About the Lecturer - IGS 1993

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Dr. Nagaraj is the recipient of IGS - Kueckelman award 1991 for his outstanding contributions to Geotechnical Engineering and the CBIP - Pt. Jawaharlal Nehru Birth Centenary Research Award, 1992. He is an elected Fellow of Indian National Academy of Engineering, Fellow of Institution of Engineers (India), Member, Indian Concrete Institute, Member of the Internatinal Society of Soil Mechanics and Foundation Engineering, Fellow, Indian Geotechnical Society, elected Member of Executive committee of the Indian Geotechnical Society and Editor, Indian Geotechnical Journal.

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Effective Stress in Soils

by

T.S. Nagaraj*

General Introduction

Among all the civil engineering construction materials, soils, due to the inherent nature and diversity of geological processes involved in their formation itself, exhibit the greatest degree of variability in their in-situ states. It is quite often necessary for the engineer to analyse and assess the engineering properties of the materials contemplated for practical usage. The manufactured materials can fulfill the stipulated requirements within certain limits. In such cases tests on a few representative samples may be adequate for utilizing large quantities of similarly composed materials. On the contrary, in soils, only a minute fraction can be sampled and tested because of time and economical constraints even though the variability is much more. For example, even if the spacing of bore hole is 10m and a 50mm sample is tested every second meter, only one millionth of the total volume of soil would have been explored.

Most often soils are engineered as they exist and hence judgement and property predictions play a pivotal role in geotechnical engineering. In contrast to many other branches of engineering where one would normally specify the requirements of materials to be used, the geotechnical engineer usually has to adjust his designs to suit the prevailing properties of the in-situ soils. Hence innovative approaches which can effect significant economies can be practised, if the soil variability in terms of soil parameters can be realistically arrived at.

In geotechnical engineering field also, a few empirical relations correlating mechanical properties with simple inferential parameters have

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been in vogue, and have almost become indispensable tools because of their simplicity. These empirical relations are essentially based on experimental observations without probing into the why or how of the correlations. The fact that these methods often lack theoretical basis restricts their usage to limited type or location or condition of soils and further generalizations could be sometimes unacceptable. Some of the relations widely used are :

- Skempton's (1944) compression index equation and its modified forms to describe the compressibility of soils.
- c_u / p vs. I_p relationship (Skempton 1953) to get the undrained strength of natural soils.
- Remoulded strength vs. liquidity index relationship (Houston and Mitchell 1969).

Though there have been many rigorous scientific attempts by molecular and phenomenological approaches, independently, to understand and model soil behaviur, the attempts have not resulted in completely satisfactory predictive models. What is desirable and practicable is perhaps to have a qualitative understanding of the mechanism involved at micro level, identifying the effects of these processes at macro-level and, finally, arriving at the analytical formulation at macro level itself involving easily measurable macro parameters. Soils being particulate materials, with void ratios quite often greater than one, are highly compressible. The shear strength which is synonymous with the strength of the soils is dependent on the compression of soils as stresses become effective. Permeability characteristics of soils control the rate at whch the applied stresses become effective. For any rational approach to solve stability, settlement and flow problems in soil engineering, strength, compressibility and permeability characteristics are to be determined. How far the effective stress principle enables to analyze the above aspects of soil behaviour merits examination.

The Principle of Effective Stress

Although soils are regarded as continua, in reality they are multicomponent, multiphase particulate materials with the possibility of water in pore spaces under pressure when subjected to stresses. An element of soil will have a set of total stresses acting on the boundaries and a pore pressure acting within the element. The priciple of effective stress (Terzaghi, 1936) determines the effect of applied stresses on the behaviour of a soil, with a given total stress. This principle is probably the single most important concept in soil mechanics and its importance needs hardly be stressed.

The Terzaghi's (1936) principles of effective stress is stated in two parts. In the first part the fundamental effective stress equation is defined as :

$$\sigma' = (\sigma - u) \tag{1}$$

As stated by Terzaghi, the principle of effective stress is deceptively simple. Of the terms in the above relation only the total stress can be directly measured. The induced pore water pressure is measured at a point away from interparticle zone. Hence the effective stress σ', is a deduced quantity.

The second part of the principle enunciates the significance of the effective stress as :

> **"All measurable changes in volume, deformation and mobilization of shearing resistance are exclusively due to changes in the effective stress"**

This formulation for quite sometime has been assumed to be valid for all types of soils, provided, they are essentially in saturated condition. Subsequently, it was recognized that the above 'classical' approach needed reexamination to consider the role of interparticle surface forces, especially when dealing with fine grained soils with appreciable clay fraction (Lambe, 1960; Parry, 1959, 1965; Trollope, 1960; Sridharan, 1968).

As typical to any of the basic construction material, the fundamental behaviour of soils can also be studied from the mechanics point of view. For this, from mechanisms point of view, an understanding of the forces at play and their origin are required. A detailed understanding of the nature of solid and liquid phases and the mutual interactions at work is a pre- requisite for the same. The discrete solid particles of soils are not strongly bonded as the crystal structure of solids, and are free to have relative movements which are themselves constrained due to internal resistance and are not as free as the molecules of a fluid. Hence the responses of soils cannot be fully characterized either by solid mechanics or by fluid mechanics. Feda (1982), has described the mechanics of particulate materials as the appropriate mechanics for characterizing the responses of soils. The particulate materials are those which exhibit dilatancy and contractancy and are sensitive to hydrostatic stresses.

Fundamentals of Soil Behaviour

Nature of Soil Solid Particles

Soils, in nature, encompass a wide range of particles of sizes, ranging from coarse sand of 2mm to clay colloids of the smallest size of stable unit being 10 AU (1 AU = 10^{-8} cm). It would be hard to encounter any other particulate material of engineering interest with the ratio of biggest to smallest size in the range of 2 million times. As an analogy, if the biggest 2mm size particle can be compared to the size of earth, then the smallest stable clay unit size would be that of medium size boulder, the ratio of which is of the same order. In soil mechanics clays are distinguished from sands mainly from particle size considerations. It so happens that along with this difference in particle size go a large number of other differences such as particle shape and specific surface, mineralogy and associated physio-chemical properties particularly the specific surface area associated with particles of smallest size units can be of the order of 800 m^2/gm compared to 2×10^{-3} m^2/gm for sand of 2mm size particles. As an analogy, the vast surface area of clays can be visualized from the fact that about 12g of bentonite clay would suffice to cover a foot ball field. Generally, sand particles are bulky and isometric in shape and exist as individual units. On the other hand, clay minerals are hydrated aluminium silicates and hydrous oxides of aluminium, magnesium and iron in a crystalline form of relatively complicated structure. Clays range in mineralogical composition from kaolins, made up of individual particles which cannot be readily subdivided, through illites to montmorillonites which consist of particles made up of a stack of more or less parallel platelets.

From an engineering stand point, soil devoid of clay and silt fraction are regarded as sands (coarse grained) and those comprising of broader range of particles with appreciable clay fraction are clays (fine grained soils).

Soil-Water Interactions

It has been very well established that the surfaces of soil solid particles carry electrical charges. Since water molecules are dipolar in nature, it is logical to expect electrostatic interactions between them with the level being defined by the quantum of surface charge per unit mass. In coarse grained soils the magnitude of surface charges per unit mass is relatively low. The presence of water does not contribute significantly to the internal force field except changing the unit weights. On the contrary, fine grained soils, especially clays, are strongly influenced by the presence of water because of their high surface activity. The clay-water interaction results in a tendency

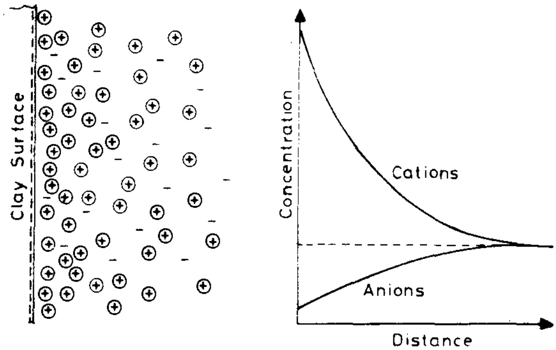


FIGURE. 1 Ions distribution adjacent to clay particle surfaces

of the counter ions (resulting from the surface ionization of clay particle) and other dissociated ions (exchangeable cations present in the water), to diffuse away from the surface, which will be counter balanced by the electrostatic attraction. This defines the concentration of ions at any point since the electrostatic attraction falls off with the distance from the surface. The concentration of the attracted cations will also diminish while the concentration of counter ions increases with distance from the surface (Fig. 1). Such a distribution of ion concentration with distance resembles 'atmospheric distribution'. The charged surface and the strongly held cation at the surface together with the relatively mobile counter ions in the medium adjacent to the surface are considered to be two layers. Hence the whole system is referred to as the 'diffuse double layer'. The concentration of charges is greater near the surface and diminishes with distance.

Interparticle Interactions

Between two units of matter, there always exist interaction forces of both attractive and repulsive nature. These can be grouped as short range and long range forces. Depending on the dominance of one group, the effect of other group may not be felt or may provide combined effect of both. While the long range forces can act over a distance of several hundred angstroms, the action radius of short range forces are only a few angstroms. The typical short range forces are the homopolar, the heterpolar, the van der Waals-Londn attractive forces, the hydrogen bonding and the born repulsive forces. The typical long range forces are the van der Waals-London attractive forces and Coulomb electric repulsive forces. The short range forces represnt the free surface energy brought into play by the interphase boundary i.e., by the failure of crystal lattices on their surface and hence are bound to that surface. The magnitude of short range forces varies as the inverse seventh power of the distance (r^{-7}) between two solids.

The long range forces originate between particles of colloidal dimensions. A typical example of this is the clay particles in an aqueous medium which conditions the dispersion of the particles themselves. The magnitude of long range attractive forces are inversely proportional to the third power of the distance between the particles (r^{-3}), while the repulsive forces are inversely proportional to the second power of the distance (r^{-2}). Hence, of the long range forces, the repulsive forces are dominant.

Physical and Physico-Chemical Interactions

In the case of coarse grained soils short range forces are dominant. The mutual interactions between particles is essentially physical as depicted in figure 2 (Leonards, 1962). While the macroscopic view of two particles in close promimity appears to be as if between two polished surfaces, in reality at sub-microscopic level the same could be jagged and irregular (Fig. 2b). This is so because of the microroughness of these solid surfaces upto heights of several thousands of angstroms. Such surfaces can touch each other only through the apexes of microroughness, and hence the potential to form contact junctions. At these points, only short range forces are at play and the long range forces are insignificant, since at every other point than in the vicinity of the contact junctions, the distance between the surfaces far exceeds the action radius of the long range forces. Due to normal stresses, contact junctions are formed as shown in figure 2c. These contact junctions offer resistance to sliding or rolling and hence they are regarded as contact or friction bonds. Generally, friction is defined as solid friction between two surfaces with negligible moisture. Since the solid surfaces are generally not clean but masked by an adsorbed layer of hydrated ions, or contaminants, friction at the contact bond will be different than solid friction. As the physico-chemical interactions between sand and water is very low, the resistance offered by contact bonds is not radically different due to the presence or absence of moisture, provided, the response is not influenced by pore water pressure.

In the case of fine grained soil solids, the micro-roughness along the basal planes is of the order of 10 – 100 AU. Further, diffuse double layer of thickness greater than that of micro-roughness originates on these surfaces in an adequate aqueous environment. This inhibits the apex contacts making the short range forces insignificant. Of the long range forces, the coulomb forces of electric double layer, repulsive in nature, are dominant over the van der Waals attractive forces. Accordingly, the long range forces can theoretically keep the fine grained stable particle units separated, preventing them from being in direct physical contact.

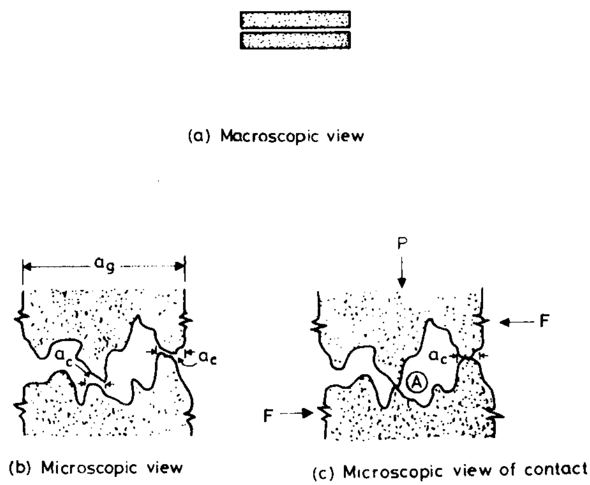


FIGURE. 2 Frictional shearing resistance due to contact junctions (Leonards, 1962)

Net force between particles

As brought out earlier there exist both attractive and repulsive forces between colloidal clay particles in dilute aqueous suspensions containing electrolyte. The net force at a given separation distance is the algebraic sum of the repulsive and atractive forces acting at this separation distance. Sinc the van der Waals - London (attractive) force is insensitive to the properties of the separating medium, while the repulsive forces are sensitive it follows that the net force of interaction between clay particles can be varied directly by varying the properties of the medium and thus the repulsive force between the interacting units. However in a dilute suspension, the concentration of solid particles may be so small that the individual platelets are separated by distances far greater than the range over which the repulsive or attractive forces operate. Interparticle forces R and A cease to exist beyond interparticle distances of 300 AU.

Although sands and clays have much in common as members of the same family particulate materials, their differences are as important as their similarities. From the above discussions, it can be inferred that in the case of sands (coarse grained soils) arising out of micro-roughness of particles and low surface activity, the short range forces are dominant. On the contrary in clays (fine grained soils), short range forces are operative only between the exposed ionic lattices (edge to face interaction) of clay particles and depressed double layers due to increase in ion concentration. Such interactions promote the growth of clay clusters towards stable units both with and without external loading. Simultaneously long range forces would be operative between particle cluster units which essentially resist the external loading. In such systems, the concepts of particulate mechanics, which are based on effective solid-solid contacts are not applicable. The water held by clay mass is essentially a balance between the urge of the clay minerals to suck in water and the tendency of applied pressure to squeeze out water. Another notable difference between sands and clays is the susceptibility of clays for volume changes with environment changes, independent of loading, as in swelling and shrinkage.

