Engineering Behaviour of Fine Grained Soils
A Fundamental Approach *

by

A.Sridharan **

INTRODUCTION

Understanding and prediction of engineering properties of clay soils is of vital importance in geotechnical engineering practice. The engineering properties of fine grained soils are strongly dependent on water content. They are susceptible to changes in external pressure, chemistry of the pore medium,wetting and drying cycles and temperature. The complexity in the engineering behaviour of fine grained soils is attributable mostly to the clay size fraction of the soil.

Most soil classification systems arbitrarily define clay particles as having an effective diameter of $2\mu m$ or less and do not account for the clay mineral type. Probably the most important grain property of fine grained soils is the mineralogical composition. The clay sized particles ($\leq 2 \mu m$) which are comprised of different clay minerals are characterised by strong electrical forces that vary in magnitude and nature depending on their mineralogical composition. The variations in mineralogy and associated surface forces impart a wide range of properties to the fine grained soils. The liquid limit of clay soils may range from a low of 30% to a high of 150% or more. The compression index of these soils may range from 0.15 to 0.85 and the swell index from 0.05 to 0.5. Typical values of coefficient of consolidation for natural fine grained soils range from 10^{-3} to 10^{-5} cm^2/sec and coefficient of permeability values vary between 10^{-4} to 10^{-7} cm/sec. Likewise the shear strength of fine grained soils is not a unique property of the soil but depends on several factors such as mineralogy, water content, nature of pore fluid, stress history, drainage during shear, stress path etc.

In comparison to the natural clay soils, the engineering properties of clay minerals exhibit a much wider range and further within a group, the range may also be significant. These variations in the properties are brought about by changes in the type of adsorbed cations/ anions and the nature of pore fluid present and need not be in the same order for different clay minerals.

The development of solutions to the intricate problems of engineering behaviour of clay soils may be achieved by formulating answers to the fundamental questions:

– What are clays and what is the nature of their surface electrical forces?

– How do the clay mineralogical and associated electrical force characteristics contribute to the soil structure, the water holding capacity, the "effective" stress, the volume change, shear strength and permeability properties of clay soils.

It has been the endeavour of the author to focus on the development of answers to these fundmental questions and in the process contribute towards a better understanding of the complex engineering behaviour of clay soils.
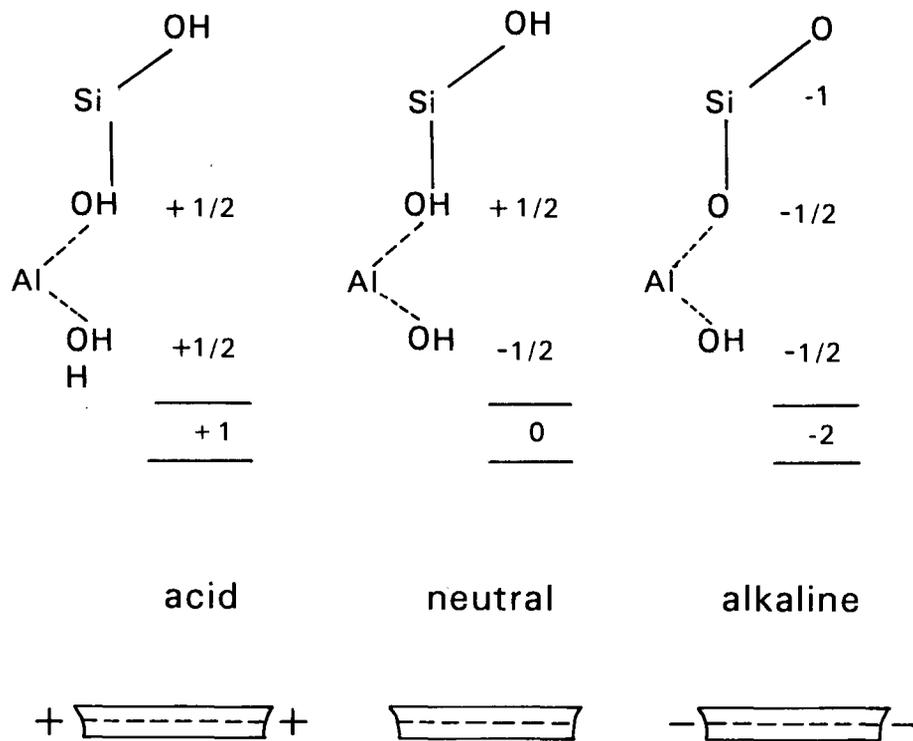
CLAY MINERALOGY

In the soil classification system, clays are defined as the particles of size less than 2 microns. But all particles less than 2 microns do not influence the engineering properties in the same way. The type of the clay mineral plays a dominant role in controlling the physical and engineering properties of fine grained soils. The clay minerals occur in particles of such small size that physico-chemical interactions with each other and with the water-electricity phase of a soil could be significant.

The clay minerals that are commonly found in soils belong to the larger mineral family termed phyllosilicates. They can be broadly grouped into kaolinite (1:1), smectite (2:1), mica (2:1) and chlorite (2:1:1). In the kaolinite group which consists of silica tetrahedral and alumina octahedral sheets, the bonding between

OH OH O
Si Si Si -1
OH +1/2 OH +1/2 O -1/2
Al Al Al
OH +1/2 OH -1/2 OH -1/2
H

_____ _____ _____
+1 0 -2
_____ _____ _____

acid   neutral   alkaline

+ ⟨‒‒‒‒‒‒‒⟩ +   ⟨‒‒‒‒‒‒⟩   − ⟨‒‒‒‒‒‒⟩ −

FIGURE 1   Charge Development on Practicle Layer

successive layers is both by van der Waals forces and hydrogen bonds. This bonding is of sufficient strength such that there is no interlayer swelling upon contact with moisture. Although controversy exists as to whether or not any isomorphous substitution exists within the structure of the kaolinites, values of 3 to 15 meq/100g have been measured. Considerable evidence also exists that kaolinite particles carry positive charges on their edges in a low pH environment and negative charges in a high pH environment (Fig. 1; Taylor 1959). The specific surface area of kaolinite is of the order of 10 to 20m²/g of dry clay.

The minerals of the smectite group (example, montmorillonite) have the octahedral sheet sandwiched between two silica sheets. The layers formed in this way are continuous in the a and b directions with bonding between successive layers by vander Waals forces and by cations that may be present to balance charge deficiencies in the structure. The bonds are weak and easily separated by cleavage or adsorption of water or other polar liquids. There is extensive substitution for aluminium and silicon within the lattice by other cations. Because of large amount of unbalanced substitution in the minerals, they exhibit high cation exchange capacities, generally in the range of 80 to 150 meq/100 g. The specific surface can range from 700 to 840 m²/g.

The most commonly occurring mica like clay minerals is illite or hydrous mica. The layered silica-gibbsite-silica sandwich have about one quarter of the silicon positions filled by aluminium and the resultant charge deficiency balanced by nonexchangeable potassium ions between the layers. Thus the cation exchange capacity of illite is less than that of montmorillonite amounting to 10 to 40 meq/100g. The interlayer bonding by potassium is sufficiently strong so that the illite mineral does not exhibit any inter layer swelling on contact with moisture. Values of specific surface in the range of about 65 to 100 m²/g have been reported for illite.

The chlorite group consists of alternative mica like and brucite like layers with basal spacing fixed at 14A°. The cation exchange capacity is in the range of 10 to 40 meq/100 g. Surface area can vary widely. (for a deetailed discussion on clay mineralogy refere Grim 1958, Mitchell 1976).

The large variations in surface area and charge characteristics of different clay minerals result in a variety of clay particle arrangements (termed clay/ soil fabric). The clay fabric together with the interparticle forces (termed soil structure) determines the consistency limits, volume change, permeability and shear strength behaviour of clay soils. The clay size fraction that are present in natural soil can thus influence its behaviour to an extent much greater than in simple proportion to the amount present depending upon their mineralogy. Thus mineralogy can be considered fundamental to the understanding of geotechnical properties. Atterberg limits, shrinkage limit and free swell index can to a great extent reflect the mineralogical characteristics of the clay size fraction. In this paper the role of type of clay minerals in influencing the physical and engineering properties of clay soils have been brought out.
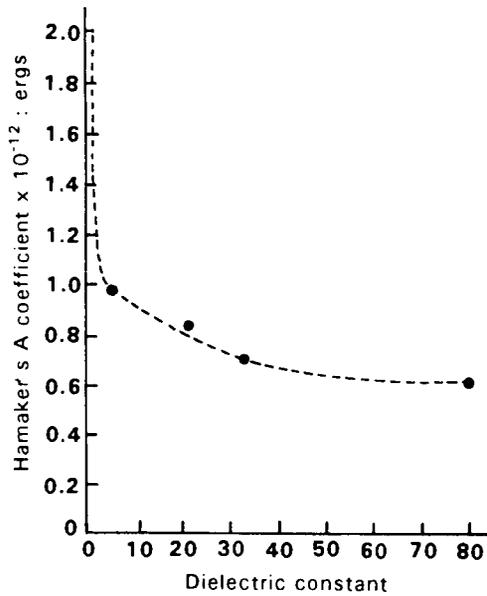
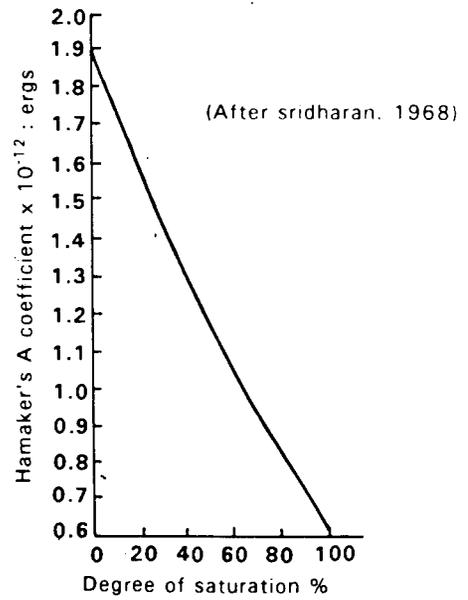**FIGURE 2b** Variation of Hamaker's 'A' Coefficient with Dielectric Constatnt

(After sridharan. 1968)

FIGURE 2a Variation of Hamaker's 'A' Coefficient with Degree of saturation

$v = 1.0$
$\varepsilon = 80.36$
$S = 800 \ m^2/gm$
$BEC = 100 \ meq \ /100gm$
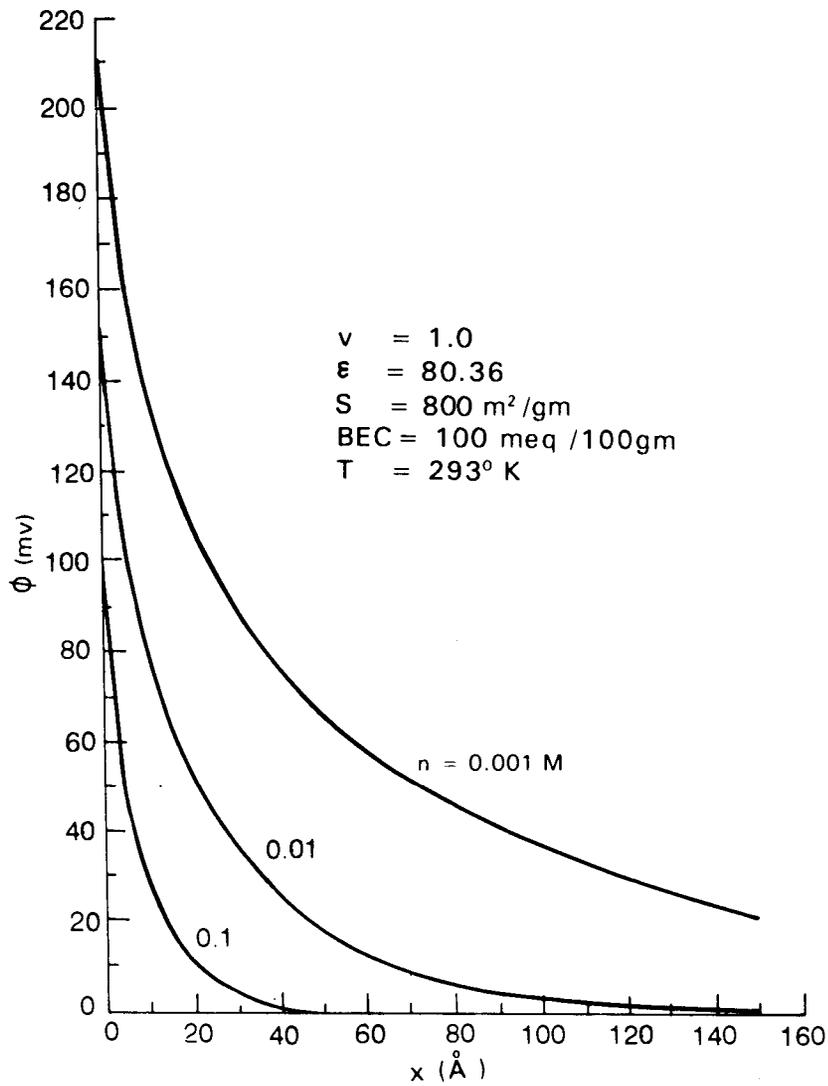$T = 293^\circ K$

$n = 0.001 \ M$

0.01

0.1

FIGURE 3 Effect of Pore Salt Concentration on Potential-Distance Relationships

449
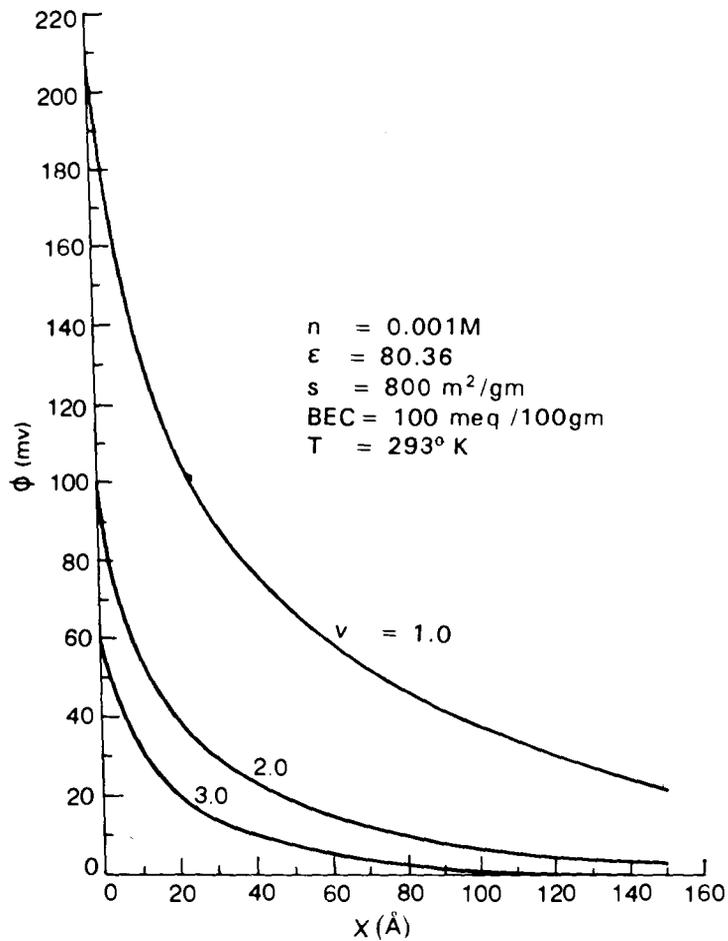
FIGURE 4 Effect of Cation Valence on Potential-Distance Relationships

In figure 4:
- $n = 0.001M$
- $\varepsilon = 80.36$
- $s = 800 \ m^2/gm$
- $BEC = 100 \ meq \ /100gm$
- $T = 293° \ K$

Curves labeled: $v = 1.0$, 2.0, 3.0
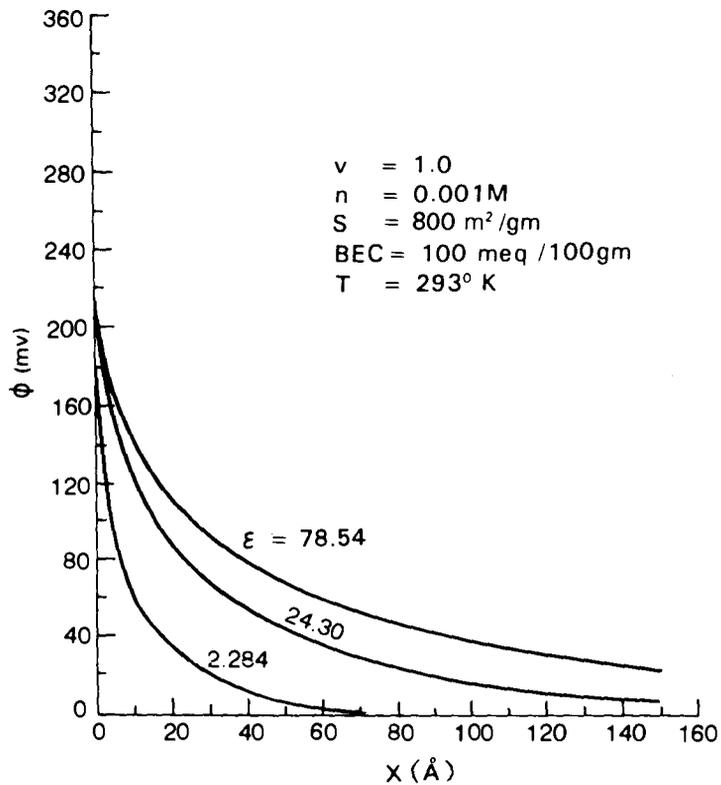
FIGURE 5 Effect of Dielectric Constant of Pore Medium on Potential-Distance Relationships

In figure 5:
- $v = 1.0$
- $n = 0.001M$
- $S = 800 \ m^2/gm$
- $BEC = 100 \ meq \ /100gm$
- $T = 293° \ K$

Curves labeled: $\varepsilon = 78.54$, 24.30, 2.284

$Z = ve\ \phi_o /KT$ at $x = O$

$u = ve\ \phi_m /KT$ at $x = d$
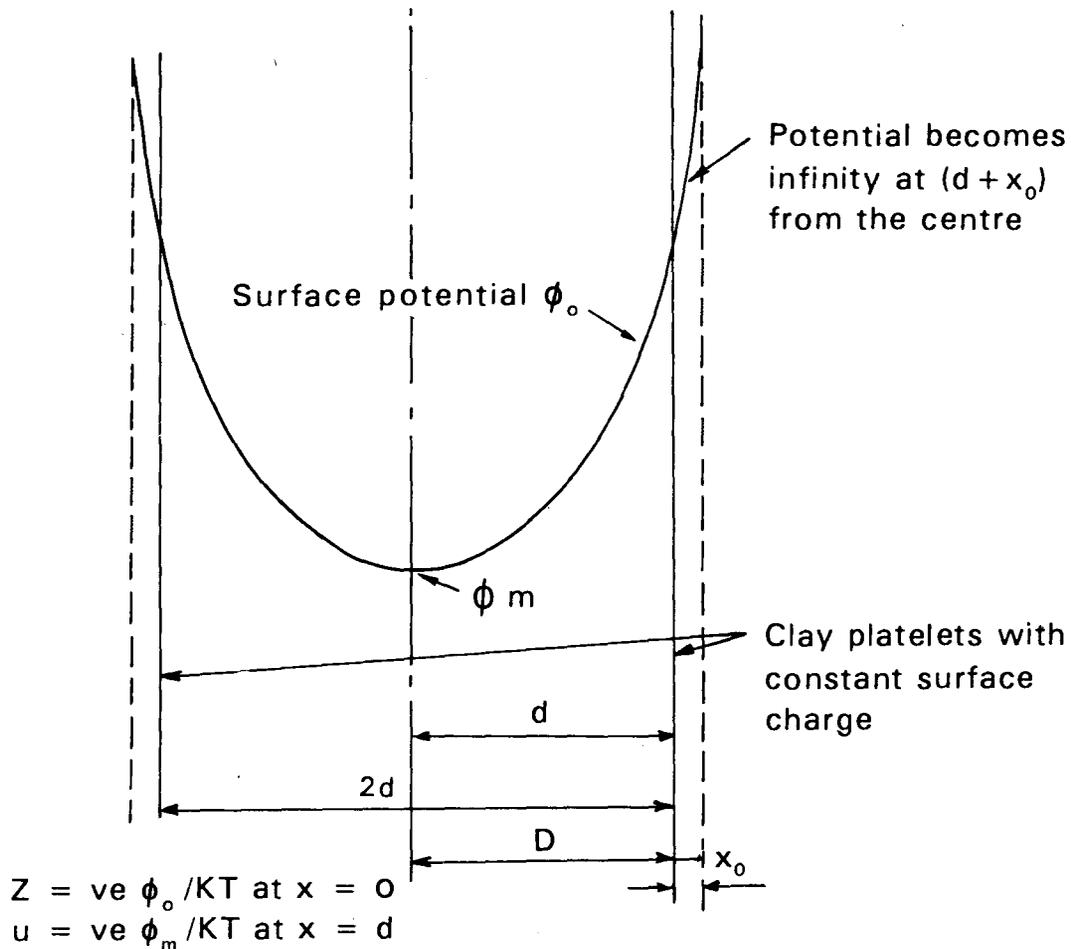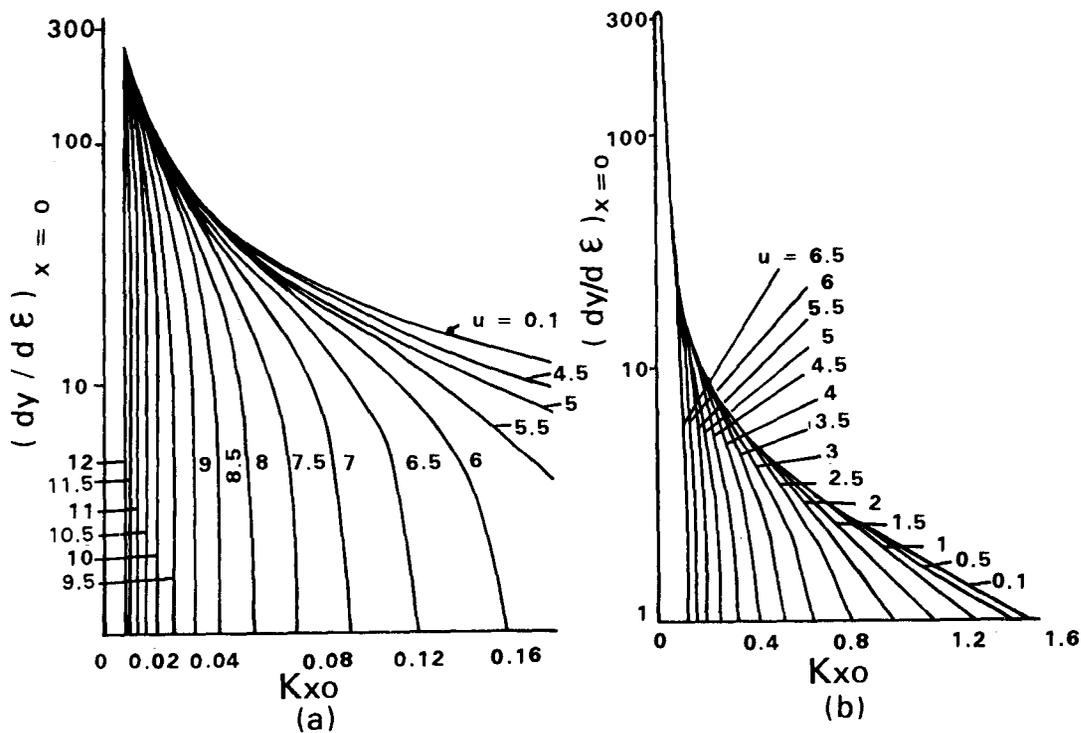
FIGURE 6   Parallel Clay Platelets

FIGURE 7a&b   Relationship between $(dy/dc)x=0$ and Kxo

ELECTRICAL FORCES OF ATTRACTION AND REPULSION

It is well established that both attractive and repulsive forces of electrical nature exist between clay particles. Many complex factors are responsible for the net attractive and repulsive forces between these particles. Number of investigators have attempted to better the understanding of the nature of these forces. Lambe (1953, 1958, 1960a), Bolt (1955, 1956), Rosenqvist (1955), Mitchell (1956), Quirk (1960), Seed, Mitchell and Chan (1960), Sridharan (1968) are among many who have contributed to the understanding of the nature of electrical forces from the view point of mechanical behaviour.

ATTRACTIVE FORCES

A number of phenomena are responsible for the existence of electrical attractive forces between clay particles. They can be primarily grouped as those relating to primary valence bonds, and secondary valence forces, The Coulombic attraction, hydrogen bonds and other possible attractions such as the ion-dipole, or induced dipole interaction, or dipole-dipole interaction (secondary valence forces) are inversely proportional to the dielectric of the medium and the distance between the units (Rosenqvist, 1955; Lambe, 1958). The secondary valence forces are of more concern to the engineer, since they are greatly influenced by applied stresses and by the changes in the nature of the clay-water system and also the fact they can act over relatively large distances. It is likely that the principal contribution to the secondary valence forces is from the mutual influence of the electronic motion between two atoms (London forces). Accoring to London's theory, these forces are universal forces which act between all pairs of atoms or molecules, varying inversely as the seventh power of the distance between them. Hamaker (1937) derived an equation for the attractive force between two plates from London's (1930) theory as

$$F = A/(6 \pi d^3) \qquad \text{..(1)}$$

·where F is the force in dyne per cm², A is the Hamaker constant, and d is the distance between the plates in cm.

There has been much discussion concerning a suitable value for the constant A and values ranging from 5×10^{-14} to 10^{-12} ergs have been estimated by varoius researchers (Quirk, 1960). The value of A can be calculated by summing up the pair potentials between volume elements having polarizability, α_1, ionization potential, I_1, and the number of interacting elements per unit volume, N_1. The summation by Hamaker (1937) gives for the parallel plates in vacuum.

$$A = A_1 = (3/4) \pi^2 N_1^2 \alpha_1^2 I_1^2 \qquad \text{....(2a)}$$

If the medium is other than vacuum, then

$$A = A_{12} = \left[\sqrt{A_1} - \sqrt{A_2}\right]^2 \qquad \text{..(2b)}$$

Where the subscripts 1 and 2 refer to the soil substance and the fluid medium respectively (Fowkes, 1964). Using the approach of Fowkes (1964) and assuming that the number of interacting volume elements is directly proportional to the degree of saturation (i.e., between the two plates there is a uniform and homogeneous distribution of water molecules which is directly proportional to the degree of saturation), Sridharan (1968) computed the values of the A parameter with degree of saturation as shown in Fig. 2a. Sridharan and Rao (1979) calculated the value of 'A' for different soil-liquid systems for which N, α and I values are readily available and is shown in Fig. 2b. It is clear that the A parameter is inversely proportional to the dielectric constant of the medium.

Thus if the soil system consists of a system of parallel plates, the dispersion force contribution to the London-van der Waals' force, which directly varies with Hamaker's A, is inversely proportional to the dielectric constant, when the change in dielectric constant is brought about by the use of the different pore fluids.

The clay-water system, as used in soil engineering, is not an ideal system. The microstructure of clay and particularly the nature of interparticle contacts is not well understood. Whether any specific force predominantly contributes to the attractive force cannot be firmly stated. The system is so complex that individual effects cannot be readily separated. However, it has been shown that the attractive forces vary inversely with the dielectric constant of the pore medium and distance between the particles.

REPULSIVE FORCES

The primary force which is responsible for repulsion between two clay platelets is due to the interaction

of their diffuse double layers. Extensive investigations have been carried out by many investigators (Bolt, 1955, 1956; Schofield, 1946; van Olphen, 1963; and Verwey and Overbeek, 1948 to name a few) on the application of Gouy-Chapman double layer theory, to understand the nature of water next to the clay mineral and the repulsive forces operating between parallel clay platelets. More recently Sridharan and Jayadeva (1980, 1982) and Jayadeva and Sridharan (1981,1982) extensively studied the nature of this electrical double layer as governed by the Gouy-Chapman theory.

The effect of various parameters on the thickness of diffuse double layer (which could be termed as water holding capacity in simple term) has been brought out by Jayadeva and Sridharan (1982). The results could be presented in the form of the electric potential and distance from the platelet. Figs. 3, 4 and 5 bring out the effect of pore salt concentration, cation valence and dielectric constant of the pore medium on the potential-distance relationship (Jayadeva and Sridharan, 1982). It could be concluded that as valence and pore salt concentration decrease, and as dielectric constant increases the double layer thickness increases. The influence of these parameters on the double layer thickness is hence significant.

DOUBLE PLATELET AND REPULSIVE PRESSURE

Bolt (1956) used the following eqn. to estimate the repulsive pressure

$$u \sqrt{(\beta C_o)} \ (x_o + d)$$

$$= 2 \sqrt{\frac{C_o}{C_d}} \int_{u=0}^{u=\pi/2} \frac{du}{\sqrt{[1 - (C_c/C_d)^2 \sin^2 u]}} \qquad \qquad ...(3)$$

$$p = TRC_o \left[\frac{C_d}{C_o} + \frac{C_o}{C_d} - 2 \right] \qquad \qquad ...(4)$$

$$e = Gr_w Sd \qquad \qquad ...(5)$$

where $x^\wedge \approx 4/\upsilon\beta$ ($0.1/\upsilon$ nm for illite, $0.2/\upsilon$ nm for kaolinite and $0.4/\upsilon$ nm for montmorillonite).

Equation (5) is valid for the parallel plate model; it neglects only the surface area contributed by the edges. Equation (4), which is the van't Hoff eqn., can be shown to be the same as the Langmuir eqn., i.e. (Sridharan and Jayadeva 1982)

$$p = 2nkt\,(\cosh u - 1)$$

Equations (3) to (5) can be used to predict the p-e relationship. To find the void ratio of a system corresponding to the repulsive pressure p, eqn. (4) is used to find C_o/C_d, and then d is evaluated from eqn. (3). It is difficult and time-consuming to find p for a given value of e. The estimation of x_o involves approximation. Sridharan and Jayadeva (1982) have given a procedure to evaluate x_o accurately and a simple procedure is given for predicting e for known pressure p or vice versa.

Two important eqns. used in the double layer theory are

$$\int_z^u (2 \cosh y - 2 \cosh u)^{-1/2} dy = - \int_0^d dc = - Kd \qquad \qquad ..(7)$$

$$(dy / dc)_{x=0} = (2 \cosh z - 2 \cosh u)^{1/2}$$

$$= \Gamma \sqrt{(2\pi / \in nkT)}$$

$$\text{at } x = 0, \ y = z$$

$$= (B / S) \sqrt{(2\pi / \in nkT)} \qquad \qquad ..(8)$$

where $\qquad \qquad y = ve'\phi / kT$

$$u = ve'\phi_m / kT$$

$$c = Kx$$

$$K = \sqrt{(8\pi e'^2 v^2 n) (\in kT)} \qquad \qquad ..(9)$$

where B is the base exchange capacity of the clay, ϕ is the electric potential at a distance x from the clay surface, ϕ_m is the electric potential at the midplane and $e' = 4.8 \times 10^{-10}$ esu.

van Olphen (1963) has given, for particular values of $(dy/dc)_{x=0}$ various values of u, z and Kd. When the void ratio e is given, d is found from eqn. (5) and K from eqn. (9). Hence u can be estimated from corresponding values of $(dy/dc)_{x=0}$, and then p can be determined using eqn. (4). The main limitation of this procedure is that tables are not given for all values of $dy/dc)_{x=0}$ and u. Hence interpolations are difficult and inaccurate.

