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Short Communication

The Effective Stress in Clays

by

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Introduction

At present 'effective stress' is regarded as the stress that controls all changes in strength and volume in the soil. It was defined by Terzaghi (1923, vide Skempton, 1961) as the excess of the total applied stress over the pore water pressure and is expressed as:

$$\sigma' = \sigma - u \qquad \ldots (1)$$

where σ' is the effective stress and σ and u are the total applied stress and pore water pressure respectively. While this equation was formulated for saturated system it can also be extended to partly saturated systems where u represents the equivalent pore pressure.

This definition of effective stress was initially considered applicable to all soil systems (both cohesionless as well as cohesive soils) although it was recognized by some investigators (Skempton, 1961) that the intergranular stress was a better representation of effective stress particularly at very high stress levels. However, the particle contact area at the usual stress levels met within geotechnical engineering is very small so that Eq. (1) is a very good approximation for intergranular stress when no other forces are assumed to be present within the soil system.

The surface forces present in clays are of sufficient magnitude, particularly when the ratio of surface charge to mass is high (Rosenqvist 1955 Lamba 1960, Van Olphen 1963, Sridharan, 1968), so as to influence the strength and volume change behaviour (Rosenqvist 1955, Olson and Mesri 1970, Mesri and Olson 1971, Sridharan and Venkatappa Rao, 1973, 1979). Equation (1) does not provide for the interaction of these surface forces between particles.

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As a result of the interaction of the electrical charges present in clays, repulsive and attractive forces are mobilized between particles. The repulsive forces are primarily due to the interaction of diffuse layers of hydrated counter ions surrounding the charged particles while the attractive forces are chiefly Van der Waal's forces when the particles are not in contact. Coulombic attraction can be the prime attractive force between oppositely charged edges and surfaces of particles if they are in contact.

Defining effective stress as the excess of the applied stress over the equivalent pore pressure u, Lambe (1960) accounted for the effect of the surface charge in clays in the following expression,

$$\sigma' = \sigma - u = \sigma_m \, a_m + R - A \quad \ldots (2a)$$

where σ_m is the intergranular stress and a_m is the mineral contact area per unit area of the reference section. R and A represent the repulsive and attractive forces due to the charged nature of the clay particles per unit area of the reference section. For well dispersed clays Lambe (1960a) suggested that interparticle contact is absent and

$$\sigma' = \sigma - u = R - A \quad \ldots (2b)$$

Other investigators (Sridharan 1968, Chattopadhyay 1972, Matsul *et al.* 1980 and Mitchell 1976) defined effective stress as the intergranular or contact stress and the expression for this stress takes the general form as

$$\sigma' = C = \sigma_m \, a_m = \sigma - u + A - R \quad \ldots (3)$$

where $C = \sigma_m \, a_m$ is the contact or intergranular stress. The expression of Matsui *et al.* (1980) includes additional terms to account for the effects of dilatancy, stress history, suction and Cementition.

Defining effective stress as the contact stress results in the electrical forces R and A playing an entirely different role in determination of magnitude of σ' in Eq. (3) compared to that in Eq. (2a). Thus, with respect to cohesive soils two entirely different expressions for effective stress exist in literature.

This note identifies the most appropriate interpretation for effective stress and investigates its role in controlling the strength and volume change behaviour of soils.

Proper Interpretation for Effective Stress in Cohesive Soils

Of the two basic forms of expressions for effective stress in cohesive soils, the most appropriate form can be arrived at by a process of elimination.

In Eq. (2), excess of the total stress over the pore pressure is the effective stress. For a system wherein interparticle contact is assured it is seen that when σ and u are kept constant, changes in the magnitudes of R and A cannot result in an effective stress change and will be compensated by appropriate changes in $\sigma_m\, a_m$. This is contradicted by reported observations on shear strength and volume change (Rosenqvist 1955, Olson and Mesri 1970, Mesri and Olson 1971, Sridharan and Venkatappa Rao 1973, 1979) which clearly indicate that changes in R and A forces result in shear strength and in equilibrium void ratio when $(\sigma—u)$ is constant.

The inability of the R and A forces to influence the effective stress when $(\sigma-u)$ is constant also exists in Eq. (2*b*) proposed for well despersed saturated clays. Further, Eq. (2*b*) specifies that the shear strength of a cohesive soil arises from the viscous resistance offered by the fluid separa-ing parallel oriented clay platelets. For such a particle configuration, wherein interparticle contact and hence friction is absent, the repulsive force R bornout of diffuse layer interactions far exceeds all possible sources of interparticle attraction which can therefore be neglected (Sridharan and Jayadeva, 1982). Although the viscosity of the fluid between the platelets increases with R force, it has been established (Allam and Sridharan, 1984) that it is too meagre to account for the shear strength of the most active clay. Therefore the shear strength of clays must be frictional in origin implying that interparticle contact exists.

