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Short Communication

Shear Strength Characteristics of Expansive Clays

by

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Introduction

Almost all the problems related to expansive soils are due to the unfavourable interactions between water and the soil. From an engineering view point, increased water content has a number of disadvantageous consequences resulting in decreased shear strength and swelling of the soil. Shear strength studies on swollen expansive soils are limited and different opinions have been reported by research workers from various parts of the world. The linear version of Mohr Coulomb's envelope is popularly practised.

However, for expansive clays, Katti (1978) reported bilinear relationship between strength and normal stress. The strength envelope consists of two straight lines with the second straight line (with a slope of ϕ_2') being flatter than the first straight line (with a slope of ϕ_1'). Katti (1978) further found that the normal stress (σ_b') at which these two straight lines interesect roughly corresponds to swelling pressure value of the soil and attributes the bilinearity to the likely changes in soil structure during shear. The direct shear test results of Babu Shanker and Sai Krishna (1989) have also shown bilinear strength envelopes for expansive clays. Fredlund *et al.* (1987) also reported bilinear nature of strength envelope for partially saturated soils saying that the expansive soils behave like partially saturated soils.

Experimental Study

The aim of the present study is to understand whether the strength behaviour of expansive clay is similar to non-expansive clay or different and also to verify the existence of bilinearity in strength envelopes, through laboratory

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consolidated undrained triaxial shear tests with pore pressure measurements. Representative soil samples collected from Tourist Guest House near Regional Engineering College, Warangal at a depth of 1.5 m have been used for the study.

The soil had the following index properties:

Liquid Limit (%)	...	63
Plastic Limit (%)	...	24
Shrinkage Limit (%)	...	10
Free Swell Index (%)	...	132
Sand (%)	...	31
Silt (%)	...	28
Clay (%)	...	41
Specific Gravity	...	2.70

The natural soil had been air dried and fraction passing 2mm was used. 38mm diameter and 76mm height cylindrical soil samples were prepared by static compaction in a constant volume mould with an initial moulding moisture content of 10% at void ratios of 0.9, 0.8 and 0.7. The soil samples were allowed to imbibe moisture under an ambient pressure of 0.07 kg/cm^2 (7 kPa) in the triaxial set up for 7 to 9 days (till saturation). Subsequently, the samples were consolidated under varying degrees of confining pressures (Selected from the range of 0.07 to 2.5 kg/cm^2 to have the soil samples tested on either side of swelling pressure values) for 2 to 3 days. After the completion of consolidation the samples were sheared under undrained conditions at a rate of 0.025 mm/min, recording the pore pressures developed.

Presentation and Discussion of Test Results

The results obtained from swelling and consolidation are shown in Fig. 1 for soil samples tested with e_i=0.8. The intersection of the confining pressure-void ratio curve with the initial moulding void ratio line gives the swelling pressure value obtained by the free-swell method.

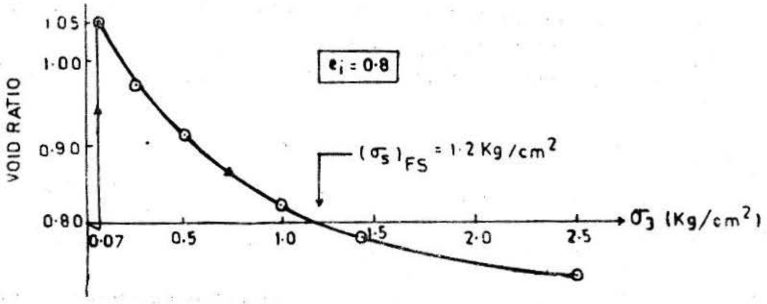

FIGURE 1 Consolidation Curve

Similarly, swelling pressure values of 0.9 and 1.8kg/cm² were obtained for soil samples moulded at initial void ratios of 0.9 and 0.7 respectively. It is observed that the swelling pressure values are higher for soil samples tested with lower void ratio. (Chen, 1975).