The value of x_o as given by Bolt (1956) involves approximation. The following procedure yields exact values. From Fig. 6, Value of $x_o = D - d$. From eqn. (7),

$$K(D - d) = \int_x^u \frac{- dy}{\sqrt{(2 \cosh y - 2 \cosh u)}}$$

$$- \int_z^u \frac{- dy}{\sqrt{(2 \cosh y - 2 \cosh u)}}$$

or

$$Kx_o = \int_x^z \frac{- dy}{\sqrt{(2 \cosh y - 2 \cosh u)}}$$

or

$$Kx_o = f\ [u,\ (dy / dc)_{x = 0}] \qquad\qquad ...(10)$$

Equation (10) has been solved and Figs 7(a) and 7(b) present Kx_o as a function of slope $(d_y/d_c)_{x=0}$ and u (all parameters are non-dimensional). The slope at the surface is $\Gamma\sqrt{(2\pi/\in nkT)}$. The charge density is related to the base exchange capacity and the surface area by

$$= B/S \qquad\qquad ..(11)$$

Use of an approximate value of x_o in eqn. (3) may lead to large errors, especially at high concentrations and low values of e.

If an accurate value of x_o is used in eqn. (3) (from Fig. 7), it can be shown that Bolt's method (eqn. 3 and 4) and van Olphen's method (eqn. 6 and 7) yield the same results (Sridharan and Jayadeva, 1982).

From Figs 7(a) and 7(b) and eqn. (10) it is clear that x_o depends not only on clay and fluid properties but also on u, the midplane potential which is a function of the distance between the platelets (or e).

The exact solution of eqn. (7) can be obtained by numerical integration and has been presented in Fig. 8 and Table 1. For any void ratio, d can be determined from eqn. (5) and, given the fluid properties, K can be evaluated from eqn. (9). Thus from Fig. 8 or Table 1, u can be determined for known values of Kd and $(d_y/d_oc)_{x=0}$.

Equation 6 relates u and p. Although Fig. 8 includes results for a wide range of values of $(d_y/d)_{x=0}$, for clay systems, the following discussion shows the unique nature of the relationship between u and Kd within the wide ranges of environmental conditions specified.

Table 2 presents typical values of base exchange capacity and surface area for kaolinite, illite and montmorillonite clays as the major components contributing to the permanent negative charge of soil clays.

u AGAINST p FOR DIFFERENT CONCENTRATIONS

Table 3 presents values of $(dy/dc)_{x=0}$ for kaolinite, illite and montmorillonite as affected by concentrations, and values of u for different pressure ranges as affected by concentrations. For all ranges of pressure and concentration, the u values are such that the u-Kd relationship (Fig. 8 and Tables 2 and 3) can be treated as unique. Hence eqn. (6) can be written as (Sridharan and Jayadheva, 1982).

$$\frac{p}{nT} = 2k \cosh f\left[\sqrt{\frac{8\ \pi\ ne^{'2}\ v^2}{\in\ kT}}\ d\right] - 1 \qquad\qquad .....(12a)$$

$$\frac{p}{nT} = 2k \cosh f\left[\sqrt{\frac{8\ \pi\ ne^{'2}\ v^2}{\in\ kT}}\ \frac{e}{G\ \gamma_w S}\right] - 1 \qquad\qquad .....(12b)$$

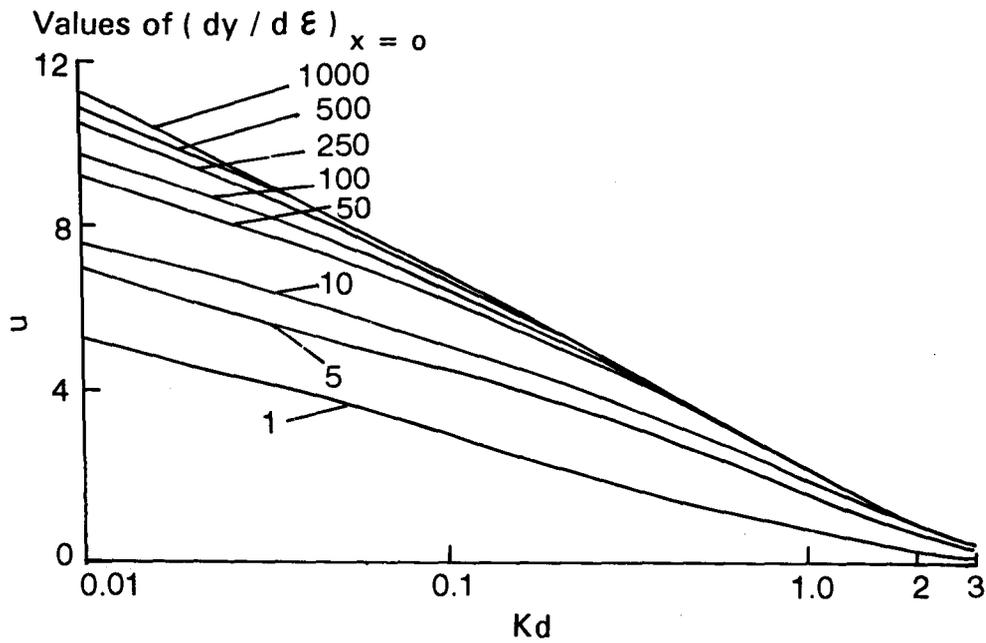Values of $(dy / d \varepsilon)_{x = 0}$

FIGURE 8   Relationship between u and Kd for Various Values of (dy/dc) x = 0

p: kg/cm²

(a)
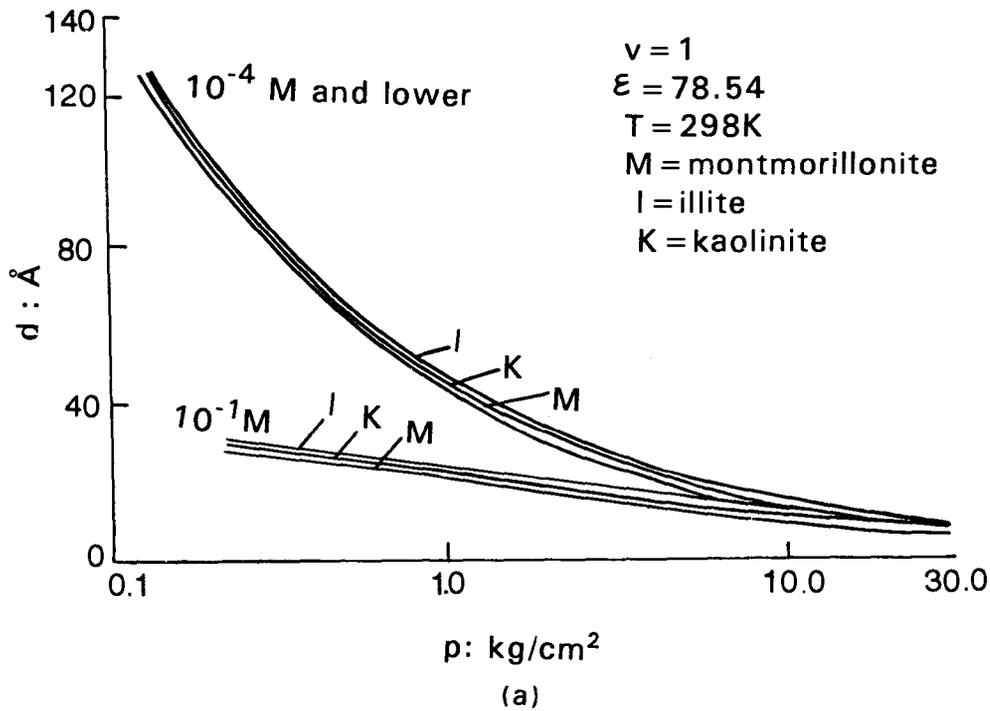
FIGURE 9a   d Plotted against log p for Various Clay Types and Concentrations

Thus a plot of p/nt against Kd should be unique. As $(d_y/d_c)_{x=0}$ represents the clay type (i.e. B/S), it has a negligible influence on the u-Kd relationship, and in the final relationship between p and d, the effect of soil type becomes insignificant. Figure 8 shows that u plotted against log Kd is linear for large values of $(d_y/d_c)_{x=0}$ and $u > 1.0$ so that

TABLE 1

VALUES OF KD FOR VARIOUS VALUES OF u AND dy/dc)$_{x=0}$

u	Values of (dy/dc)$_{x=0}$							
	1	5	10	50	100	250	500	1000
0.1	2.938	3.989	4.181	4.339	4.359	4.371	4.375	4.377
0.5	1.351	2.346	2.536	2.695	2.715	2.727	2.731	2.733
1.0	0.735	1.590	1.777	1.935	1.955	1.967	1.971	1.973
1.5	0.435	1.125	1.306	1.463	1.483	1.495	1.499	1.501
2.0	0.264	0.795	0.966	1.121	1.141	1.153	1.157	1.159
2.5	0.161	0.553	0.709	0.862	0.882	0.894	0.898	0.900
3.0	0.098	0.375	0.513	0.662	0.681	0.693	0.697	0.700
3.5	0.060	0.249	0.365	0.506	0.526	0.538	0.542	0.544
4.0	0.036	0.161	0.253	0.385	0.405	0.417	0.421	0.423
4.5	0.022	0.102	0.171	0.292	0.311	0.323	0.327	0.329
5.0	0.013	0.064	0.113	0.219	0.238	0.250	0.254	0.256
5.5	0.008	0.040	0.073	0.162	0.181	0.193	0.197	0.199
6.0	0.005	0.024	0.046	0.118	0.137	0.148	0.152	0.154
6.5	0.003	0.015	0.029	0.085	0.102	0.114	0.118	0.120
7.0	0.002	0.009	0.018	0.060	0.076	0.087	0.091	0.093
7.5	0.001	0.006	0.011	0.041	0.055	0.066	0.070	0.072
8.0	0.001	0.003	0.007	0.027	0.039	0.050	0.054	0.056
8.5		0.002	0.004	0.018	0.027	0.037	0.041	0.043
9.0		0.001	0.002	0.011	0.019	0.027	0.031	0.033
9.5			0.001	0.007	0.012	0.020	0.023	0.025
10.0			0.001	0.004	0.008	0.014	0.017	0.019
10.5			0.001	0.003	0.005	0.010	0.013	0.015
11.0				0.002	0.003	0.007	0.009	0.011
11.5				0.001	0.002	0.004	0.006	0.008
12.0				0.001	0.001	0.003	0.004	0.006

$$u = 2.35 - 4.375 \log_{10} Kd \qquad ....(13)$$

Thus from eqn. 6 and Table 1 or eqn. 13 one can obtain d-p relationship and with eqn. 5, e-p relationship using the Gouy-Chapman double layer theory.

Fig 9(a) shows the d-log p relationship for the three clays (Table 2) for different pore salt concentrations. It is seen that clay type has no effect irrespective of pore cation concentration. Fig 9(b) shows the influence of clay type on e-log p behaviour. The effect is indeed significant. This brings out the inference that the specific surface is the parameter which significantly influences the e-log p behaviour as per double layer theory and takes care of the soil type when pore medium characteristics are same.

In the later sections results of diagnostic experiments performed to elucidate the mechanisms governing the engineering behaviour of fine grained soils are presented. Experimental results strongly suggest that the diffuse double layer theory is capable of explaining the engineering behaviour of swelling montmorillonites alone and does not answer the behaviour of non-swelling kaolinites.
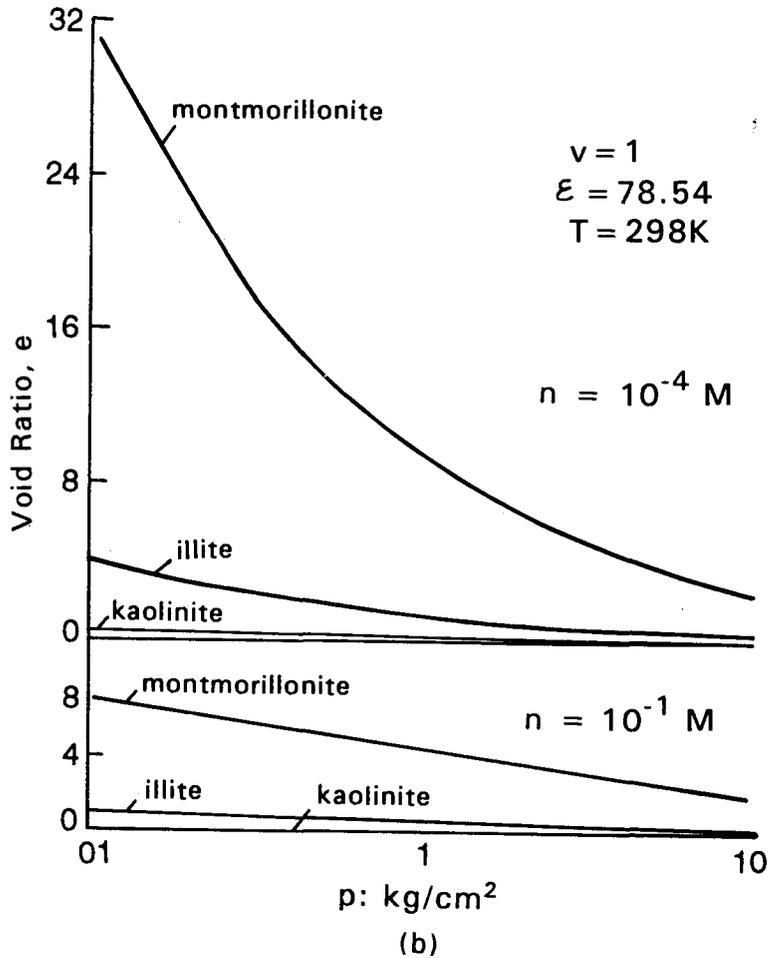
FIGURE 9b e   Plotted against log p for Various Clay Types and Concentrations

TABLE 2

CLAY PROPERTIES

Clay type	Base exchange capacity B : μ eq/g	Surface area S : m^2/g	B/S μ eq/m^2
Kaolinite	30	15	2.00
Illite	400	100	4.00
Montmorillonite	1000	800	1.25

TABLE 3

VALUES OF $(dy/dc)_{x=0}$ FOR DIFFERENT CLAY TYPES AND VALUES OF u FOR DIFFERENT PRESURE RANGES AND DIFFERENT ELECTROLYTE CONCENTRATIONS

n	Values of $(dy/dc)_{x=0}$ [eqn. (8)]			Values of u for different pressure ranges [eqn. (6)] : kg/cm^2		
	Kaolinite	Illite	Montmo-rillonite	0.1	1.0	10.0
10^{-1}	10.4	20.7	6.5	0.20	0.62	1.77
10^{-2}	32.8	65.5	20.5	0.62	1.77	3.74
10^{-3}	103.6	207.2	64.8	1.77	3.74	6.00
10^{-4}	327.6	655.2	204.8	3.74	6.00	8.30
10^{-5}	1036.0	2072.1	647.5	6.00	8.30	10.60

SOIL STRUCTURE

The term soil structure refers to inter-particle forces operative in a soil system as well as the geometric arrangement of clay particles (Mitchell, 1976). In granular or cohesionless soils, the inter-particle forces are very small so both the fabric and structure of gravels, sands and to some extent of silts are the same. On the contrary, as has been brought out earlier, inter-particle forces are relatively large in fine grained cohesive soils, and thus both these forces and the fabric of such soils must be considered as the structure of the soil. A knowledge of soil structure is essential because :

(i) Inter-particle forces arising from physico-chemical mechanisms have been observed to have a profound influence on a wide range of soil engineering properties which include; consistency, consolidation, permeability and shear strength characteristics of clay soils (Lambe, 1958; Leonards and Altschaeffl, 1964; Quirk 1960; Yong and Warkentin, 1966; and Mitchell, 1976). Hence a knowledge of these physico-chemical forces is essential to understand the soil behaviour and predict the soil's response to changes in chemical and physical environments.

(ii) The arrangement of soil particles influences the response behaviour of a soil to an external set of constraints. The inter-particle forces as discussed earlier determine the arrangement of clay particles (soil fabric).

SOIL FABRIC

The term fabric refers to the arrangement of particles, particle groups and pore spaces in a soil. The particle associations in clay suspensions can be described as follows (van Olphen, 1963; Yong and Warkentin, 1966 and Mitchell, 1976) :

(i) *Dispersed* : No face to face association of clay particles. Possibility of this arrangement arises, in the presence of high inter-particle repulsion. In dispersed fabric, particles have a parallel orientation with respect to each other. Such a fabric is obtained in a low salt environment.

(ii) *Aggregated* : Face to face association of clay particles. This fabric is realised in the presence of salt concentration in the surrounding aqueous environment or when the clay is saturated with divalent or trivalent exchangeable cations; the net result being reduction of inter-particle repulsion and aggregation of particles.

(iii) *Flocculated* : This involves edge to edge or edge to face association of clay particles. Attraction exists between the oppositely charged ends of the particles. Such a particle arrangement arises in an acidic environment.

(iv) *Oriented* : Oriented fabric arises primarily from consolidation effects. The degree of consolidation will depend on the surcharge loading or consolidation process, i.e., time and resultant volume change. In oriented fabric, the clay particles will be arranged parallel to each other.

Besides the four basic modes of particle interactions, the other various modes of particle associations in clay suspensions possible are dispersed and deflocculated, aggregated but deflocculated, edge to face flocculated but dispersed, edge to edge flocculated but dispersed, edge to face flocculated and aggregated, edge to face and edge to edge flocculated and aggregated.

In the latter sections the engineering behaviour of fine grained soils have been discussed in relation to the soil fabric.

PORE SPACES AND FABRIC

Two kinds of pores or pore spaces can readily be identified in clay soils. The pore spaces between fabric units are larger than the pore spaces between particles of the fabric units.

The pores between fabric units are termed as macropores and the pore spaces between particles within the fabric units as micropores (Yong and Warkentin, 1975). In identifying and characterizing the clay fabrics, it is necessary to take into account particle and fabric unit spacing in defining the solid network of fabric units. A knowledge of distribution of pore spaces provides an appreciation of 'packing' of fabric units and the gradation of units.

Soil behaviour in relation to water flow, pore-water extrusion, soil deformation and consolidation requires a knowledge of characteristics of water movement in the macro–and micro–pores and also of the rearrangement

of fabric units and deformation of fabric units. For example, Sridharan (1968) and Sridharan, Altschaefl and Diamond (1971) have shown that for partly saturated soils, the finer pores have significant bearing on the soil shear strength characteristics. The narrow pores mobilize significant matric suction leading to large shear strength values. They have also shown that soils compacted dry of optimum have wider pores which has significant bearing on the permeability of compacted clays. Measurement of pore-size distribution are obtained through techniques involving forced intrusion of a non-wetting fluid such as mercury, or through capillary condensation methods which involve interpretation of adsorption and desorption isotherms (Sridharan, Altschaeffl and Diamond, 1971; Garcia-Bengochea, Lovell and Altschaeffl, 1979).

EFFECTIVE STRESS CONCEPT

It has been now widely accepted that Terzaghi's concept of effective stress (Terzaghi, 1923) provies a satisfactory basis for understanding the strength and deformation characteristics of saturted soils. The effective stress concept can be stated as

$$\sigma' = \sigma - u \qquad \qquad ...(14)$$

where σ' = effective stress, σ = applied external pressure and u = pore water pressure. The concept as given in eqn. (14) tacitly assumes that no other forces except the external applied pressure σ and the hydrostatic pressure u exist within the particulate system. It may be noted here that σ' is the contact stress at mineral to mineral contact zone which is also called conventionally as the inter-granular stress and it is assumed that the area through which this stress acts is negligible. One can discuss at great length what is the nature of this contact. For purpose of brevity it is assumed that the role of contact is to transfer the stresses.

It has been brought out earlier that both attractive and repulsive forces of an electrical nature exist between clay particles. It has been shown that the repulsive forces between clay particles are primarily attributed to an interaction between electrical double layers. The principle contributions to the attractive force come from van der Waal's forces. It has been brought out that many factors are responsible for the net attractive and repulsive forces between clay particle and that the force system responsible for them is complex. The magnitude of these forces mainly depend on the inter-particle separation, clay mineral type, exchangeable cation type and pore fluid characteristics.

Lambe (1960a) introduced an equation relating the total external load to the internal stresses in the particulate soil system. His generalized eqn. could be stated as :

$$\sigma = \bar{\bar{\sigma}} \, a_m + \bar{\bar{u}} + R - A \qquad \qquad ....(15)$$

where σ = total stress

$\quad \bar{\bar{\sigma}}$ = mineral to mineral contact stress

$\quad a_m$ = fraction of the total interparticle contact area that is mineral to mineral contact

$\quad A$ = total interparticle attraction divided by total interparticle area

$\quad \bar{\bar{u}}$ = equivalent pore pressure

$\quad R$ = total interparticle repulsion divided by total interparticle area. Accordingly, the conventional effective stress becomes (Lambe, 1960a).

$$\sigma' = \sigma - \bar{\bar{u}} = \bar{\bar{\sigma}} \, a_m + R - A$$

It is equal to the intergranular or contact stress plus the net repulsive pressure. From this eqn., it is seen that the effective stress increase with repulsive forces and decreases with attractive forces. This does not agree with the physical behaviour. Sridharan (1968) rewrote Lambe's eqn. in the following manner.

$$\bar{c} = \bar{\bar{\sigma}} \, a_m = \sigma' - \bar{u}_a - \bar{u}_w - R - A \qquad \qquad ...(17)$$

where $\bar{c}$ = effective contact stress

σ = externally applied pressure on unit area.

$\bar{u}_w$ = effective pore water pressure ($=u_w \, a_w$, a_w, being fraction of the total area through which pore water pressure u_w acts)

459

$\bar{u}_a$ = effective pore air pressure (=$u_a a_a$, a_a being fraction of the total area through which pore air pressure u_w acts)

$a_m + a_w + a_a = 1$

Equation (17) may be rewritten as

$$c = \sigma' + \sigma'' \qquad \qquad ....(18)$$

where σ' is the conventional effective stress $\sigma - u_a - u_w$ (= $\sigma - u$ for fully saturated soils) and σ'' is the intrinsic effective stress = $A - R$ = net electrical attractive pressure. The effective contact stress $\bar{c}$ between particles is defined as the modified effective stress and it was hypothesized that $\bar{c}$ is the stress controlling the shearing resistance. The role of R and positive pore water pressure U_w tend to make the particles to move away from each other whereas the effect of A and σ tend to bring the particles closer to each other. If the pore water pressure is negative as in the case of partly saturated soil, its effect would be to bring the particles closer to each other.

Equation (17) is a general one and considers all the possible forces that exist in a clay-electrolyte system. Equation (17) has been studied extensivly for its validity on volume change behaviour (Sridharan, and Rao, 1973, Sridharan, Rao and Pandian 1973); on strength behviour (Sridharan, Narasimha Rao and Venkatappa Rao, 1971; Sridharan and Rao, 1979; Sridharan, Sreepada Rao and Makan, 1983); on shrinkage phenomena (Sridharan and Rao, 1971) and on secondary compression (Sridharan and Sreepada Rao, 1982). Equation (17) is further examined with examples in the latter part of this paper.

Although eqn. (17) is considered also to be valid in partly saturated soils, wherein the pore water pressure is negative, Allam and Sridharan (1978b), examined this aspect and suggested that the stresses imparted by the contractile skin should also be considered. The effective stress eqn. was rewritten as :

$$\bar{c} = \sigma - u_a + (1 - a_a)(u_a - u_w) - R + A + \Gamma T \qquad \qquad ....(19)$$

where ΓT referes to the contractile skin stress per unit area of a reference plane, and is a product of the interface perimeter and sufrace tension T. At the particulate level it can be visualised that the contractile skin stress, matrix suction and the attractive pressure bring the soil grains closer to each other while the repulsive pressure tends to separate the particles away from each other. It has been brought out that the magnitude of contractile skin stress is in the range of negative pore water pressure and hence should not be neglected. A further discussion on the effective stress in clays and its limitation can be found in Allam and Sridharan (1987a).

INDEX PROPERTIES

LIQUID LIMIT

Atterberg limits are perhaps the oldest and most commonly used tests in the field of soil engineering. Their determination is by strictly empirical testing procedure and this would seem to be contrary to the very foundations of scientific procedures. however, the tests are relatively simple and the results are generally acceptable and valuable in almost every conceivable use of soil from engineering standpoint. it is widely accepted that the liquid limit test is essentially a measure of the viscous resistance or shear strength of a soil as it approached the liquid state. The soils at their liquid limit possess definite but small shearing strength and are considered to be nearly same for all soil types. The results obtained by the use of direct shear and vane shear apparatus indicate that the strength at liquid limit is of the range of 15-30 g/cm^2. (Sridharan and Rao,1975).

Great deal of work has been reported on the effects of different test procedures and tools on the results obtained. The one point method has received wide attention as a simplified procedure. Several attempts have been made in the past to use alternate test methods like the cone penetration method, the vane shear method and the soil moisture tension method (Sridharan and Rao, 1975), but it remains a fact that the physical mechanisims governing the liquid limit of a soil were not throughly investigated. Although, the Atterberg limits were devised originally for purposes of soil classification, or in particular to know the plastic nature of cohesive soils, in the recent past various attempts have been made to correlate them with various soil properties/or parameters like surface area and cation exchange capacity, the mineralogy and geological history (through

460

'activity'), swelling behaviour, California Bearing Ratio, compressibility, shear strength, and compaction characteristics; and to use these correlations for purposes of checking new data/or to predict soil behaviour for design work (Nagaraj and Srinivasamurthy, 1983a, 1983b). Such correlations should be attempted only after throughly understanding the mechanisms involved in these test procedures and the properties of the soil that control these behaviour. Sridharan and Rao (1975) have discussed the possible mechanisms governing the liquid limit of kaolinite and montmorillonite types of clays. Commercially available kaolinite (LL = 49%, PL = 29%) and montmorillonite (LL = 306%, PL = 44%) clays have been used in this study. These clays have been chosen as they represent the extreme types of clay minerals and any natural soil is likely to behave in between these two extremes. Table -4a and Fig.10 present the liquid limits of kaolinite and montmorillonite determined with six organic pore fluids and water, mainly with a view to study variation in the force fields governing the particulate system. The properties of these fluids are listed in Table 4a.

The liquid limits of kaolinite and montmorillonite clays with the various organic pore fluids were determined using the cone penetration method. The cone penetrometer method was chosen because of its simplicity, quickness and reproducibility of test results, especially with the organic pore fluids.

In Table 4a, the liquid limit values for kaolinite and montmorillonite clays on volume basis (the ratio of the volume of fluid to the volume of solids expressed as a percentage) for the various organic pore fluids are tabulated. Since the unit weight of the fluids used differ from one another, the liquid limit was calculated on volume basis, as it facilitates comparison. The variation of liquid limit (volume basis) with dielectric constant is shown in Fig.10. From Table 4a, and from Fig.10, it is distinctly seen that the two clays viz., kaolinite and montmorillonite behave in strikingly opposite manner with respect to the change in pore fluids. While a decrease in liquid limit is observed for kaolinite with an increase in dielectric constant of the pore medium, an increase in liquid limit values have been recorded for montmorillonite. For montmorillonite, the liquid limit decreased significantly from a value of 866% for water (dielectric constant =80.4) to 149.2% for hexane (dielectric constant = 1.89) whereas the liquid limit increased from 127.0% (for water) to 230.0% (for hexane) for kaolinite. Interestingly, the liquid limit values of kaolinite with hexane, carbon tetrachloride and ethyl acetate are more than the corresponding values of montmorillonite. At the outset, these observations may look paradoxical but the following study into the mechanisms controlling the liquid limit behaviour will show that these results are order.

MECHANISMS CONTROLLING THE LIQUID LIMIT BEHAVIOUR

It has been brought out that both attractive and repulsive froces of an electrical nature exist between clay particles. Further the electrical attractive forces vary inversely with the dielectric constant of the pore medium and the repulsive forces increase directly with dielectric constant other factors remaining same (Sridharan, 1968; Sridharan and Rao, 1971, 1973). It has been realised that an increase in attractive forces increase the shearing resistance at the interparticle level and an increase in repulsive forces decreases the shearing resistance. Thus the effect of a decrease in dielectric constant is to bring in an increase in the attractive forces and a decrease in the repulsive forces and the cumulative effect is additive, increasing the shearing resistance (Sridharan and Rao, 1973). Sridharan and Rao (1973) clearly demonstrated through experiments that the strength decreases rapidly with increase in dielectric constant of the pore medium.

In addition to its influence on the shearing resistance at the inter-particle level, an increase in A force in lowering the dielectric constant of the pore medium also promotes the extent of particle flocculation and the amount of voids space available for fluid entrapment at liquid limit. If the attractive force were the only factor governing the liquid limits of clays, then it is logical to conclude that the variation in the liquid limit for both clays should follow the same trend with increasing dielectric constant. But the results presented (Fig. 10 and Table 4a) evidently show that only kaolinite behaves in this manner.