Equation (3) and expressions of a similar nature (Matsui *et al*, 1980) regard intergranular or contact as effective stress. Changes in the R and A forces at constant $(\sigma-u)$ result in changes in effective stress. Thus there is a scope for the R and A forces to play a role in determining the effective stress.

It can therefore be concluded that for clays contact stress is the correct interpretation of effective stress—as has been established for cohesionless soils at high stress levels (Skempton, 1961).

Relationship between Effective Stress and Shear Strength

It has been well established that the drained shear strength increases with the applied normal stress on the failure plane for saturated choesion-less soils and also for those saturated cohesive soils in which the pore fluid dharacteristics have kept constant. It can be inferred that the shear strength increases with effective (intergrannular or contact) stress.

At constant applied normal stress, the contact or intergranular stress in saturated clays (Eq. 3) can be also increased either by reducing the R forces (e.g. by increasing the valency of the adsorbed cations and/or the ion concentration in the pore fluid) or by increasing the A forces while simultaneously reducing the R forces (e.g. by decreasing the dielectric constant of the pore fluid). Strength increases on substitution of the adsorbed cation with one of higher valency have been reported (Rosenqvist, 1955). Likewise, a decrease in the dielectric constant of the pore fluid results in greater shear strength (Sridharan and Venkatappa Rao, 1979).

It can therefore be concluded that all changes in the different variables (i.e. σ, u, R and A) which result in an increased effectives (contact) stress bring about an increase in the shear strength—changes in shear strength are consistent with changes in effective (contact or intergranular) stress.

Volume Change Behaviour Related to Changes in Effective Stress

Abundant experimental data exists which establishes that as the applied normal stress σ increases the equilibrium void ratio decreases in saturated cohesionless and cohesive soils. In these experiments only the applied normal stress was altered while other varlables (e.g. u in cohesionless soils or u and pore fluid properties in cohesive soils) influencing effective (intergrannular) stress were unchanged. Thus increases in effective stress brought about soley by increasing σ result in reductions in the void ratio.

However, there are other ways of affecting a change in effective stress apart from varying σ a stated in the preceding sections.

Collapse of partly saturated silt occurs on soaking under constant load σ (Jennings and Burland, 1962). This has been attributed (Leonards, 1962) to following a decrease in the intergranular stress as a result of the negative pore water pressure in the partly saturated silt vanishing on soaking. Thus a volume decrease occurs when the effective (intergranular) stress decreases for cohesionless soils.

Experimental investigations have shown that reductions in the ion concentration of the pore fluid results in saturated homoionic kaolinite and montmorillonite equilibrating at higher void ratios for constant σ and u (Olson and Mesri 1970, Mesri and Olson, 1971). Since a reduction in ion concentration results in increased interparticle repulsion arising from the interection of diffuse layers of counter ions while the interparticle attractive forces are marginally affected, it is seen from Eq. (3) that a decrease in effective stress takes place. Thus an increase in void ratio is observed for a decrease in effective stress achieved by increasing the R force.

On the other hand an increase in the dielectric constant of the pore fluid (attained by using different organic pore fluids) which brings about an increase in the R forces and a decrease in the A forces has been reported (Olson and Mesri 1970, Sridharan and Venkatappa Rao, 1973) to result in higher equilibrium void ratios in the case of montmorillonite and lower equilibrium void ratios for kaolinite at constant σ and u. Equation (3) indicates a reduction in the effective stress results in these cases.

Similarly for low to moderate values of σ, montmorillonite or clays containing a high proportion of montmorillonite swell on soaking where as kaolinite collapses (Sridharan *et al.* 1973). Soaking results in an increase in the R forces and a decrease in the A forces since the dielectric constant increases from unity (for air filled voids) to nearly 80 (that for water in the saturated voids). A net decrease in the effective stress under constant σ results on soaking (Eq. 3). Therefore, depending upon the clay mineral, both a volume increase and a volume decrease can accompany a decrease in effective stress achieved by simultaneously varying the R and A forces.

It is thus seen from an examination of published data that the types of volume change brought about under constant σ in both cohesionless and cohesive soils by altering the other component stresses of effective stress are not consistent. Both volume increases and decreases are brought about by a reduction in effective stress depending on the manner in which the change was effected. While a consistent relationship exists between strength behaviour and effective stress change, this is not true for volume change behavior. Therefore effective stress may be redefined as the stress controlling strength changes only.