Fig. 2 gives the stress-stain relationships for soil samples tested with an initial moulding void ratio of 0.8. The deviator stresses were found to increase with increasing axial strain in a hyperbolic form (Kondner, 1963). The failure was observed to occur at 6-12% axial strains. The arrows in Fig. 2 indicate the maximum deviator stress values.

Shear strength envelope obtained for soil samples tested with initial moulding void ratio of 0.8 is shown in Fig. 3. The shear strength envelopes were observed to be of bilinear nature for soil samples tested with initial void ratios of 0.9 and 0.7. Table 1 gives the values of effective shear parameters obtained from the present study. The results obtained in an earlier study from consolidated drained direct shear tests on the same soil (Babu

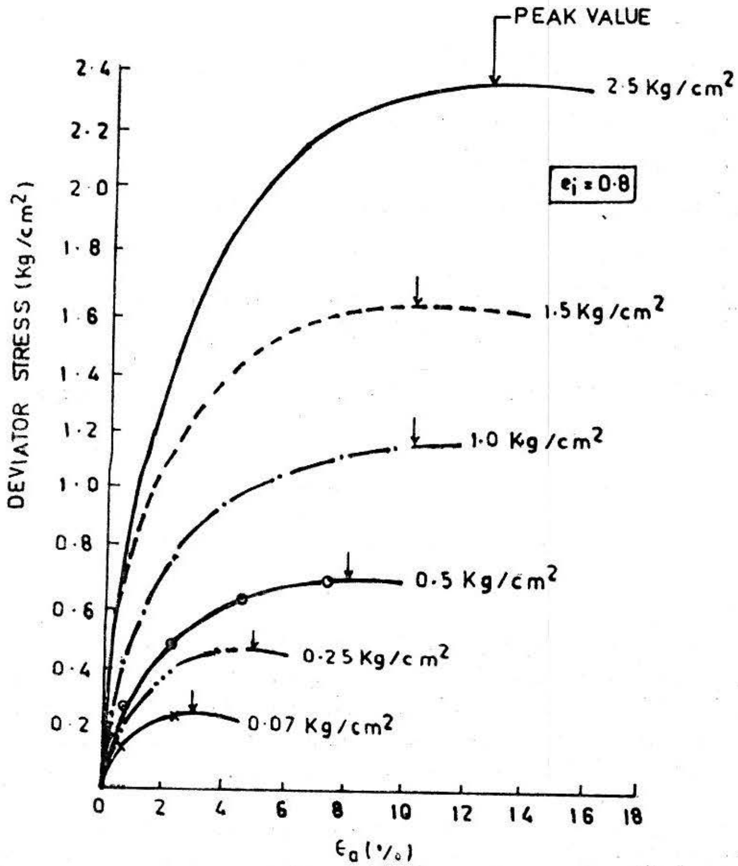

FIGURE 2 Stress Strain Relationships

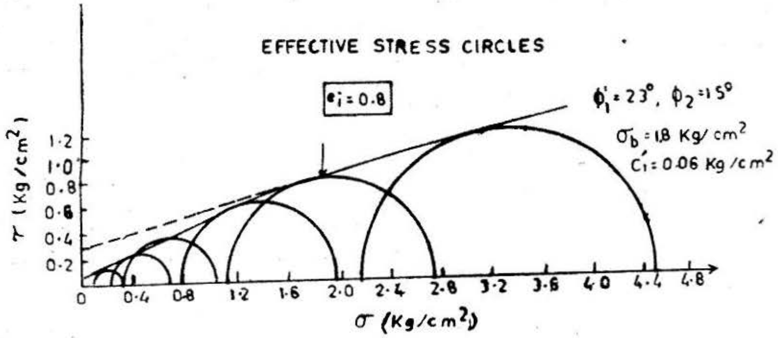

FIGURE 3 Mohr's Strenth Envelopes

TABLE 1

Effective Shear Parameters

Moulding void ratio, e_i	0.9	0.8	0.7
Cohesion intercept, c_i (Kg/cm^2)	0.05	0.06	0.09
First angle of internal friction ϕ_1' (degrees)	21.3	23.8	25.00
Normal stream at bilinearity σ_b' (kg/cm^2)	1.1	1.5	1.66
Second angle of internal friction ϕ_2' (degrees)	16.1	15.0	14.0

Shanker and Sai Krishna, 1989) have shown effective shear parameters values equal to $\pm 10\%$ of the values obtained in this study.