A change in dielectric constant brings out other changes in the particulate system too. When the clay particles come in contact with an electrolyte, a diffuse double layer is formed around each flake. Other factors remaining same, the thickness of the diffuse double layer directly varies with the dielectric constant of the pore medium (Fig.5). After some distance, the fluid is 'free' and is no longer under the influence of the surface charge potential. Fig.11 shows the effect of variation of dielectric constant of the pore medium on the swelling of montmorillonite and kaolinite clays under a normal pressure of 0.0625 kg/cm². The swelling tests were conducted in a conventional oedometer with compacted dry samples of 76mm diameter and 12.5 mm height. The results in Fig.11 show clearly the significant swell of montmorillonite with increase in dielectric constant, whereas kaolinite is virtually unaffected. The large increase in percentage swell with dielectric constant for montmorillonite is primarily attributed to the enormous increase in double layer thickness.
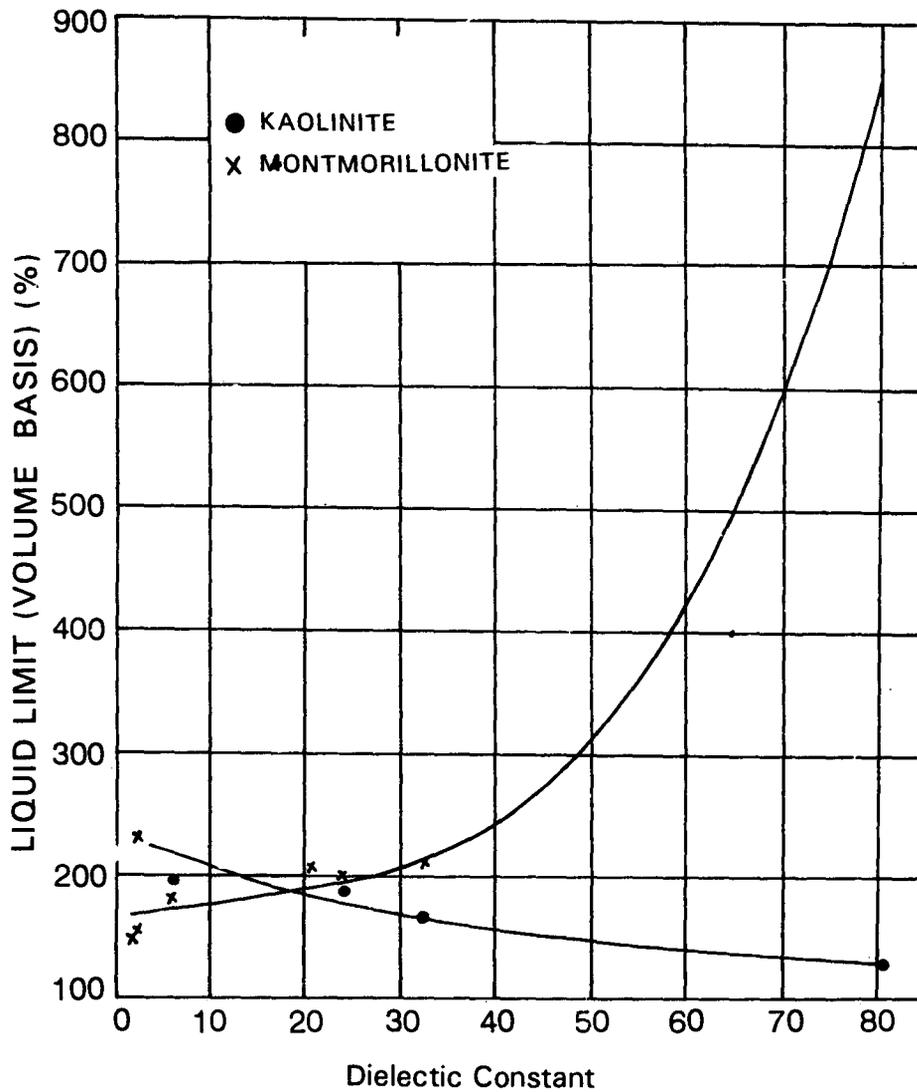
461

FIGURE 10 Effect of Dielectric Constant on Liquid Limit

TABLE 4a

LIQUID LIMIT TEST RESULTS ON KAOLINITE AND MONTMORILLANITE

S. No.	Pore fluid	Density g/cc.	Dipole moment Debyes	Dielectric Constant	Kaolinite		Montmorillonite	
					Liquid Limit (wt.basis) %	Liquid Limit (vol.basis) %	Liquid Limit (wt.basis) %	Liquid Limit (vol.basis) %
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Hexane	0.655	0.08	1.89	58.2	230.0	34.6	149.2
2	Heptane	0.680	0.00	1.92	58.5	231.0	35.0	150.0
3	Carbon tetra chloride	1.585	0.00	2.284	140.0	228.5	86.7	155.2
4	Benzene	0.867	0.00	2.284	140.0	229.0	87.0	156.0
5	Ethyl acetate	0.895	1.81	6.02	67.5	195.2	57.1	180.8
6	Acetone	0.785	2.72	20.70	61.4	202.2	56.9	205.0
7	Ethanol	0.785	1.68	24.30	57.4	189.2	55.5	200.0
8	Methanol	0.787	1.664	32.63	50.8	167.0	59.5	214.0
9	Water	0.999	1.84	80.4	49.0	127.0	306.0	866.0
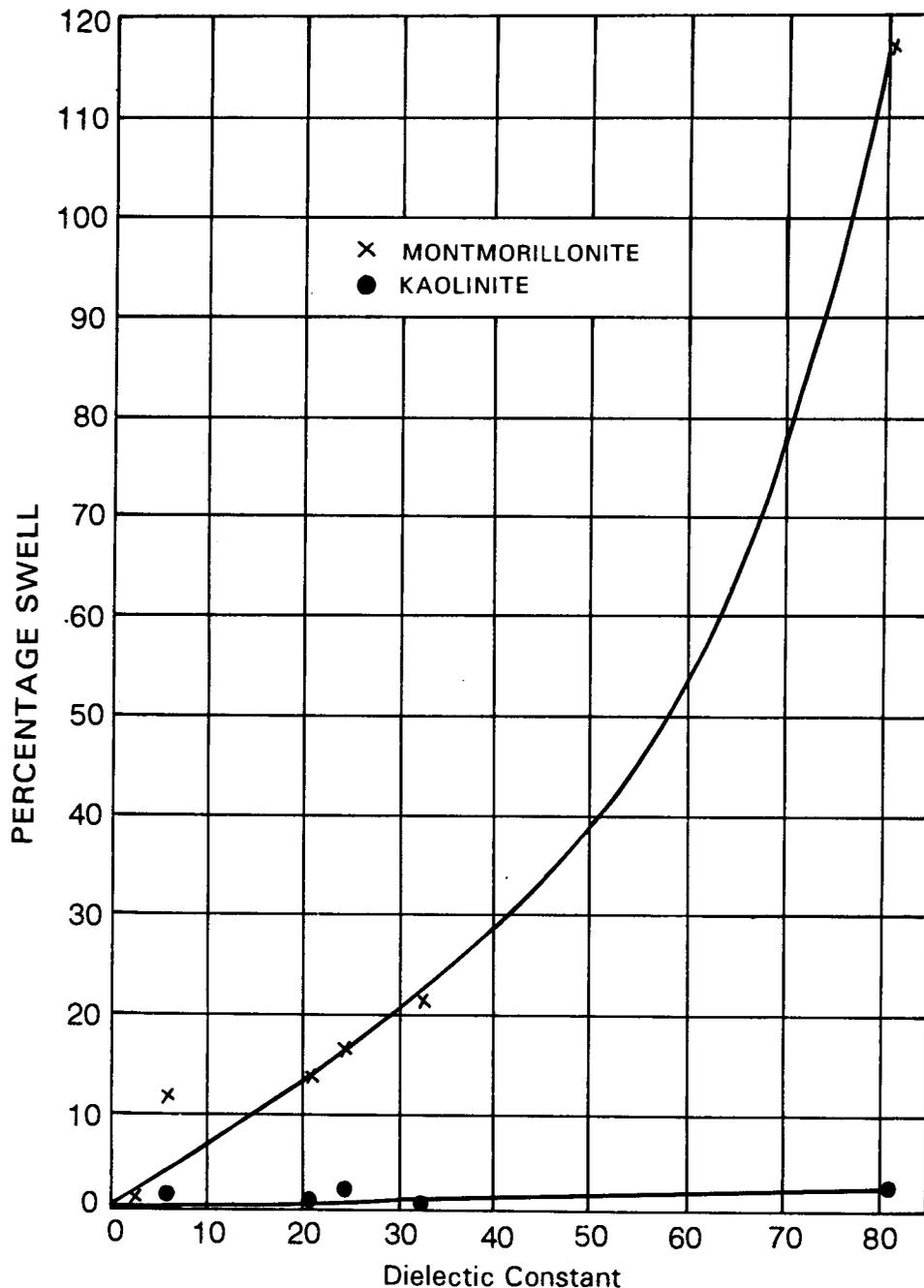
FIGURE 11 Effect of Dielectric Constant on Percentage Swell

The direct dependence of the liquid limit of montmorillonite on the dielectric constant of the pore medium (Fig.10) then suggests that this consistency limit of montmorillonite is primarily contributed by the diffuse double layer fluid content. Hence an increase in dielectric constant of the pore medium has two effects :

(i)   it enhances the double layer thickness leading to increase in liquid limit,

(ii)   it lowers the inter-particle attraction leading to decrease in the extent of particle flocculation and consequently the amount of void space available for water entrapment at liquid limit.

These effects are obviously opposite in nature and the liquid limit behaviour of a particular clay depends upon the predominating influence between the two. With the help of the foregoing discussion, the liquid limit behaviour of the two extreme types of clays can be satisfactorily explained as follows.

For kaolinite, a change in dielectric constant does not bring about appreciable change in double layer thickness; the decrease of dielectric constant however enhances the inter-particle attraction, the inter-particle shearing resistance and the extent of particle flocculation. The increase in void space for fluid entrapment

facilitated by the enhanced particle flocculation on lowering the dielectric constant of the pore fluids is responsible for the higher liquid limits of the non-swelling clay specimens.

For montmorillonite, the effect of dielectric constant on the thickness of diffuse double layer is more significant when compared with its effect on the clay fabric. Thus, an increase in dielectric constant of the pore medium causes an increase in the diffuse double layer thickness, the same is reflected as higher liquid limit of the swelling clay.

Comparison of the liquid limits of Kaolinite and montmorillonite clays at very low dielectric constants (for example hexane and carbon tetrachloride)brings out further evidence to the proposed mechanisms. Because of low dielectric constants, these fluids develop very thin or practically no double layer at all on the clay particles. Hence, the liquid limit values should primarily be governed by the interparticle shearing resistance. Because of its relatively higher shearing resistance and enhanced particle flocculation kaolinite has a higher liquid limit than montmorillonite for these fluids i.e., it has liquid limits of 230% and 228.5% when compared to 149.2% and 155.2% for hexane and carbon tetrachloride respectively.

HOMOIONISED CLAYS

Table 4b presents the effect of cation type, valence and size of the ion on the liquid and plastic limits of a bentonite homoionised with the respective cations (Sridharan, Rao and Murthy, 1986b). The results presented in the table illustrate the pronounced effect of cationic valence and size in varying the plasticity characterisitics of bentonite from about 60% to 630%.

TABLE 4b

INFLUENCE OF VALENCY AND SIZE OF THE ADSORBED CATIONS ON THE INDEX PROPERTIES OF BENTONITE

Bentonite type	Specific gravity	Liquid limit :%	Plastic limit :%	Hydrated ionic radius A° *
Lithium	2.61	675	49.1	7.30-10.30
Sodium	2.81	495	49.2	5.60-7.90
Ammonium	2.59	223	55.8	5.37
Potassium	2.72	233	57.8	3.80-5.32
Magnesium	2.65	129	49.9	10.80
Calcium	2.65	125	40.6	9.50
Barium	2.73	108	45.8	8.80
Aluminium	2.43	108	60.5	--
Iron **	2.70	120	63.5	–

*Mitchell (1976)

**Owing to the hydrolysis of the Al^{3+} and Fe^{3+} ions in the presence of water, the hydrated radii of these ions cannot be evaluated.

The specific gravity values for the various homoionized bentonite specimens show slight variations and do not follow any consistent pattern with respect to cation valency or size. It has been shown earlier that the liquid limit of bentonite is controlled by the thickness of the diffuse double layer surrounding the clay particles. Another noticeable feature is that the hydrated ionic radius induces significant changes in the liquid limit values. The size effect is more pronounced for the monovalent bentonites than for the divalent and trivalent clays. In general, for a constant valency the liquid limit increases with the hydrated radii of the adsorbed cations.

The liquid limit of monovalent bentonites decreased in the order ammonium bentonite <potassium bentonite <sodium bentonite <lithium bentonite, and there is no unique value as anticipated from Gouy-Chapman theory. Stern (1924) proposed two corrections to the Gouy-Chapman theory. He pointed out that the centre of a positive charge in a cation can come on close to the surface of a particle than the radius of the cation and that some of the cations may be specifically adsorbed into the Stern layer (i.e. reside in a compact layer adjacent to the clay surface) and do not participate in the diffuse portion of the double layer. The number of such specifically adsorbed ions increase with a decrease in hydrated ion size.

Shainberg & Kemper (1966) calculated the hypothetical amounts of specifically adsorbed ions in the Stern layer (designated as Q_u) for lithium, sodium and potassium bentonites to be 16%, 36% and 49% respectively. Such an increase causes a corresponding decrease in the thickness of their diffuse double layers and hence in the liquid limit values.

Lithium bentonite has a liquid limit of 675% with 16% of its exchangeable cations residing in the Stern layer. An increase in the amount of specifically adsorbed ions to 36% in sodium bentonite ($Q_{uk+}-Q_uN_a+=13\%$) results in a 55% decrease in its liquid limit with respect to that of sodium bentonite. The large decrease in the liquid limit of potassium bentonite cannot be solely explained by specific adsorption of cations in the Stern layer.

The unhydrated size of potassium ions (diameter $2.66A°$) is such that they can partly fix in the hexagonal holes in the surface configuration of the silicate layer (Kardos, 1964). As the partially fixed ions are close to the seat of negative charge, they suppress the diffuse double layer to a greater extent than if they were present in the Stern layer. The partial fixation of potassium ions in the hexagonal oxygen holes together with their higher amount of specific adsorption in the Stern layer is presumably responsible for the large decrease in the liquid limit of the clay.

The unhydrated size of ammonium ions ($2.86A°$) is also such that they can partially fix in the hexagonal lattice holes. The percentage of specifically adsorbed ions is approximately 53%, computed from the data of Shainberg & Kemper (1966). The liquid limit of ammonium bentonite is 223%, which is slightly lower than that of potassium bentonite.

The liquid limits of divalent magnesium, calcium and barium bentonites are 129%, 125% and 108% respectively, while their hydrated radii are $10.84A°$, $916A°$ and $8.8A°$. The results indicate that the effect of ionic size on the liquid limits of divalent bentonites is small in comparison with that noted for the monovalent clays.

The liquid limits of trivalent iron and aluminium bentonites are 120% and 108% respectively, which are slightly lower than those of the calcium and magnesium clays. Further, the liquid limit of aluminium bentonite is comparable with that of barium bentonite, while iron bentonite has a higher liquid limit than barium bentonite. The ionic size of the barium ion has a dominant effect over the valency of iron and aluminium ions in reducing the liquid limit of bentonite.

Thus it can be seen that besides valency, the hydration radius of the adsorbed cations have a marked influence on the liquid limit of bentonite. While the thickness of the diffuse double layer is the primary reason, the Gouy-Chapman theory is inadequate to explain the observed variations.

Having discussed in detail the liquid limit behaviour of purified systems, namely kaolinite and montmorillonite minerals, it is proposed to examine natural soils which are essentially kaolinitic and montmorillonitic soils.
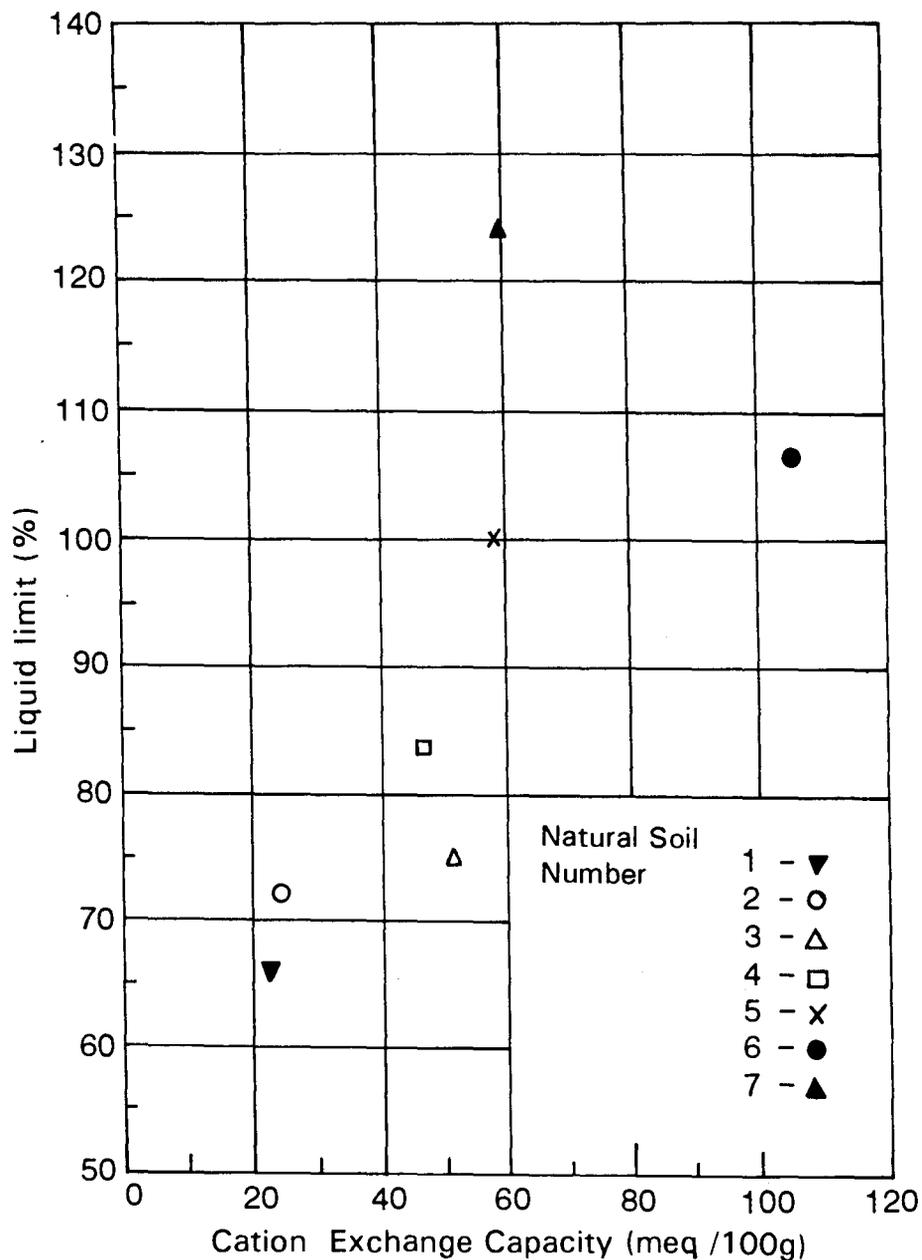
MONTMORILLONITIC SOILS

Table 5 and Table 6 (Sridharan, Rao and Murthy, 1986c) present the cation exchange capacities, mineralogical composition and physical characteristics of montmorillonitic soils obtained from different parts of India. A closer examination of the liquid limit behaviour with other physical and chemical characteristics indicate that no correlation exists between

(i) clay fraction and liquid limit, and

(ii) surface area and liquid limit.

The thickness of the diffuse double layer surrounding the clay particles in an aqueous environment chiefly depends on compositional factors, which include clay minenral type, cation exchange capacity, and exchangeable cation type, environmental factors, that include pore-water characteristics. Excluding the enviornmental factors, the thickness of the double layer and the liquid limit of the soils should depend only on the compositional factors. As the clay mineral type is common for all the soils, the factors affecting the double layer thickness and hence the liquid limit should be cation exchange capacity and the exchangeable cation type. With the exception of highly acidic and sodic soils, calcium and magnesium are the cations that mainly balance the clay's negative charge, while sodium and potassium ions are less common. This was also true for the soils studied (Sirdharan, Rao and Murthy, 1986c).

The plot in Fig. 12 showed that a poor relationship existed between liquid limit and cation exchange capacity of the montmorillonitic soils. Fig.13, however, shows that the liquid limit of these soils has a near perfect relationship with the amount of exchangeable sodium ions present. The plausible reasons for the dependence of the double layer thickness and in consequence the liquid limit, on the exchangeable cation type may be

Liquid limit (%)

140

130

120

110

100

90

80

70

60

50

Natural Soil
Number

1 - ▼
2 - O
3 - △
4 - □
5 - ✗
6 - ●
7 - ▲

0   20   40   60   80   100   120

Cation Exchange Capacity (meq /100g)

FIGURE 12 Plots of Liquid Limit Versus Cation Exchange capacity of Montmorillonite Soils, (Identity of the Soils can be found in Table 6.)

explained as follows.

The exchangeable cations, usually present, as previously stated are calcium, magnesium, sodium, and potassium. The divalent calcium and magnesium ions by virture of their higher valency are strongly adsorbed by The clay surface (Bolt, 1976) and do not undergo appreciable dissociation in the presence of water to contribute significantly to the number of ions in the double layer. Besides, the Langmuir theory also indicates that contribution to the double layer thickness is much less for divalent ions (Fig.4). In the case of potassium ions, their size (unhydrated diameter 0.266 nm) is such that they fit partly into the hexagonal holes (diameter 0.28 nm) in the surface configuration of the silicate layers and, as they are close to the seat of negative charge, they are held tightly by electrostatic bonding. The resultant high adsorption affinity and also the minimal concentration of potassium ions prevent them from contributing significantly to the thickness of the double layer. Unlike the potassium ions, the sodium ions cannot be fixed, partly because of their smaller size (unhydrated diameter, 0.19 nm) and partly because of their greater hydration energy, which prevents their close approach to the surface. As a result, the sodium ions are weakly held by the surface and readily dissociate to contribute significantly to the thickness of
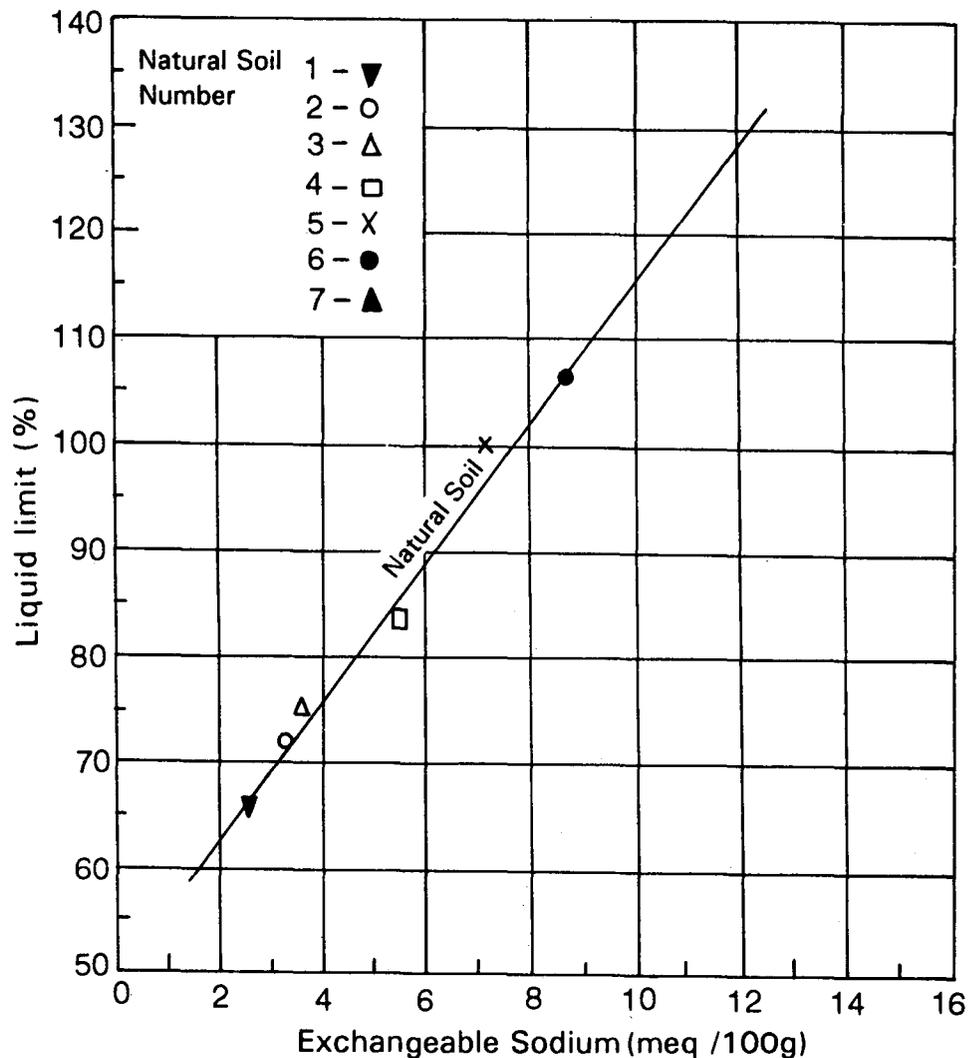
FIGURE 13 Plots of Liquid Limit versus Amount of Exchangeable Sodium Ions in Mont morillonite soils. (Identity of the Soils can be found in Table 6)

TABLE 5

CATION EXCHANGE CAPACITIES AND MINERALOGICAL COMPOSITION OF SOILS STUDIED

Speci-men	Location	Exchangeable cations, meq/100g					X-ray Diffraction Mineralogy
		Na	Ca	Mg	K	Total	
1	Koraput District Orrissa	2.59	11.04	4.05	1.05	23.01*	montmorillonite, kaolinite, muscovite, quartz.
2	Madras District Tamil Nadu	3.27	9.53	9.32	2.32	24.44	montmor lonite, kaolinite, quartz, feldspar, diaspore ilmenite.
3	Chitradurga District Karnataka	3.72	32.94	14.18	0.84	51.68	montmorillonite, antigorite, quartz diaspore.
4	Dharwar District Karnataka	5.52	30.56	10.13	1.33	47.54	montmorillonite, quartz
5	Bijapur District Karnataka	7.15	41.15	8.72	1.45	58.47	montmorillonite, feldspar, quartz, calcite
6	Dharwar District Karnataka	8.68	81.10	13.17	1.59	104.54	montmorillonite, calcite, quartz, hematite.
7	Chittoor District Andhra Pradesh	13.32	34.29	11.15	1.18	59.94	montmorillonite, feldspar, quartz, diaspore, rutile

*Includes 1.20 meq./100 g of exchangeable Al^{+3} and 3.08 meq/100 g of H^{+} ions.

TABLE 6

SOME PHYSICAL CHARACTERISTICS OF THE SOILS STUDIED

Specimen	Surface Area m²/g	Liquid Limit %	Plasticity Index %	Particle Size Distribution%			Unified Soil Classifi- cation
				Sand	Silt	Clay	
1	150.61	66.0	26.0	12.0	58.0	30.0	MH
2	155.16	72.0	40.6	13.0	39.0	48.0	CH
3	290.61	75.0	32.5	18.0	32.0	50.0	MH
4	399.22	84.0	42.0	8.0	55.0	37.0	MH
5	306.17	100.0	54.8	4.0	65.0	31.0	MH
6	369.12	106.4	62.3	7.0	51.0	42.0	MH
7	124.73	124.2	101.0	33.0	28.0	39.0	CH

responsible for the dependence of the double layer thickness and in turn the liquid limit on the amount of exchangeable sodium ions present. The relative ease of dissociation of cations is governed by valency as in the calcium and magnesium and by ionic size and hydration energy in the case of sodium and potassium ions (Sridharan, Rao and Murthy, 1986c).

TABLE 7

SOME PHYSICO-CHEMICAL CHARACTERISTICS OF SOILS STUDIES

Soil specimen	Soil Location	Soil pH	Exchangeable cation content					X-ray diffraction mineralogy
			Ca	Mg meq/100g	Na	K	Total CEC	
1	Singroli, Uttar Pradesh	7.05	–	–	1.8	–	1.8	Kaolinite, muscovite, quartz
2	Nat. Aeronautic Lab., Bangalore	7.10	2.31	0.7	1.7	0.2	4.9	Kaolinite, quartz
3	Dommalur Market Bangalore	7.30	9.00	5.5	1.0	–	15.5	Kaolinite, montmorillonite, muscovite, quartz
4	Indian Inst. of Science, Bangalore	7.40	9.30	1.8	0.7	0.4	12.2	Kaolinite, montmorillonite, muscovite, quartz
5*	Mangalore Chem. Fertilizers, Manglore	3.20	–	–	1.2	–	33.5	Kaolinite, feldspar, hematite, montmorillonite, muscovite.
6	Central Power Research Inst., Bangalore	7.30	10.80	4.0	2.6	1.0	18.4	Kaolinite, montmorillonite, muscovite, quartz
7	Indian Inst. of Science, Bangalore	7.70	13.90	2.7	1.0	0.6	18.2	As for Soil 4
8	Dommalur Market, Bangalore	7.60	12.90	2.0	1.3	0.6	16.8	As for Soil 3
9	Dommalur Market, Banglaore	7.80	15.80	3.1	1.6	1.5	22.0	As for Soil 3

* Includes 12.0 meq/100 g of exchangeable Al^+ and 20.3 meq./100 g of exchangeable H^+ ions
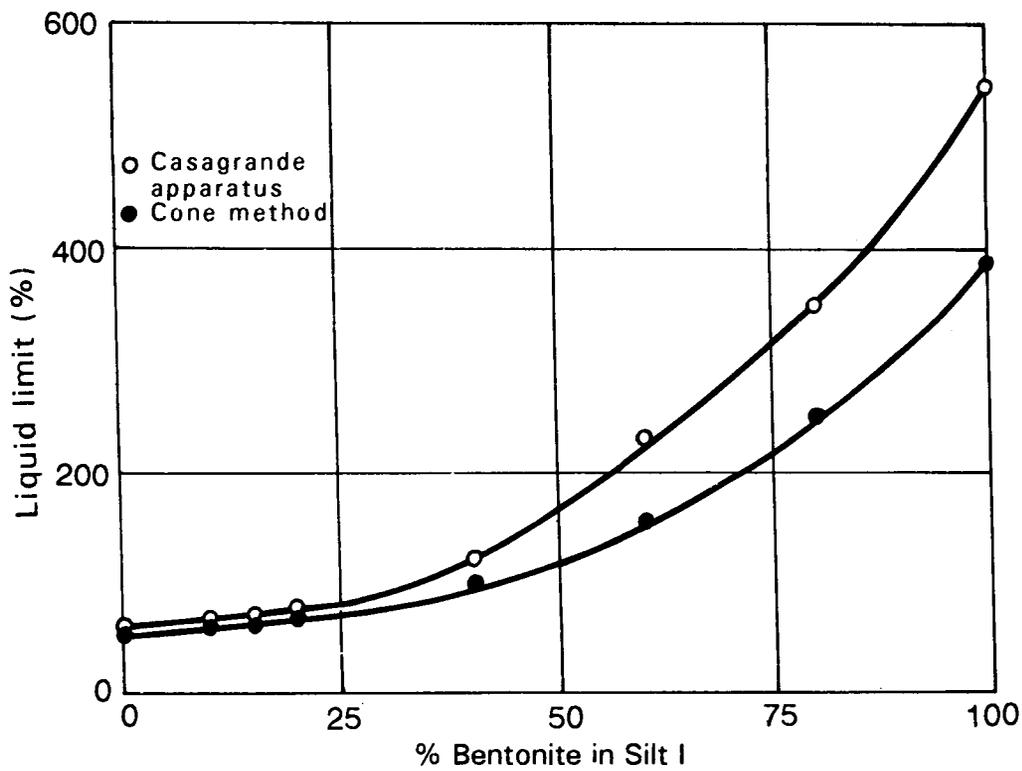
FIGURE 14 Liquid Limit versus Percent Bentonite in Silt I
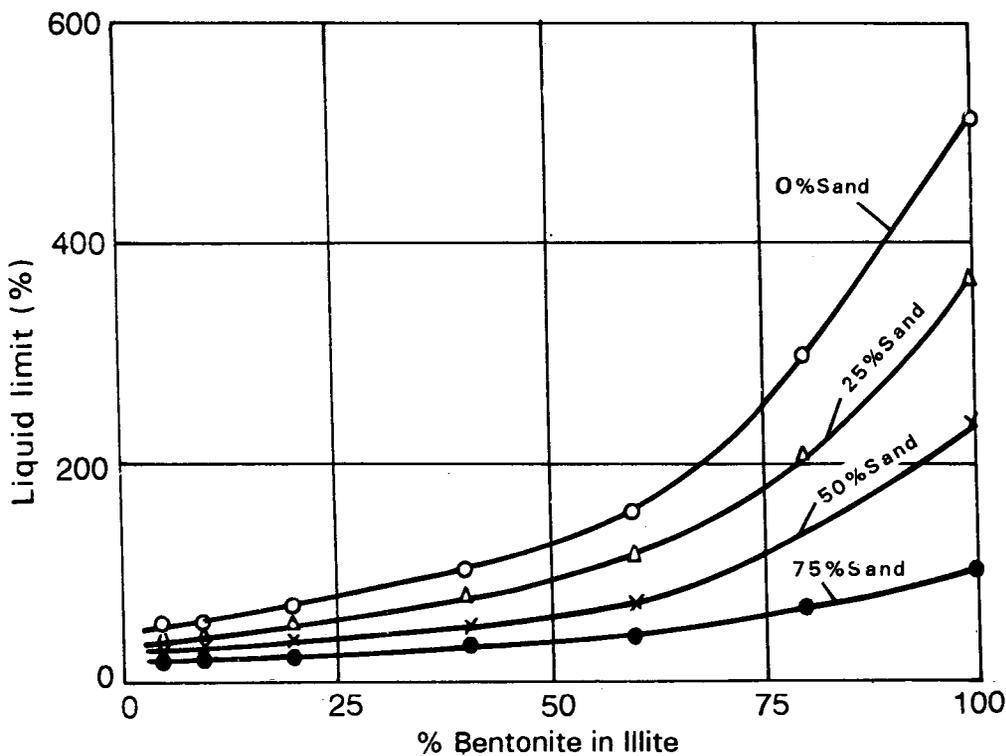