From considerations of soil mineralogy and the interactions with other phases of the multiphase system, soil systems devoid of clay minerals can be regarded as non-interacting systems. On the other hand, reasoning based on the structure and mineralogy of clays and their interactions with water has established that all clay minerals have a tendency to adsorb water and/or exchangeable cations, if available, from the fluid phase. It can be recognized that the degree of adsorption depends on the expanding and non-expanding clay minerals, the distinction of which is only relative without any inherent difference between them (Bolt, 1956; Grim 1968). The engineering properties which clays can exhibit due to variation in mineralogy extends over a wide range. But the striking factor is that, despite the wide differences that can exist between one clay and the other within a given range of water contents, they exhibit essentially the same mechanical behaviour from a qualitative point of view. Differences in mineralogy, adsorbed ions etc. manifest themselves in differences in quantitative behaviour. Although all fine grained soils exhibit the phenomenon of consolidation swelling and shrinkage sols with kaolinite clay mineral show the much smaller range than those containing illite and montmorillonite.

TABLE 1

Non-Interacting Soil System (Sand and Silt)	Interacting Soil System (Clays)
Each phase has an independent continuous stress field.	Independent continuous stress field in each of the multi-phases is not tenable
The principal of superposition of coincident equilibrium stress field is valid	Not tenable
In addition to stress fields in each phase an overall total stress field can be assumed	The overall total stress field is the only acceptable physical stress field

This is mainly due to the low surface activity of kaolinite clay mineral. The physico-chemical interaction between the clay particles leads to the existence of both interparticle repulsive and attractive forces resulting in the separation of stable particle units preventing any direct contact, in the strict sense, to associate the measured shear strength to frictional properties derived from physical solid particle contact (including interlocking). Still the continuity in the clay-water system is maintained through the interacting assorbed water layers and/or exchangeable cations. Hence, soil systems in which solid constituents are fully or partly clay minerals can be regarded as interacting soil systems.

The above considerations permit an examination of the fundamentals of soil behaviour from consideration of multiphase continuum mechanics. Briefly the differences in the specific characteristics of both the systems (Nagaraj, 1981) have been given in Table 1.

Effective Stress Relations with Interparticle Forces

The preceding discussions indicate that to formulate the effective stress relations with interparticle forces, the simplest one can consider is the equilibrium state between two clay particles in parallel configuration. This parallel plate model has to be such that it should accommodate possible short and long range forces by assuming certain separation between them for the mobilization of forces and their changes upon incremental loading. On the premises that :

i) the soil behaviour is governed primarily by inter-granula[illegible]

(R – A) is net interacting force on unit area of total c/s

and a_m, a_a, a_w are the fractions of the total area in contact with each phase of solid, air and water respectively.

In the later discussions Lambe, (1960) visualizes two circumstances for a mechanistic interpretation of the conventional effective stress $(\sigma - u)$.

(a) in a highly plastic saturated dispersed clay $a_w = 1$, $a_w = 0$, the effective stress would be

$$(\sigma - u) = (R - A) \tag{3}$$

(b) for conditions of mineral to mineral contact and the contribution of interparticle forces being negligible the intergranular stress would be

$$(\sigma - u) = \overline{\overline{\sigma}}_m a_m \tag{4}$$

Still, the above distinction did not persist for long mainly due to the fact that most natural soils consist of far higher percentages of soil constituents coarser than clay sized fractions. The modifications of the effective stress relations for fine grained soils, within the basis of well established intergranular friction model, attempted by several investigators are critically reviewed elsewhere (Nagaraj 1981).

Based on the premise that even in fine grained soils the contact stress between solid particles is the effective stress and principle of superposition valid for all stress components Sridharan (1968) proposed the equilibrium [illegible] the form trends observed in the volume change response of the soil under changed physico-chemical environments, i.e., different pore fluids such as acetone, benzene, methanol. It has been shown elsewhere (Nagaraj, 1992) that these are not the diagnostic experiments to prove or otherwise the existence of contact between the particles in clays. In an interacting system, variation of pore fluid for a soil constitutes altogether differ system. Hence comparison of the similarity in responses of different clays with change in pore fluid is not tenable. In the equilirbrium equation 5, u, $\overline{C}$, R and A are all reactions to the applied stress. While u can be an independent variable the other components are only dependent upon the mutual interations between soil particles and pore fluid, they can only be grouped together along with c on one side of the equilibrium equation. In a detailed discussion on the transmission of force through soil, Lambe and Whitman (1969) have shown that a stress of about 5500 kg/cm^2 is requirred to squeeze out the adsorbed water completely between the two interacting clay platelets so that $\overline{c}$ to be operative. Hence it may be appropriate to consider eqn. 3 itself to characterize the behaviour of fine grained soils.

.More specifically the effective stress consideration is to recognize the soil system with appropriate predominance of gravitational or surface forces as distinctly different in controlling the soild behaviour (Nagaraj 1981, 1993) viz.,

1. Non - interacting particular materials (sands) with negligible long range forces.

$$\sigma' = \overline{\overline{\sigma}} a_m = (\sigma - u) \tag{6}$$

$\overline{\overline{\sigma}} a_m$ the intergranular stress is the effective stress.

2. Interacting particulate materials (soil containing clay sized fractions) where mobilized long range forces maintain equilibrium at all stages.

$$\sigma' = (\sigma - u) = (R - A) \tag{7}$$

where σ' is the effective stress.

The mode of effective stress consideration in the case of non interacting particulate materials needs no substantiation. The relation cited for interacting particulate materials, eqn. 7, does not violate the conditions stipulated for interacting soil systems in table 1. Further, it has been brought out by Bloch (1978) that in an interacting system such as clay-water system, in partly saturated state arrived at by monotonic loading consideration of independent stress fields for different phases and their superposition to get the total response are not physically tenable. Extending the same logic to saturated systems, although pore fluid pressure is an independent stress field, the mobilized interparticle forces are only the consequences of the conventional effective stress ($\sigma - u$) variations. Accordingly, in the case of clays (interacting systems) the long range forces (internally mobilized stresses) balance the externally applied stresses as the pore fluid pressure dissipates progressively causing changes in the state (void ratio) of clays. Manifestation of interparticle forces due to changes in applied effective stress is essentially the reaction of the soil system dictated by its physico-chemical factors.

State Parameter Approach

Since soils are particulate materials, when an external loading is applied, there would be changes in interparticle orientations and spacings to mobilize interparticle forces to resist the stresses imposed. This is particularly so, in the case of fine grained soils with appreciable clay fractions. The dominant macro- parameter to reflect the particulate material state is the void ratio. As early as 1937, Hvorslev (1937) recognized that there exists for a soil, a unique relationship between the effective stress and void ratio at equillibrium condition. Subsequently, Casagrande (1944), Rutledge (1947), Leonards (1953) and others showed that the void ratio at failure is of considerable significance when relating strength of clays to applied stresses while pore water pressures are zero. The elasto-plastic Cam-clay model developed by the Cambridge group to obtain the complete stress-strain behaviour of soils for different loading conditions. (Roscoe et al., 1958) clearly recognizes the need to consider the void ratio-pressure relation of the clays along with the consideration of energy dissipation factor, M. This factor is the ratio of deviator stress q to effective mean principal stress p′ at ultimate state of shearing and related to the friction angle of the soil.

Analysis of Soil Behaviour from Effective Stress Considerations

It is now examined about the validity of the principle of effective stress with regard to both the parts enunciated by Terzaghi (1936).

Normally Consolidated Soils

The simplest clay-water system one can consider is that when it has

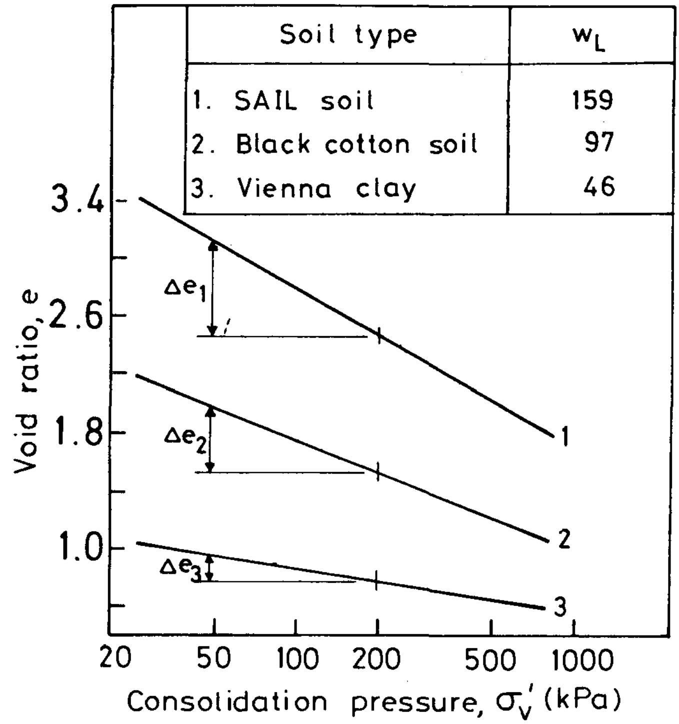


FIGURE 3. Compression paths of uncemented high water content soils

no previous stress history effects and is devoid of any cementation. It is possible to have such a situation when clay from its powdered state is mixed with water in excess of its liquid limit water content and compressed from that state. Such soils can be regarded normally consolidated soils.

Compressibility : Figure 3 shows the compression paths of three fine grained soils. It can be seen that soil with a higher liquid limit has a steeper slope than the one with a lower liquid limit water content. It is further interesting to observe that although water content changes are markedly different. the change in water content over the same consolidation pressure increment is proportional to its liquid limit, the ratio of which is constant. For the compression paths shown in Fig. 1, the void ratio- effective stress relation can be represented by the linear relation of the form :

$$e = a - b\log(\sigma - u) \quad (8)$$

In figure 4 the compression paths of four soils whose liquid limit water contents range over 106 to 48 show the same characteristic response

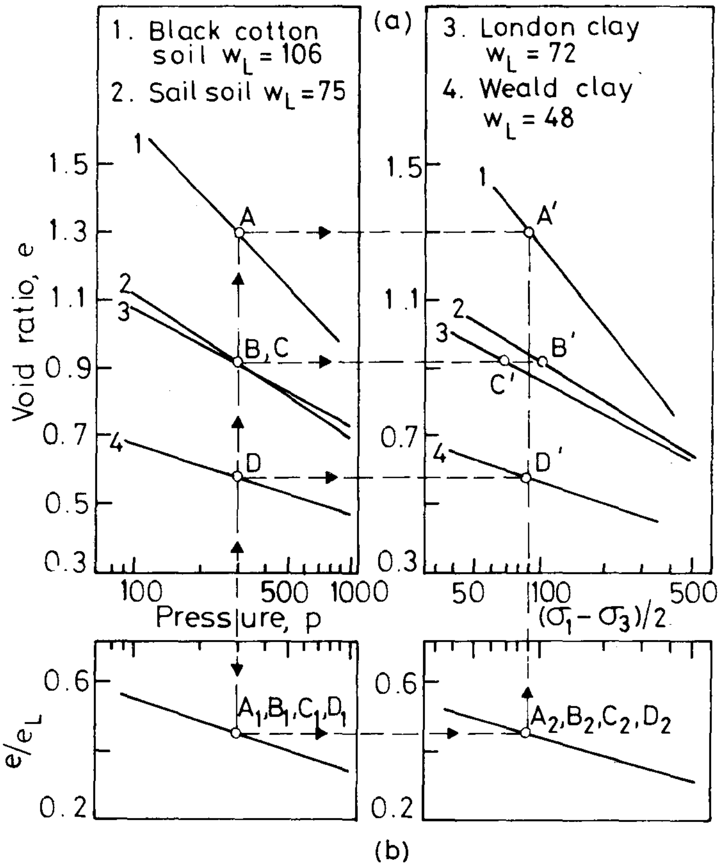


FIGURE. 4 Shear strength of different soils at the same consolidation pressure

even under isotropic effective stress $p' = (\sigma_1' + \sigma_2' + \sigma_3')/3$ conditions. Consider the equilibrium void ratio at consolidation pressure of 300 kPa. Although the void ratio of soils are distinctly different it collapses to a narrow band (fig 4b) when the change is reckoned with respect to their corresponding void ratio at liquid limit state. Accordingly the combined macro- and micro- state at that effective stress level merits examination. This needs consideration of the micro-structure at the liquid limit state and the subsequent change induced to the microstructure due to the change in effective stress. This enables to elucidate the micro-structure of the four soils at a specific level of conventional effective stress.

Micro structural Considerations : Clay particles of colloidal size in an electrolyte suspension are subjected to chance impacts by water molecules which cause the clay platelets to move randomly (Brownian movements). Such random movements (together with grosser fluctuations in water currents) will, from time to time, bring clay particles together within distances in the range of interparticle forces. As a result, the subsequent behaviour of the particles will depend on the net forces between them at a separating distance to which the chance movement has brought them.

If the net force is repulsive, the particles will remain separate and the random movements will separate them still further. If all the particles consist of the same clay mineral and the suspension is uniform, we can then assume that all the individual platelets will remain separate, even after a long time. This results in a 'dispersed structure'. As water content decreases the internal resistance arises due to long range forces only.

On the contrary if the net force is attractive due to higher electrolyte concentrations, the chance approach of particles in the suspension may bring them still closer. With increasing attraction as the distance diminishes, the chance encounter will lead to coagulation of the particles. The subsequent behaviour is similar to that of a large particle. If were of different sizes, the larger particles would probably meet and coagulate with smaller ones as they settle more rapidly through the suspension. Since, each smaller particle encountered would be gobbled to the group due to attractive forces of particles, the entire assembly would fall at an increasing speed, probably reaching a constant size and velocity when the fluid shearing resistance at the periphery of the aggregate removes smaller particles at the same rate at which they are accumulated. This results in the flocs of nearly same size, though initially the particles were of different sizes (i.e., poly dispersive system). The coagulation, flocculation or aggregation process would proceed simultaneously at many zones in the suspension. Under these circumstances, the clay particles, in groups or flocs, would settle relatively rapidly, resulting in a sediment of loosely knit assemblages of large number of individual particles.