The shear strenth equations for expansive clays may be written as:

$$S = C_1' + \sigma' \tan \phi_1' \quad \text{for } \sigma' < \sigma_b'$$

$$= C' + \sigma_b' \tan \phi_1' + (\sigma' - \sigma_b') \tan \phi_2' \quad \text{for } \sigma' > \sigma_b'$$

It may be observed from Table 1, that the value of ϕ_2' decreases with increase in initial moulding void ratio value. It is of interest to note from Fig. 4 that inspite of bilinearity the failure envelopes obtained from the soil samples tested with the initial void ratios of 0.9, 0.8 and 0.7 were not overlapping for a wide range of confining pressures.

For all the tests conducted it was also observed that the bilinearity occurred (approximately) at a value equal to swell pressure on normal stress axis. Similar findings were reported by Katti (1978). The lower

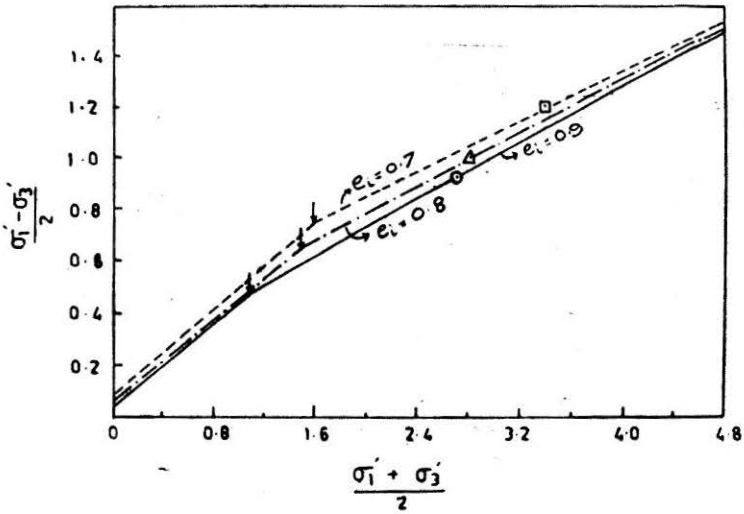

FIGURE 4 Modified Failure Envelopes of Triaxial Shear Tests

values of ϕ' for samples tested with confining pressures greater than swelling pressure value were attributed to structural changes by Katti (1979) during shear. However, Nagraj *et al.* (1980) discussing Katti's results attributed the bilinearity to Pseudo equilibrium state of opposite trend existing on either side of swelling pressure.

It was observed in this study that the pore pressure and hence pore pressure Coefficient (A) values were not uniformly increasing with the confining pressure as is generally the case for non-expansive clays (Fig. 5). The pore pressure changes may be due to the volume changes (dilatancy effects) exhibited by the soil during the shearing stage. Under confining pressures less than swell pressure, the soil behaves as a normally consolidated clay with loose structure having a tendency of developing higher pore pressure whereas under higher confining pressures soil behaves as over consolidated clay and has a tendency to undergo less volume changes and hence develops lower pore pressures. The variation of failure pore pressure parameter A_f with confining pressure shows the same tendancies (Fig. 6). Thus, it looks that the bilinearity due to expansive nature of soil is similar to that exhibited by non-expansive clays because of consolidation stress history. The swell pressure or the normal stress corresponding to bilinearity in expansive clays is similar to preconsolidated pressures in over consolidated clays.

