimum degree of saturation beyond which it decreased. This has been qualitatively explained by the variation of effective negative pore water pressure at failure with the degree of saturation S_r.

The drained angle of shearing resistance remained almost constant but the cohesion intercept, c varied with the degree of saturation. This finding has significant practical implications. For the stability calculations of earth structures on effective stress basis, possibly the drained angle of shearing resistance in its partly saturated state should be considered neglecting the cohesion intercept if these earth structures are likely to get saturated in their life time.

Assuming that all shearing resistance is of frictional nature and that for a constant dry density and fabric the effective angle of shearing resistance is not affected by the degree of saturation, the effective negative pore water pressure at failure, $\overline{U_{wf}}$ has been estimated by relating it to the cohesion intercept. The values of $\overline{U_{wf}}$ are compared with some of the experimental results available from the literature and they are found to be in agreement within reasonable limits. Also it has been reasoned out why the theoretically calculated values are likely to be overestimated.

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Notations

A = Total interparticle electrical attraction divided by total interparticle area.

c = Cohesion, total stress

$\overline{C}$ = Effective contact stress

E_2 = Secant modulus at one-half of the maximum principal stress difference.

R = Total interparticle electrical repulsion divided by total interparticle area.

$\overline{U_a}$ = Effective pore air pressure

$\overline{U_w}$ = Effective pore water pressure

$\overline{U_{wf}}$ = Effective pore water pressure at failure

σ = Total stress

ϕ = Angle of shearing resistance, total stress

ϵ_a = Axial strain

ϵ_v = Volumetric strain