FIGURE 15 Liquid Limit Versus Percent Bentonite in Illite

KAOLINITIC SOILS

Representative soils from various locations (mainly in southern India) which are essentially kaolinitic soils (Sridharan, Rao and Murthy 1988) have been used in the following discussion. Table 7 and 8 present some physico-chemical characteristics and some physical properties. Close examination of the variation of liquid limit or activity with clay size fraction shows no systematic behaviour.
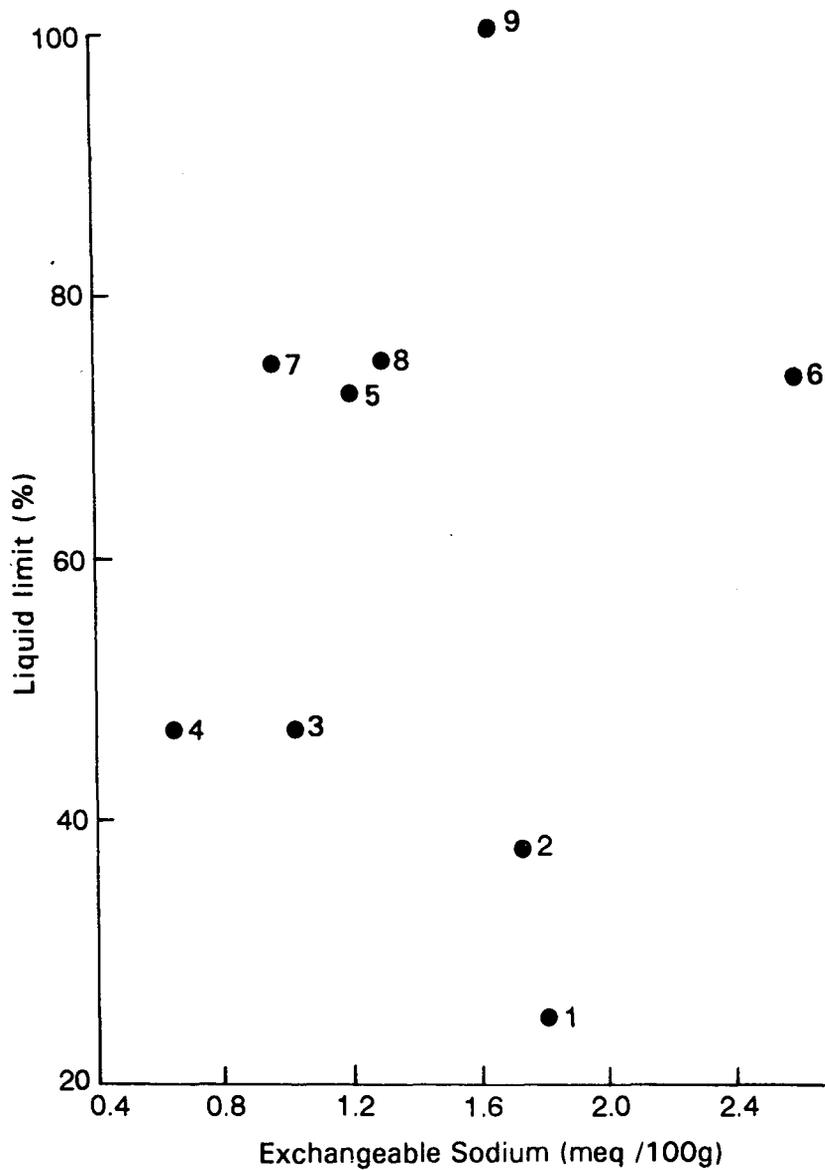
FIGURE 16 Variation of Liquid Limit as a Function of the Exchangeable
Sodium Content of the Soils

TABLE 8

SOME PHYSICAL PROPERTIES OF SOILS STUDIED

Soil Specimen	Particle size distribution			Liquid Limit %	Plastic Limit %	Shrinkage Limit %	Plasticity index%	Activity	Surface area: m²/g	Sediment volume in water cm³/g
	Sand %	Silt %	Clay %							
1	36	39	25	25	13.8	10.8	11.2	0.45	40.2	1.00
2	34	39	27	38	15.3	11.5	22.7	0.84	42.5	1.25
3	59	16	25	47	21.5	16.4	25.5	1.02	73.6	1.30
4	40	36	24	47	25.4	16.8	21.6	0.90	84.3	1.40
5	13	66	21	73	40.4	18.5	32.6	1.55	172.8	1.75
6	12	58	30	74	25.0	15.7	49.0	1.63	224.0	1.85
7	11	53	36	75	33.8	20.1	41.2	1.14	125.8	1.85
8	22	32	46	75	36.3	19.8	38.7	0.84	147.4	1.90
9	10	39	51	100	35.8	27.3	64.2	1.26	167.2	2.40

Seed et.al.(1964) presented data showing linear variations of the liquid limit with percentage clay size fraction for quarz with kaolinite mixtures. The percentage clay size and liquid limit of artificial soil mixtures presumably varies in a predictable manner (linear law) in relation to each other, as physical blending of soil constitutents would have a calculable effect on the properties concerned. Natural soils which have been subjected to physical and chemical weathering and other depositional processes, have a wide distribution of particle sizes and physico-chemical and mechanical properties. Consequently, natural soils not only represent an assemblage of sand, silt and clay fractions but also a system, the properties of which depend on the composite effects of several interacting and interrelated factors. The result is that the different constituents of a soil may not influence properties directly, in whatever quantity they are present (Sridharan, Rao and Murthy, 1988).

This is indeed illustrated from the data of Sivapulliah and Sridharan (1985) for bentonite-silt mixtures. Fig.14 presents the variation of liquid limit for bentonite-silt mixtures. The "linear law" is not found to be valid. The liquid limit of mixtures are found to be less than what one can calculate from the individual values. This shows the soil fabric becoming more of aggregated one for mixtures giving rise to reduction in the liquid limits. Fig.15 shows similar results for bentonite + illite. The effect of physico chemical interaction is very significant. The bentonite-illite mixture can result in a liquid limit value much less than the one obtained from calculating from their individual values. This leads to the conclusion that the liquid limit is not a physical property. More results of this nature could be seen in the paper by Sivapulliah and Sridharan (1985).

It was earlier brought out that the liquid limit of montmorillonitic soils which is recognized as a function of electrical diffuse double layer should be a function of exchangeable sodium content (Fig.13). Fig.16 shows the plot of exchangeable sodium vs liquid limit for the Kaolinitic soils.

Unlike what has been noticed for montmorillonitic soils (Fig.13) no correleation exists between the liquid limit and exchangeable sodium content of kaolinitic soils. Wide scatter between the two parameters suggest that the diffuse double layer does not contribute to the liquid limit of kaolinitic soils.

In view of these observations, it is likely that the geometric arrangement of clay particles (clay fabric) regulates the liquid limit of kaolinitic soils. Soils with a relatively greater degree of particle flocculation will enclose larger void spaces for water entrapment and exhibit higher liquid limit values, while soils with a lesser degree of particle flocculation and with smaller void spaces will possess lower liquid limit values.

Direct measurements of particle flocculation within a clay are difficult to make. Lambe (1958) observed that the amount of shrinkage upon drying could be used as a measure of average particle orientation and that any soil with a parallel arrangement of particles should undergo more volume reduction upon drying than the same soil with its particles in a random/flocculant array. Lambe examined the relationship between particle orientation at the start of drying and the amount of shrinkage upon drying for a compacted Boston blue clay specimen. It was shown that the more nearly parallel the particles are, the greater the shrinkage of the soil upon drying. It was, therefore, thought that a kaolinitic soil with a greater degree of particle flocculation and higher liquid limit should undergo lesser shrinkage than a soil with a lesser extent of particle flocculation and even lower liquid limit.

Figure 17 plots the liquid limit of kaolinitic soils as a function of their shrinkage limit values. As expected, an increase in liquid limit is accompanied by an increase in shrinkage limit, suggesting that the liquid limit of kaolinitic soils is strongly influenced by the clay fabric. The latter is a function of interparticle forces of attraction and repulsion determined in turn by the exchangeable cation content and soil pH.

Besides the correlation with shrinkage limit, the influence of soil fabric in regulating the liquid limit may also be seen in the latter's relationship with the sedimentation volumes of the soil specimens in water. In general, the larger the sediment volume after sedimentation, the higher the degree of flocculation (Lambe, 1958). Sediment volumes of flocculated suspensions are usually large, as the strong attractive forces between settling particles causes each random collision to be successful, leading the particles to 'stick' in the position of volume. If there is a barrier to flocculation owing to weak interparticle repulsion, the sediment volume decreases as the particles are much more mobile and pack more tightly (van Olphen, 1963). It was therefore anticipated that soils with a higher liquid limit, possessing stronger interparticle attraction and a greater degree of particle flocculation should occupy a higher sediment volume in water. Fig.18 shows the liquid limit of the kaolinitic soils as a function of their sediment volumes in water. An increase in liquid limit is accompanied by a near perfect linear increase in sediment volume substantiating the view that liquid limit of kaolinitic soils is a function of clay fabric.

The results of the various studies thus bring out the distinctive role of clay mineralogy, inter-particle forces and clay fabric in influencing the liquid limits of clays. The liquid limit of non-swelling kaolinites is mainly contributed by the water entrapped in the void spaces of its flocculated structure which in turn is dependent on the attractive forces. The greater the degree of flocculation of a soil, higher is the liquid limit exhibited by it. The
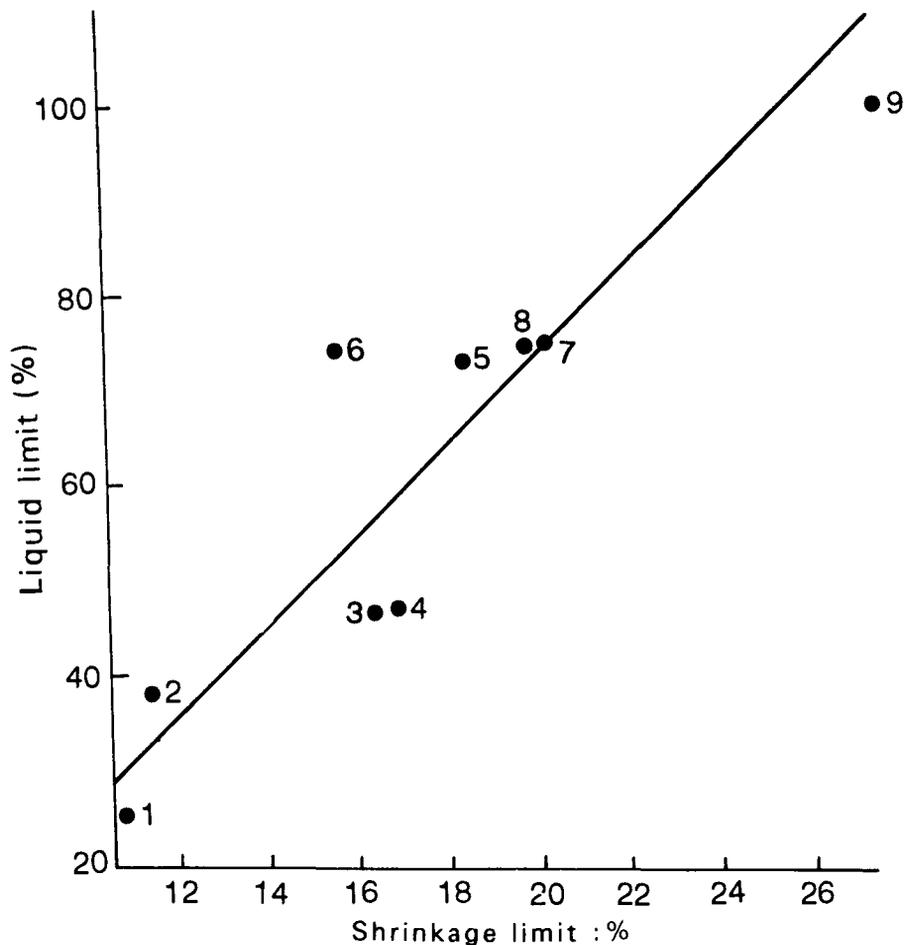
471

FIGURE 17 Plot of Liquid Limit as a Function of the Shrinkage Limit of the Soils

liquid limit of montmorillonites is mainly contributed by the diffuse double layer water. The diffuse double layer thickness of natural montmorillonitic soils in turn is chiefly contributed by its exchangeable sodium content.

INFLUENCE OF DRYING

While detailed discussion on the mechanisms governing the liquid limit of kaolinitic and montmorillonitic soils have been made, an aspect which is also important is the influence of drying on the liquid limit behaviour. The causes of reduction in Atterberg limits on drying of a clay specimen are recognised to be the presence of halloysite mineral, hydrated sesquioxides and organic matter. Carbonate is also recognised as a major cementing agent. Rao, Sridharan and Chandrakaran (1989) have brought out in detail the physical and chemical mechanisms controlling the liquid limit behaviour of soils upon drying. They have reasoned out that in marine clays the presence of calcium and magnesium as predominent exchangeable ions facilitates strong inter-particle attraction and small particle separation leading to aggregation of particles on drying and resulting in reduction in the Atterberg limits.

SHRINKAGE LIMIT

One of the important physical properties of soil in which an engineer is interested is the volume change caused by change in water content on climatic variations. The conventional shrinkage limit to a certain extent qualitatively helps to identify the soil for its potential from volume change considerations. Sridharan and Rao (1971) discuss at great length the mechanism controlling the shrinkage limit of soils and the role of effective stress concept. Based on theoretical considerations and extensive tests carried out on kaolinite with different pore fluids listed in Table 4a, it has been brought out that the shearing resistance at particle level controls the shrinkage limit of kaolinites. The shearing resistance at particle level is in turn controlled by the contact stress/effective stress as defined in eqn.17.
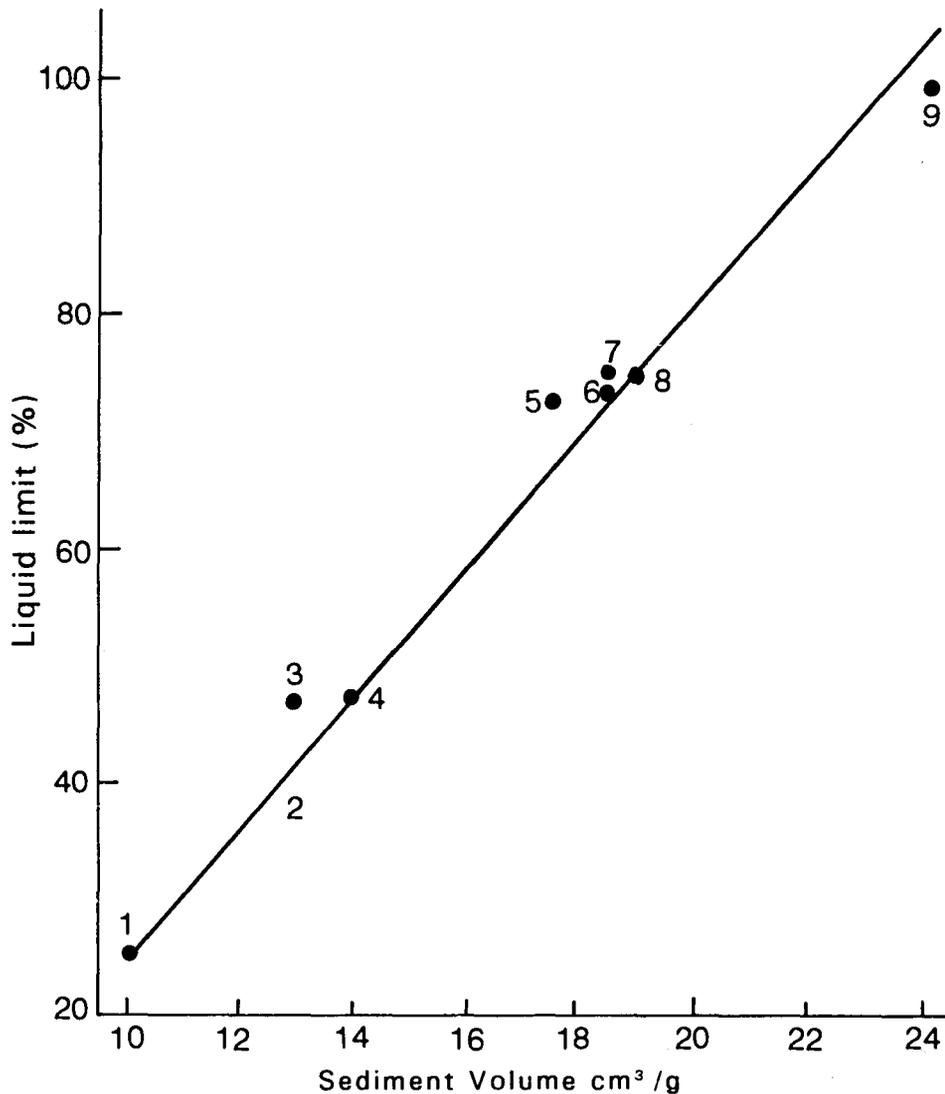
FIGURE 18 Plot of Liquid Limit as Function of the Sediment Volume of the Soil in Water

PHYSICAL MECHANISM INVOLVED

During a shrinkage limit test the external load is zero and, as the dead weight of the clay can be considered negligible in comparison to the intensity of the capillary forces at the surface and interparticle electrical forces dealt with, the hydrostatic pressure in the interior of the clay is zero. Since a hydrostatic stress difference is required for the flow of the capillary water towards the surface of evaporation, the hydrostatic pressure acting on the vicinity of the surface of evaporation must be negative. During shrinkage, as evaporation removes water from the surface, a meniscus begins to form in each of the pores at the surface with the resulting tension in water. Therefore, the hydrostatic stress difference required for the outward flow is caused by the surface tension of the capillary water.

Equation 17, which, when $\bar{u}_a = 0$ (for an initially saturated soil mass), becomes

$$\bar{c} = \sigma - \bar{u}_w - R + A. \qquad ....(20a)$$

The external load σ in this case is none other than the force due to surface tension, and the pore water pressure term becomes zero. Denoting the surface tension forces as σ_{TS}, the effective stress equation controlling the shrinkage phenomena, now reads as

$$\bar{c} = \sigma_{TS} - R + A \qquad .....(20b)$$

The resistance against volume change that a soil exhibits during shrinkage test is a manifestation of the shear resistance that can be mobilized between particles at the particle contacts for any surface tension force, the greater the shear resistances at the particle contacts and (or) between particles, the greater is the resistance

473
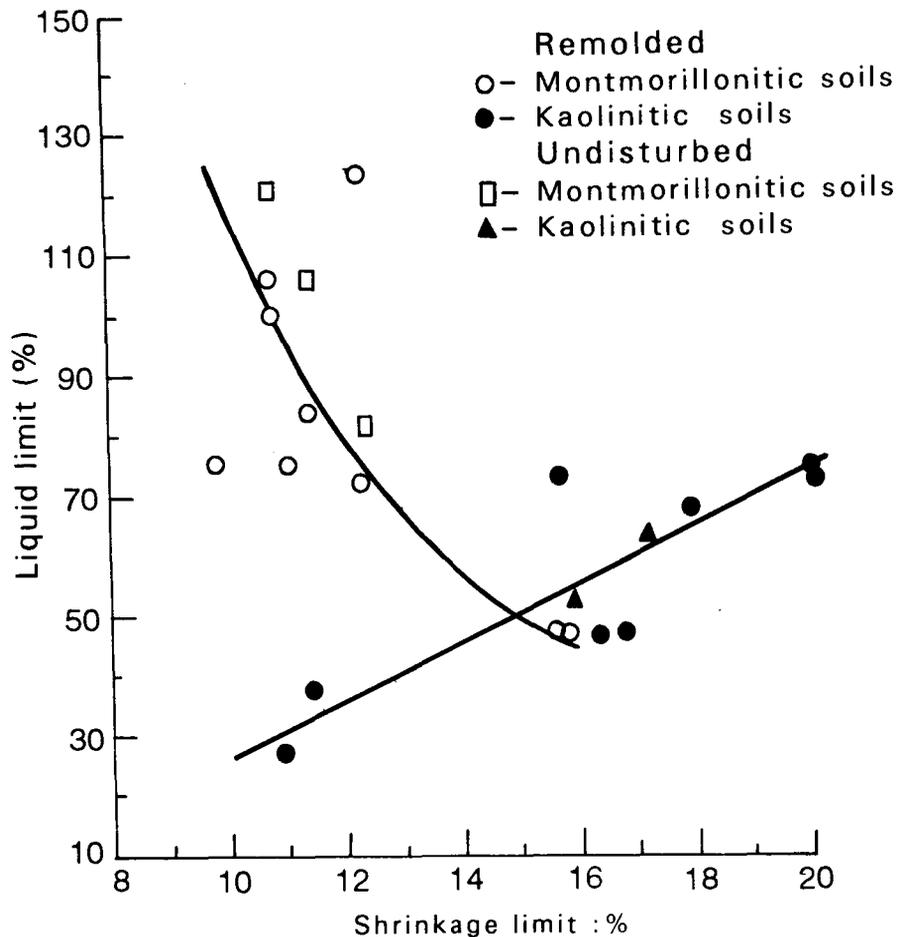
FIGURE 19 Variation of Liquid Limit with Shrinkage Limit for Kaolinitic and Montmorillonitic Soils

against volume change and the smaller is the volume change, and vice versa. These shear resistances are functions of normal forces acting between particles and (or) at the particle contacts, the frictional properties, and the electrical attractive and repulsive forces.

During shrinkage, the volume change occurs as long as the capillary forces are larger than the internal shearing resistance as governed by Eq.(17). As the volume change occurs, the internal resistance becomes larger and larger because of closer particle contact, and the capillary forces also increase because of the decrease in the radius of curvature of the menisci. The shrinkage reaches its limit when the total internal shear resistance becomes equal to that of capillary forces.

Hence, shrinkage phenomena can be broadly explained as being initiated by the capillary forces due to the surface tension of the pore fluid, but the resultant shrinkage void ratio is dependent on the resistance offered between the soil particles and (or) at the particle contacts which is governed by eqn. (17).

It has been further concluded that flocculated fabric yields a higher shrinkage void ratio and the dispersed fabric a lower shrinkage void ratio. When attractive force is more, the effective stress is more and hence the lower shrinkage or higher shrinkage void ratio. If the double layer thickness is more, repulsive pressure is more and the fabric is a relatively oriented one in resulting higher shrinkage or lower shrinkage void ratio. Sridharan and Rao (1971) have also shown that samples consolidated to higher pressures shrink more. This has been attributed to a relatively more oriented fabric.

Shrinkage limit is an extremely useful parameter in qualitatively identifying the soil fabric (Lambe,1958). In the earlier section, the importance of residual fabric of remolded kaolinitic soils in influencing the soil's liquid limit was demonstrated using the shrinkage limit values. The liquid limit of kaolinitic soils in general increases with the shrinkage limit values (Fig.19) confirming the influence of soil fabric on the liquid limit of the non-swelling soils.

In contrast, the liquid limits of montmorillonitic soils decreases with increase in shrinkage limit (Fig. 19). With montmorillonitic soils, an increase in liquid limit results from an expansion of the diffuse double layer

474

thickness (Sridharan, Rao and Murthy, 1986c). Such a situation leads to the development of diffuse double layer repulsion and a more parallel particle arrangement. The relatively oriented particle arrangement in turn allows for greater development of capillary stresses and an easier movement of particle and particle groups during drying leading to lower shrinkage limits (Shridharan and Rao, 1971, Mitchell, 1976).

FREE SWELL INDEX

The presence of swelling clay in a soil introduces complications in a wide variety of engineering problems. Analytical techniques, such as X-ray diffraction, infrared spectroscopy, and thermogravimetry are employed to detect the presence of expansive clays in soils. However, these tests are not particularly suited to soil engineering practice, as they require sophisticated and expensive equipment, which may not be available for practical purposes. Procedures have been developed in soil engineering investigations that serve as indicators for identification of swelling soils. Studies had shown that expansive clays can be recognized characteristics, activity of clay, and clay fraction. The consolidation test data can be analysed suitably to determine the swelling potential of soils. Besides these methods, the simple free-swell test is used for estimating the expansivity of clayey soils. This study compares the sediment volumes of 10 g of dry soil in polar water and nonpolar carbon tetrachloride (CCl_4)/kerosene, respectively, under no external constraint. The difference in the sediment volumes occupied by the soil in water and carbon tetrachloride, respectively, divided by the sediment volume of the soil in carbon tetrachloride and expressed in percentage, is termed as free-swell index of the soil.

Current concepts of the causes and effects of swelling in clay soils postulate the existence of forces of attraction A and repulsion R between soil particles whose origin and nature have been earlier discussed.

In the free-swell test of kaolinite, the clay material occupied a higher sediment volume in carbon tetrachloride than in water, contrary to the anticipated observation. This property of kaolinite in occupying a higher sediment volume in a nonpolar solvent than in a polar solvent has been analyzed by Rao and Sridharan (1985). The above mentioned property of kaolinite is utilized to provide a quick and simple identification test for the presence of nonswelling clays, swelling clays, or a mixture of swelling and nonswelling clays in the soil. A definition of free swell index of soil has been given by Sridharan, Rao and Murthy (1986a) which is simple, less time consuming and more realistic.

Tabel 9 presents the physical and mineralogical properties of clays and soils from different parts of India which have been considered for the following discussions.

Fig. 20 shows the plot of sediment volumes occupied by the clay minerals and soils in water versus their corresponding volumes in carbon tetrachloride. The results are disposed in two distinct groups: the swelling bentonite clay (Number 13) and the soil samples 8 to 12 occupied minimum sediment volumes of 1.0 to 1.1 cm³/g in carbon tetrachloride with all the points lying on a plateau, the non-swelling kaolinite and chlorite clays (Numbers 1 and 2) and the soil samples 3 to 7 were positioned well above the plateau and occupied sediment volumes > 1.1 cm³/g in carbon tetrachloride, the values ranging from 1.3 to 2.8 cm³/g. This property of occupying a higher sediment volume in carbon tetrachloride is typical of the non-swelling clays with exposed gibbsitic surfaces, namely, kaolinite and chlorite; such a situation arises as these clay particles are randomly oriented in a non-polar solvent caused by their weak bonding (Rao and Shridharan, 1985). The higher sediment volumes of >1.1 cm³/g observed with soils 3 to 7 in carbon tetrachloride, then indicates the presence of kaolinite or chlorite clays in the soils. Although soils 4 to 7 were positioned above the plateau, they differed from the non-swelling clays in that their sediment volumes in water were higher than those in carbon tetrachloride. Such a behaviour may be explained on the basis of two types of clays present in the soil : a non-swelling clay belonging to kaolinite or chlorite group responsible for the sediment volume ≤ 1.1cm³/g in carbon tetrachloride and a swelling clay with significant R forces, favouring a high sediment volume of the soil in water. The higher sediment volume of soil 3 in carbon tetrachloride, than in water, points to the presence of non-swelling clay alone in the soil. Soils 8 to 12 occupied minimum sediment volumes of < 1.1 cm³/g in carbon tetrachloride and large sediment volumes in water ranging from 2.1 to 8.0 cm³/g. Their sediment volume pattern in water and carbon tetrachloride was similar to that of the swelling bentonite clay, namely, 1.05 cm³/g in carbon tetrachloride and 13.4 cm³/g in water, indicating the presence of the swelling smectite clay in the soil. In the non polar carbon tetrachloride, as R forces are insignificant, the net inter-particle interaction $(R - A)$ is attraction, facilitating the settling particles to approach closely and pack densely under the gravitational field. In the ploar water, the dominant R forces push the particles apart at points of close proximity, and a large equilibrium volume is finally attained when the repulsive R forces are balanced by the attractive A forces.

X-ray diffraction patterns of the magnesium saturated glycerol solvated clay fractions of soils 4 to 7

TABLE 9

PHYSICAL AND MINERALOGICAL PROPERTIES OF CLAYS AND SOILS

Sample Location	Liquid Limit %	Plasticity Index %	Particle Size Distribution %			Unified Soil Classification	X-ray Diffraction Mineralogy
			Sand	Silt	Clay		
1 Shimoga District,Karnataka	47.0	17.3	0	78	22	ML	Kaolinite, Quartz
2 Hassan District, Karnataka	36.0	non-plastic	25	68	7	ML	Chlorite
3 Bangalore District Karnataka	53.0	33.0	65	15	20	CH	Kaolinite, muscovite, Quartz, feldspar, diaspore.
4. Mangalore District Karnataka	72.6	32.2	13	66	21	MH	smectite, kaolinite, muscovite, hematite, quartz,feldspr
5. Bangalore District Karnataka	75.0	41.2	11	53	36	CH	kaolinite, smectite, muscovite, quartz.
6. Bangalore District Karnataka	75.0	38.7	22	42	36	MH	kaolinite, smectite. muscovite, quartz.
7. Korapet District Karnataka	66.0	26.0	12	58	30	MH	smectite, kaolinite, muscovite, quartz.
8. Chitradurga District, Karnataka	75.0	32.5	18	32	50	MH	smectite, antigorite, quartz, diaspore.
9. Dharwar District Karnataka	84.0	42.3	8	55	37	MH	smectite, quartz.
10. Bijapur District Karnataka	100.0	54.8	4	65	31	MH	smectite, feldspar, quartz, calcite.
11. Dharwar District Karnataka	106.4	62.3	7	51	42	MH	smectite, calcite, quartz, hematite.
12. Chitoor District Andhra Pradesh	124.2	101.0	33	28	39	CH	smectite, feldspar, quartz, diaspore, rutile.
13. Kolar District Karnataka	495.0	445.7	0	0	100	CH	monotmorillonite, quartz

Sample 1, 2, and 13 are pure clays
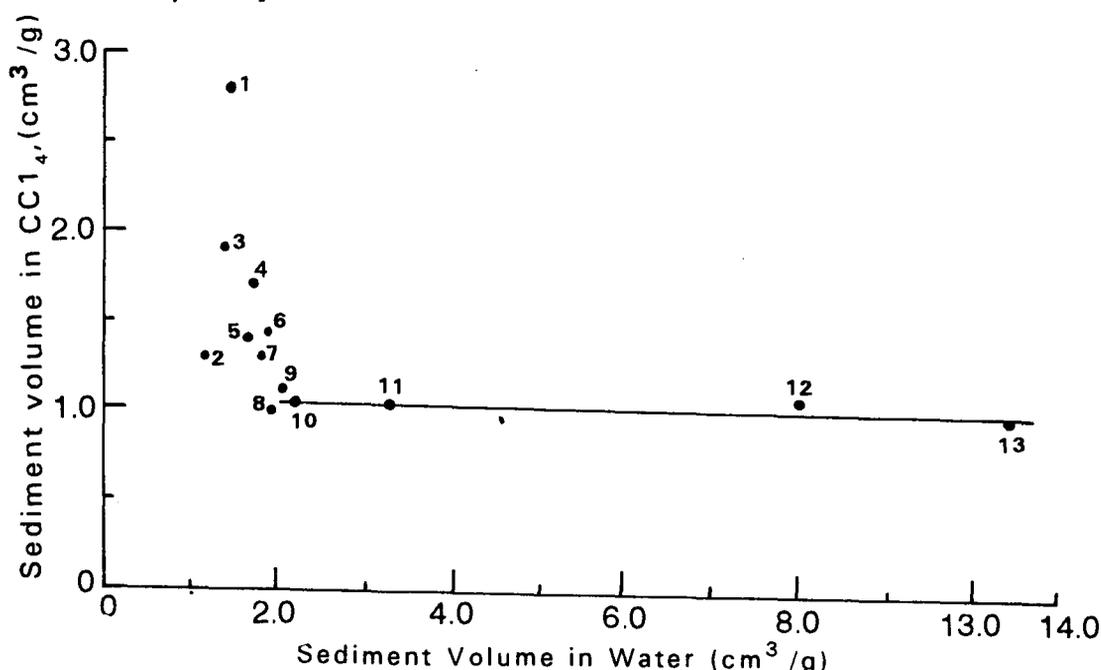
FIGURE 20 Plot of Sediment Volume of Clays and Soils in Water Versus their Sediment Volumes in Carbon Tetrachloride.