Mechanism for Volume Change

Volume changes take place in cohesionless soils as a result of sliding at particle contacts. The mobilization of shear strength in clays has been attributed to development of friction at particle contacts (Allam and Sridharan, 1984). Therefore, the contact model should be also valid for examining the volume change behavior of clays mechanistically.

Discussing the phenomena of collapse in partly saturated silt on saturation under constant normal stress, Leonards (1962) concluded that the collapse was the resultant deformation in the direction of the applied normal stress arising from individual slippages at the particle contacts which occurred when the shearing resistance at these points decreased. This shearing resistance decrease was ascribed to a decrease in the contact or intergranular stress at these points as a result of the pore water pressure in the partly saturated silt increasing from a negative value to zero on soaking.

From the collapse behavior of partly saturated silt the following features are observed : Firstly, a volume change resulted when a change in the contact or intergrannular stress existing in the system occurred. Secondly, this contact stress change was not brought by a change in the applied normal stress but by a change in the pore water pressure which is not unidirectional. Thirdly, the volume change occurred in the direction of the applied normal stress and resulted from sliding deformations at the contact points between particles.

From these observations a mechanism for volume change can be postulated.

Volume Change Mechanism

Volume changes occur primarily as a result of sliding deformations at particle contacts. The stresses in the soil system can be grouped into a deforming component consisting of the excess of the applied normal stress over opposing it (for example a positive pore water pressure or interparticle electrical repulsion) and a resisting component that imparts an attractive force at particle contacts (for example electrical attractive forces between particles, negative pore water pressures and cementing bonds). When the soil system is in equilibrium (that is no volume change takes place) the ratio of the resisting to the deforming components has a limiting value. This ratio is henceforth termed the "Slide Ratio". A volume decrease takes place if the slide ratio decreases and vice versa.

For partly saturated silt, a negative pore water pressure imparts an interparticle attraction at the contacts and thereby forms the resisting component. Thus when the silt was saturated the resisting component decreased while the deforming component (σ) remained unchanged so that the slide ratio decreased from the limiting value and collapse occurred.

Using this mechanism the volume change behaviour reported for clays can be analysed. For any value of σ a decrease in the ion concentration of the pore fluid results in greater electrical repulsion between particles. No significant change in the A forces is anticipated since the system's dielectric constant is paractically unaltered (Gouy-Chapman theory of the double layer). Hence the slide ratio increases. Swelling is predicted by the mechanism and this is corrobarated by the experimental data on kaolinite and montmorrillonite (Olson and Mesri 1970, Mesri and Olson, 1971).

Similarly an increase in the dielectric constant (treating pore fluid ion concentration as constant) results in greater R and lesser A forces. For a clay whose positively charged edge area is very small compared to the negatively charged surface area, the increase in the R forces and hences decrease in the deforming component is comparably in excess of the decrease in the A forces and thereby in the resisting component. The slide ratio increases and swelling is predicted. This prediction is verified by the behavior exhibited by montmorillonite when the dielectric constant of the pore fluid is increased (Olson and Mesri 1970, Sridharan and Venkatappa Rao, 1973). For a clay (e.g. kaolinite) whose positively charged edge area is significant compared to the negative surface area, a decrease in the deforming component is accompanied by a significant decrease in the resisting component. If, as a result, the slide ratio is less than the limiting value a volume decrease can occur. This has been observed for kaolinite when the dielectric constant is increased (Sridharan and Venkatappa Rao 1973, Olson and Mesri, 1970).

It is of interest to note that the deforming and resisting components can have different magnitudes for the same effective stress. Further, different fabrics can have different values for the slide ratio at equilibrium. Thus a soil can equilibrate at different void ratios for the same effective stress (although the magnitudes of σ, u, A and R will be different). Since the shear strength is related to effective stress, the soil at these void ratios will possess the same shear strength. The data in Table 1 illustrate this feature and present some of the infinite equilibrium void ratios that yield the same shear strength for different values of σ. The forces A and R are different for each case while u is constant (atmospheric pressure).

TABLE 1

Equilibrium Void Ratios for Kaolinite and Montmorillonite at Constant Shear Strength (τ^{**} = constant = 41.4 k Pa)

Soil / Pore fluid	Kaolinite		Montmorillonite	
	σ^* (*k Pa*) (Total stress)	e	σ^+ (*k Pa*) (Total stress)	e
Hexane	27.3	1.340	52.50	1.580
Heptane	30.45	1.335	53.55	1.570
Carbon Tetra-chloride	35.42	1.330	—	—
Benzene	42.00	1.327	—	—
Ethyl Acetate	57.40	1.310	—	—
Acetone	63.00	1.170	—	—
Ethyl Alcohol	*66.85*	*1.185*	64.75	1.850
Methyl Alcohol	75.95	1.120	67.55	1.950
Water	88.20	0.895	86.45	3.850

(* σ vs. e—Sridharan and Venkatappa Rao, 1973)
** σ vs. τ—Sridharan and Venkatappa Rao, 1979)

It is also conceivable to vary the resisting and deforming components while the effective stress remains constant. This will result in volume change occurring since the slide ratio value changes from limiting value for that void ratio and fabric. Hence a changes in effective stress is not always required to effect a change in volume. The different failure void ratios in Table 1 for the same shear strength (and hence effective stress) indirectly substantiate this argument.