A dilute suspension of soil particles (or the basic units i.e., flocs) in water does not constitute the engineering material, soil. It would not be possible to ascribe a definite state to the soil. Only when a small but measurable stress is applied, the particles are brought into closer distances at which interaction forces operate, to be in equilibrium with the applied pressure. Then the particles units get themselves arranged in a defined pattern and the soil acquires engineering properties. Consider an isotropic stress, p as applied to the soil. Obviously, the particles cannot be parallely placed as assumed by the Guoy–Chapman theory throughout the soil mass, as there is no preference to be oriented along any direction under an isotropic state of stress. The net repulsive force (R – A) between two units

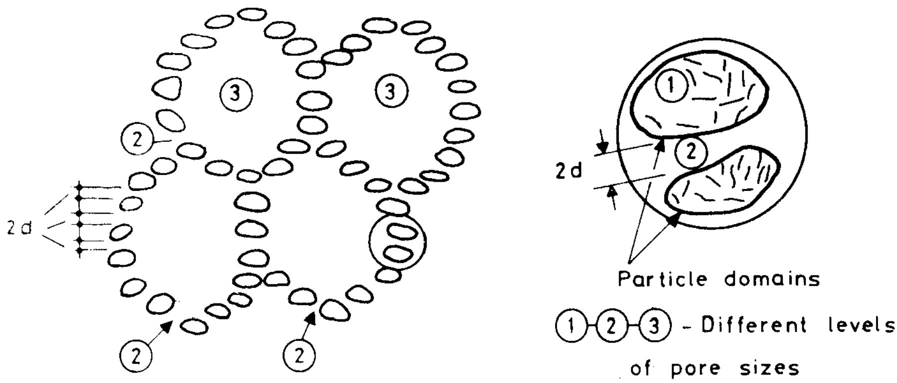


FIGURE. 5 Possible microstructure of clays (Nagaraj et. al., 1994)

creates an osmotic suction on the adjacent fluid equal in magnitude to p. When the fluid is subjected to a suction pressure, p at innumerable points around, due to several pairs of interacting particle units, it settles to an equilibrium forming a spherical pore as shown in Fig. 5. The maximum size of this pore is also dictated by the applied suction from surface tension considerations by the equation $p = 2T/R$. A very clear evidence for the existence of this type of structure is the pore size distribution data of several soils (Griffiths and Joshi, 1989) using mercury porosimeter. The data shows (Fig. 6) that there are pores as large as 10000 AU and that the volume of such pores amounts to nearly 90 to 95 % of the total void volume of the soil. It is known that the interparticle force R and A cease to exist beyond a distance of about 300 AU. From the pore size distribution data, it can be seen that the volume of pores less than 300 AU is only 5 to 8 % of the total void volume, while the rest of the volume, which is more than 90 % is due to pores of sizes greater than 300 AU.

The existance of pores of sizes greater than 10000 AU together with double layer interactions between particles being restriced to less than 300 AU, can be explained with probably three distinct levels of pores forming the microstructure (Nagaraj et al., 1990).

a) Intra-aggregate (or cluster) pores with sizes, less than about 20 AU between the individual platelets within a cluster. Particles within the cluster would have crossed the repulsive energy barrier and come to distances at which the net interaction force is one of attraction so that they are stable units.

b) Inter aggregate pores between two interacting aggregates (where double layer interactions prevail) of sizes greater than 20 AU and less than 300 AU depending on the applied equilibrium pressure.

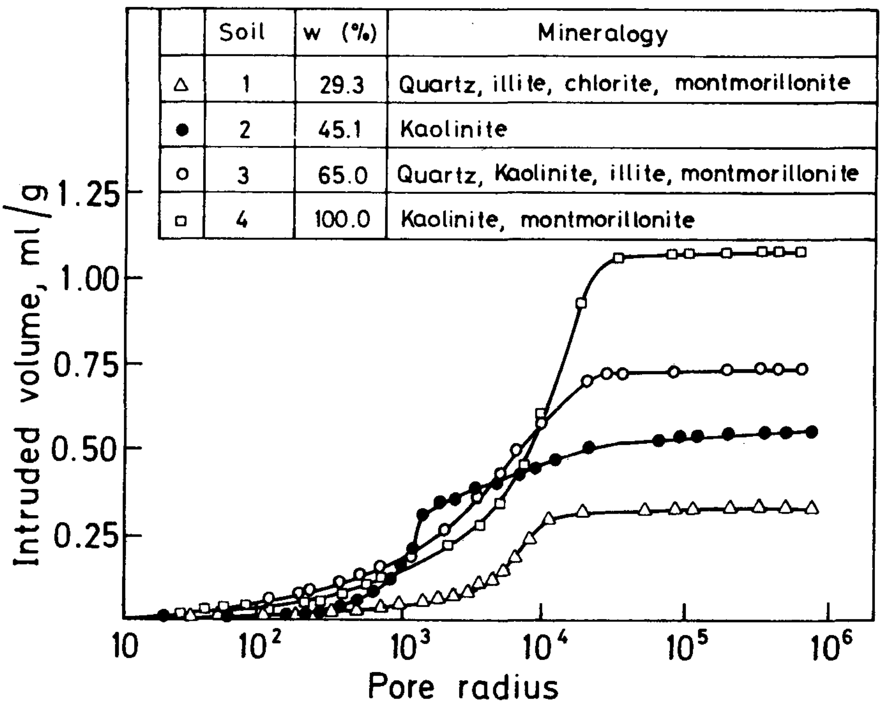
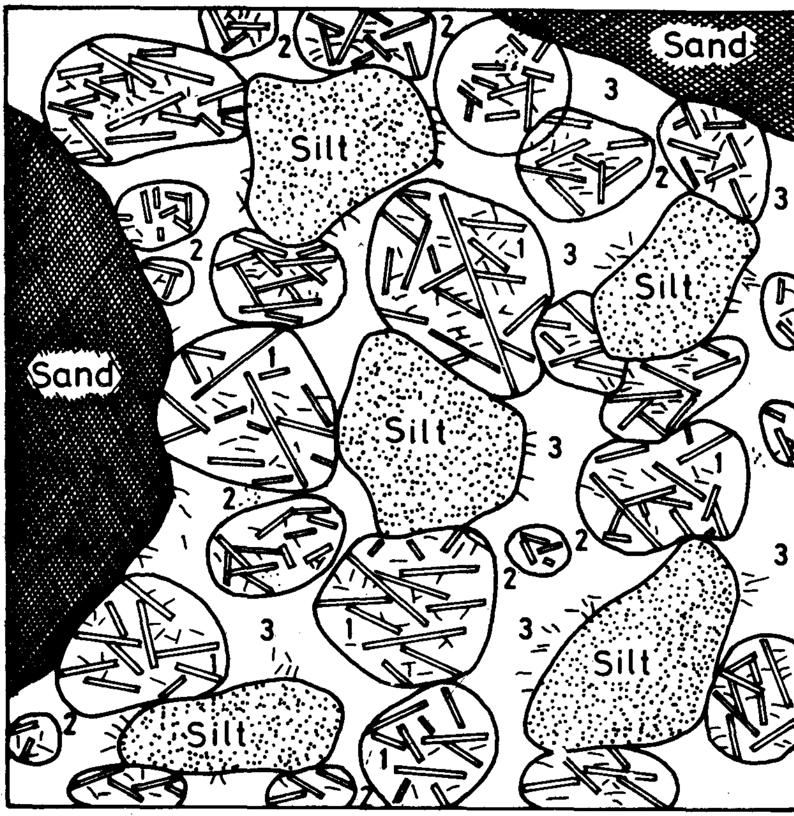


FIGURE. 6 Pore size distribution of soils at their liquid limit state (Griffiths and Joshi, 1989)

c) Inter aggregate large pores held within a group of clusters by surface tension with sizes far greater than 300 AU.

Figure 7 is a schematic representation of the modified diagram of microstructure of fine grained soils proposed by Casagrande (1932) with the relative disposition of different fabric units consistent with the above interpretation. Similar microstructural picture, comprising of unequal pores, (Fig. 8) was assumed by Olsen (1962) as early as 1962, to interpret experimental data on hydraulic conductivity. Subsequently, extensive mercury intrusion porosimetry and electron microscopic studies (Griffiths and Joshi, 1989, 1990; Shear et. al., 1992) have confirmed the above fabric model. Further it has also been possible to show that differential compression takes place both at levels of inter-cluster and inter-assemblage pore structure without violating the static equilibrium at all locations.

Consider the pore size distribution of four soils at their liquid limit state whose water contents vary from 30 to 100 (Fig. 6). With three distinct



1. Intra aggregate pores
2. Inter aggregate pores
3. Large enclosed pores within a group of aggregates

FIGURE. 7 Modified diagram of Casgrande's schematic representation of clay microstructure (Nagaraj et al., 1990)

levels of pores, the ideal pore size distribution curve should be as shown in figure 9(a). Even with the inherent complexities in natural soils, the plots show the same trend (Fig. 9(b)). In this plot the pore size distribution data of the four soils (Fig. 6) are replotted in terms of intruded pore volume per unit volume of the soil instead of per unit weight of dry soil, so that the total pore volumes for different soils are normalized for compresion. It can be seen that the relative distribution is similar for all the soils. This indicates

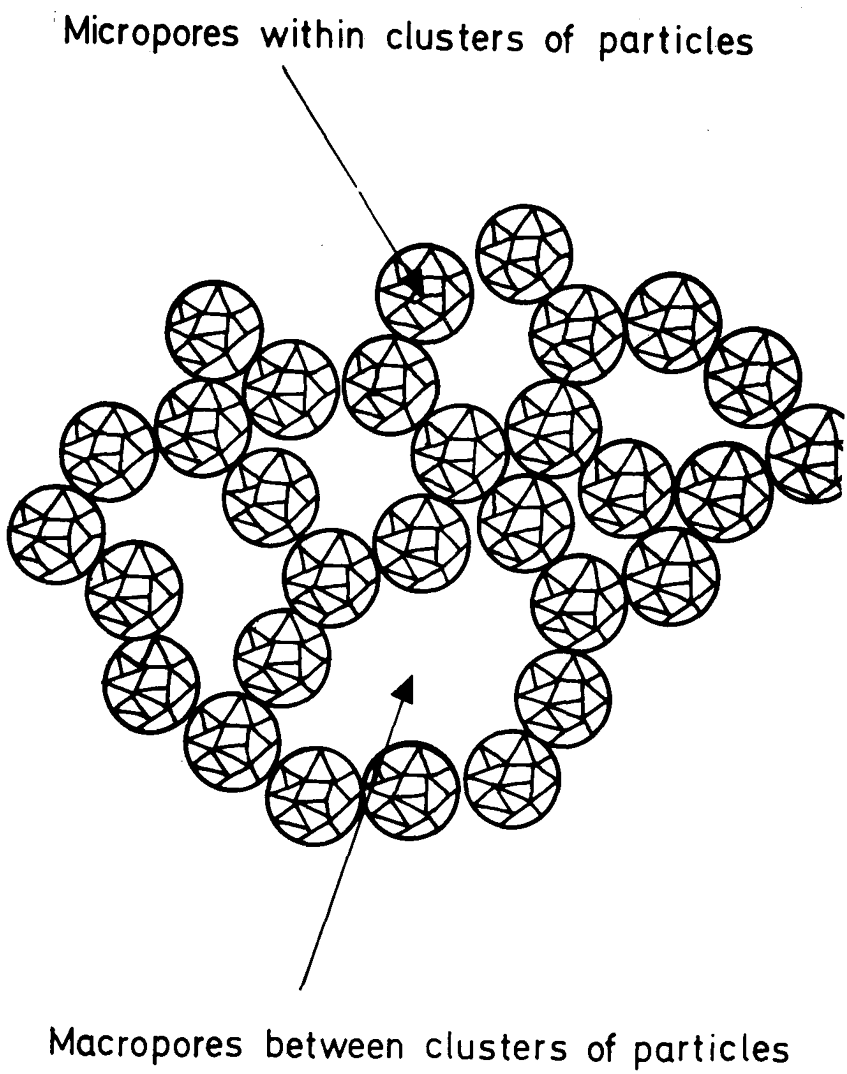
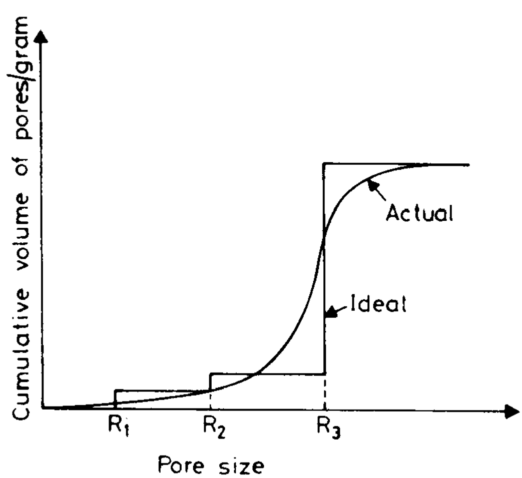


FIGURE. 8 Idealized unequal clay pore structure (Olsen, 1962)

the micro-structure which reflects the distribution of pores, i.e., intra-aggregate, inter-aggregate pores and large pores held within a group of clusters in an unit volume would be of similar pattern for different clays exhibiting wide variations in their water holding capacities. Another supporting evidence for the existence of the same pore structure has been observed by premeability data (Nagaraj et al., 1991, 1993) (Table 2).

The important aspect of these data is that although the water contents



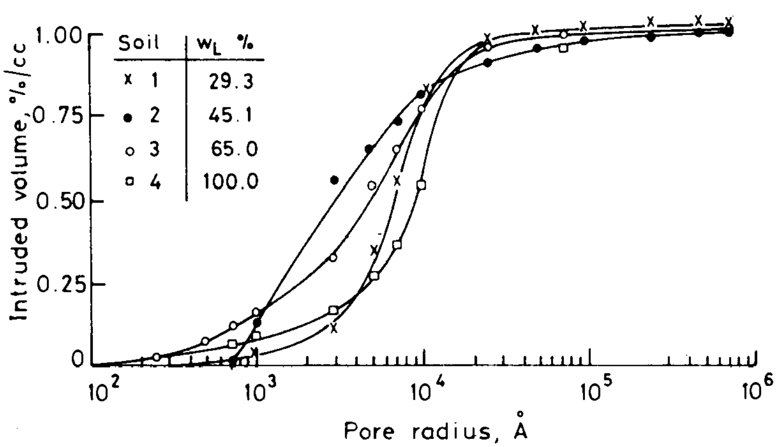


FIGURE. 9
a) idealized pore size distribution and
b) Normalized intruded volume versus pore size distribution of Figure 6
(Nagaraj et. at., 1990)

and void ratios at liquid limit state for different soils vary over a wide range, the hydraulic conductivity is nearly the same for all of them. This implies that effective pore sizes controlling the fluid flow must be the same for all the soils and hence the same pattern of micro-fabric. For the formation of such a self supporting micro-structure, the forces of interaction between clay particle surfaces and adsorbed water should be about the same for different clays. The amount of water held per unit area of surface has to be the same and should correspond to the same magnitude of pore water suction. Worth (1979) on the basis of earlier investigations (Russel and Mickel, 1970, Worth and Wood, 1978 and Whyte, 1982) and critical state concepts has indicated that all fine grained soils equilibrate to their respective liquid limit water contents at a suction of about 6.3 kPa. For this condition to prevail, for all soils at their liquid limit water contents, the same order of physico-chemical potential should be available for physico-chemical interactions. If the state of soil is considered in terms of volume basis instead of weight basis, the weight of solid particles present in unit volume would be inversely proportional to the surface area, i.e., the weight of solid particles in unit volume would be such as to provide the same order of surface area and hence the same order of physico-chemical potential for all soils. Hence the volume of large pores enclosed (which essentially controls the volume changes under further loading or permeability) will also be nearly same. At this value of suction which is the effective stress of the fine grained soils according to the effective stress principle, shear strength should be of the same order. The shearing resistance C_{uL}, of the soils at their liquid limit state water contents varying between 36 and 159 measured by laboratory vane has been reported by Federico (1983). The measured values fall within limits of 1.7 to 2.8 kPa. Based on the above discussion the micro-structure of clays at their liquid limit water contents can be considered as the initial microstructure, which takes into account the inter-particle forces resulting in the same pattern of micro-fabric for all fine grained soils.