476

showed moderately strong reflections around 0.177, 0.702 to 0.719 nm, and around 0.349 to 0.356 nm. This indicates the presence of smectite and kaolinite clays, respectively, in the soils. The presence of kaolinite in these soils was also indicated by the 3690 cm^{-1} absorption in their infrared spectra. X-ray diffraction pattern of the Mg-saturated, glycerol solvated clay fraction of soil 3 showed very strong basal reflections at 0.719-nm and 0.358-nm characterstic of kaolinite. The presence of kaolinite in the soils was also shown by the strong 3690- and 3610-cm^{-1} absorptions in the infrared spectrum of the soil. The X-ray diffraction patterns of the Mg-saturated, glycerol solvated clay fractions of soils 8 to 12, showed basal reflections around 1.697 to 1.78 nm and ~0.45 nm corresponding to the presence of smectite alone in the soils. Hence, the inferences on clay types in soils as obtained by their sediment volumes in water and in CCl$_4$ agreed well with the result of X-ray diffraction and infra red analysis.

Limitation of existing Definition of Free-Swell Index of Soils and the Proposed Modification (Sridharan, Rao and Murthy 1985, 1986a)

Table 10 presents the percent free-swell index of soils as calculated from eqn. 21.

$$\frac{\text{Sediment Volume in water} - \text{Sediment Volume in kerosene}}{\text{Sediment Volume in kerosene}} \times 100 \qquad ....(21)$$

This eqn. uses the assumption that the non-polar liquid does not cause any swelling of the soil, and the volume occupied by unit weight of the soil in this solvent may be read as its original volume. The property of fixed lattice clays belonging to kaolinite or chlorite group, to occupy a higher sediment volume in non-polar liquid than in water, results in a negative free-sweel index for such clays, as can be seen in Table 10. Besides, soils 4 to 7, which contain both the non-swelling kaolinite and sweling smectite, occupy a higher sediment volume in carbon tetrachloride, than would have been the case, if smectite alone was present in the soils. This higher sediment volume occupied by such soils in carbon tetrachloride, results in a supression of their percent free-swell values.

With a view to overcome the above shortcomings, it has been proposed by Sridharan, Rao and Murthy 1985, 1986a, that the free-swell index may be calculated as :

$$\text{Free - swell index} = (V_d /10) \ cm^3/g \qquad ....(22)$$

where V_d is the sediment volume of 10 g of the dry soils specimen read from the graduated cylinder containing distilled water. The sediment volume of a soil in a non-polar solvent could however be utilized to give information on the clay type present in a soil, namely, non-swelling, swelling, or a combination of both.

TABLE 10

FREE-SWELL INDEX VALUES OF CLAYS AND SOILS

Sample	Free-swell Index,% (from Eq.21)	Free-Swell Index, cm^3/g (from Eq.22)
1.	−48.21	1.45
2.	−11.53	1.15
3.	−26.32	1.40
4.	2.94	1.75
5.	20.00	1.68
6.	31.03	1.90
7.	42.31	1.85
8.	96.00	1.95
9.	90.90	2.10
10.	109.52	2.20
11.	214.28	3.30
12.	627.27	8.00
13.	1240.00	13.40

The free-swell index of soils as calculated from eqn.22 is presented in Table 10. The values range from 1.15 cm³/g for soil 2 to 13.5 cm³/g for soil 13. The plot of sediment volumes of soils in water versus their corresponding volumes in carbon tetrachloride (Fig.20) helps to identify the type of clay, namely, swelling, non-swelling, or a combinatation of both, present in the soil. Based on the free-swell volumes of the soils in water and their clay type (reflected from their sediment volumes in water and carbon tetrachloride), a system of classification of a soil's expansivity under no external load condition is given in Table 11.

Correlations are also available to predict the degree of soil swelling from Atterberg limits, activity of clays, and colloid content (<2 μm) of soils (van der Merwe, 1964; Gibbs 1969; and Skempton, 1953). The probable expansion of soils as estimated from the liquid limit, plasticity index, activity, and free-swell index are given in Table 12. It is readily seen that, with the exception of soils 12 and 13, the free-swell index predicts a lesser expansivity than the existing evaluation technique.

The clay mineralogical composition of the soils from Table 9 is as follows.

TABLE 11

CLASSIFICATION OF SOIL EXPANSIVITY

Free-swell Index in Water,cm³/g	Sediment Volume in Carbon tetrachloride cm³/g	Clay Type	Soil Expansivity
<1.50	1.10 to 3.00	non-swelling	negligible
1.50 to 2.00	>1.10 and < sediment volume in water	mixture of non-swelling and swelling	low
1.50 to 2.00	≤ 1.10	swelling	moderate
2.00 to 4.00	≤ 1.10	swelling	high
>4.00	≤ 1.10	swelling	very high

TABLE 12

COMPARISION OF SOIL EXPANSIVITY FROM VARIOUS EVALUATIN TECHNIQUES

Soil	Clay Type	Classification of Soil Expansvity			
		van Der Merwe (1964)	Liquid limit (Gibbs 1969)	Plasticity Index (Skemption ,1953)	Free-swell (Sridharan, Rao and Murthy, 1985- 1986a)
1	K	medium	medium	low	negligible
2	Ch	low	low	low	negligible
3	K	high	high	high	negligible
4	Sm, K	high	very high	high	low
5	K, Sm	very high	very high	very high	low
6	K, Sm	very high	very high	very high	low
7	Sm, K	high	very high	very high	low
8	Sm	high	very high	high	moderate
9	Sm	very high	very high	very high	high
10	Sm	very high	very high	very high	high
11	Sm	very high	very high	very high	high
12	Sm	very high	very high	very high	very high
13	Sm	very high	very high	very high	very high

K = Kaolinite, Ch = Chlorite, and Sm = Smectite.
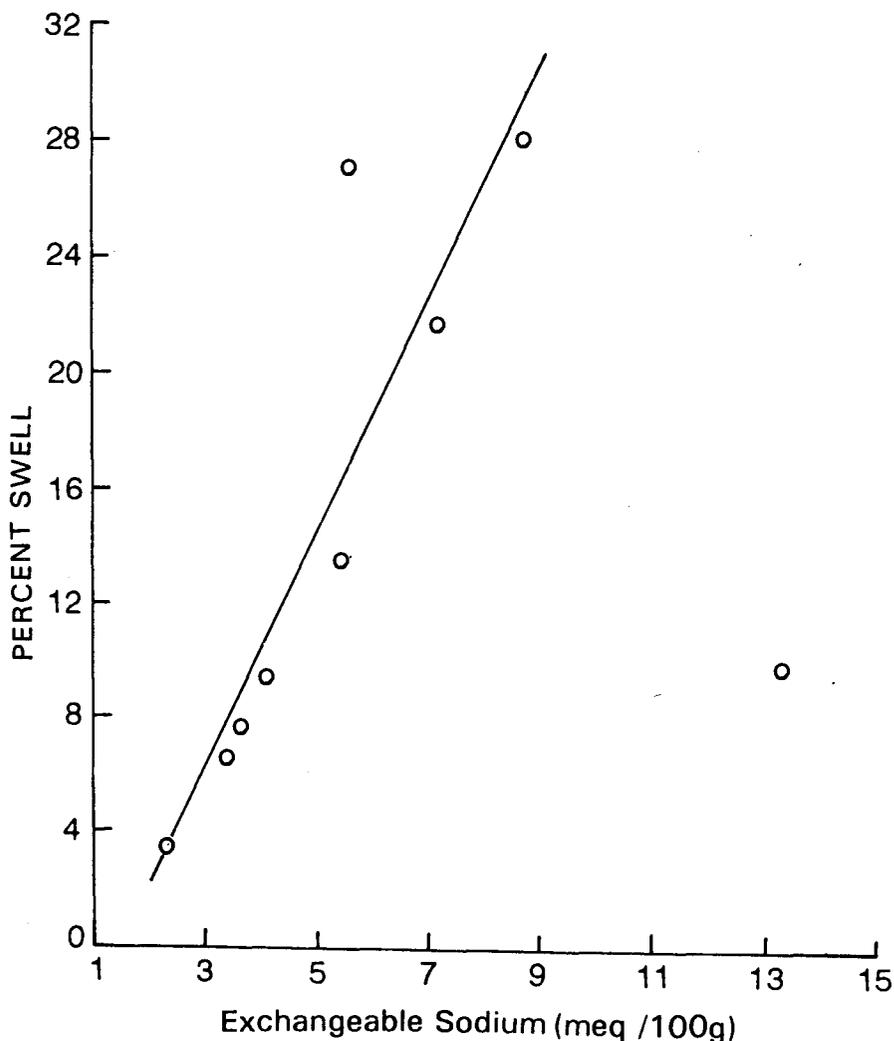
FIGURE 21 Plots of Percent Swell Versus Exchangeable Sodium Content for Montmorillonitic Soils

The clay fractions of soils 1 and 3 are comprised of the non-swelling kaolinite alone. soils 4 and 7 refer to acidic soils (indicated from pH values) containing smectite and kaolinite clays. Acidic soils have polyvalent aluminium as the exchangeable cations with a consequent low swelling capacity (Yong and Warkentin, 1966). Soils 5 and 6 are the commonly termed red earth whose clay fractions are dominated by non swelling kaolinite, with the presence of small amounts of smectite group of clays. The clay fractions of soils 8 to 15 contained only the smectite clay group.

It may be inferred from the clay mineralogical considerations that the existing evaluation techniques do not give a realistic picture of a soil's swelling ability. In particular, classification of soils 1 and 3 as moderately and highly swelling, though their clay fractions are constituted by kaolinite, is indeed questionable. Similarly soils, 5 and 6, which have a predominantly kaolinitic character, are categorized as very highly swelling. Further, the acidic soils 4 and 7, which have a low swelling capacity caused by the presence of polyvalent aluminium ions, are classified as very highly swelling.

The usefulness of the existing evaluation methods in estimating the probable expansion of soils is perhaps restricted, since the mechanisms controlling Atterberg limits and swelling may not be the same (Sridharan and Rao, 1973-1979; Rao and Sridharan, 1985; Sridharan and Rao, 1975; Mitchell, 1976). The free-swell index gives a truer picture of the soil's swelling ability as it considers the type of clay minerals present in the soil and their associated R and A forces, reflected in the sediment volume of the soil in carbon tetrachloride and water.

479
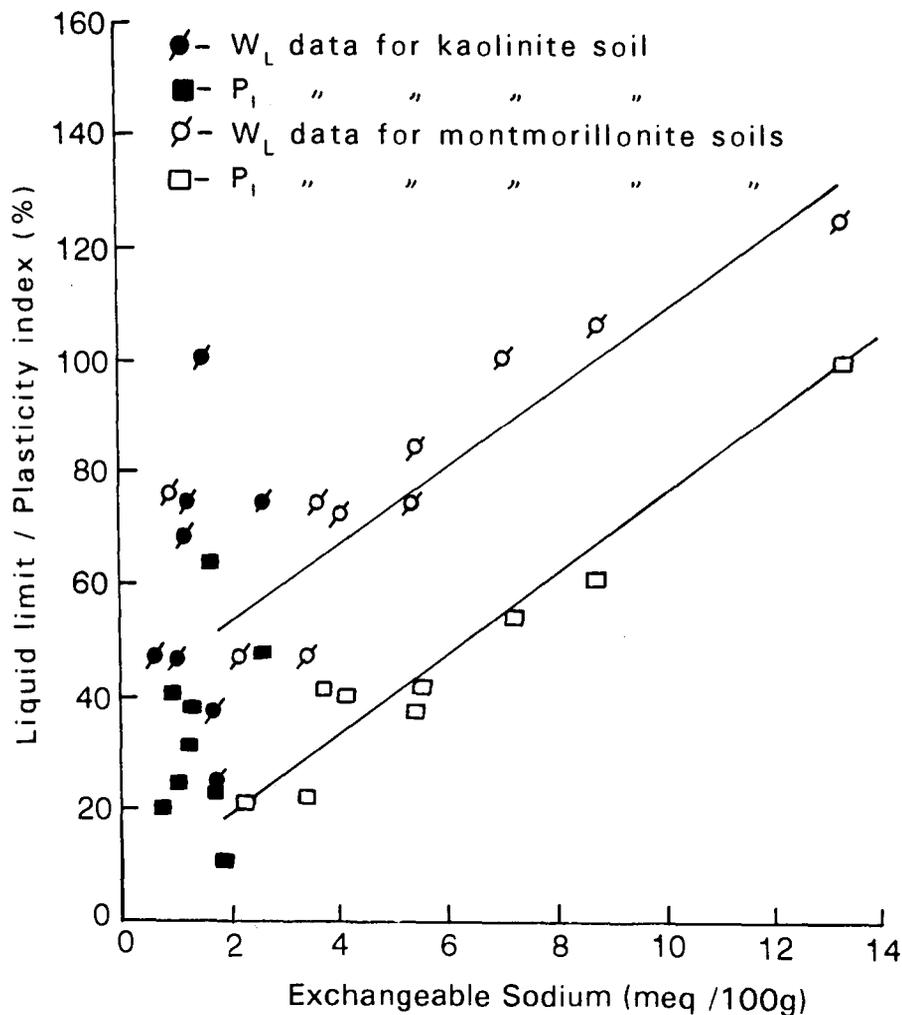
FIGURE 22 Plots of Liquid Limit/Plasticity Index Versus Exchangeable Sodium Content of Kaolinitic and Montmorillonitic Soils

CLASSIFICATION OF FINE GRAINED SOILS AS KAOLINITIC AND MONTMORILLONITIC

Having discussed about the clay mineralagy, the electrical attractive and repulsive forces, index properties as affected by physico chemical environment and the fr e swell index tests, it emerges that the behaviour of kaolinitic and montmorillonitic soils are governed by different mechanisms. While the engineering properties of montmorillonitic soils are primarily governed by the thickness of diffuse double layer, the kaolinitic soils properties are mainly influenced by fabric and attractive forces. An approximate but a simple procedure of identifying these groups of soils has been indicated previously (Fig. 20)

In the following paragraphs some more soils will be examined from the view point of classifying them as montmorillonitic and kaolinite soils.

Table 13 (Sridharan and Rao, 1988) presents the physical characterstics of 18 natural soils collected from different parts of South India from a shallow depth of 2 to 3m. Their X-ray diffraction analysis indicate that soils 1 to 9 are soils with primary clay mineral as kaolinite and soils 10 to 18 with montmorillonite as the primary clay mineral.

In a recent study, Sridharan, Rao and Murthy (1986c) have highlighted the role of exchangeable sodium in governing the diffuse double layer thickness and the liquid limit of montmorillonitic soils. Since the swelling forces in a clay soil are a function of diffuse double layer repulsion, it is expected that the exchangeable sodium content also governs the former. Fig.21 presents percent expansion from air-dry to saturated condition for the montmorillonitic soils (Nos. 10 to 18) obtained using an oedometer under a load of 7 kPa as a function of the exchangeable sodium content of the soils. A strong correlation exists between the two parameters indicating that the amount of swelling of the soil specimens is a function of the concentration of exchangeable sodium ions available. Hence, in order that predictions of relative swelling abilities based on index tests be reliable,

TABLE 13

PHYSICAL CHARACTERISTICS OF SOILS STUDIED

Soil Number	Grain Size Distribution,%			Liquid Limit, %	Plastic Limit, %	Plasticty Index, %	Activity	Sediment Volume		
	Sand	Silt	Clay					In water cm³/g	In Carbon Tetra-chloride, cm³/g	Expansi-on from Oedo-meter, %
1.	36	39	25	25	13.8	11.2	0.56	1.00	1.10	2.4
2.	34	39	27	38	15.3	22.7	1.03	1.25	1.15	2.1
3.	59	16	25	47	21.5	25.5	1.28	1.30	1.15	1.0
4.	40	36	24	47	25.4	21.6	1.14	1.40	1.23	0.8
5.	19	60	21	68	35.4	32.6	2.04	1.75	1.70	4.2
6.	12	58	30	74	25.0	49.0	1.96	1.85	1.35	4.1
7.	11	53	36	75	33.8	41.2	1.33	1.85	1.40	1.3
8.	22	32	46	75	36.3	38.7	0.94	1.90	1.45	1.5
9.	10	39	51	100	35.8	64.2	1.40	2.40	1.58	2.0
10.	49	20	31	47	25.0	22.0	0.85	1.30	1.00	3.5
11	54	32	15	47	24.5	22.5	2.25	1.45	1.05	6.7
12.	13	39	48	72	31.4	40.6	0.94	1.90	1.18	9.7
13.	18	32	50	75	32.5	42.5	0.94	1.80	1.00	7.6
14.	27	49	24	75	36.6	38.4	2.02	1.95	1.00	13.6
15.	8	55	37	84	41.7	42.3	1.32	2.10	1.10	27.3
16.	4	65	31	100	45.1	54.9	2.11	2.20	1.05	21.8
17.	7	51	42	106	44.1	61.9	1.67	3.30	1.05	28.4
18.	33	28	39	124	23.2	100.8	2.96	8.00	1.10	13.6

correlations should exist between the exchangeable sodium content that influences the swelling magnitude and the index property of the soil concerned (Sridharan and Rao, 1988).

Fig. 22 presents the liquid limit and plasticity index of kaolinitic and montmorillonitic soils as a function of their exchangeable sodium contents, and Fig. 23 plots their activity values as a function of the latter. With kaolinitic soils, the variations appear as a scatterogram suggesting that the diffuse double layer repulsion does not influence their index properties. In the case of montmorillonitic soils strong correlations exists in all the plots indicating that their index properties are governed by diffuse double layer forces.

Fig. 24 plots the results of free-swell oedometer tests for kaolinite clay and a 75% kaolinite +25% bentonite mixture. On innundation with water (under the nominal load of 7kPa), the kaolinite clay under goes compression with time rather than swelling highlighting the incapability of the mineral to swell due to moisture absorption process. Such a situation presumably arises owing to the insignificant contribution from osmotic repulsion forces produced in the diffuse electrical double layers in the aqueous phase occupying the voids of the solids. In comparison, the 75% kaolinite+25% bentonite mixture on innundation with water, initially exhibits some compression and later shows net expansivity. Extending the results to natural soils it follows that soils containing kaolinite alone have no inherent ability to swell on exposure to moisture, the presence of an expansive clay however imparts an expansive nature to the soil.

The results of the study bring out that the presence of an expansive clay is essential for swelling by moisture absorption. The seat of swelling forces, namely, diffuse double layer repulsion, governs the index properties of montnorillonitic soils as well; hence the use of index tests in estimating expansivities of montmorillonitic soils is mechanisticallly justified. The index properties of kaolinitic soils are not a function of diffuse double layer repulsion, and hence their employment in prediction of soil expansivities has no scientific basis.

VOLUME CHANGE BEHAVIOUR

SATURATED CLAYS

The importance of the prediction of volume change behaviour of clays in soil engineering practice needs hardly be stressed. Several factors are responsible for volume changes in natural and compacted clays. Volume change could occur due to changes in effective stress brought about either by changes in external loading applied to the soil, or due to changes in environmental conditions during and after construction like changes in water table, seepage forces temperature and leaching action. In the recent, past, considerable progress has been made in predicting settlements caused by consolidation, particularly if the compression is approximately one-dimensional.

The concept of effective stess, as formulated by Terzaghi has made possible the application of rational principles to many soil engineering problems that previously could only be treated in an empirical manner. Probably the most important 'tool' that has been in the contemporary development of soil mechanics is the concept of effective stress. It has been brought out earlier that the effective stress eqn. 14 as formulated by Terzaghi need to be modified to take into consideration the electrical attractive and repulsive forces. Since the understanding of the volume change behaviour of clays appears to depend on a better understanding of the physical mechanisms which are involved and since the effective stress concept is a useful tool in explaining the engineering behaviour, this paper is concerned with the elucidation of the phenomena that produce one-dimensional volume changes (compression and swelling) in clays in the light of the modified effective stress concept (eqn.17) which takes into consideration the electrical attractive and repulsive forces in the particulate system.

Based on a review of the existing mechanisms controlling the volume change behaviour of clays and on the experimental data obtained, the modified effective stress concept has been qualitatively examined by Sridharan and Rao (1973), Sridharan, Rao and Pandian (1973), and Rao and Sridharan (1985). Their study shows that the modified effective stress concept qualitatively explains the basic mechanisms controlling the volume changes of clays, and indicates the attention needed to develop a consistent method for applying the concept quantitatively.

BASIC MECHANISMS CONTROLLING COMPRESSIBILITY OF CLAYS

Terzaghi (1929) is perhaps the first to make an attempt to understand the mechanisms controlling the compressibility of clays. He concluded that large void ratios and high compressibilities of clays were caused by the presence of 'scale shaped' mineral particles. He has also postulated the existence of semi-solid layers of absorbed water on clay surfaces for explaining the low permeabilities of clay and secondary consolidation. Leonards and Altschaeffl (1964) proposed a mechanism for compression in which the magnitude of the inter-particle bonds at the near contact points (which are not equal at all contacts) and their dispersion plays a decisive role in the stress-strain response. Change in effective stress produces the sliding between particles which results in volume changes and the sliding of particles is controlled by the bond strength.

Form a laboratory investigation on the natrure of bonds in Canadian clay, Kenney et al. (1967) concluded that the resistance against deformation that a soil exhibits during a consolidation test is a manifestation of the shearing resistances that can be mobilized at the contacts between individual particles; for any applied pressure, the greater are the shearing resistances at the particle contacts, the greater is the resistance against deformation and the smaller is the deformation.

Bolt (1956) explained the compressibility of 'pure' clays by the consideration of long-range repulsive forces between the particles. Due to the diffuse type of ion distribution around a clay particle in a clay electrolyte system, the system can be regarded as an osmometer, the semi-permeable membrane of which is formed by the clay particles themselves. Upon applying a load to such a system, a certain amount of pore fluid is pressed out until the difference between the osmotic pressure of the system and of the free liquid phase pressed out equals the loading pressure. Contrary to the situation in coarse-granined soils, this repulsion between clay particles does not therefore depend upon a direct contact between the particles. It was hence concluded that the compressibility will essentially be a function of the double layer repulsive force which is primarily dependent on the type of clay and the electrolyte content of the system. As the lattice structure of both monrillonite and illite is basically the same, Bolt(1956) opined that the behaviour of the two will be similar, varying only in magnitude. Mitchell (1960) performed compression tests on Na-kaolinite, Na-illite and Na-montmorillonite to study the applicability of the above theory to the compression of clays. Based on the test results, he concluded that double layer theory is not
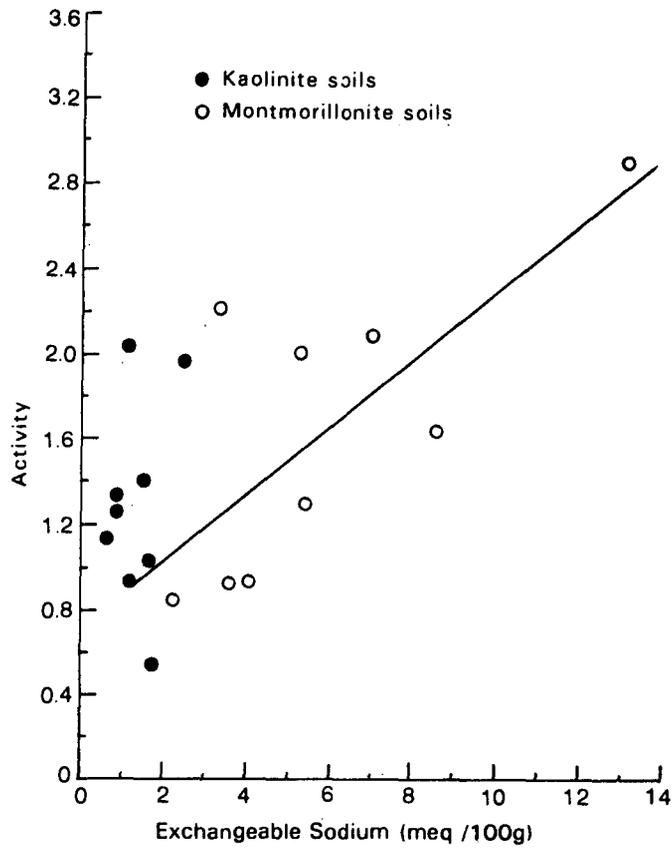
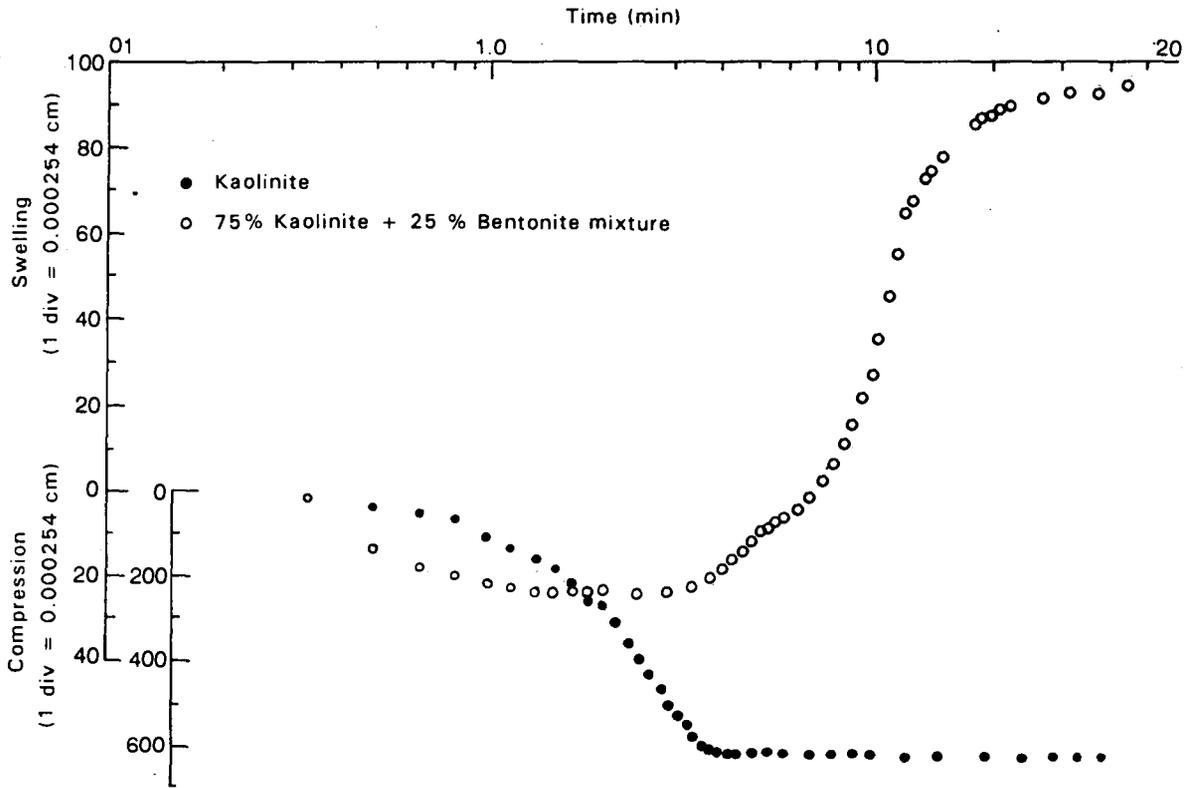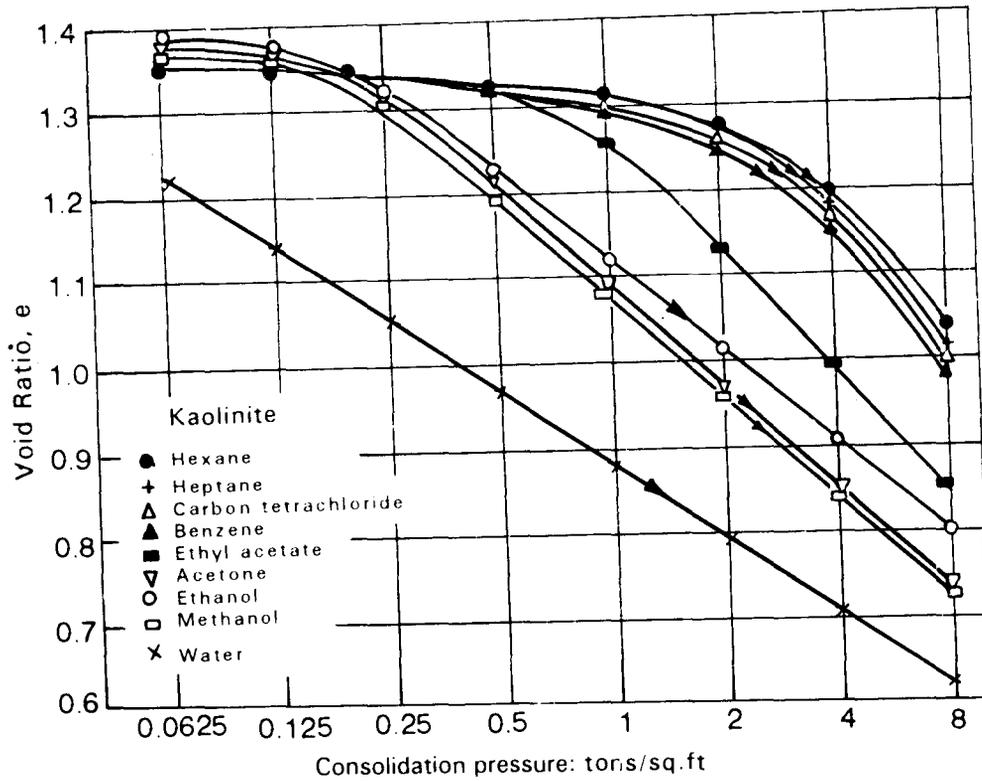FIGURE 23 Plots of Activity Versus Exchangeable Sodium Content for Kaolinitic and Montmorillonitic Soils
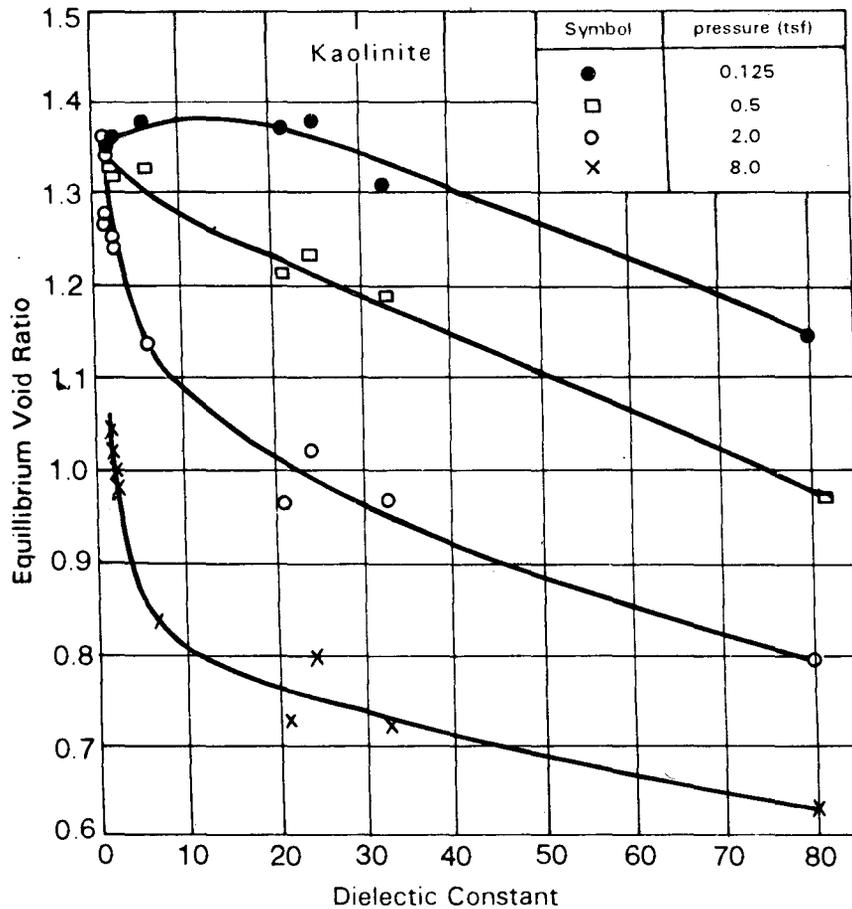