It is seen that ϕ_1' values increase, with the decrease in initial moulding void ratio which is to be expected but ϕ_2' values are found to decrease with decrease in void ratio. This shows that ϕ_1' and ϕ_2' will probably merge at some initial moulding void ratio called as terminal void ratio (Fig. 7). The terminal void ratio value obtained from an earlier study in direct shear

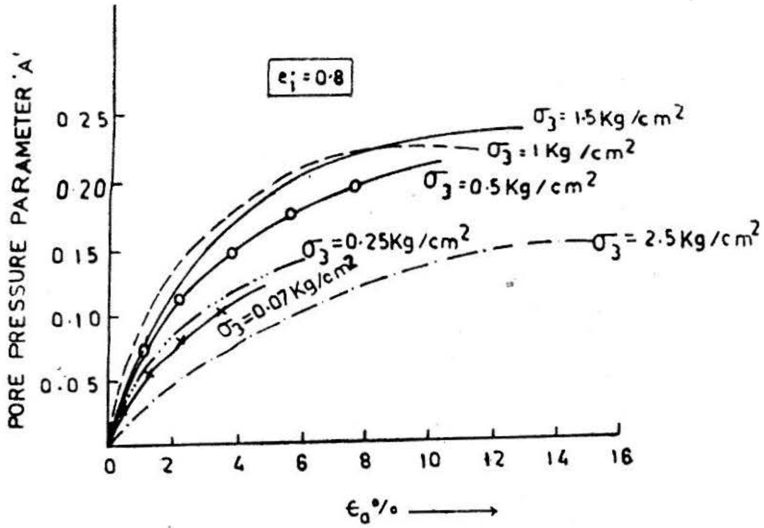

FIGURE 5 Pore Pressure Parameter A Vs. Axial Strain

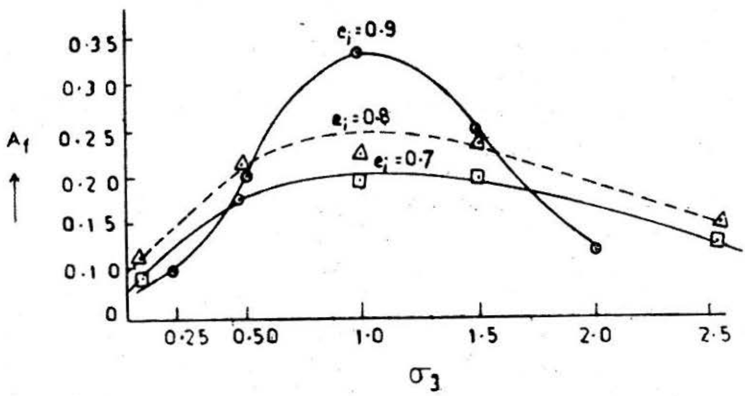

FIGURE 6 Failure Pore Press Parameter A_f Vs Confining Pressure

drained tests (Babu Shanker and Sri Krishna, 1989) was equal to 1.0. The bilinearity trends may not be observed for moulding void ratios higher than terminal void ratio. The absence of bilinearity for these samples may be reasoned due to low or negligible swell pressures. The results have been confirmed by taking up studies on soil samples with an initial moulding void ratio of 1.2 both in drained direct shear tests and triaxial shear tests.

While the phenomena of bilinearity and the reasons behind it have to be further microscopically studied, the macroscopic implications of such a phenomena have to be assessed. For the lightly loaded structures have a tendency to heave (swell) and hence in foundation design heave will be the governing criteria and not the strength. For this case $\phi' = \phi_1'$, since normal