It is worthwhile to examine the effect and changes to the initial micro-structure with consolidation, i.e., change in effective stress. To examine this aspect, published literaturre (Griffiths and Joshi, 1989) on micro-structural

Table 2
Hydraulic Conductivity at Liquid Limit State for Several Clays
(Nagaraj et al., 1991; Mitchell, 1993)

Soil Type	Liquid Limit (percent) w_L	Void Ratio at liquid limit e_L	Hydraulic conductivity (10^{-7} cm/sec)
Bentonite	330	9.240	1.28
Bentonite + Sand	215	5.910	2.65
Natural marine soil	106	2.798	2.56
Air dried marine soil	84	2.234	2.42
Oven dried marine soil	60	1.644	2.63
Brown soil	62	1.674	2.83

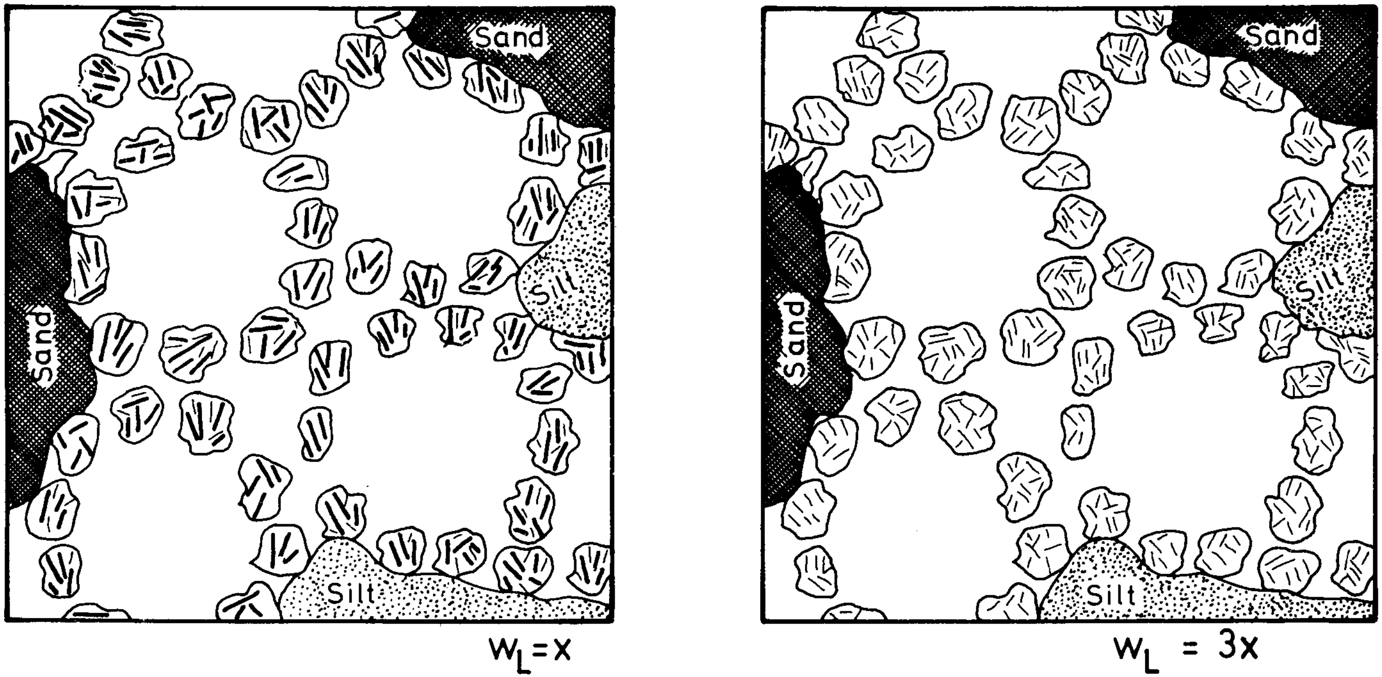


FIGURE. 10 Schematic picture of fine grained soil microstructure for the same coarse fraction but with different clay minerals (Clay surface area contribution per unit volume is same in either cases with distinct variations in weight contribution due to massive and other clay minerals)

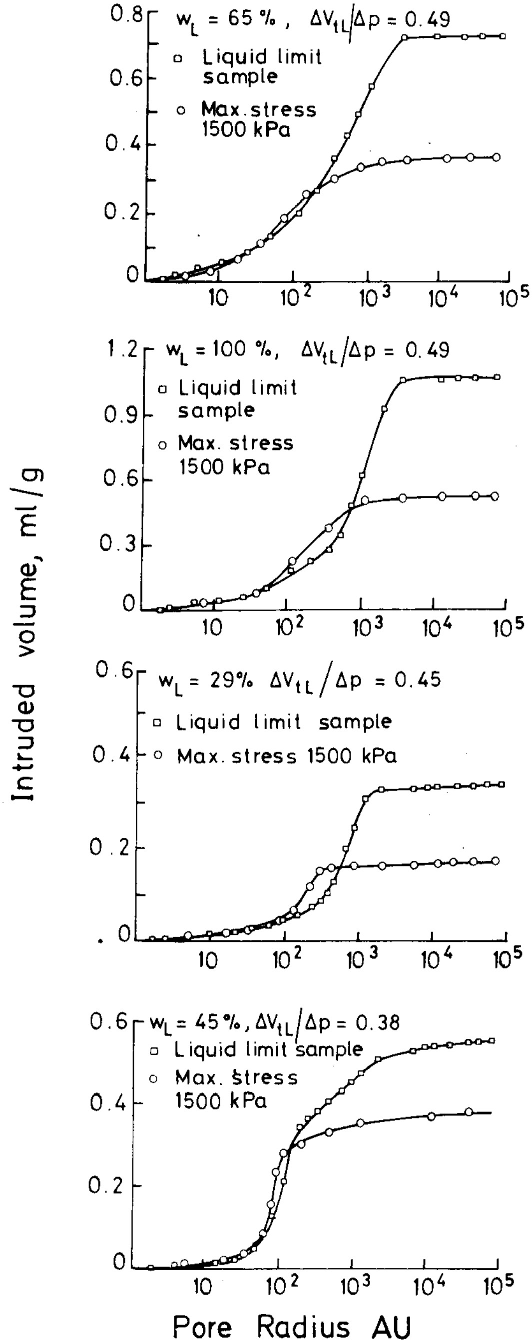


FIGURE 11 Intruded pore volume versus pore radius of four soils at their liquid limit state and compressed to 1500 kPa (Data from Griffiths and Joshi, 1989)

changes due to consolidation on four soils ($w_L = 29$ to 100), each stressed to four stress levels in one dimensional consolidation, freeze dried, and pore size distribution determined by mercury intrusion porosimetry are reviewed. In general terms the same micro-structural changes have been observed in the pore size distribution for all the four types of soils upon increasing the consolidation stress. Fig. 11 is a typical pore size distribution data for the four soils at their liquid limit states and consolidation stress upto 1500 kPa. It can be seen that the ratio of ΔV_t which is the change in the intruded pore volume between the liquid limit state that at consolidation stress of 1500 kPa, to total pore volume at liquid limit state, V_{tL} is constant.

The relationship between the normalized pore volume of soil with respect to that at its liquid limit state and consolidation stress is of the form (Griffiths and Joshi, 1989)

$$\frac{V_t}{V_{tL}} = 1.193 - 0.2060 \log(\sigma - u) \qquad (9)$$

Since micro pore volume distribution at liquid limit state of different soils are of the same pattern, its change with stress is proportional. This implies that the micro-pore structure of different soils monotonically stressed to various levels is of similar pattern.

In retrospect, the above discussions clearly indicate that in the case fine grained soils, without stress history effects and devoid of cementation, response to changes in effective stress follows an unique pattern. This happens inspite of wide variation in void ratio changes over the same effective stress change (compatible with their liquid limit water contents). The effect of inter-particle forces are simultaneously taken care of by the compatible macro and micro-states. Even if the physico-chemical environment changes, with the effective stress remaining constant, as induced interparticle forces change, the compatible void ratio and the microstructure so as to attain the statical equilibrium.

Another supporting evidence regarding the possibility of obtaining same permeability which is a reflection of the micro-structure can be seen from the analysis of consolidation- permeability data of normally consolidated soils (Nagaraj et al., 1993). Figure 12 shows the void ratio-consolidation pressure and void ratio-coefficient of permeability relationships of fine grained soils. Consider the equilibrium void ratios at a consolidation stress of 100 kPa. Soils equilibrate to differ void ratios in proportion to their initial void ratios. The permeability coefficient is the same reflecting the same micro- structure of different soils at the same consolidation stress.

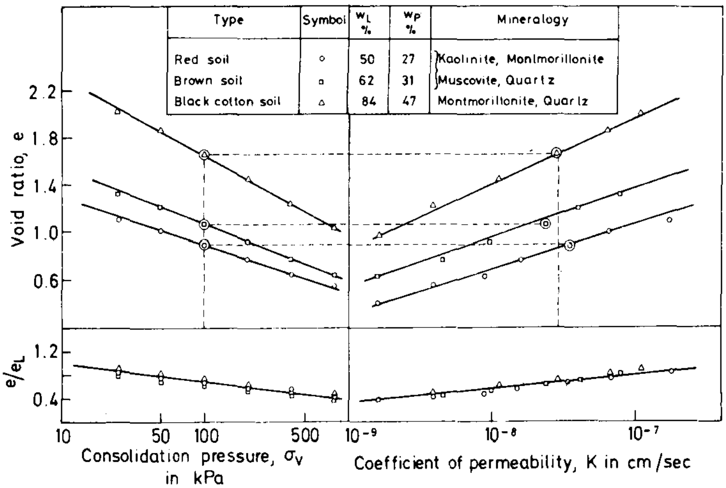


FIGURE. 12 Stress-state-permeability relations for fine grained normally consolidated uncemented soils.

Shear Strength : Obviously, another level of reflection for the validity of effective stress principle is that for the same level of effective stress, the shearing resistance ought to be same provided the structure is same. Reverting to figure 11, it can be seen that at the same level of consolidation pressure of 300 kPa all the four soils develop the same order of shearing resistance even though the equilibrium void ratios at that consolidation stress are different. The combined micro- and macro- state is the same. As the same order of micro-structural changes takes place with change in effective stress, the change in shearing resistance of soils is also of the sme order.

It is believed that during shearing the micro-structure will change. The identical shearing resistance at a given effective stress and the same initial micro-structure for different fine grained soils clearly indicates that the changes in micro- structure during shearing are also identical. Perhaps identical micro-structure of soils at their critical state is a consequence of this postulation.

Overconsolidated Soils

Most often sedimented soils exist in a state of overconsolidation. Since the present magnitude of effective stress is less than the level to which the soil has already been subjected to, the maximum past effective stress has a

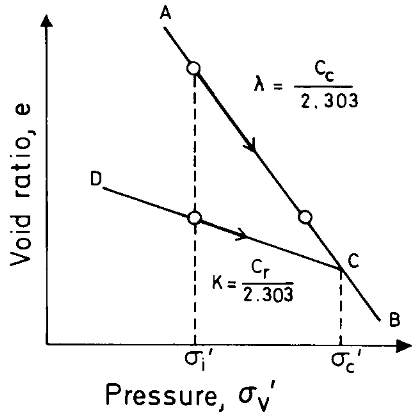


FIGURE. 13 Typical void ratio consolidation stress paths for normally and overconsolidated conditions.

significant role to play on the compressibility and strength behaviour of such soils. This necessitates to consider and account for the stress history effects in the analysis of soil behaviour from effective stress considerations.

Compressibility : Consider a typical compression–rebound–recompression path of fine grained soil unloaded at effective stress level, σ_c' (Fig. 13). Along the compression path AB the soil is in a normally consolidated state. At an effective stress level of σ_c' if the soil is unloaded, the rebound path is far different from CA. The path followed is CD. Although rebound–recompression paths are different CD is the linearized average path. As such at any effective stress level less than σ_c', say σ_i' two possible void ratios, one on the normally consolidated path and the other on the rebound–recompression path reflect the equillibrium state. Upon further compression of the soil from these two situations as the effective stress changes, the path followed would be different implying that for the same incremental change in effective stress the compressibility is not the same. Can this response be explained by conventional effective stress principle from inter-particle forces and fabric considerations merit examination.

Micro-structural Considerations : It is needless to stress that the void ratio realized as the effective stress changes reflect the compressibility behaviour of saturated clays. For an ideal situation of perfect parallel plate configuration, an increase in effective stress should decrease inter-particle separation at particulate level with a consequent decrease in void ratio at macro-level. Upon unloading from a particular effective stress level as the effective stress reduces, the particles should rebound back to original level (Fig. 14a). Obviously this does not happen. Hence a micro-structural

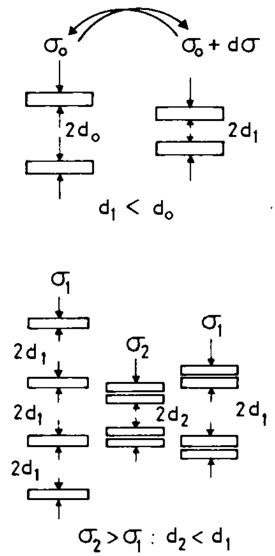


FIGURE. 14 Schematic representation of clay cluster growth and their separation with loading and unloading

examination is needed.