Conclusions

An examination of the definition and interpretation of effective stress as applied to clays indicates that,

(1) For the electrical surface forces present on clay particles to have a role in determining the behaviour of clay, the proper interpretation of effective stress is the contact or intergranular stress between particles.

(2) Effective stress may be defined as the stress controlling all changes in streagth alone. There is no consistent relation between the nature of volume change and change in effective stress. Volume changes can also result under a state of constant effective stress.

(3) Volume changes can be explained mechanistically using a contact model. The different constituents of effective stress can be classed into a resisting component and a deforming component. When the ratio of resisting to deforming components termed 'slide ratio' decreases below the limiting value at equilibrium compression take place and vice versa.

References

ALLAM, M.M. and SRIDHARAN, A. (1984): "The Shearing Resistance of staturated Clays—Technical Note," *Geotechnique*, 34 : 1.

JENNINGS, J.E.B. and BURLAND, J.B. (1962): "Limitations to the use of Effective Stresses in Partially Saturated Soils." *Geotechnique*, 12 : 2 : 125-144.

LAMBE, T. W. (1060): "A Mechanistic Picture of Shear Strength in Clay." *Proc. Am. Soc. Civ. Engrs. Res. Conf. on Shear Strength of Cohesive Soils, Boulder*, Colorado, pp. 555-580.

LAMBE, T.W. (1960a): "Discussion on Factors Controlling the Strength of Partly Saturated Cohesive Soils." By A.W. BISHOP, I. ALPAN, G.E. BLIGHT and I.B. DONALD, *Proc. Am. Soc. Civ. Engrs. Res. Conf. on Shear Strength of Cohesive Soils, Boulder*, Colorado, pp. 1094-1095.

LEONARDS, G. (1962): "Correspondence". *Geotechnique*, 12 : 4 : 354-355.

MATSUI, T., ITO, T., MITCHELL, J.K. and ABE, N. (1980): "Microscopic Study of Shear Mechanics in Soils." *Journal of the Geotechnical Engineering Division*, ASCE, 106 : GT 2 : 137-152.

MESRI, G. and OLSON, R.E. (1971): "Consolidation Characteristics of Montmorillonite." *Geotechnique*, 4 : 341-352.

MITCHELL, J. K. (1976): "*Fundamentals of Soil Behavior.*" John Wiley and Sons, New York.

OLSON, R.E. and MESRI, G. (1970): "Mechanisms Controlling the Compressibility of Clay." *Journal of Soil Mechanics and Foundations Divisions*, ASCE, 96 : 6 : 1863-1878.

ROSENQVIST, I. TH. (1955): "Investigations in the Clay-Electrolyte-Water System." *Publication No. 9, Norwegion Geotechnical Society*, Oslo.

SKEMPTON, A.W. (1961): "Effective Stress in Soils, Concrete and Rocks." *Conference on Pore Pressure and Suction in Soils,* London, Butterworths, pp. 4-16.

SRIDHARAN, A. (1968): "*Some Studies on the Strength of Partly Saturated Clays.*" Ph. D. Thesis, Purdue University, Indiana.

SRIDHARAN, A. and JAYADEVA, M.S. (1982): "Double Layer Theory and Compressibility of Clays." *Geotechnique*, 32 : 2 : 133-144.

SRIDHARAN, A. VENKATAPPA RAO G. and PANDIAN, R.S. (1973): "Volume Change Behaviour of Partly Saturated Clays during Soaking and the Role of the Effective Stress Concept." *Soils and Foundations*, 13 : 3 : 1-15.

SRIDHARAN, A., and VENKATAPPA RAO, G. (1973): 'Mechanisms Controlling Volume Change of Staturated Clays and the Role of the Effective Stress Concept,' *Geotechnique*, 23 : 2 : 359-382.

SRIDHARAN, A., and VENKATAPPA RAO, G. (1979): 'Shear Strength Behaviour of Saturated Clays and the Role of the Effective Stress Concept,' *Geotechnique*, 29 : 2 : 177-193.

VAN OLPHEN, H. (1963): "*Clay Colloid Chemistry*." Wiley Intersc ience New York.

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