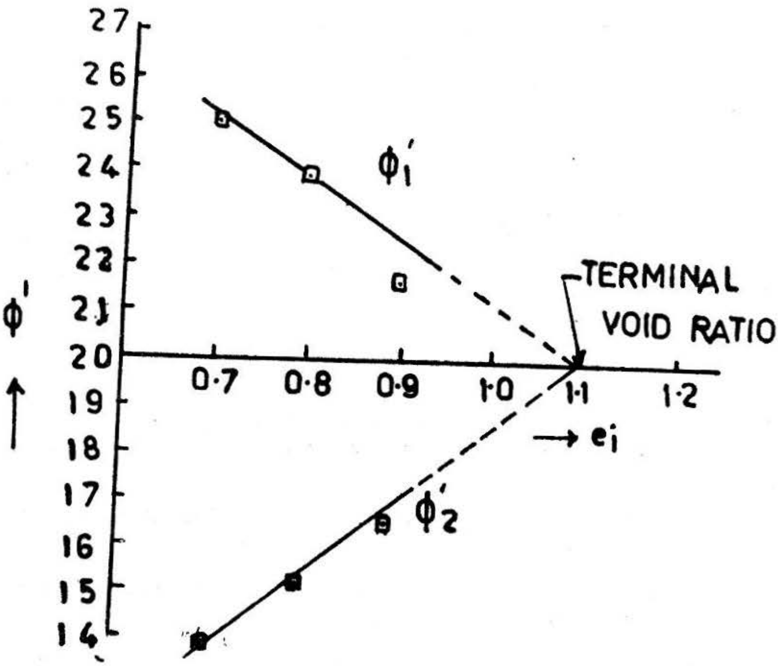

FIGURE 7 Effective Angle of Internal Friction Vs Void Ratio (From Modified Failure Envelopes)

stress is less than swell pressure ($\sigma' < \sigma_s' = \sigma_b'$). However, for heavily loaded structures including high earthdams, where $\sigma' > \sigma_s' = \sigma_b'$, the structures would not swell and hence strength and bearing capacity would be the governing criteria. For this range ϕ' value is equal to ϕ_2', which is less than ϕ_1' and hence there is a risk of over stressing the foundation if bilinear trends are not taken into consideration. In other workds, for the design of foundations is expansive soils, the shear strength has to be evaluated by conducting tests in the laboratory where the range of nromal or confining pressures must be wide enough such that they lie on either side of swell pressure values and the appropriate shear parameters are to be used in the design of foundations.

Conclusions

(1) Effective shear strength envelopes of expansive clays are of bilinear nature with the bilinearity occuring approximately around swell pressure value of the soil sample on normal stress axis.

(2) It is observed that ϕ_2' values are always lower than ϕ_1' for a particular void ratio. However, the values of ϕ_2' are found to increase with an increase in the initial moulding void ratio

(3) The bilinearity is attributed to the type of soil structure and the

nature of pore pressures developed during undrained shear. For the range of normal stress lower thsn σ_b', the expansive clays behave similar to normally consolidated clays and for normal stresses greater than σ_b', they behave as over consolidated clays.

(4) For expansive clay, the swell pressure (σ_s) as well as normal stress value at bilinearity (σ_b') decrease with increasing initial void ratio and hence there is a possibility that at some high initial moulding void ratio (called terminal void ratio) the soil may not exhibit any swell pressure and hence bilinearity will not be apparent and the strength envelope will be linear.

References

BABU SHANKER, N. and SAI KRISHNA, V. (1989): "Laboratory Studies on Shear Strength of Expensive Clays", *Proc. Indian Geotechnical Conf.*, Visakhapatnam, pp. 155-158.

CHEN, F.H. (1975) : "Foundations on Expansive Soils" Elsevier Scientific Publishing Company, Amsterdam.

FREDLUND, D.G. *et al.* (1987) : "Non-linearity of strength envelope for unsaturated Soils", Tech. Sec. II, *6th Int. Conf. on Expansive Soils*, India, pp. 48-55.

KATTI, R.K. (1978) : "Search for Solutions of Problems in black cotton soils", 1st IGS Lecture, *Indian Geo-Tech. Jl.*, Vol. 9, No. 1, pp. 1-78.

KATTI, R.K. *et al.* (1979): "Shear Strength Development in Expansive B.C. Soil media with and without a cohesive non-swelling soil surcharge", Applications to Stability of Canals in Cuts & Embarkments, Final Report, CBIP, New Delhi.

KONDNER, R.L. (1963) : "Hyperbolic Stress-Strain, response in cohesive soils", *Proc. ASCE*, Vol. 89, SMI pp. 115-144.

NAGENDER RAO, A. (1989) : "Shear Strength Characteristics of Black Cotton Soil", M.Tech. Thesis submitted to Kakatiya Univrsity, Warangal.

NAGARAJ, T.S., *et al.* (1980) : "Discussion on Selection of design parameters for expensive soils of six composite dams in Maharashtra" by Katti, R.K., *et al.*, *Proc. Indian Geotechnical Conf.*, Bombay.