With the above picture of microstructure, examine the role of induced interparticle forces under different imposed loading conditions. Due to incremental loading, clay particles/clusters are forced into closer proximity such that short range forces are operative with forces of attraction being dominant promoting the growth of clusters. As a result of this, there is a progressive shifting of inter-cluster double layer interactions to a different level to enclose compatible large pore. The possibility has been clearly cited in a different sense, by Parry (1959) as early as 1959. The attractive forces mobilized due to the growth of clusters were considered to be latent since, the same component was not available for inter- cluster interactions. A compression curve has been considered to illustrate this point (Fig. 15). Each point on the virgin compression curve and on any rebound curve is an equillibrium point with zero pore water pressure. At any void ratio, e, the external stress may have value within a wide range (e.g. σ_a, σ_b, σ_c) the maximum being the virgin consolidation value σ_{NC}. With this possibility, the general equilibrium equation would be

$$\sigma_E - \sigma_1 + \sigma_{AL} = 0 \qquad (10)$$

where σ_E is the external stress

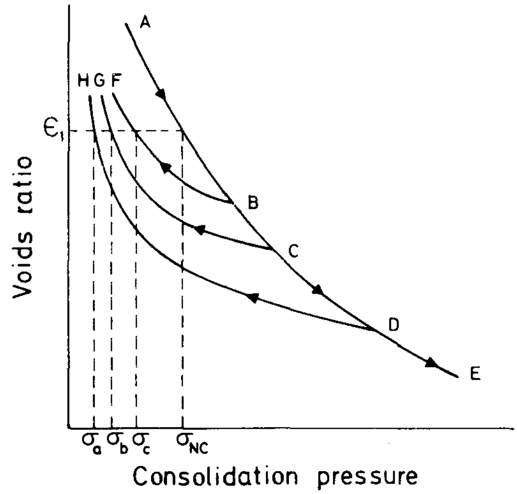


FIGURE. 15 Compression curve and rebound curves from different consolidation stress levels (Parry, 1959)

σ_I is the inter-cluster repulsion

σ_{AL} is the internal latent attractive stress

This internal attractive stress has been termed as the latent stress, whose origin has been attributed to London – van der Waals attractive forces. Since this component tends to be irreversible even upon the release of the external stresses, the equilibrium of clay-water system is possible at the same void ratio even under a reduced external stress.

In retrospect the above analysis and interpretations of the microstructural aspect of clays point to the fact that the fine grained soil (clay) – water system is a dynamic system with progressive internal changes in the microstructure occurring at all levels of application of external loading and/or environment due to short and long range forces operative juxtaposed. As such, it is very unlikely to idealize the microstructure of clays with a simple model such that the quantitative estimations of interparticle forces can be made by computations even if the mode of incorporation of interparticle forces in statical equilibrium equation is resolved.

By reiterating the discussions of Parry (1959, 1960) on latent interparticle forces mobilized along the virgin compression path, it is possible to infer that there is progressive jumping over of interaction between particles to cluster which grows with effective stress due to short range attractive forces. Based on the Gouy-Chapman diffuse double layer theory, Klausner and Shainberg (1967) generated $e - \log \sigma$ curves for different number of particles possible in a cluster. Comparing the experimental $e - \log \sigma$ curve of a Na-Montmorillonite soil with these curves, Nagaraj and Srinivasa Murthy (1968a) logically showed that there is a gradual grouping of particles into clusters with increase in effective stress level. Due to grouping of particles into clusters, the process of which being dominantly irreversible (Fig. 14b) results in a reduced volume change in the unloading and reloading over the same range of effective stress change. This reasoning can be extended to the micro-structural picture arrived for mercury pore size distribution data since the large pore enclosed by clusters depends upon the inter-cluster adsorbed water interactions in a similar manner as explained for normally consolidated soils. The above discussion indicates that although the pattern of micro-structure is same, the cluster characteristics are distinctly different at the same level of effective stress so as to result in different void ratios at equilibrium, compatible with the initial potential reflected by the initial specific surface of the soil which indirectly is reflected by the liquid limit water content of the soil. Hence further compression from this state would be at different rates compatible with the initial micro-structure as the effective stress increases. The compressibility responses from practical stand point, is charcterized by compression and recompression indices. This circumvents the need to consider the inter-particle forces separately.

There are specific situations, when compressibility responses can be of unique pattern for different overconsolidated soils. Different fine grained soils compressed from their liquid limit state to the same level of effective stress, unloaded to the same level (same OCR) would have identical initial micro-structure, although the void ratios are proportional to void ratios corresponding to water content at their liquid limit state. The ratio of change in void ratio over the same incremental change in effective stress to that corresponding to the void ratio at liquid limit water content is constant i.e., $\Delta e/e_L$ is constant.

From the above discussions, it can be inferred that for the analysis of compressibility behaviour of overconsolidated soils, consideration of maximum past pressure is required to account for micro-structural changes. In the $e - \log(\sigma - u)$ relations incorporation of σ_c can be made resulting in relation of the form (Nagaraj et. al., 1994)

$$e = a - b \log \sigma_c' + c \log(\sigma_c'/\sigma') \tag{11}$$

where $\sigma' = \sigma_c$ for normally consolidated state of the soils.

Shear Strength : It is now examined as to how the shear behaviour of overconsolidated soils both under undrained and drained conditions can be interpreted from the conventional effective stress considerations, with due regard to inter-particle forces and micro-fabric of soils.

Consider an isotropic compression and rebound stress path as shown in Fig. 16a. Consider three specimens on this path. Specimen (1) is normally consolidated to a mean pressure p_1' and void ratio e_1. Specimen (2) is on the rebound line from a maximum past pressure of p_p' having the same void ratio of e_1 i.e. $e_1 = e_2$, and the corresponding confining mean pressure is p_2'. The specimen (3) is on the normally consolidated line with the consolidation pressure same as p_2 and the corresponding void ratio e_3 $(e_3 > e_1)$. If the deviatoric stress, is applied incrementally, under undrained condition on samples (1) and (2), figure 16b shows schematically the responses of the two specimens during shearing in the usual q versus ε_a (axial strain) plot and the pore pressure, u versus ε_a plot. For specimen (1) both shear stress and the excess pore water pressure (positive) increase with increase in axial strain. For specimen (2), the shear stress increases in the same way as for specimen (1) but may be with a little slower rate due to lower confining pressure. However the final strength value is nearly the same. But the pore pressure in specimen (2) is positive and increases but after a particular strain level decreases and becomes even negative with increase in axial strain.

Let incremental deviatoric stress be applied on new specimens, specimen (2) and specimen (3) allowing complete drainage of water so that the excess pore water pressure in the sample at any stage is zero. The samples undergo volume changes. Fig. 16c represents the q vs ε_a and ε_a vs ε_v plots for both the specimens. The shear stress for specimen (2) rapidly increases with a higher modulus, reaches a peak and softens with increase in axial strain. The specimen (3) will have shear stress non-linearly increasing to reach an asymptotic value at large strains. It is interesting to note that, both the specimens have the same ultimate shear strength. Regarding volumetric strain-axial strain plot, there is a continuous compression for specimen (3) while, specimen (2) undergoes initial compression and then after a particular strain level expansion or dilation. It is equally interesting to note that final void ratio of both the specimens are nearly same though they had different initial void ratios. This final void ratio is generally referred to as the critical void ratio.

In fine grained soils during undrained shearing the clusters gradually get dismembered in an attempt to erase out the effects of stress history (Dafalias et al., 1980; Srinivals Murthy et al., 1991). To sustain a deviatoric stress, q and still keep the stress in the pore fluid isotropic, the cluster configuration should change to a non-spherical pore. For a given volume of pores, since the perimeter of a non-spherical pore is more than that of

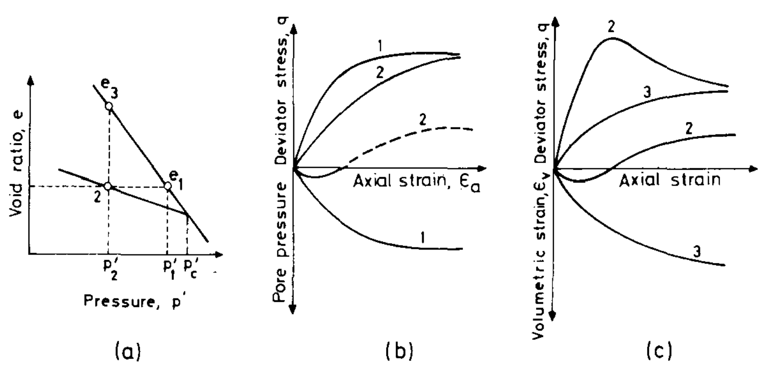


FIGURE. 16 Typical paths of stress-strain-pore pressure at the same void ratio and stress-strain volumetric strain at same pressure for normally and overconsolidated soil states.

spherical pore, clusters tend to breakdown. A breakdown of clusters induces additional stresses causing dilation to the system. In the undrained case, since the volume is kept constant, the broken units are forced to a closer d spacing, and hence the internal osmotic stress (R – A) increases. Negative pore pressure will develop to keep the system in equilibrium with the external pressure. Shear strength will also increase corresponding to the increased effective stress. In the drained case, there is adequate scope for volume change compatible to the effective stress in the system. Therefore, upon breakdown, the broken units move apart to mainain the distance between them compatible with the stresses. Hence, dilation is observed. In retrospect, in overconsolidated soils, the conventional effective stress principle can be used with due consideration to microstructural changes which takes care of interparticle forces appropriately. Conventionally, this is already in built in the analysis of compressibility and shear strength through reduced compression index, C_r for the change in effective stress in overconsolidated stress range and shear strength by negative pore water pressures in undrained shearing and dilational component in drained shearing.

Cemented Soils

Soils can acquire cementation bonding, when they are in a normally or overconsolidated state. Depending upon the intensity of cementation and the initial state of the soil, different states ranging from soft highly sensitive to highly stiff are possible (Fig. 17). Cementation bonds between particles/ or their aggregates are developed due to solid amorphous links of precipitates

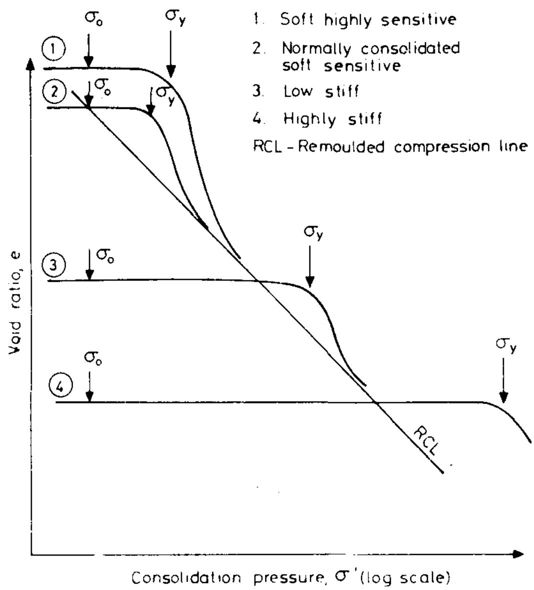


FIGURE. 17 Schematic representation of soils of different degrees of cementation

of calcites, iron oxides, alumina and other inorganic and organic matter. These bonds impart non-particulate charcteristics to the system with additional rigidity to the soil fabric against deformation. If cementation takes place while clays are normally consolidated, the soil acquires a metastable state in the sense that void ratios would be higher than those corresponding to the equilibrium state under the prevailing effective overburden pressure.

This state is regarded as 'metastable' in the sense that under any given stress, if the bonds are leached or loaded beyond its bond strength, the soil would collapse in order to develop commensurate resistance to sustain the imposed level of stress. Stiff cemented soils are normally formed under overconsolidated state of the soil and as such are less problematic due to the inherent high strength and low compressibility in the stress regime of engineering interest. Hence the discussions in this section mainly pertain to soft cemented soils.

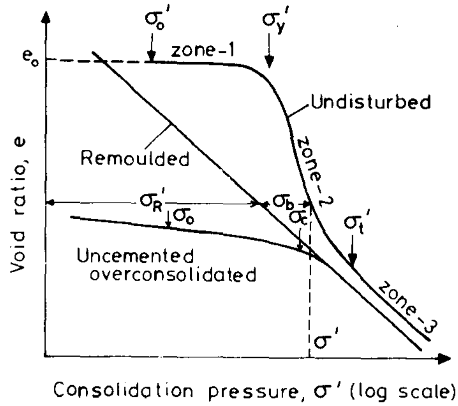


FIGURE. 18 Typical compression path of soft cemented soil

Characteristics of Soft Sensitive Cemented Soils

Fig. 18 depicts a typical compression path of soft cemented soil. Three zones can be considered in the compression path (Nagaraj et al., 1990). In zone 1 i.e., upto the yield stress, σ_y' the compressions are negligible, beyond which in zone 2, compressions are marked in the narrow range of stress increment and further beyond the transition stress σ_t. Compressions attain normal levels comparable to the remoulded soil compressed in the entire stress regime. Since the compression path of sensitive soils are often considered to be that of overconsolidated state for relative comparisons, compression path of uncemented overconsolidated soil is also indicated.

The compression test data of Louiseville clay (Lapierre et al., 1989) (Fig. 19) shows the yield stress to be about 180 kPa. The liquidity index of in-situ soil is greater than 1.0. As such this yield stress is entirely due to bonds since uncemented skeleton resistance at such high water contents greater than liquid limit can be less than only 6 kPa. Upon further loading from 180 to 360 kPa, high order of compressions take place. On comparison, stress components at equilibrium void ratios both on intact clay and compression path of the same clay in its undisturbed state are 170 and 190 kPa. Even at a higher stress level, although the stress component increases corresponding to equilibrium void ratio, the cementation bond component is either of the same order or slightly more. These features are observed with many other sensitive clays (Fig. 20) (Quigley and Thompson, 1966; Lo, 1972; Yong and Nagaraj, 1977).