FIGURE 24 Free-Swell Oedometer Plots for Kaolinite and 75% Kaolinite + 25% Bentonite Mixture

FIGURE 25 One-Dimentional Consolidation Curves for Kaolinite with Different Pore Fluids (1 ton/sq.ft. ≈ 1 kg/cm²)

FIGURE 26 Effect of Dielectric Constant on Equilibrium Void Ratio at Different Consolidation Pressure for Kaolinite (1 ton/sq.ft. ≈ 1 kg/cm²)
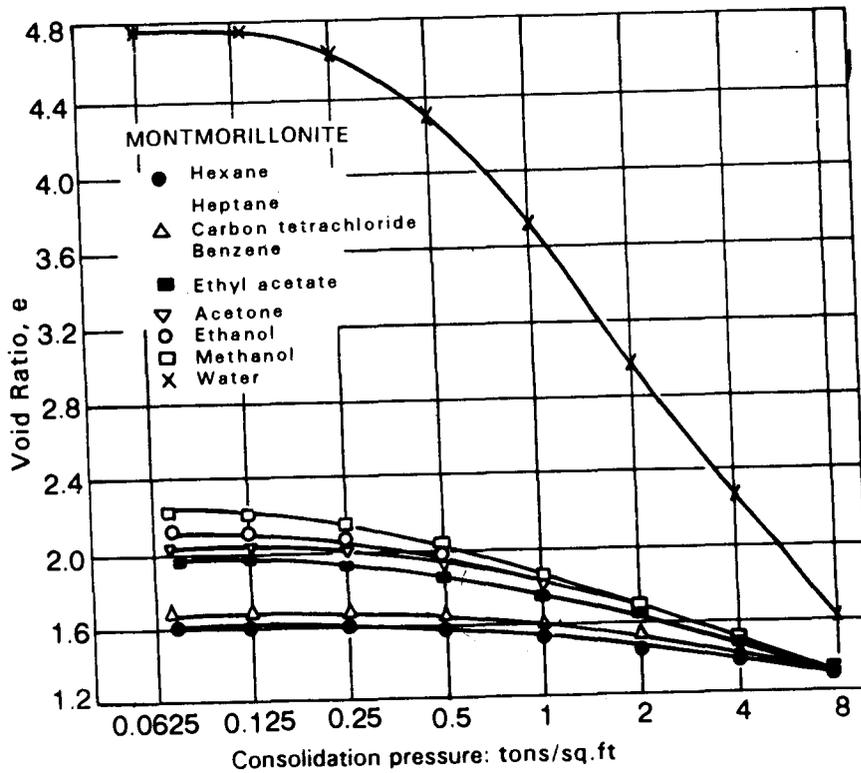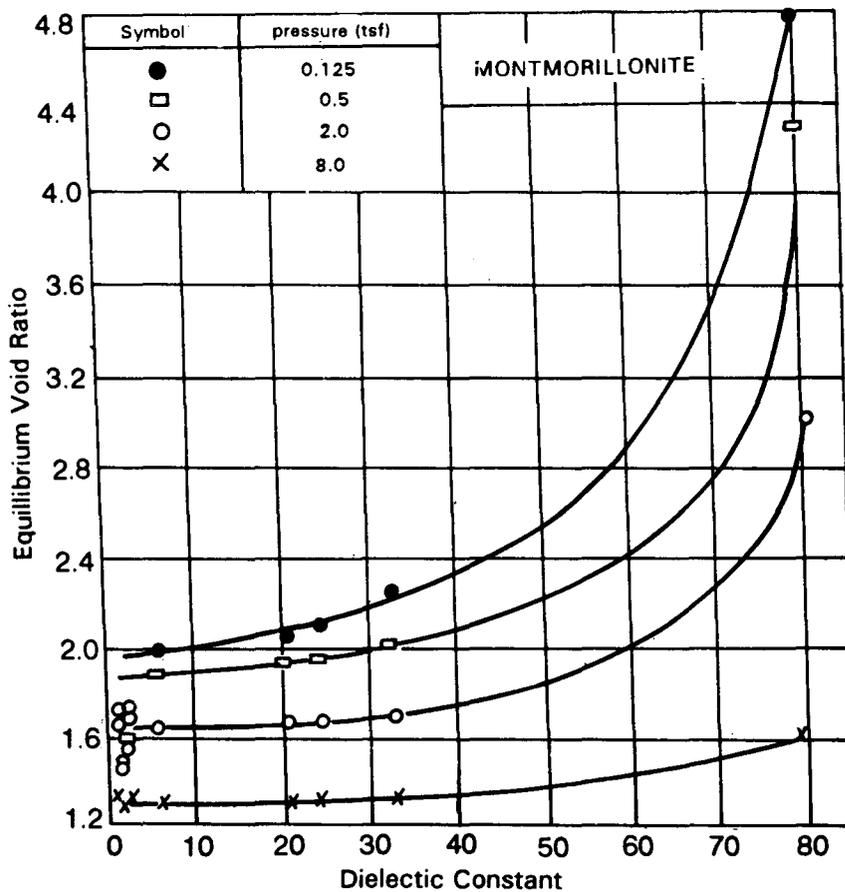
484

FIGURE 27 One-Dimentional Consolidation Curves for Montmorillonite with Different Pore Fluids (1 ton/sq.ft $\approx$ 1 kg/cm^2)

FIGURE 28 Effect of Dielectric Constant on Equilbrium Void Ratio at Different Consolidation Pressure for Montmorillonite (1 ton/sq.ft. $\approx$ 1 kg/cm^2)

485

TABLE 14

PROPERTIES OF THE SOILS

Properties	Kaolinite	Montmorillonite	Black cotton soil
Liquid limit (%)	49	305	105
Plastic limit (%)	29	44	28
Plasticity index (%)	20	261	73
Specific Gravity	2.59	2.83	2.71
% retained on BSS sieve No.200	0	0	38
silt size fraction (%)	46	0	43
Clay size fraction (%)	54	100	29

applicable to clays containing clay particles greater than some size between 0.2 and 1.0 μm diameter. In those clays which do not meet the assumptions of the theory (e.g. sytems with perfectly flat clay particles having no irregular or terraced surfaces), the initial particle arrangement appeared to be of overriding importance. On the basis of one-dimensional consolidation tests performed using specimens of ground muscovite, kaolinite, illite and smectite and varying the valency of the adsorbed cations, the pore water electrolyte concentration and the nature of the pore fluid, Olson and Mesri (1970) discussed the mechanisms controlling the compressibility of clays and concluded that smectite is primarily controlled by the physico-chemical model and kaolinite by the mechanical model.

To verify the above mechanisms, Sridharan and Rao 1973, Sridharan, Rao and Pandian, 1973 conducted diagnostic experiments and explained the behaviour in the light of the modified effective stress concept (eqn.17). Experiments were conducted with commercially pure kaolinte and montmorilloniite clays and a natural black cotton soil containing essentially montmorillonite clay mineral. The physical properties of these soils used are given in Table 14. Table 4a list the various fluids used.

Several series of one-dimensional consolidation tests were conducted in the conventional oedometers using various organic pore fluids. Test were also conducted by replacing the remoulding pore fluid with another. Only seleted test results are presented in the following paragraphs.

COMPRESSION BEHAVIOR

Void ratio-external applied pressure relationships for kaolinite obtained with various organic pore fluids are shown in fig. 25. From this figure, it is seen that the e-log p curves are placed in the order of their dielectric constant, the uppermost curve being that of hexane (dielectric constant = 1.89), whereas the lower-most one is that of water (dielectric constant = 80.4). With the exception of acetone, the e-log p curves for fluids with low dielectric constants show higher dipole moment (as seen from Table 4a) and this may possibly be the reason why the é-log curves for acetone do not conform to the general pattern of behaviour. In order to bring out the significant influence of dielectric constant more clearly the results of Fig. 25 have been replotted as equilibrium void ratio when there is an increase in dieletric constant (Fig.26).

Fig. 27 presents the e-log p curves for montmorillonite with various fluids. The compression curves obtained for both hexane and heptane are almost identical, hence they are represented in Fig. 27 as a single curve. This is also the case for the curves of carbon' tetrachloride and benzene. Contrary to what has been observed for kaolinite (Fig. 25), it is seen from Fig.27, that for montmorillonite, the uppermost e-log p curve is that for the highest dielectric constant (i.e water), whereas the lowermost one is that of hexane of the lowest dielectric constant . Fig. 28 illustrates the equilibrium void ratio-dielectric constant relationships at different externaly applied pressures in which the significant influence of dielectric constant is brought out. Comparison of Figs. 25 and 26 and Figs. 27 and 28, shows clearly the distinctly opposite types of behaviour in kaolinite and montmorillonite systems.

It will be shown later that the shear strength, friction angle and cohesion intercept decrease as the dielectric constant of the pore medium increases, irrespective of the soil type. It has been noted earlier that the electrical attractive forces, A decrease and the repulsive forces R increase with an increase in the dielectric constant of the pore medium. Thus an increase in the dielectric constant decreases the shear strength, the friction

angle and the cohesion intercept. Since macro-behaviour is the manifestation of micro-behaviour, these results indirectly show that the shearing resistance at micro-level decreases with increase in dielectric constant.

If mechanism 1, in which the compressibility of clays is primarily controlled by the shearing resistance at the near contact points, governs the behaviour then an increase in the dielectric constant of the pore medium decreases the shearing resistance and hence should bring about lower equilibrium void ratios at the same externally applied pressure. The results of kaolinite (Figs 25 and 26) are qualitatively consistent with mechanism 1 and the modified effective stress concept (eqn. 17). It should be pointed out here that the conventional effective stress eqn. (eqn. 1) fails to explain this behaviour because it does not take into account the interparticle electrical forces.

In an another study on the mechanisms controlling volume change behaviour of kaolinite, Rao and Sridharan (1985) have brought out that (1) the dipole moment of the pore fluid is responsible for the geometric arrangement of the kaolinite clay particles and (2) the shear resistance at particle level which governs the volume change behaviour is a function of the clay fabric. Consequently in a nonpolar solvent, the randomly oriented kaolinite particles exhibited maximum shear strength and low compressibility.

It has already been observed that the behaviour of montmorillonite is quite opposite to that of kaolinite. In spite of a decrease in shearing resistance with increase in dielectric constant, the equilibrium void ratio for a particular externally applied pressure increases with dielectric constant, unlike what has been noticed for kaolinite. The montmorillonite clay is able to withstand the external pressure at higher void ratio with a pore medium of higher dielectric constant and vice versa. As in mechanism 2, in which compressibility is primarily governed by the long range electrical repulsive forces which means that the compression at a given externally applied pressure is primarily resisted by the double layer repulsive forces. As has been shown earlier, this primary force of repulsion increases with an increase in dielectric constant of the medium. Hence, the experimental observations regarding montmorillonite (Fig. 27) are consistent with mechanism 2.

Applying the modified effective stress concept as given in eqn. 17 an increase in the repulsive force R, due to an increase in dielectric constant reduces the effective contact stress. It then follows (from a concept similar to the conventional approach) that a decrease in effective stress as defined in equation (17) results in an increase in volume. The analysis made for kaolinite (i.e. a decrease in effective stress causing a decrease in volume) may look paradoxical. But it should be emphasized that in the case of kaolinite, the decrease in volume is due to a decrease in shearing resistance at interparticle level. Thus, it is emphasized here that the effective stress concept should be applied only in conjunction with the physical mechanisms involved. Thus, the results presented in Fig. 27 for montmorillonite clay can be explained in terms of the effective stress concept in conjunction with mechanism 2.

REBOUND BEHAVIOUR

Fig. 29 presents the relationship between dielectric constant and the rebound (as change in void ratio) due to the reduction of consoildation pressure from $8kg/cm^2$ to $0.0625 kg/cm^2$. It is evident that in kaolinite clay the dielectric constant has little effect on the magnitude of rebound, demonstrating that the rebound was primarily due to the mechanical effects, e.g. the elastic rebound and the hydrostatic pressure deficiency due to the release of pressure from $8 kg/cm^2$ to $0.0625 kg/cm^2$. For montmorillonite, however, the dielectric constant has a significant effect on the magnitude of rebound showing that apart from mechanical effects, the magnitude of rebound is significantly influenced by the diffuse double layer thickness which increase directly with dielectric constant. It may be seen from Fig. 29 that the magnitude of rebound at very low dielectric constants(e.g. using hexane, heptane, carbon tetrachloride or benzene) for both kaolinite and montmorillonite clays is almost in the same range. These pore fluids, because of their low dielectric constants and zero dipole moments (except for heptane, which has a small dipole moment of 0.08 debyes), develop very little or no double layer at all. Hence, the rebound for these cases is primarily due to the hydrostatic pressure deficiency which is same for both kaolinite and montmorillonite.

TESTS WITH REPLACEMENT OF PORE FLUIDS :

KAOLINITE

In order to further confirm the two basic mechanisms together with the applicability of the modified effective stress concept controlling the volume change behaviour of clays, experiments were designed in such
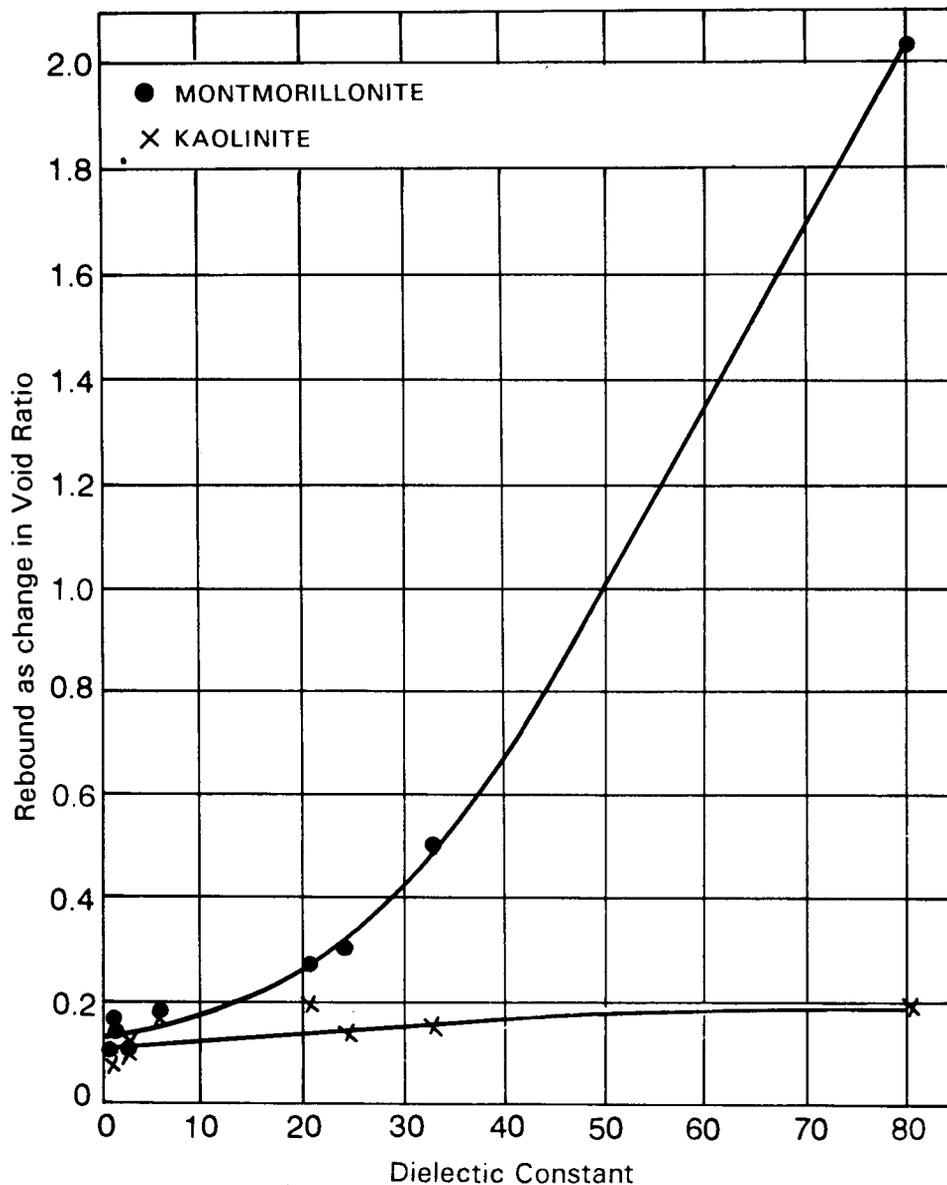
FIGURE 29   Effect of Dielectric Constant on Rebound

a way that one fluid could be replaced by another, in a consolidation sample under an external load. The details of experimentation have been described by Sridharan and Rao (1973).

Fig. 30 shows the e-log p relationships for kaolinite clay remoulded with water (curve 3), series 2. In the direct method, the water is replaced directly by CCl_4. In the indirect method, it is replaced first by ethyl alcohol then by acetone, and then by CCl_4. For comparison, the e-log p curve for the same clay remoulded with water (curve 1) is also shown in the same figure. The indirect method is quite successful in replacing the carbon tetrachloride by water, possibly for the following reasons.

(a)   Carbon tetrachloride is completely miscible with acetone as is acetone with ethyl alcohol and ethyl alcohol with water. Hence there is a possibility for better replacement than when one attempts to replace carbon tetrachloride directly by water, because these two liquids are immiscible.

(b)   The absorptive capacity of clay surfaces for water is very high. This further aids replacement.

(c)   The surface charge density and presence of positive edge and negative face charges, promote edge-face contacts in the kalonite clay carbon tetrachloride system to a greater measure due to the absence of double layers. When an attempt is made to replace the pore fluid by water, due to the mobilization of double layers to the extent governed by the clay's potential, the system collapses due to the reduction in the shearing resistance at interparticle level. The microstrains developed due to the magnitude of external loading induce reorientation of clay fabric units to stable configuration towards minimum potential position.

488
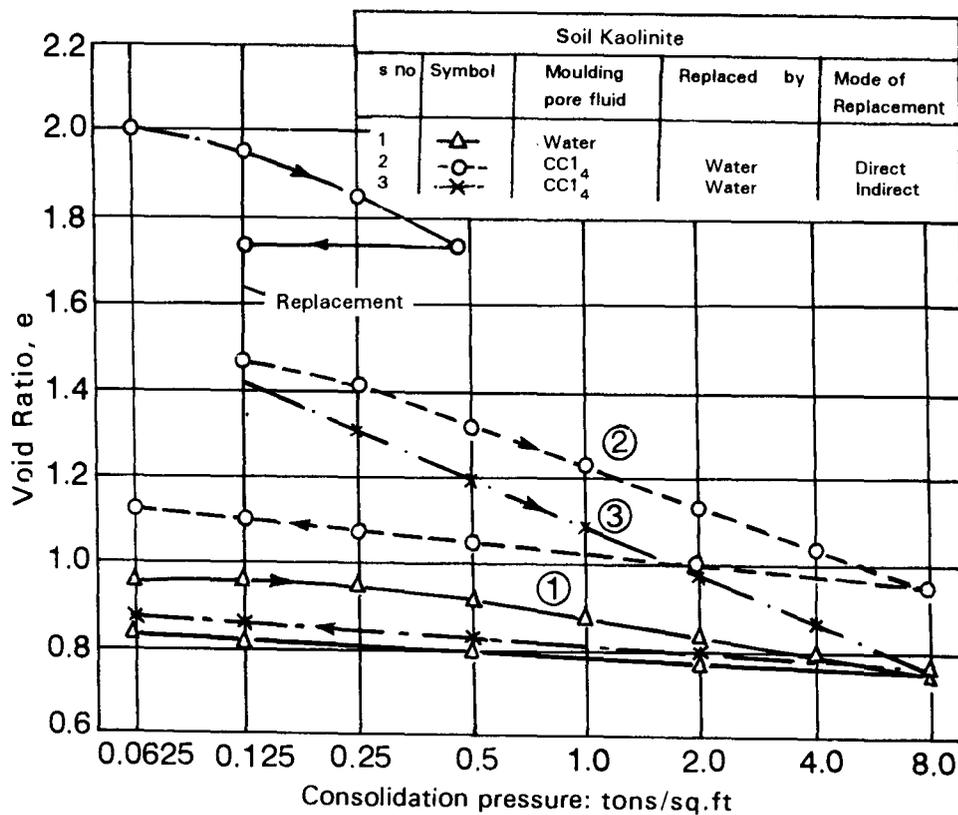
FIGURE 30 One-Dimentional Consolidation Curves for Kaolinite with Replacement of Pore Fluids (1 ton/sq.ft. $\approx$ 1 kg/cm²)

For these reasons, the e-log p curve of the kaolinite system originally remoulded by carbon tetrachloride and subsequently replaced by water indirectly (curve 3, Fig. 30) finally joins the e-log p curve for water as pore fluid (curve 1). The direct method (curve 2) is only partly successful.

The dominant influence of mechanism 1 is clearly shown in Fig. 30, which gives the e-log p curves of kaolinite clay remoulded with carbon tetrachloride and subsequently replaced by water at a pessure of 0.125 kg/cm². Because of the low dielectric constant of carbon tetrachloride, the electrical attractive forces predominate causing higher shearing resistance at interparticle level governed by equation (17) and the clay with carbon tetrachloride as a fluid is able to resist the external pressure of 0.125 kg/cm² at a relatively higher void ratio. Once the fluid is replaced by water, there is a loss in shear strength due to the decrease in effective stress because the electrical repulsive forces predominate (equation 17) and thus there is a collapse of the structure, resulting in a volume decrease.

MONTMORILLONITIC SOILS

Figures 31 and 32 present the e-log p relationships obtained by using black cotton soils, a natural clay soil containing, montmorillonite. Fig. 31 shows the e-log p relationships for black cotton soil remoulded with water and subsequently replaced by carbon tetrachloride. For the sake of comparison., the e-log p curve for the same soil remoulded with carbon tetrachloride is also shown in the same figure. As in kaolinite, the replacement through acetone and ethyl alcohol was better than the direct method. The e-log p curve of the sample in which the water is replaced by carbon tetrachloride indirectly (Fig. 31, curve 3) almost coincides with that of the sample wherein carbon tetrachloride is fluid (curve 1), whereas the one obtained by direct method (curve 2) shows a marked difference. Although the results presented in Fig. 32 are in order and there is a better agreement with the indirect method, the consolidation curves after replacement (final pore fluid-water) do not join the one in which the consolidation has been carried out with water itself. The possible reason for this behaviour is that in the black cotton soil-carbon tetrachloride system, the potential energy is lower, as is evident from the lower equilibrium void ratio. Even when the pore fluid is replaced completely, the energy imbalance is still of insufficient magnitude to induce rebound equal to that of the state corresponding to that of the black cotton soil-water system.

The predominant influence of mechanism 2 is clearly demonstrated by the results shown in Fig. 31. As black cotton soil is essentially a montmorillonite type clay, double layer repulsive forces are predominant when
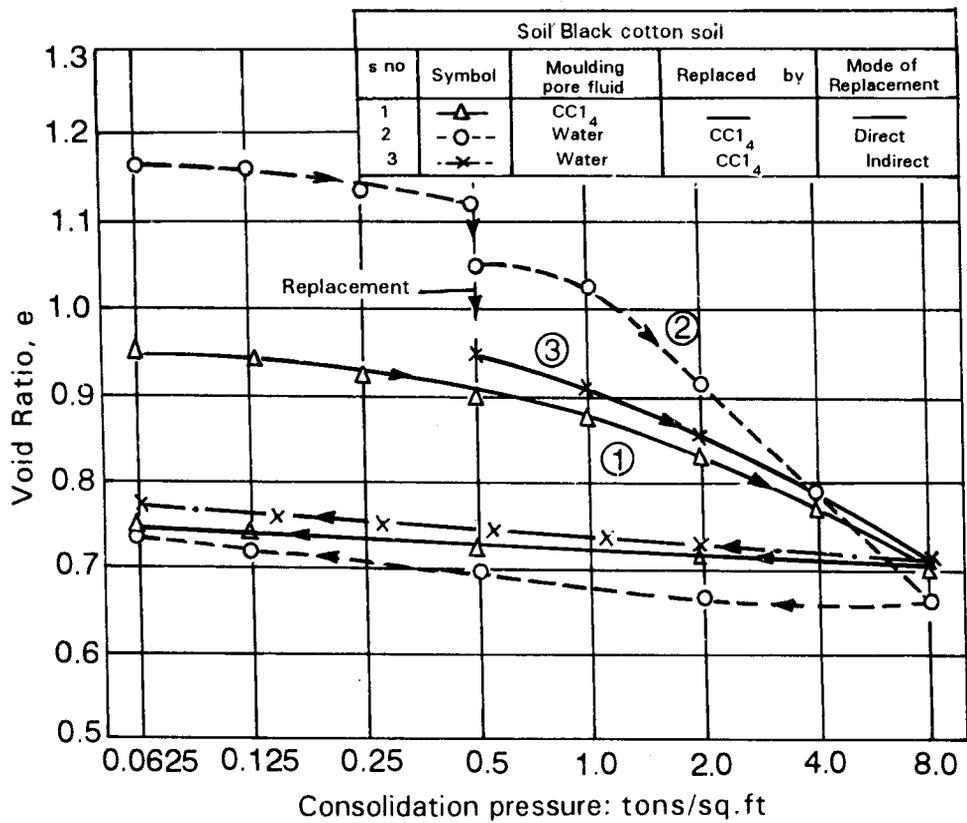
489

FIGURE 31 One-Dimentional Consolidation Curves for Black Cotton Soil with Replacement of Pore Fluids (1 ton/sq.ft. $\approx$ 1 kg/cm^2)
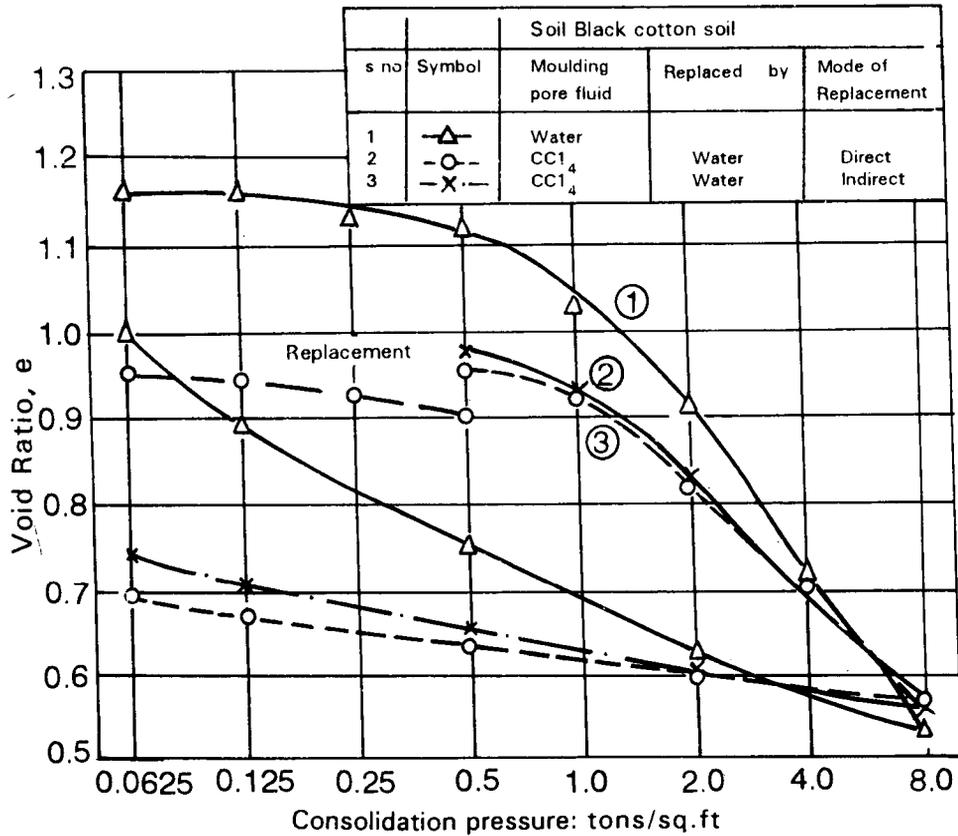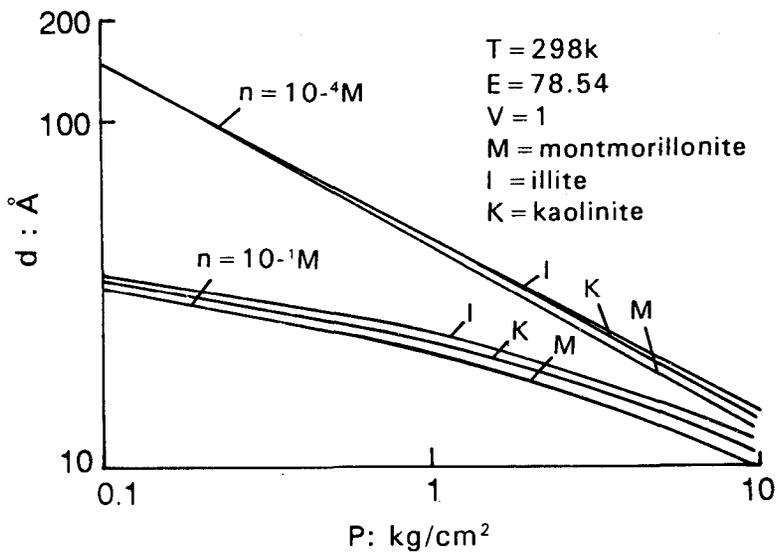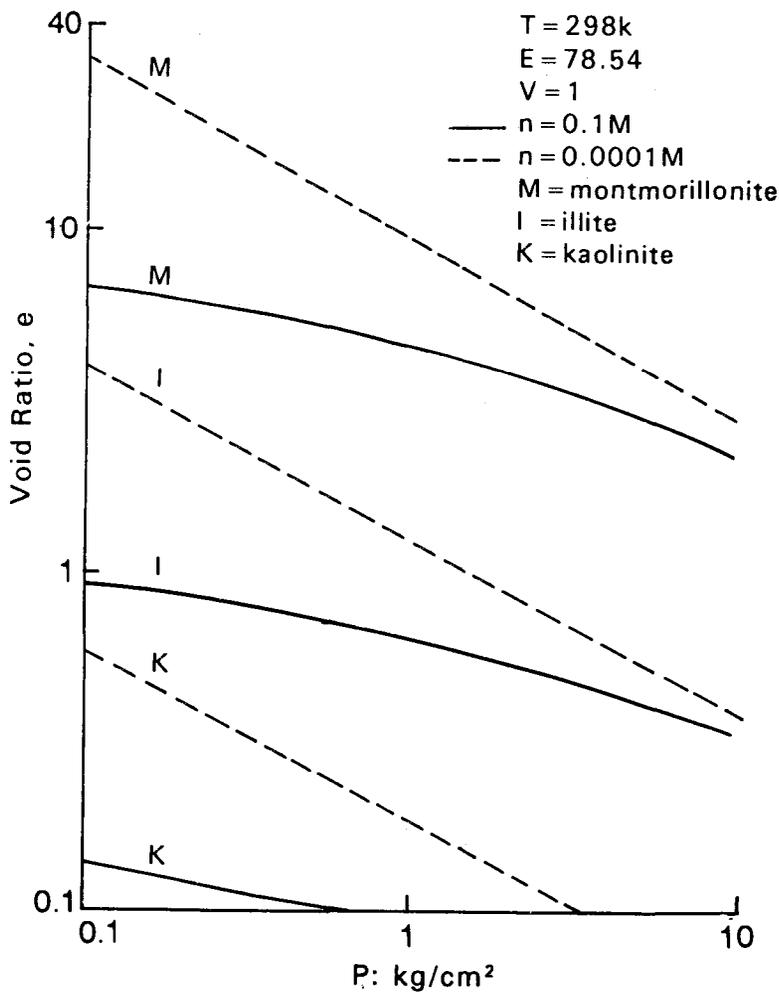

FIGURE 32 One-Dimentional Consolidation Curves for Black Cotton Soil with Replacement of Pore Fluids

490

FIGURE 33 (a) log d plotted against log p for Various Clay Types and Concentrations
(b) log e plotted against log p for Various Clay Types and Concentration

water is the pore fluid. Once the water is replaced by carbon tetrachloride, there is a reduction in the double layer repulsion, the effective stress increases and the sample undergoes a volume reduction. This phenomenon is the same as the one discussed earlier in the case of tests conducted on montmorillonite clay with various organic pore fluids. The results given in Fig. 32 are also consistent with this discussion. Because of the replacement of carbon tetrachloride by water, double layer repulsive forces increase, resulting in a net reduction in the effective

491

contact stress (eqn 17). This decrease in effective stress causes an increase in volume.