It is not clear at this stage as to why the cementation bond strength component of the same magnitude persists at different stress levels of

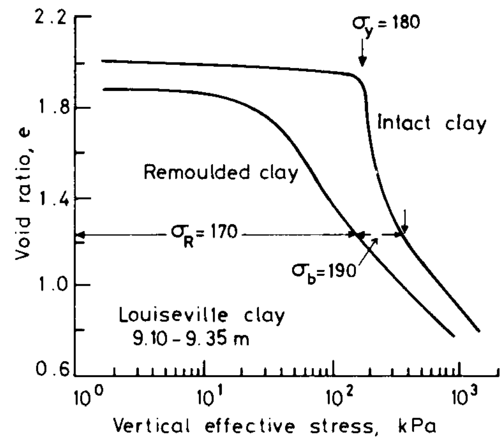


FIGURE. 19 Compression paths of Louiseville clay (Lapierre et. al., 1989)

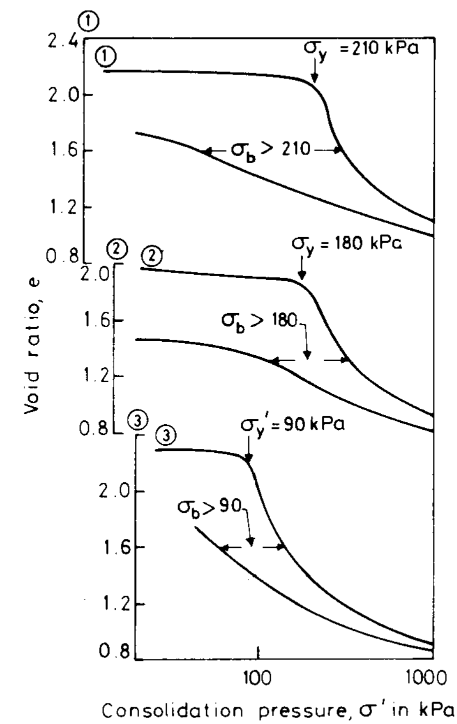


FIGURE. 20 Compression paths of three cemented soils (Data from 1. Quigley and Thomson, 1966, 2. Yong and Nagaraj, 1977 and 3. Lo, 1972)

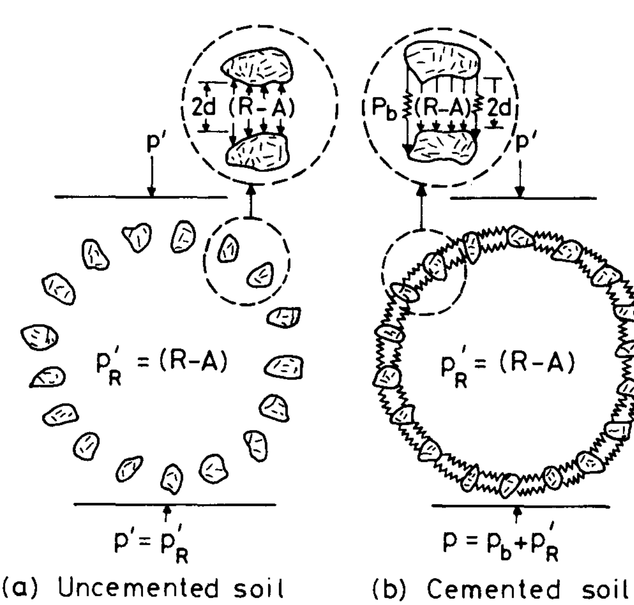


FIGURE. 21 Possible microstructure of cemented soils

loading. Perhaps it implies that despite disruption of bonds due to loading, number of bonds per unit volume might remain same as it was initially, so that the stress carrying capacity of the bonds does not reduce with compression. Despite complete clarity regarding the reasons for this constant bond strength at different stress levels of loading, the observations that compressions are essentially due to the changes in stresses on the unbonded skeleton, with the cementation bonds providing additional constant resistance are unambiguous. Perhaps considerations at micro-structural level might provide further clarity and reinforcement to the observations at engineering level.

Micro-structural Considerations : It has' already been substantiated that in a clay-water system, the stress transfer is through an interacting fluid phase. The soil state realized as the pore water pressure dissipates can be assumed to be dictated by the requirement of equilibrium between the long range forces mobilized between the interacting clusters and the externally applied stress. There is nothing in principle to bar the coexistence of long range forces operative between interacting units and cementation due to the extraneous cementing material. As such components of resistance mobilized due to particulate nature of the material which is due to conventional effective stress and cementation can operate simultaneously (Fig. 21). The same mechanism of cementation bonding can exist even for the complex

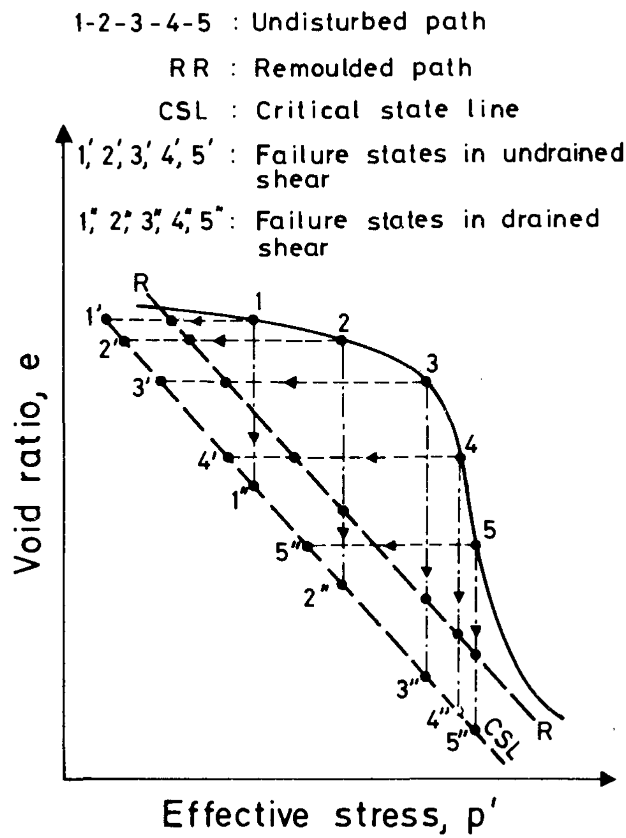


FIGURE. 22 Drained and undrained shear test paths of cemented soil in e-p plane

microstructure of particle cluster enclosing large pores, as depicted in Fig. 20. In this depiction, the geometrical aspects of micro-structure of uncemented and cemented soil are same with additional cementation component between the clusters. The pore size distribution and permeability data of the cemented undisturbed soil and of the remoulded soil at the same void ratio lend supporting evidence to the above micro-structural postulations. Since, the pore size distribution is same and premeability is of the same order, it indicates that overall the micro-fabric of a cemented soil is not very different from that of the remoulded state (Delage and Lefebvre, 1984; Lapierre et al., 1989).

Compressibility : From the above discussions, it is indicative that compressibility can be attributed to the deformation of soil due to changes in the components of stresses on the unbonded skeleton and hence this component can be regarded as the equivalent effective stress on the soil and can be written as (Nagaraj et.al., 1991 Vatsala et. al., 1995)

$$\sigma' = \sigma_R' = (\sigma - \sigma_b - u) \qquad (12)$$

The stress state relation would be the same as for uncemented soil of the form

$$e = a - b \log \sigma_R' = a - b \log (\sigma - \sigma_b - u) \qquad (13)$$

The importance of this approach lies in the possibility of its extension to shear strength and the stress-strain behaviour of cemented soils leading to a better understanding and predictive capability than what is currently possible.

Behaviour under shear : Under shear, cemented soils exhibit a steep rise in strength or a high tangent modulus at low strains, and a softening beyond peak strength. The usual elasto-plastic models can explain only the strain softening associated with volumetric dilation or negative pore pressure which occurs with uncemented overconsolidated soils. But in soft cemented soils (with high liquidity index), softening is always associated with continued increase of positive pore pressure or volumetric compression. Further, a pronounced softening in strength is observed only in undrained shearing wheareas in overconsolidated soils it occurs in drained shearing. A notable feature with undrained softening of sensitive soils is that it is observed even under confining pressures much greater than the yield stress. Figure 22 schematically shows the paths in e–p plane, of drained (constant p tests, for simplicity) and undrained shear tests at different confining pressures. Obviously, at very large strains, when the bonds are completely broken, the soil should reach the same failure states on the critical state line as remoulded soil would do under similar conditions (same void ratio in undrained test and same confining pressure and stress path in drained test). It can be clearly seen why the volumetric compression increases for tests with increasing confining pressures upto the yield stress and then decreases (3-3" > 2-2" > 1-1" and 3-3" > 4-4" > 5-5"). It is also clear that the magnitude of pore pressure or volume change for cemented states is far greater than what would be observed for the corresponding remoulded states.

It appears that all the above features can be explained by extending the hypothesis, proposed earlier, to shear behaviour that the yielding or deformation is entirely due to changes in the component of stresses on the unbonded soil skeleton excluding the bond resistance, and that the actual shearing resistance of the soil at any stage i, is the sum of the unbonded skeleton resistance and the cementation bond resistance, i.e., at any strain level.

$$q_i = q_{ri} + q_{bi} \tag{14}$$

This mode of superposition was suggested earlier by Conlon (1966) and then by Feda (1982).

An approach to predict the shear behaviour of soft cemented clays by suitable constitutive model has been developed (Vatsala et. al., 1994) wherein the above mode of superposition has been adopted. The behaviour of unbounded soil skeleton is predicted and the cementation bond resistance quantified by experiments, is added at different strain levels to get the overall response. A quantitative prediction of the undrained behaviour of Osaka clay has clearly indicated the viability of this approach and hence provides a base for the mathematical formulation of elastoplastic models for sensitive soils.

Partly Saturated Soils

Most of the natural soils above ground water table (or above capillary zone) and invariably all soils in their as compacted state are partly saturated. Some of the specific problems associated with such soils are :

i) compression due to applied stresses
ii) heave or collapse and swelling pressure development upon inundation
iii) drying shrinkage and
iv) shear strength and its change upon soaking

The early efforts (Atchison and Donald, 1956; Croney et al., 1958; Bishop, 1961; Jennings, 1961 and others) were, in general, the attempts made mainly to modify Terzaghi's conventional effective stress relation to account for partial saturation in the form :

$$\sigma' = -u_s + \chi(u_a - u_w) \tag{15}$$

where u_a and u_w are the pore air and water pressures respectively, and χ is a constant ranging between 0 and 1, and is a function of the degree of saturation. But, such effective stress equations failed to answer satisfactorily the volume changes under different loading conditions.

The difficulties in finding a unique relation led to the acceptance of two independent stress fields of the prevalent three i.e., $(\sigma - u_s)$, $(\sigma - u_u)$ and $(u_a - u_w)$ in the overall framework (Alonso, 1990; Coleman, 1962; Fredlund et. al., 1978; Morgenstern, 1987; Lloret and Alonso, 1985; Matyas and Radhakrishna, 1968; Tolls, 1990) for the analysis of the shear behaviour of partly saturated soils. There are specific situations such as unloading from virgin compression wherein with the knowledge of two stress components the effective stress can be reckoned since the pore water tension mobilizes to keep the system in equilibrium with effective stress equal to preconsolidation stress. Satisfactory answers by the consideration of the above stress variables are not possible for other total stress and suction probes such as increase or decrease of suction under constant load, and drying of prestressed soils.

Micro-structural Considerations : To understand the mechanism of stress transfer in partly saturated fine grained soils it is necessary to visualize the probable micro-structure. In an interacting partly saturated system, for monotonic loading, independent stress fields for different phases are not valid and the overall stress field is the only acceptable stress field (Block, 1978). The pore water and pore air pressures and the physico-chemical inter-particle forces are all interdependent and are consequences of the total stress variations. It is necessary to consider the level of partial saturation, before considering pore air-water interfaces affecting the inter-particle forces directly dependent or independent of total stresses acting on the system.

Generally, three broad classifications of partly saturated states can be recognized. For simplicity, let us consider the state of compacted soils. For soils with low degree of saturation (< 50%) which have been regarded as extremely dry clays (Barden, 1965) the water is firmly attached to the skeleton by capillary forces. The air voids are completely interconnected and only air can flow out of the system. The next range of degree of saturation prevalent in soils when compacted dry of optimum is between 50% to 85%. The water still does not flow from the soil to any appreciable degree possibly because of internal drainage. The air voids are still continuous and air is again the only fluid that can flow out of the system. For the soils wet of optimum ($S_r > 85\%$) the value of air- permeability drops off rather abruptly due to the sealing by water of the thin necks between the air filled larger voids. At high degrees of saturation beyond 85 percent, water phase is continuous but, the air which would tend to be in the form of discrete bubbles within water phase, is discontinuous. Such occluded bubbles just exist within the liquid phase without altering the structure of the saturated soil matrix. Their presence does not affect the basic nature of the soil behaviour except the responses being affected due to the altered compressibility of pore fluid.

While soils of very low degrees of saturation are not of practical interest in geotechnical engineering and effects of high degrees of saturation can be encompassed within the framework of the behaviour of saturated soils, it is the intermediate range that attracts the attention of geotechnical

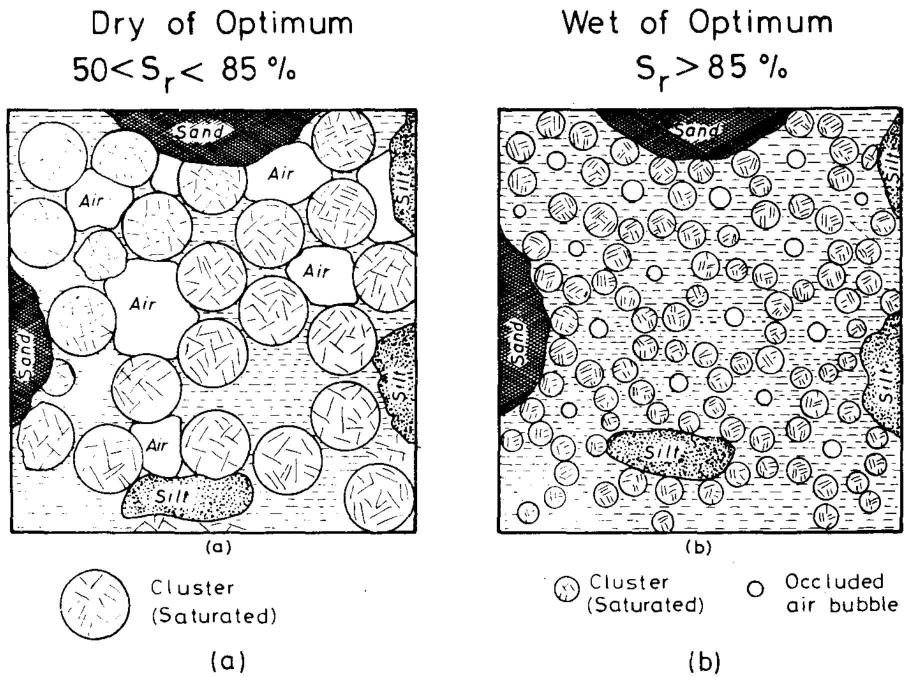


FIGURE 23. Possible microstructure of uncemented partly saturated soil both at low and high degree of saturation

engineers in the development of appropriate approaches to analyze and predict soil behaviour.

In retrospect, the clay particles are likely to be grouped together to form clusters which can get arranged enclosing large pores within themselves as shown in Fig. 7. Diffuse double layer interactions and stress transfer through osmotic repulsion prevail between two such clusters. The structure of partly saturated soils is likely to be very much similar, except that some or all of the large pores may be filled with air. The possible microstructural details of partly saturated fine grained soil in the intermediate and high degree of saturation are shown in Fig. 23. The sub-microscopic pore spaces within the clusters are likely to be completely saturated because of the very high affinity of particle surfaces to water and also the amount of water required to fill all these spaces is quite small, being only 5 to 10 percent of total void volume. The air-water interphase is formed by the menisci which bridges the space between two cluster units around the air pore. A capillary suction $(u_s - u_w)$ is sustained by these menisci, the magnitude of which is a function of the curvature of the menisci, as per the relation :

$$\left(u_s - u_w\right) = 2T/R_c \tag{16}$$

This capillary suction $-u_w$ (u_u being atmospheric) will be equal, in equilibrium, to osmotic pressure (R – A) which may be same or different from total stress at that state.

For a monotonic loading, from an unstressed state, the net osmotic pressure (R – A) between clusters is equal to applied stress, and the same osmotic suction is transformed into capillary suction, $-u_w$ at the air-water interface. Hence the total stress, σ is the equilibrium stress in the entire system and is the effective stress.

$$\sigma' = \left(R - A\right) = \left(\sigma - u_w\right) \tag{17}$$

where (R – A) is the osmotic suction balancing σ with excess u_w being 0.

On the contrary when the equillibrium stress is decreased with no access to water, due to lack of dilation, (R – A) remains practically the same with the stress decrement being balanced by the enhanced capillary suction resulting in a relation :

$$\sigma' = \left(R - A\right)_{@\sigma c} = \sigma_a - \left(\sigma - u_w\right) \tag{18}$$

where σ_c is the maximum past stress and σ_a is the present stress level which is less than σ_c. Upon nullifying the capillary suction by inundation in the first case, collapse takes place whereas, heave is the result in the second case.

Compressibility : The analysis of compressibility data (Nagaraj and Srinivasa Murthy, 1985 and Pandian et al., 1992) show that the e vs log σ_v' paths lie above that of the saturated e vs log σ_w' path, the deviation increasing with decrease in degree of saturation 50 to 85%. It is clear that no unique relation exists between e and σ_v for partly saturated soils and that it depends upon the degree of saturation. But the same data when plotted in $e\sqrt{S_r}$ vs log σ_v plot, shows a unique relation irrespective of the initial water content of the form :

$$e\sqrt{S_r} = a' - b'\log(\sigma_v') \tag{19}$$

If during shearing, the sample gets saturated (or $S_r > 85 - 90\%$ for practical purposes) further shearing may follow an undrained path, if there is no drainage. Hence, it may be logically inferred that the unconfined compressive strength of a compacted soil is unique for a given compactive effort, since the effective stress induced is same. Its magnitude can be related to the parameter $e\sqrt{S_r}$ since this parameter is uniquely related with shear strength or the compactive effort.

Behaviour under shear : As already stated, even if there is no provision for external drainange, it will always be a drained condition under shear for partly saturated soils because of internal drainage. Though a truly undrained condition can be achieved in the laboratory under controlled conditions it is not a likely field situation. Therefore, only the drained shear behaviour under constant water condition has to be considered. To describe the failure shear strength or the complete stress-strain behaviour, the internal friction or the friction factor M, should be known in addition to the compressibility coefficient or the plastic modulus. The general present day understanding (Fredlund et al. 1978; Tolls, 1990 and others) on this aspect is to consider two different friction angles ϕ_a' and ϕ_b' or equivalently the friction factors M_a and M_w for total stress and suction probes so that the shear strength can be expressed as

$$\tau = (\sigma - u_a)\tan\phi_a' + (u_a - u_w)\tan\phi_b' \tag{20}$$

or

$$q = M_a(p - u_a) + M_w(u_a - u_w) \tag{21}$$

However, a few researchers (Alonso et.al, 1990) have considered an increased cohesion due to suction. Since the effective stress which is the net interparticle repulsive pressure is the same for both total stress and suction probes, it may not be reasonable to have two different friction probes. The friction coefficient would be a single value with respect to effective stress which may however be different from that for the remoulded saturated state because of the basic difference in the structure. An initially unsaturated soil, will have a structure with a greater number of soil pocket which are not enclosing large pores. Thus at a given void ratio, there may be a greater number of interacting particle units on a given cross section, which are actually the sites of shearing resistance. Hence the friction factor for the partly sturated state M_{ps} may be higher than that for remoulded state.

The magnitude of this M_{ps} may turn out to be a function of initial water content which determines the initial structure. With continued shearing, the value of M_{ps} may decrease, and reach the normal value of M for remoulded state at very large strains. These aspects are yet to be established with extensive experimental programme. Once the compressibility parameters λ and κ, and the friction factor M_{ps} are established, the stress-strain behaviour can be modelled by the usual elasto-plastic models.

While analysing the behaviour of partly saturated soils, we see that the degree of saturation is an additional state parameter in addition to void ratio which is directly related to water content for saturated states. Hence the equations can be solved only if one of them is kept constant, or can be expressed as a function of the other, as under a constant water condition or known final degree of saturation which are infact the real practical situations.

If during shearing, the samples get saturated (or $S_r > 85 - 90\%$ for practical purposes), further shearing may follow an undrained path, if there is no drainage. The corresponding strain computations for each stress path can be done using the appropriate parameters. From the discussions presented above, it may be logically inferred that the unconfined compressive strength of a compacted soil is unique for a given compactive effort, since the effective stress induced is same. Its magnitude can be related to the parameter $e\sqrt{S_r}$ since, this parameter is uniquely related with p or the compactive effort (Nagaraj and Tiwari, 1985).

The above mode of analysis of partly saturated fine grained soil behaviour cannot be directly extended to partly saturated insitu soils due to the cementation effects (desiccation bonding and/or residual cementation in the formation of tropical soils due to weathering) apart from partial saturation.

Development of Generalized Soil State Parameter - Effective Stress Relationship

In a clay water system wherein the stress transfer is assumed to be through the interacting fluid phase the change in the soil state when the pore water pressure is dissipated can be assumed to be dictated by the requirement of equilibrium between the long range forces, (R – A) mobilized between the interacting units and externally applied stress. Accordingly from Gouy-Champman diffuse double-layer theory, based on the significant finding (Sridharan and Jayadeva, 1982) that for the three basic sheet clay minerals namely kaolinite, illite and montmorillnoite for the assumed parallel plate model d vs log (R – A) is practically same for the same physico-chemical environment. Accordingly, the average adsorbed water layer thickness is about the same for all particle surfaces. Hence, we can express the equilibrium condition by the relation of the form.

$$d = a - b\log(R - A) \tag{22}$$

and for equilibrium

$$(R - A) = \sigma - u = \sigma' \tag{23}$$

where d is the average half space distance between particles

(R – A) is the net internal repulsive pressure and

a & b are constants

With the use of Langmuir equation and the modified relationships of van Olphen (1963) between mid-plane potential and half space distance in the assumed parallel plate model, the half space distances corresponding to various magnitudes of repulsive pressure have been computed for three clays for the properties as detailed in Table 3 (Nagaraj and Srinivasa Murthy, 1983)

Table 3
Properties of Soils used in Analytical Study

Soil type	Sp. surface m^2/gm	Base exchange capacity μ eq/gm
Kaolinite	15	30
Illite	100	400
Montmorillonite	800	1000

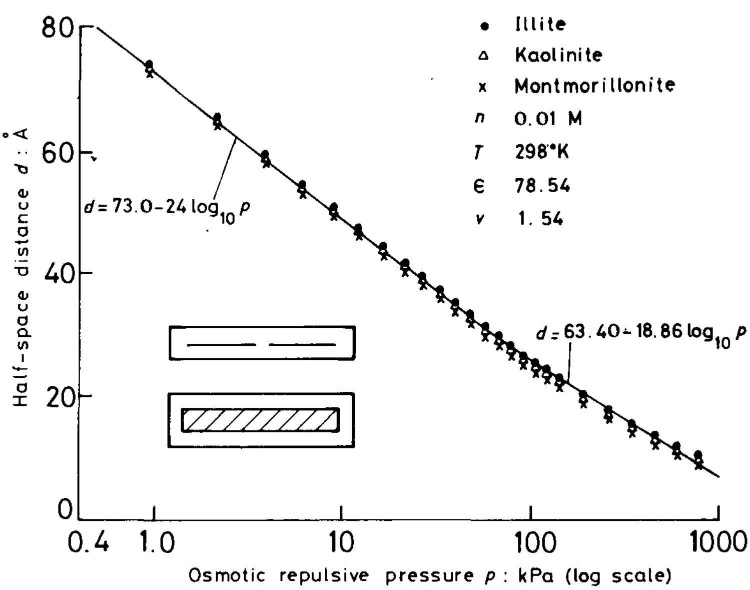


FIGURE. 24 Linearized half space distance, d versus osmotic repulsive pressure (R - A) plot (Nagaraj and Srinivasa Murthy, 1983)

Fig. 24 shows the plot between half space distance and Osmotic repulsive pressure. Irrespective of clay type, for the osmotic repulsive pressure range of 40 – 800 kPa, the $d - \log_{10}(R - A)$ plot can be linearized in the form

$$d = 63.40 - 18.86 \log_{10}(R - A) \tag{23}$$

with a correlation coefficient of 0.992

In the above equation the effect of clay type on the $d - \log_{10}(R - A)$ relationship is subdued by the respective specific surfaces. To ensure generality of the (R – A) relationships further investigations were carried out by considering the possible field variation in the values of electrolyte concentration n from 0.007 to 0.013 and that of valency v as 1 and 2 (Nagaraj and Srinivasa Murthy, 1986).

Examination of the above analytical approaches reveals that fundamental relations between half space distance, d vs (R – A) would be general with respect to principal types of clay sheet minerals having very wide variations in specific surface which get subdued in these relations.

How the above relations can be transformed for practical use to encompass the parameters commonly determined in geotechnical engineering practice merits examination.

For the parallel plate model, which is the basis in the above analytical formulation, Bolt (1956) and Nagaraj and Jayadeva (1981) have inter-related void ratio, a macro-parameter with geometrical characteristics of clay units, external surface area and separation distance by the relation

$$e = G\gamma_w S d \tag{24}$$

When γ_w is in g/cm^2

S is in m^2/g and

half-distance space d is in AU

then

$$e = G\gamma_w S d \times 10^{-4}$$

For saturated soils $e = wG$; when water content w is expressed as percentage, the above equation reduces to

$$w = 0.01Sd \qquad \text{and} \qquad d = w/0.01S \tag{25}$$

The analytical relationship can be expressed as

$$\frac{w}{0.01S} = 63.40 - 18.86\log(R - A) \tag{26}$$

$$\frac{w}{S} = 0.6340 - 0.1886\log(R - A) \tag{27}$$

Reiterating again the microstructure considerations of soil water systems at their liquid limit state, it is possible to ascribe a value of 6 kPa to osmotic suction (R – A), the corresponding equilibrium water content being w_L

$$\frac{w_L}{S} = 0.6340 - 0.1886\log_{10} 6 \tag{28}$$

$$w_L = 0.4872\,S \tag{29}$$

or

$$S = 2.052\,w_L \tag{30}$$

On substitution of this value in Eq. 26 and replacing (R – A) by (σ – u) the resulting equation is the following generalized state – effective stress relation

$$\frac{w}{w_L} = \frac{e}{e_L} = 1.3 - 0.387\log(\sigma - u) \tag{31}$$

As it is obvious, parallel plate model is not representative of the actual situation of particle dispositions in natural soils and individual particle interactions do not prevail during the entire stress range of engineering interest. Hence the possible recourse is to consider the monotonic compression paths of soils whose liquid limit water contents encompass the range encountered in geotechnical engineering practice, to reevaluate the constants in the Eq. 31. The compression paths of eleven soils with wide range of liquid limit water contents from 36 to 160% where the consolidation has been started from water contents corresponding to liquid limit state collated from published literature has been considered (Nagaraj and Srinivasa Murthy, 1986). The curves are spread out, with the position of each curve being in the order of its liquid limit water content. When the water contents at different consolidation pressures are normalized by the corresponding liquid limit water contents, all the normalized points fall within a narrow band. (e/e_L) versus consolidation pressure within the stress ranges of 25 to 800 kPa has been represented by a linear equation in the form (Nagaraj and Srinivasa Murthy, 1986) :

$$\frac{e}{e_L} = 1.122 - 0.234\log_{10}(\sigma - u) \tag{32}$$

with a correlation coefficient of 0.962 and standard error of estimate of 0.025.

In this phenomenological relation, it is presumed that the initial microstructure at liquid limit state is identical. This possibility with distinctly different liquid limit water contents, with the same fraction of sand, silt as coarse particles has been schematically shown in Fig. 10. In a more recent investigation (Nagaraj et al., 1994) it has been shown that water contents far higher than but proportional to liquid limit state of soils would also reflect the same order of physico-chemical potential per unit weight of soil for different soils. For the range of liquid limit water contents considered, the sediment void ratio is 1.76 times the void ratio corresponding to liquid limit water content. It has not yet been possible to establish that at this void ratio the microstructure is same for all clays. Further, the possibilty of formation of self-supporting structure as is in the case of liquid limit stage cannot be easily established. The principles and potentials of considering water content of fine grained soils at their liquid limit state in the analysis

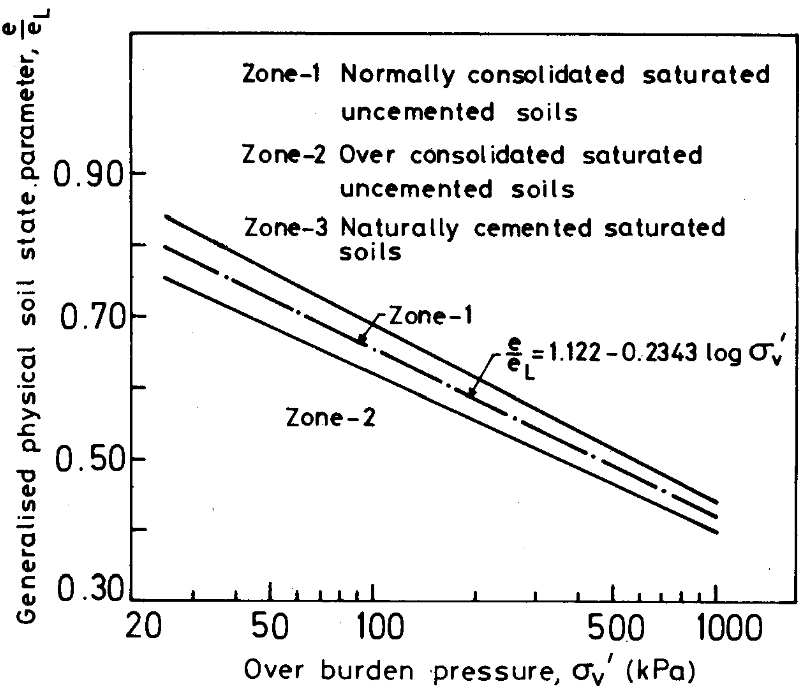


FIGURE 25. Generalized state parameter versus effective overburden pressure

of soil behaviour has been discussed in detail elsewhere (Nagaraj and Srinivasa Murthy, 1988; Nagaraj, 1993).