The results discussed above further confirm the existence of the two distinct basic mechanisms controlling the volume change behaviour of clays which are in qualitative conformance with the modified effective stress concept.

Although this discussion dealt with specific systems in the laboratory, because only in that case it is possible to determine significant parameters and understand the mechanisms involved, the inferences should be applicable to natural soils as well. The volume change behaviour of clays, in general, is influenced by both mechanisms. but one or the other usually dominates, depending on the soil type.

VOLUME CHANGE BEHAVIOUR AND DOUBLE LAYER THEORY

It has been shown above that the double layer repulison controls the volume change behaviour of montmorillonitic soils. The repulsive force due to interacting double layers has been discussed in the earlier section. Sridharan and Jayadeva (1982) examined in detail the various parameters affecting the distance between the platelets, and the pressure relationship as well as the void ratio-pressure relationship as brought out by Gouy-Chapman theory.

Factors Influencing the Compressibility Behaviour of Montmorillonite :

It has been brought in the previous section, that the electrical diffuse double layer does not have a prominent role in the compressibility behaviour of kaolinites. It should be realised that the diffuse double layer computations for compressibility are essentially valid for the swelling montmorillonite clays. The factors involved in the double layer theory are :

(a) soil properties represented by base exchange capacity and surface area :

(b) fluid properties,i.e. ion concentration, cation valency, v, dielectric constant ε and temperature T.

Hydrated ion size and anion adsorption, which influence the beehaviour of clays, are not considered in the Gouy-Chapman double layer theory. The attractive force between platelets is also neglected because it is negligible compared with the repulsive force in normal pressure ranges.

ION CONCENTRATION

As ion concentration increases, the swelling pressure decreases because the double layer is compressed The concentration factor is directly related to the left-hand side of equation (12) but its relationship with right-hand side is more complex. Fig.9 shows that the effect of concentration is large at low pressure and when the concentration is greater than 10^{-4} M. Also, when the concentration is less than 10^{-4} M its effect is negligible it all pressures. Fig 9a shows that the d-log p relationship is linear for high concentration ($n > 10^{-1}$ M). At low concentrations ($n > 0.0001$ M), the log d-log p relationship is linear (Fig 33 a) At all concentration levels, the d-log p and log d-log p relationship are essentially independent of clay type (Fig. 9a and Fig. 33a). Fig. 33a shows log e plotted against log p. The effect of clay type is evident. The curves for different clays are parallel.

For very low concentrations ($n < 10^{-4}$ M) the log d-log p relationship can be written as (Sridharan and Jayadeva, 1982).

$$Log_{10}d \ = \ f - g \ log_{10}p \qquad\qquad ..(23)$$

where f and g are constants. f is a function of the fluid properties n, E, v and T, and g is independent of fluid and clay properties and is equal to 0.5263.

For $n < 0.0001$ M, $v = 1$, $E = 80.36$ (for water at·20°C) and $T = 293°K$, the value of f is 0.65, where d is in nm and p is in kg/cm^2.

Differentiating equation (23) with respect to $log_{10}\ p$ gives

$$\frac{dd}{d \ log_{10} \ P} \ = \ - \ 1.21 \ d$$

The compression index C_c is the slope of the conventional e-log p curve, Therefore

$$C_c \ = \ \frac{de}{d \ log_{10} \ P} \ = - \frac{de}{dd} \ \frac{de}{d \ log_{10} \ P}$$

However, $e = G\gamma_w Sd$ (equation 5) and so

$$C_c \quad G\gamma_w S(-1.21d) = 1.21e \qquad \qquad ....(24)$$

Thus C_c is shown to be a function of void ratio, e and independent of clay type. Based on a compliation of consolidation test results, Rendon-Herrero (1980) also concludes that C_c is a function of e only.

When the concentration is greater than 10^{-1} M, the d-log p curve is linear (Fig. 9a), C_c is constant and is not a function of e. It can be shown (Sridharan and Jayadeva 1982) that

$$C_c = \frac{G\gamma_w S \times 10^{-6}}{0.4367 \sqrt{(n/\in T)v}} \qquad \qquad ....(25)$$

Thus for higher concentration, C_w is independent of the void ratio but is directly related to the surface area and hence the clay type.
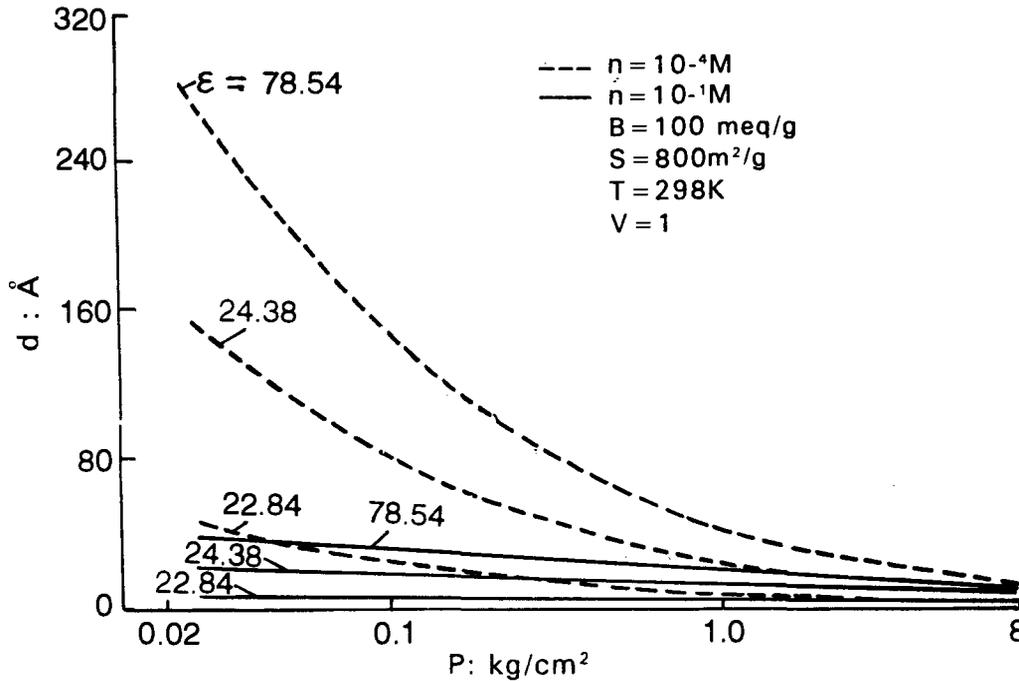
FIGURE 34 d plotted against log p for Various Di-electric Constants and Concentration
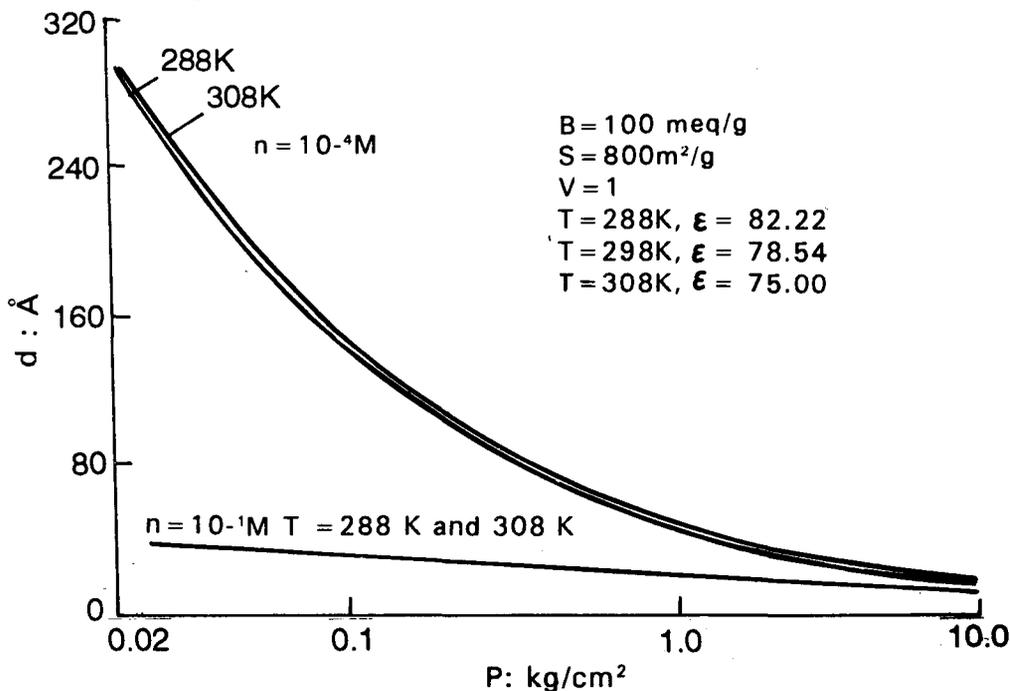
FIGURE 35 d plotted against log p for Various Temperatures and Concentrations

493

CLAY TYPE

It has been shown that the effect of clay type on the d-log p relationship is neglligible, whereas it is reflected in the e-log p relationship by the surface area. In the double layer theory, the clay type is considered in the term $(dy/dc)_{x=0}$. For $(dy/dc)_{x=0} > 250$, its influence on the u-kd and p-d relationships is negligible. Although the clay is represented in the double layer theory by two parameters-base exchange capacity and surface area, in the mathematical relationship it appears as B/S. For clays this ratio has a small range (Table 2). Figs 9 and 33 show the effect of clay type on the d-log p and e-log p relationships.

CATION VALENCY

The valency term is directly related to d in the d - p relationship (equation 12a). For the same pressure p, the half distance d decreases as the valency increases. At any pressure the equilibrium d values for two ion valencies are related by

$$d_1 v_1 = d_2 v_2$$

where d_1 and d_1 are the equilibrium half distances for ions of valencies v_1 and v_2.

It follows that, at any pressure, $e_1 v_1 = e_2 v_2$, where e_1 and e_2 are the void reatios for cations of valencies v_1 and v_2.

Thus, the void ratio of any natural soil could be decreased in direct proportion by leaching with a cation of higher valency and vice versa.

DIELECTRIC CONSTANT

In most cases water is the only fluid in soil engineering practice and so the dielectric constant is not important. However, its influence on the equilibrium half distance is clear from equations 12.

AT any pressure the equilibrium d or e for two fluids of dielctric constant ϵ_1 and ϵ_2 are relaterd by

$$d_1 / \sqrt{\epsilon_1} = d_2 / \sqrt{\epsilon_2}$$

Thus, the void ratio or d-log p relationship can be obtained for any fluid if the relationship for any other fluid is known.

Fig. 35 shows the theoretical relationship of d plotted against log p for three fluids (water, ethanol and carbon tetrachloride) of dieletric constants of 78.54, 24.3 and 2.284 for cation concentrations of 10^{-1} M and 10^{-4} M. These relationships are essentially independent of clay type.

TEMPERATURE

The temperature term occurs on both sides of equation (12). Also, the dielectric constant changes with temperture. For water, Fig. 35 shows that the effect of temperature on the d-log p relationship for the concentration range 10^{-1} M to 10^{-4} M is negligible and, as the temperature increase the equilibrium distance at any pressure increases by ±1-2% for a change in temperature of ±15°C. Conventional consolidation tests at room temperature (20°C) and at 75° C showed an increase of 0.04 mm in the height of the kaolinite clay sample (an increase in the void ratio of 0.003) and 0.11 mm (an increase in the void ratio of 0.014) for a montmorillonite clay sample, at a constant pressure of 1 kg/cm².

Hence, for all practical purposes, the effect of temperature can be neglected when volume changes are involved. Mesri (1973), discussing the temperature effect on the coeffcient of secondary compression and on consolidation test results, also concludes that temperature has a neglilgible effect. Yong, Chang and Warkentin (1969) report that in the swelling pressure test on a clay soil, where complete saturation exist, an increase in temperature results in a slightly increased water content (an increase in the void ratio) for the same swelling pressure, which is consistent with the prediction using the double layer theory. Also, the increase in void ratio is about the same as reproducibility of the measurement.

EXPERIMENTAL RESULTS

Using Bolt's (1956) experimental results, Fig. 36 shows the vd-p relationship for montmorillonite and illite clays saturated with Na+ and Ca++. Equation 12 predicts a linear relationship. Considering the various

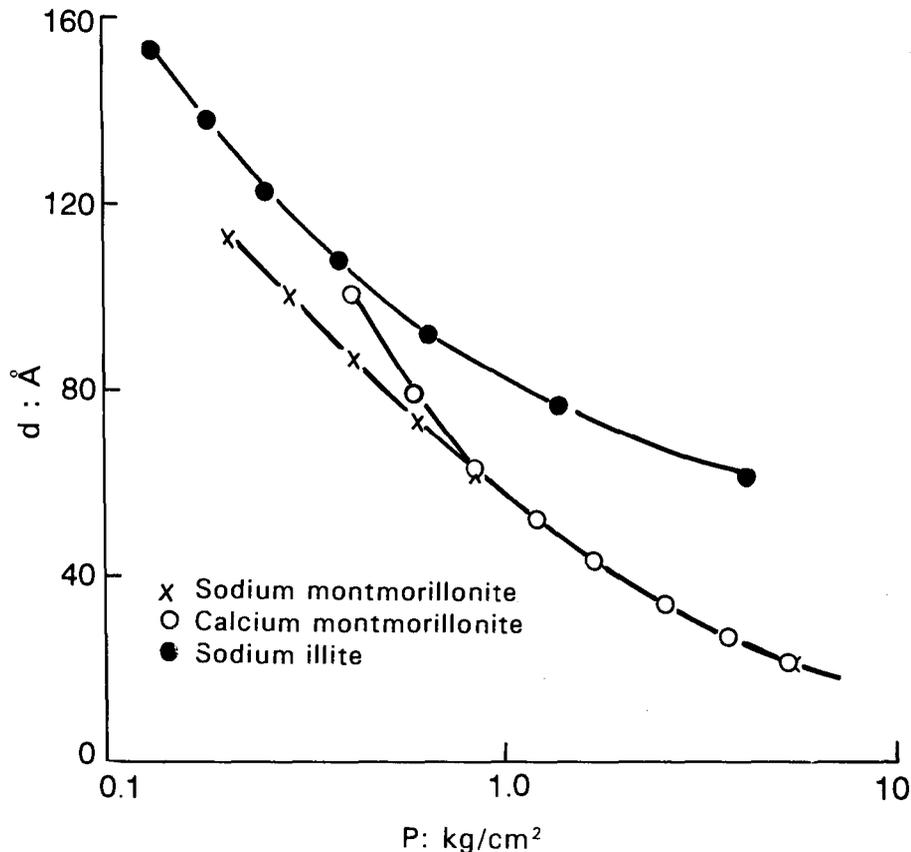**FIGURE 36 vd plotted against log p using Bolt's (1956) Data**

assumptions made in the development of the double layer theory, Fig. 36 agrees reasonably well with theory.

The $d\sqrt{n}$ log (p/n) relationship (Fig. 37) shows the influence of concentration using the results of Bolt (1956) and Mesri and Olson (1971). As with Table 3 and Fig.8, the theoretical curve is essentially independent of clay type and cation concentration for the ranges of values involved.

Although all the experimental results do not coincide with the theoretical line, it is seen that there is an almost parallel shift. Considering the variations in concentration (100 times) and clay type (illite and montmorillonite) the results obtained agree satisfactorily with the prediction of the double layer theory.

Low (1980) reports consolidation test results on 35 sodium-saturated montmorillonitic clays with surface areas varying from 288 m²/g to 800 m²/g. Fig 38 shows six of the typical void ratio-pressure curves obtained from his results lie in a narrow band as shown in Fig. 39, which also shows the theoretical line and the line for the average of the experimental points. Thus Low's results support the predictions of the double layer theory.

Fig. 40 shows the results of 32 tests (Sridharan, Rao and Sivapulliah, 1986) corresponding to 4 initial molding moisture contents, and relates swelling pressure determined experimentally at different dry densities of a black cotton soil, from Bennihalla (Dharwar Dit. Karnataka State). It is seen that the swelling pressure and dry density could be uniquely related.

Thus one could conclude that electrical double layer theory could be effectively used to qualitatively understand the compressibility and swelling behaviour of montmorillonitic soil.

HOMOIONIZED BENTONITES

In the previous section the compressibility behaviour has been studied with regard to Gouy-Chapman theory. This theory has been successfully applied to explain the compressibility of expansive clays (Bolt, 1956). In the following paragraphs the limitations of the Gouy-chapman theory as brought out by Sridharan, Rao and Murthy (1986b) using the results on homoionised bentonites.

Table 4b presents the results on liquid limit, plastic limit and the ionic radii of different cations used for the homioinisation. Standard oedometer tests were carried out by Sridharan, Rao and Murthy (1986b) on different homoionized clays.
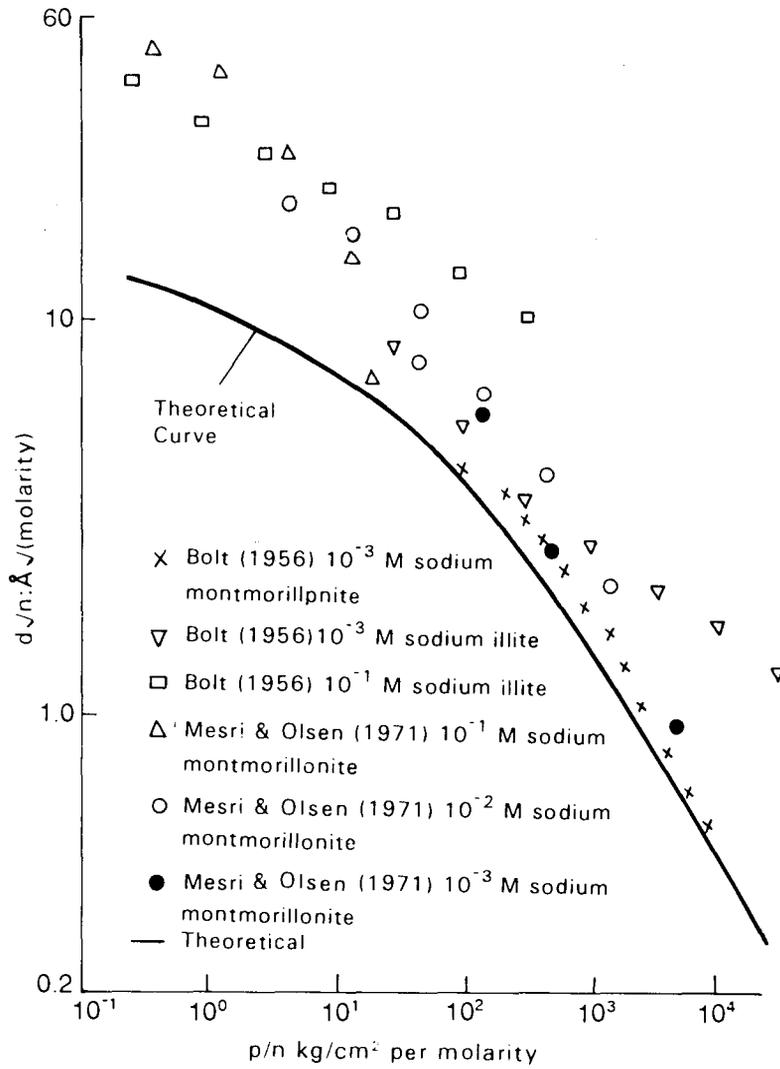
495

FIGURE 37 Comparison of Experimental and Theoretical Relationships between log d√n and log p/n
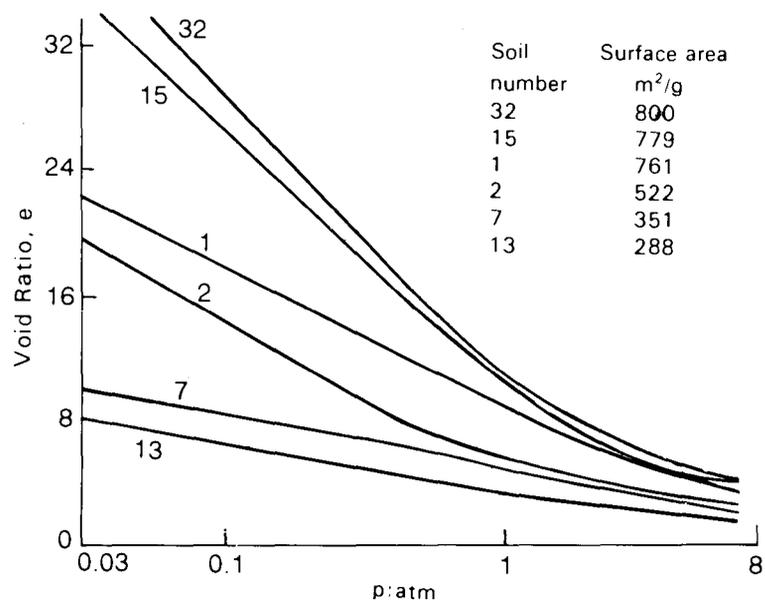

FIGURE 38 Void Ratio-Pressure Relations using Low's (1980) Data

Fig 41a presents typical compression versus time curves for the monovalent bentonites for a pressure increment of 1-2 kg/cm². Similar curves are obtained for other load ranges. The curves illustrate that potassium and ammonium bentonites consolidate faster than the sodium and lithium clays.

Fig. 41b presents the compression-time plots for the divalent and trivalent clays, for the same pressure increment. The consolidation rates of divalent bentonites are comparable with those of potassium and ammonium clays but higher than those of sodium and lithium bentonites. The trivalent bentonites in turn consolidate more quickly than the divalent clays and compress much less.
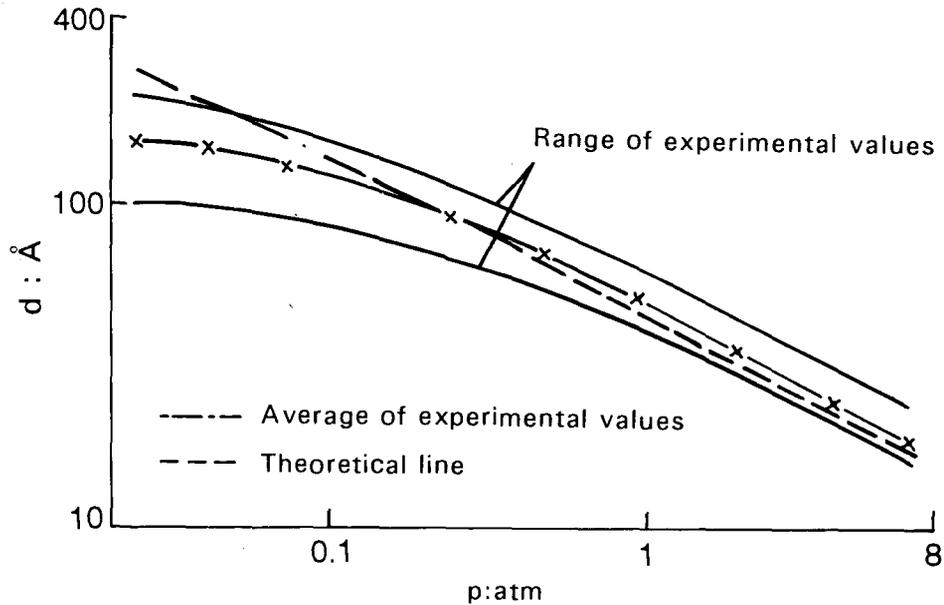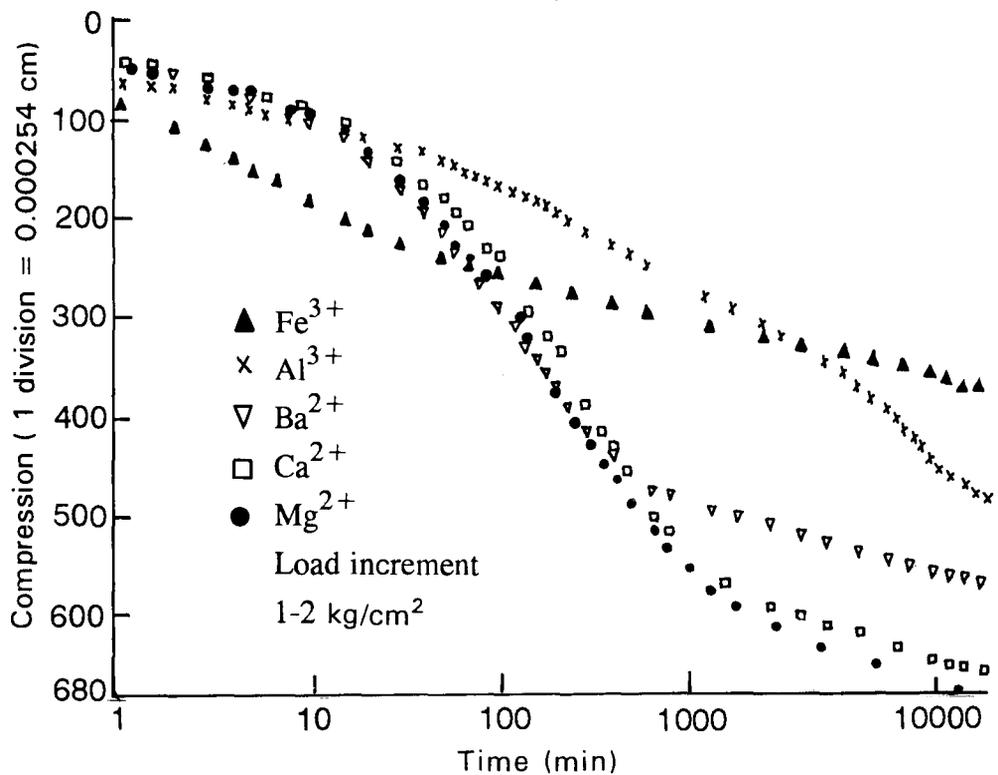

FIGURE 39 Comparison of Experimental Results with Theory of log d-logp Relation using Low's (1980) Data
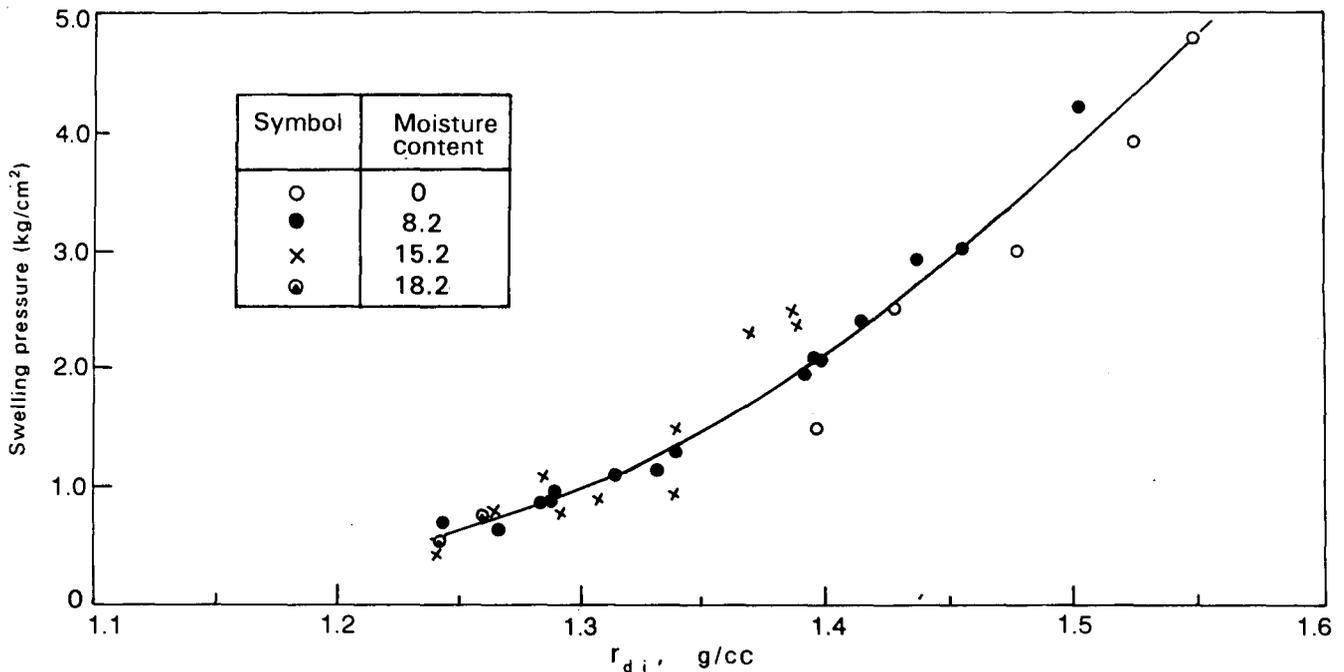
FIGURE 40 Variation of Swelling Pressure with Ydi