Recently, Burland (1990) has tried to characterize the intrinsic compressibility behaviour of normallally consolidated soils in terms of a new soil parameter, the void index $(e-e_{100})/(e_{100}-e_{1000})$ where e_{100} and e_{1000} are the void ratios at consolidation pressures of 100 and 1000 kPa respectively. From an analysis of an extensive set of data it has been shown that the required parameters e_{100} and C_c can be related to the liquid limit void ratio e_L of the form :

$$e_{100} = 0.109 + 0.679\, e_L - 0.089\, e_L^2 + 0.01\, e_L^3 \tag{33}$$

$$C_c = 0.257\, e_L - 0.04 \tag{34}$$

The resulting C_c and e_{100} are shown to agree very well (Fig. 34) with those predicted from the generalized equations over a wide range of liquid limit values.

It is very interesting to note that computed ($e_{100}/e_L = 0.653$) obtained from value of e at 100 kPa from the Eqn. 32 is very close to the average value of 0.662 of all the soils considered by Burland (1990). As such, the generalized relation of Eqn. 32 obviates the need to conduct consolidation test and hence can be a good practical tool to assess the compressibility characteristics of uncemented saturated fine grained soils.

Fig. 25 is a plot of generalized state parameter versus effective oberburden pressure σ_v' from both the Eqn. 32. It can be seen that a narrow band can be identified to encompass minor variations. As such three distinct zones can be identified for classification of the in-situ state of soil as normally consolidated, over consolidated and naturally cemented soft clays. Apart from classification the generalized state-effective stress relation can be used to trace the compression path of saturated uncemented soil devoid of any stress history effects. For all practical purposes this forms generalized one dimensional compression path of the soil.

Generalized Approach for Coarse Grained Soils

Having so far discussed in all its ramification the generalized state-effective stress relations for fine grained soils in order to encompass the entire spectrum of particulate materials, it would be logical to examine the possibility of having similar state parameter approach for coarse grained soils also.

To reiterate again, the stress transfer in coarse grained granular medium is essentially through physical contacts between grains by frictional bonds. Under an applied load increment, the particles slide finally to reach an arrangement that is most stable under the new set of loading with the force at each of the contacts being normal to the contact area. Cohesionless coarse grained soils can exist in nature or can be placed under degrees of density although this range compared to fine grained soils, is small.

The existence of coarse grained soils, most often than not, would be in their denser state. Nothing prevents the grains for physically being close to each other so that the void spaces are minimum. As such a sandy soil, under isotropic stress state, may withstand pressure far higher than the range of engineering interest before undergoing appreciable compressions. Hence it becomes difficult to obtain the linear portion of the compression path similar to that of normally consolidated clays. In clays the interparticle forces prevent the particles from being forced into closer proximity. As a result, fine grained soils exist in natural state at void ratios far higher than sands and undergo appreciable compressions which are of engineering

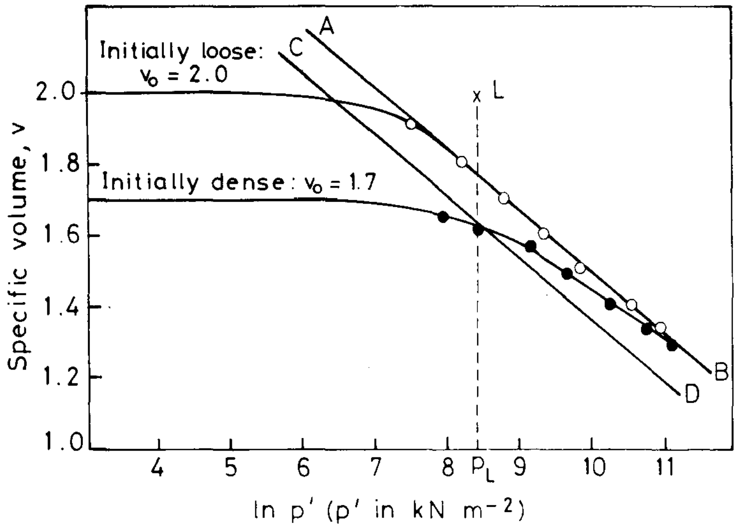


FIGURE 26. Specific volume, v versus initially loose and dense sands (Date by Vesic and Clough, 1968)

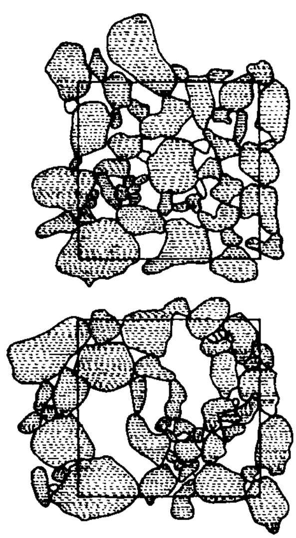


FIGURE. 27 Variation of local porosity with the same overall porosity

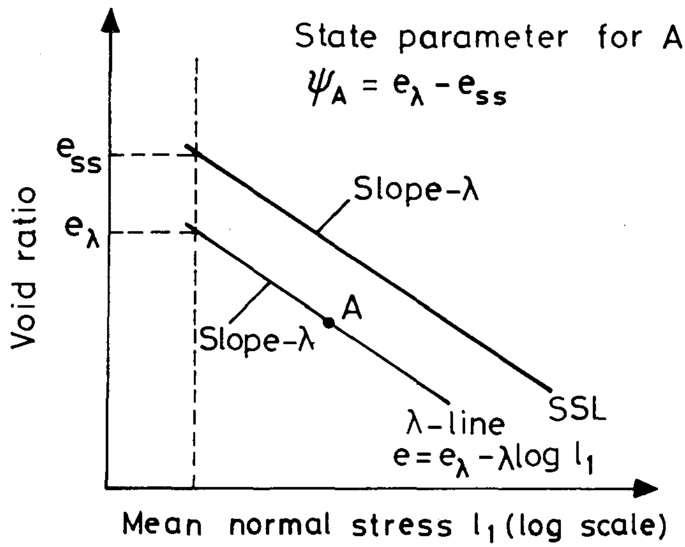


FIGURE. 28 Definition of state parameter (after Been and Jefferies, 1985)

interest. On the contrary, the occurrence of sand at exceptionally loose state, such as point L in the Fig. 26, is rather difficult to comprehend.

Another factor that makes the generalization of the behaviour of coarse grained soil difficult is that coarse grained soil can have large variations in local porosity even though gross porosity may be the same (Fig. 27) (Mogami, 1965). All the above discussions point to the fact that the response of sand is primarily density dependent and pressure dependent and not completely inter-independent as in fine grained soils. Hence the formulation of State-effective stress relation to encompass a wide spectrum of coarse grained soils is very unlikely.

Not withstanding this predicament, extensive studies on the combined influence of stress level and density on strength of sands have been reported by Been and Jefferies (1985) and Bolton (1986). The state parameter, identified by them as a single parameter to combine the effects of density and mean effective stress is calculated as the difference in void ratio, e between the present state and that the sand would have if brought to a critical state (constant volume shearing without further change in effective stress) at the same mean effective stress (vide Fig. 28). At this stage, the data for which generalization of behaviour has been attempted with this state parameter, is very limited and quite often with large scatter. Moreover, there is no logical basis as to how different sands can be generalized with just one state parameter when their steady state lines have significantly varying slopes.

Analysis and Prediction of Soil Behaviour

The proof of pudding is in eating. It has been possible to recognize the role of interparticle forces in fine grained soil behaviour and to encompass the effects of the same in the generalized state-conventional effective stress relations

$$\frac{w}{w_L} = \frac{e}{e_L} = a - b\log_{10}(\sigma - u) \tag{34}$$

in assessing the engineering properties viz., compressibility, strength and permeability (Nagaraj et al., 1994).

Further the independent parameters required to identify and predict the behaviour of each of the soil state would progressively be more as the degree of variability increases. For example, preconsolidation pressure term is to be incorporated in the generalized relation in addition to in-situ water

Analysis and prediction of soil behaviour with Minimum Input Parameters (Nagaraj et al., 1994)

1.	Normally consolidated uncemented saturated soils (Reference state)	– w_L or e and σ'_v
2.	Overconsolidated uncemented saturated soils	– w_L, e and σ'_v
3.	Partly saturated uncemented soils	– w_L, e, S_r and σ'_v
4.	Naturally cemented saturated soils	
	a. Soft and sensitive	– w_L, e, S_u and S_t
	b. Stiff cemented	– w_L, e, S_u and σ'_v
5.	Partly saturated cemented soils	– w_L, e, S_r, S_u and σ'_v
6.	Layered or fissured in combination with any one or more of the above compexities	– Not possible from micromechanistic approach

content, effective overburden pressure and liquid limit of soils to be applicable to overconsolidated soils. Degree of saturation will be an additional parameter in the predictive model to encompass partly saturated residual and compacted soils. Cementation bond strength or yield strength would be the additional parameter in the generalized relation for prediction of cemented soil behaviour. It is stressed that none of the above relations would be adequate to predict the behaviour of layered/fissured in-situ soils. The minimum input parameter for the assessment of engineering parameters are summarized below

Concluding Remarks

The principle of Effective Stress as enunciated by Terzaghi (1936) consists of two statements.

The first part states the fundamental effective stress equation as :

$$\sigma' = (\sigma - u)$$

The second part of Terzaghi's statement enunciates the importance of effective stress as :

> **"All measurable changes in volume, deformation and mobilization of shearing resistance are exclusively due to changes in the effective stress."**

The detailed indepth probing so far made and the supporting evidence drawn from the published literature, suggest that the conventional effective stress i.e., total stress minus pore water pressure, is the only effective stress that can be determined experimentally both in the case of coarse grained and fine grained soils.

The fact that volume change and shearing resistance of coarse grained soils are exclusively due to changes in the effective stress needs no substantiation. In the case of fine grained soils for the same level of effective stress, volume changes and shear strength can be different due to stress history effects, partial saturation and changes in the physico-chemical environment. Since the inter-particle forces mobilized are only the consequence due to external stimuli and not independent parameters, the principle of superposition of inter-particle stress components in equilibrium equations as independent parameters is not tenable. As such conventional effective stress considerations in the analysis of volume change and shear behaviour are essential but not sufficient to encompass different insitu states of fine grained soils.

Hence, consideration of the state of the soil, realized due to the mobilized interparticle forces due to external or internal stimulus along with stress components, would provide a better means to analyze soil behaviour due to changes in effective stress. It has been shown that Effective Stress–State Relation of the form

$$e = a - b \log(\sigma - u)$$

is convenient to accommodate the effects of internal stress changes which arise due to mobilized interparticle forces as a result of various environmental factors.

Infact the kernel of the development of critical state concepts by Cambridge Group is to consider both the stress components and the state realized in the analysis of the soil behaviour. With this framework it has been possible to encompass the effects of stress history but not cementation and partial saturation. Perhaps, this is due to not explicitly distinguishing the mode of stress transfer in coarse grained soils and clays. To quote (Schofield and Worth, 1968)

> "Consider a random aggregate of irregular solid particles of diverse sizes which tear, rub, scratch, chip and even bounce against each other during the process of cintinuous deformation. If the motion were viewed at close range we could see a stochastic process of random movements, but we keep our distance and see a continuous flow. At close range we could expect to find many complicated causes of power dissipation and some damage to particles; however, we stand back from the small details and loosely describe the whole process of power dissipation as 'friction', neglecting the possibilities of degradation or of orientation of particles."

Although sands and clays have much in common as members of the same family of particulate materials, their differences are as important as their similarities. In the case of coarse grained soils the mutual interactions between particles is essentially physical and hence the stress transfer is through physical contacts between grains by short range force induced frictional bonds. In clays, the charged particles invariably have a film of water adsorbed around them inhibiting a direct contact of the Bowden-Tabor type. As such in the working range of stresses, the stress transfer can be assumed to be through the interacting long range forces induced friction bonds, with the change in the soil state as the pore water pressure dissipates. This is dictated by the requirement of equillibrium between long range forces (R – A) and the effective stress. Apart from stresses, environmental factors can also bring about changes in the quilibrium state even under the same level of imposed effective stress.

Further it has been possible to characterize the microstructure of fine grained soils and identify the initial effective stress state when the microstructure is of the same pattern for different fine grained soils in relation to which microstructure at other states can be characterized and accounted for in the analysis of soil behaviour. At engineering level, even now, without probing microstructural details the effects of microstructure and its change is taken care of by appropriate parameters such as compression and recompression indices for compressibility, negative pore water pressure and dilational components during shearing. Within the framework developed to analyze the behaviour of fine grained soils, the possibility of analysis of volume change and shear strength behaviour of cemented soils and partly saturated soils has been elucidated.

The basic State-effective stres relation can even be modified to take care of different practical situations which would otherwise not be possible to consider only from the first part of Terzaghi's enunciation. For example, overconsolidated soils, preconsolidation pressure σ'_c can be incorporated in the void ratio – effective stress relation resulting in the form

$$e = a - b \log \sigma'_c + c \log(\sigma'_c / \sigma')$$

where $\sigma' = \sigma'_c$ for normally consolidated soil.

In the case of cemented soils the total stress component has to be reduced to the extent of cementation bond strength to analyze the compressibility response of such soils. The relation can be expressed as

$$e = a - b \log(\sigma - \sigma_b - u)$$

For partly saturated soils the compressibility behaviour can be characterized by

$$e\sqrt{S_r} = a' - b' \log(\sigma')$$

The recognition of the above distinctly different modes of stress transfer in coarse grained and fine grained soils has further permitted even to extend the considerations in the analysis of soil behaviour to predictive mode by appropriate phenomenological models. The methods developed are innovative, simple and employ parameters determined only in routine investigations as input parameters (Nagaraj et al., 1994). The pore fluid holding capacity of fine grained soils, reflected at their liquid limit state has been recognized as the appropriate potential parameter to account for inter-particle forces in the analysis and prediction of soil behaviour.

In retrospect, it can be inferred that in the case of fine grained soils, effective stress mainly reflects the relative magnitude of total stress and pore water pressures as independent parameters and its combined level to which the soil is being subjected to. Consequently, the ocmpressibility and shear strength mobilization are due to this effective stress provided the clay-water-electrolyte system remains unaltered. The mobilized surface force are only reaction to the effective stress. Since this reaction can be varied by varying the physico-chemical environment, the observed volume changes and shearing resistance need not be necessarily a direct reflection of this effective stress. The changed clay-water-electrolyte system would be altogether a different system.

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