497
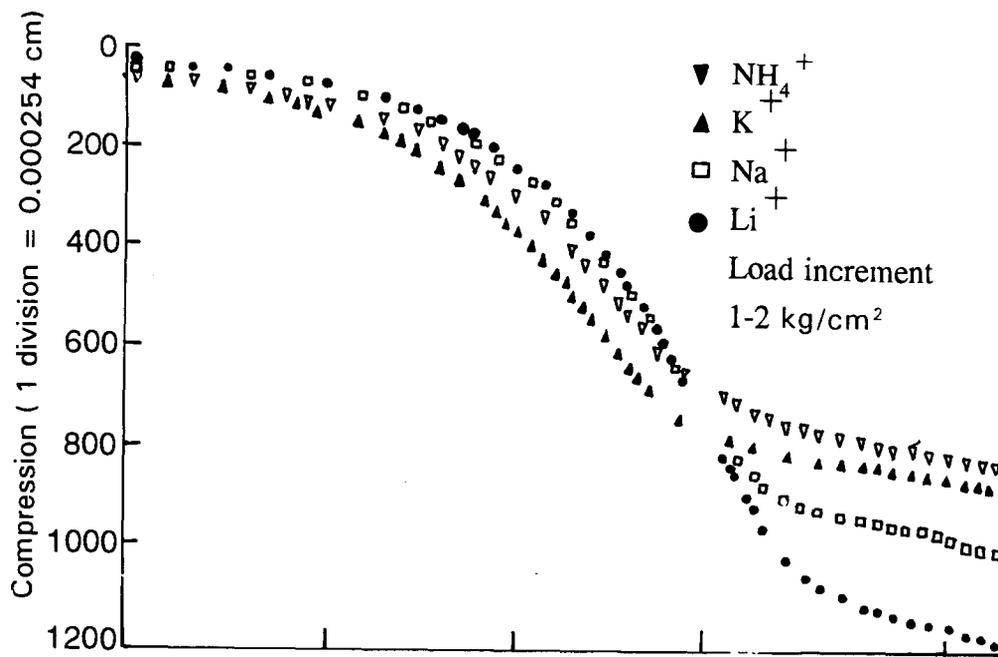
FIGURE 41 Time-Compression Curves (a) For Monovalent Bentonites (b) For Divalent and Trivalent Bentonites

Fig. 42a presents the e-log p relationships for the monovalent, divalent,and trivalent bentonites, the curves are convergent at higher stresses (at or above $2kg/cm^2$) and the effects of cationic size and valency are less perceptible. The initial moisture contents used for remoulding the specimens were less than their liquid limit values. A reduction in remoulding water content produces an effect that was similar to preconsolidation in the pressure-void ratio relationships and at high pressures both the magnitude and rate of change of void ratio is independent of soil characteristics and initial void ratio. The preconsolidation-like effect which arises because the remoulding water content is lower than the liquid limit is almost negligible for lithium and sodium bentonites; a
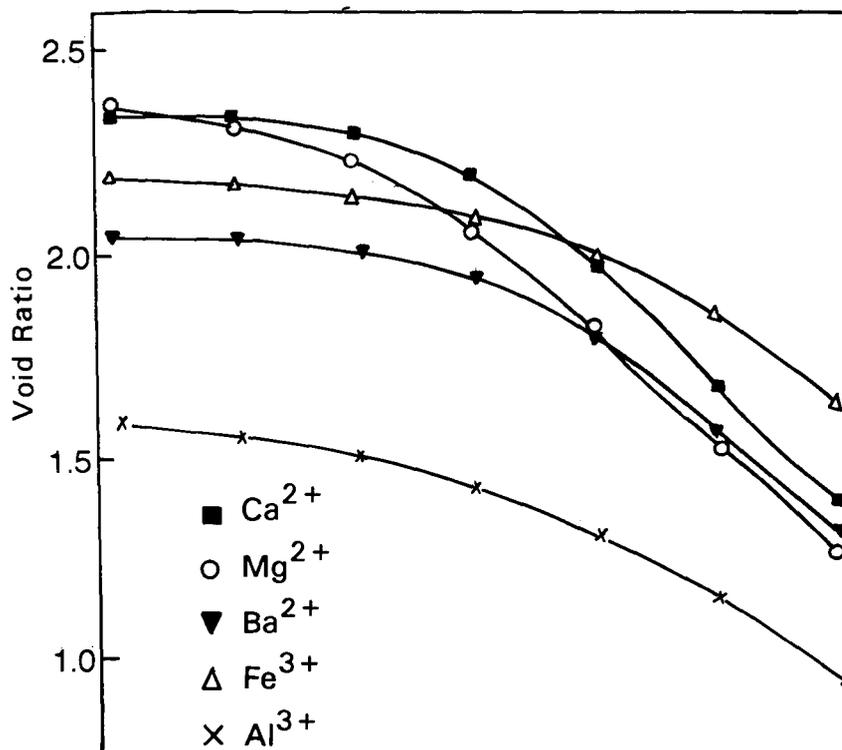
498

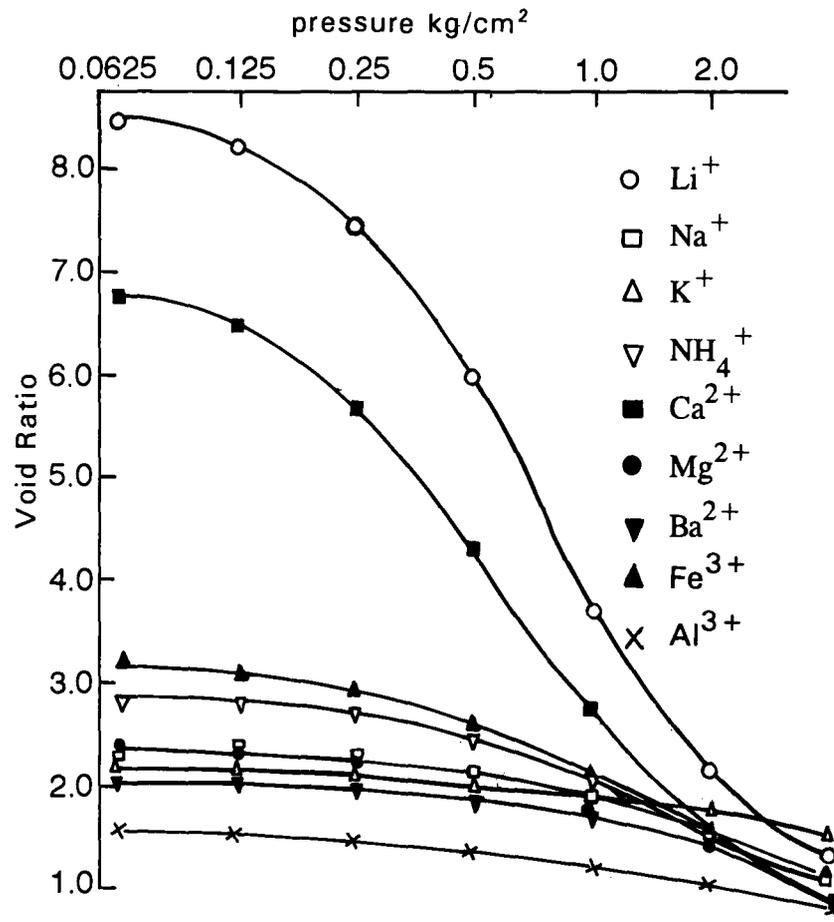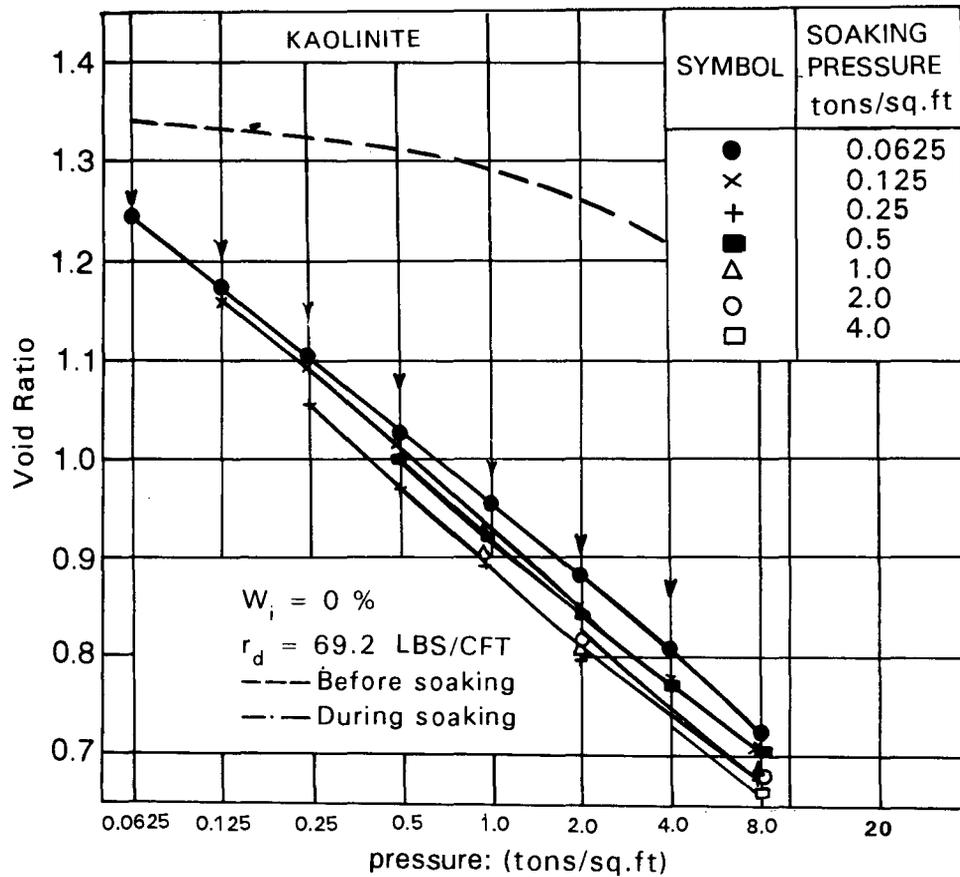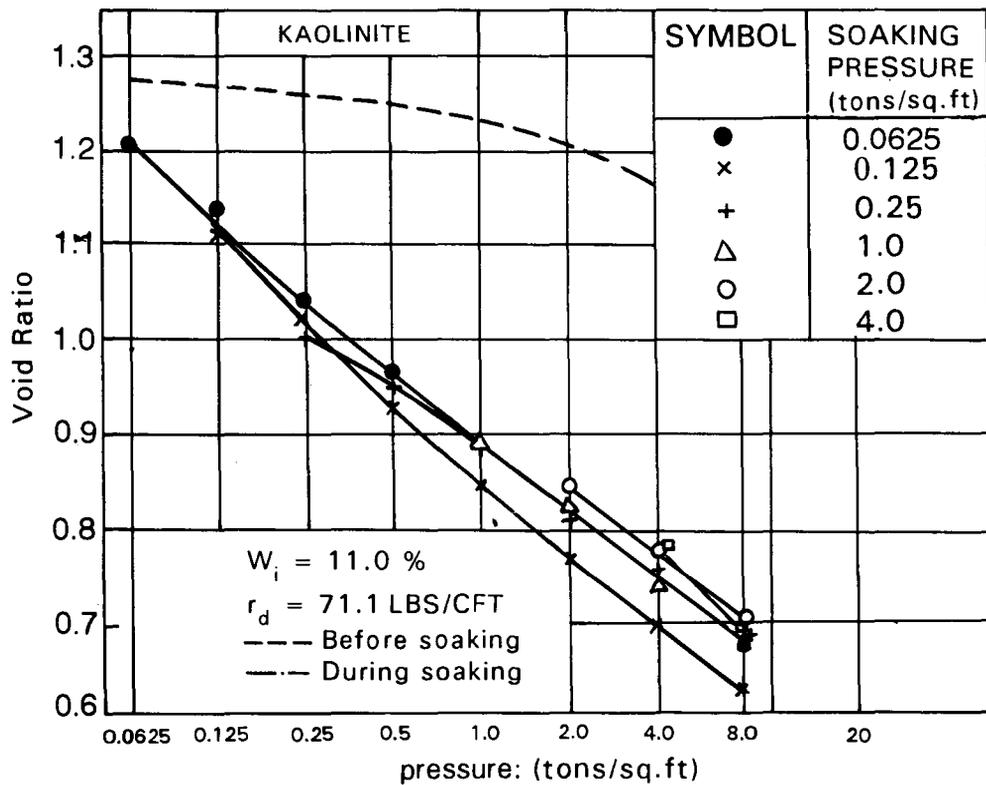**FIGURE 42** Void Ratio-Pressure Relationship (a) For Homoionized Bentonites (b) For Divalent and Trivalent Bentonites

499

FIGURE 43 Pressure-Void Ratio Curves for Kaolinite (wi = 0 % yd = 1.11 g/cc) Soaked at Various External Pressure (1 ton/sq. ft. = 1 kg/cm²)

FIGURE 44 Pressure-Void Ratio Curves for Kaolinite (wi = 11 % yd = 1.15 g/cc) Soaked at Various External Pressure (1 ton/sq. ft.≈ 1 kg/cm²)

500

value of approximately 0.15 kg/cm² is observed for the potassium and ammonium clays and a value of aproximately 0.4 Kg/cm² is observed for the divalent and trivalent bentonites. Remoulding with water contents lower than the liquid limit leads to a small preconsolidation-like effect. Hence in comparing the consolidation data values obtained at low preconsolidation pressures have been avoided.

The *e-log p* curves of ammonium and potassium bentonites are closer to those of the divalent clays than to the curves of lithium and sodium bentonites. The behaviour of the potassium and ammonium bentonite curves high-lights the importance of ionic size in influencing the compressibility of bentonites and brings out the shortcomings of Gouy-Chapman theory as it does not make provision for the ion size. The variations in void ratio with pressure for the divalent and trivalent bentonites are relatively small in comparision with the void ratios observed for lithium and sodium clays.

Figs 42b presents the void ratio-pressure relationsships for the divalent and trivalent bentonites. The order of compressibilities, computed from the decrease in void ratio, over the entire load range, is magnesium bentonite > calcium bentonite > barium bentonite > alumimium bentonite > iron bentonite. The absorption of divalent and trivalent cations by the bentonite surface if known to reduce the thickness of the diffuse double layer by one-half and one-third respectively (Mitchell, 1976). The changes in ionic size can bring out perceptible variations even in the thickness of these suppressed diffuse double layers.
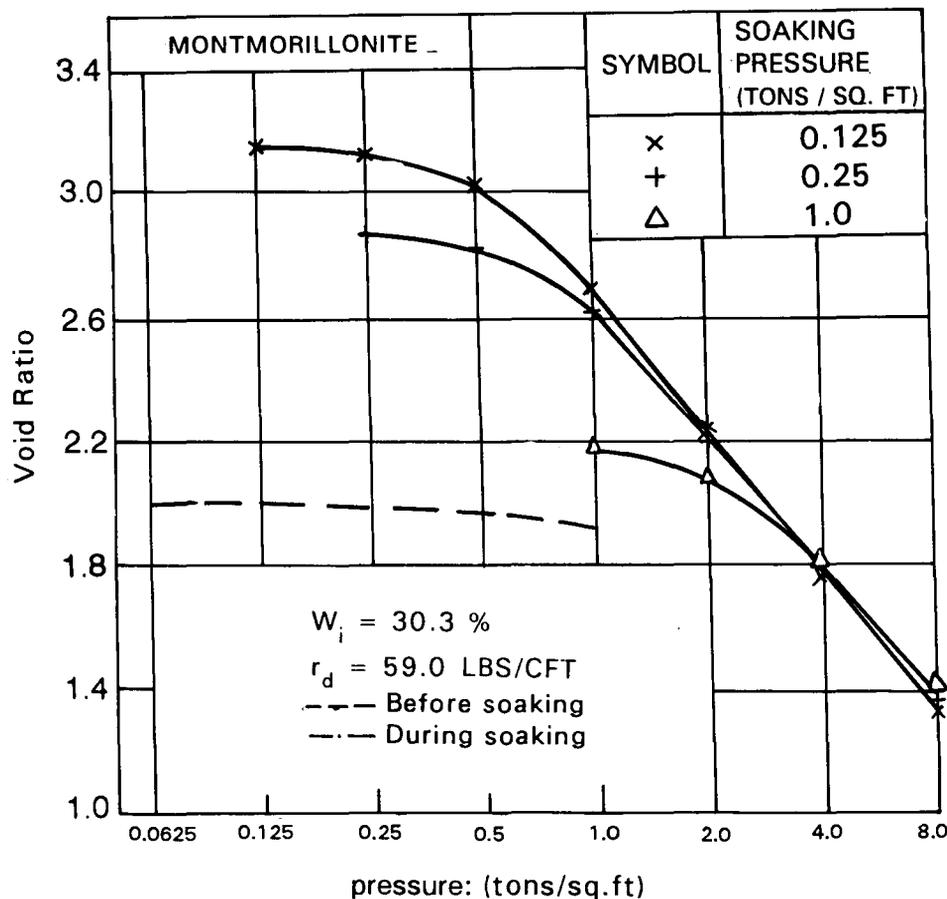
A larger compression is realized with aluminium bentonite than with iron bentonite, although iron bentonite possesses a higher liquid limit (Table 4b). The initial moisture contents of iron bentonite and aluminium bentonite are 82% and 70% respectively. If the difference in their initial moisture contents was responsible for their different compressibilities, then aluminium bentonite with a lower initial moisture content should have compressed less than iron bentonite. The experimental results indicate an opposite behaviour. The observed behaviour may be explained by considering the absorption mechanisms of aluminium and iron ions by the clay surface.

Hydrolysis of iron (III) and aluminium (III) ions in the presence of water has long been recognized (Sridharan, Rao and Murthy, 1986b). It has been shown that aluminium montmorillonite contains a mixture of unhydrolysed aluminium ions (Al^{3+}) and hydrolysis species $Al(OH)_{2.5}^{0.5+}$ Hydrolysis of ferric chloride solution at the experimental pH and concentration leads to the presence of $Fe_2(OH)_2^{4+}$ species, in addition to the unhydrolysed Fe^{3+} ions (Atkinson and Mc Bryde, 1961). Blackmore (1973) has proposed that the positively charged $Fe_2(OH)_2^{4+}$ species adhere strongly to the clay surface, undergo polymerization and subsequently form films of iron oxide on the clay surface. Aggregation of clay particles is then achieved by the process of cohesion between films or parts of films of iron oxied, creating bridges of oxide between particles. It is likely that the positively charged Al^{3+} ions and $Al(OH)_{2.5}^{0.5+}$ species satisfy the negative charge on the bentonite surface to a greater extent than do the $Fe_2(OH)_2^{4+}$ species which subsequently form the uncharged iron oxide flims. Further, aluminium hydroxide formed stronger electrostatic bonds with the montmorillonite surface than did iron hydroxide. As a consequence, a greater reduction in the diffuse double-layer thickness and a lower liquid limit value results for a aluminium bentonite than for iron bentonite. However, it is likely that the cementation bonds formed in iron bentonite resist compressive stresses more effectively than the electrostatic bonds in aluminium bentonite leading to a lower compression for iron bentonite.

The variations in the initial remoulding water contents of the different homoionized specimens result in different origins for their *e-log p* curves. To obtain compressibility curves starting from the same locus, the total compression undergone by the homoionized specimen at the end of each load increment (δ), divided by the height of the soil solids (H_s) expressed as a percentage, has been studied and it has been found out that the compression of potassium and ammonium bentonites are approximately three times lower than those for the lithium and sodium clays. Saturation of bentonite by the divalent cation reduces its compressibility by about 5-7 times with respect to that of lithium and sodium clays. The trivalent bentonite undergoes less compression than the divalent clays. Further discussion on C_c, C_v of homoionized clays can be found in Sridharan, Rao and Murthy (1986b). The above discussion brings out the effectiveness of ionic size in influencing the compressibility of swelling clays and the shortcomings of the Gouy-Chapman theory as it does not account for the ionic size.

PARTLY SATURATED SOILS

When fine grained soils are involved for building earth structures like dams and embankments, they are in partly saturated state. In many parts of the world, particularly in India, fine grained soils are encountered in a partly saturated state to a considerable depth. Since partly saturated soils are influenced by atmospheric and hydrological conditions, apart from the usual boundary loading conditions, a prediction of its behaviour in the field

	MONTMORILLONITE			SYMBOL	SOAKING PRESSURE (TONS / SQ. FT)
				×	0.125
				+	0.25
				△	1.0

W_i = 30.3 %

r_d = 59.0 LBS/CFT

— — Before soaking

— · — During soaking

pressure: (tons/sq.ft)

FIGURE 45 Pressure-Void Ratio Curves for Montmorillonite (wi = 30.3% yd = 0.95 g/cc) Soaked at Various External Pressure (1 ton/sq. ft. $\approx$ 1 kg/cm²)

should be based on an overall consideration of the probable variations in environment during and after construction for e.g. exposure to water may cause swelling or breakdown of soil fabric. In view of the importance of this aspect, the volume change behaviour of partly saturated kaolinite and montmorillonitic clays will be examined upon soaking and the role of modified effective stress in controlling the behaviour.

Selected consolidation experimental results published by Sridharan, Rao and Pandian, 1973, (Table 14) will be discussed in the following paragraphs.

While presenting the void ratio-pressure relationships (Figs. 43 to 46) the following notation has been adopted. The e-p relation before soaking is represented by a single dashed curve. During soaking the change in void ratio is shown by a dash and dot line.

The compression curves for the soaked soils are plotted in terms of conventional effective stress, whereas, those for partly saturated condition are in terms of applied stress. In a partly saturated soil, under specific external loading, the effective stress component due to pressure deficiency and the electrical attractive and repulsive forces (Eqn. 17) are not known quantitatively.

Fig.43 shows the pressure-void ratio relationships for kaolinite initially in a dry condition compacted to a dry density of 1.11 g/cc (69.2 lbs/cft) and soaked at different external pressures with distilled water. Fig. 44 shows similar curves for kaolinite compacted with a water content of 11.0% and a dry density of 1.15 g/cc (71.1 lbs/cft.). Similar results were obtained for higher densities also. From these figures, it is clear that additional compression occurred at all soaking pressures. Irrespective of soaking pressures, it is observed that all the compression curves join almost into a single curve after soaking. The differences if any could be attributed to experimental errors. There is no definite trend in these differences indicating that the effect of initial fabric difference due to the different soaking pressures adopted is not significant; in other words the effect of stress path seems to be negligible on the e-p curve.

Fig. 45 and 46 report results on montmorillonite. Fig. 45, presents for initial water content of 30.2% and a dry density of 0.946g/cc (59.0 lbs/cft). Similar results were obtained for other densities and water contents.
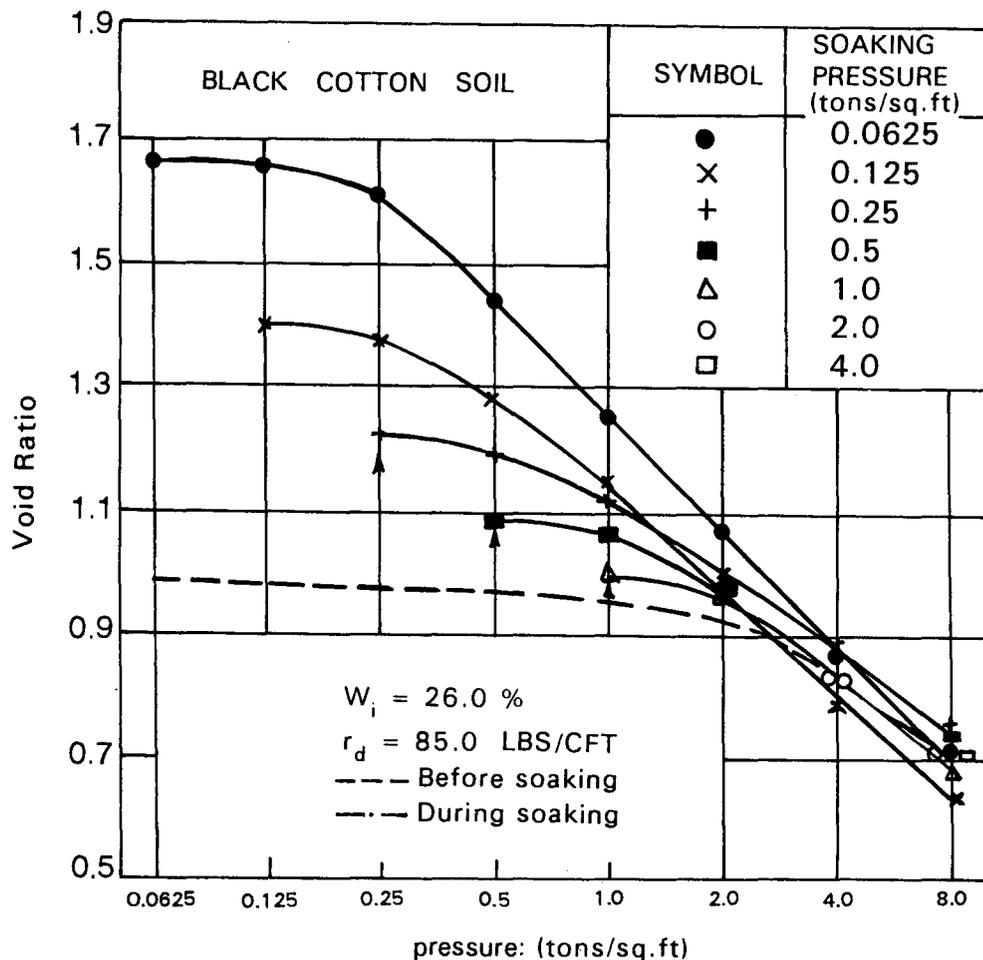
FIGURE 46 Pressure-Void Ratio Curves for Black Cotton Soil (wi =36.0% yd = 1.36 g/cc) Soaked at Various External Pressure (1 ton/sq. ft. $\approx$ 1 kg/cm²)

Unlike kaolinite, in all these cases, even up to externally applied pressures in the range of 1-2 kg/cm² soaking caused swelling. Soaking at higher external pressures caused compression. Similar results are obtained for black cotton soil, which are shown in Fig. 46. As one could expect, for both montmorillonite and black cotton soil, the amount of swelling is more when the externally applied pressure at which soaking was done is less. This is quite consistent for all the cases reported in Figs. 45 and 46. The e-p relationships after soaking are distinctly different, although they tend to merge at pressures of 4-8 tons/sq. ft ($\approx$ 4-8 kg/cm²). Should the system behave ideally, irrespective of soaking pressure, the e-p relationship after attainment of equilibrium at the end of swelling, should be unique. Since soil is not an ideal material, and since hysteresis effects are present in soils, the stress path effect is distinctly seen. By an analogy with saturated clays it may be expected that stress path dependence will be introduced by swelling, but that it may be lost under subsequent straining or loading.

It has been brought out earlier that mechanism I and II controls the volume change behaviour of saturated system depending upon the nature of soil present.

The volume change behaviour of clays, in general, is influenced by both the mechanisms, but one or the other usually dominates: Mechanism I, primarily controls the non-expanding lattice type of the clays like kaolinite, and Mechanism II, in the case of expanding lattice type of clays like montmorillonite. In the following. the volume change behaviour during soaking presented in Figs 43 to 46 has been examined from the point of view of the modified effective stress concept, in the light of two basic mechanisms presented.

Referring to the modified effective stress equation as given in equation (17), a process of soaking the soil will bring about change in $\overline{u}_w$ R and A. Since the system is open to atmosphere and since atmospheric pressure is taken as datum, there will be no change in the value of u_a. The effective pore water pressure $\overline{u}_w$ which has contributed a positive component to $\overline{c}$ will become zero and hence $\overline{c}$ will decrease. Due to soaking, the effective dielectric constant of the pore medium increases, hence R increases, A decreases (Sridharan, 1968) and hence the net effect is that $\overline{c}$ decreases. Thus, due to soaking, the value of $\overline{c}$ which is the effective contact stess controlling the shearing resistance at interparticle level decreases, thereby there is a decrease in shearing resistance at the

503

particle contacts. According to Mechanism I, this decrease in shearing resistance brings about compression. The results reported in Figs 43 and 44 show that kaolinite with initial water contents of 0% and 11.0% underwent additional compression or collapsed when they were soaked under constant external pressure. The reduction in shearing resistance should be quite significant considering the compression results reported in Figs 43 and 44. The soil could not even resist an external pressure of 0.0625 tons/sq. ft.($\approx$ 0.0625 kg/cm^2), after soaking had taken place. As one could expect, the change in void ratio (or amount of collapse) is more when soaking pressure is more. The soil undergoes compression to develop enough shearing resistance under the soaked environment so as to resist the shearing stresses developed at the particle level due to the externally applied load. Thus the results of kaolinite are consistent with the modified effective stress concept together with Mechanism I.

From the analysis presented above, it can be concluded that for non-expanding lattice type of clays, shearing resistance governed by the effective stress Eqn. 17 at the interparticle level controls the volume change behaviour during soaking. This is in conformity with the earlier conclusion arrived at on volume change behaviour of non-expanding lattice type of clays using various organic fluids. With a co-ordinated look of the modified effective stress concept together with the appropriate mechanism (viz., Mechanism I) it is possible to explain the volume change behaviour reported by Jennings and Burland (1962) for silts, silty sand and silty clays (which are basically of non-expanding type) also.

The results reported in Fig. 45 for montmorillonite and in Fig. 46 for black cotton soil (an essentially montmorillonitic type of soil) show swelling for externally applied pressures up to about 2 tones/sq. ft. ($\approx$ 2 kg/cm^2). These results are interesting and show a distinctly opposite behaviour when compared to kaolinite at pressures up to about 1 ton sq.ft. ($\approx$ 1 kg/cm^2). For montmorillonite although the initial moisture content is about 30% and dry densities around 0.91-0.95 g/cc, significant swelling occurred even at soaking pressures up to 1 ton/sq.ft. In spite of a decrease in shearing resistance due to soaking (eqn. 17) the equilibrium void ratio after soaking increases for externally applied pressure up to 1-2 tons/sq.ft. ($\approx$ 2 kg/cm^2), unlike for kaolinite. If shearing resistance at interparticle level were to be the criterion for controlling the volume change behaviour as it has been with kaolinite, then compression should have taken place. As the soil here is of the expanding lattice type, repulsive forces predominate and these forces resist external pressures up to about 1-2 tons/sq.ft. ($\approx$ 1-2 kg/cm^2), at relatively high void ratios. Increase in void ratio or swelling occurs, until the net repulsive force decreases to a level where equilibrium is established between this repulsive force and the externally applied load.

When once the externally applied pressure becomes large, compression takes place due to soaking in order that this pressure may reach equilibrium with the internally mobilized repulsive forces. Hence, the experimental observations as regards montmorillonite (Fig. 45) are consistent with Mechanism II.

Applying the modified effective stress concept as given in equation (17) an increase in the repulsive force R, due to soaking, brings down the effective contact stress. It then follows (from a concept similar to the conventional approach) that a decrease in effective stress (as defined in eqn. 17) results in an increase in volume. The analysis made for kaolinite i.e., a decrease in effective stress causing a decrease in volume may look paradoxical. But it should be emphasized that in the case of kaolinite, the decrease in volume is due to a decrease in shearing resistance at interparticle level. Thus it is emphasized here that the effective stress concept should be applied only in conjunction with the physical mechanisms involved.

The results presented in Fig. 46 for black cotton soil, which is an essentially montomorillonitic clay, are consistent with the above discussions.

The results discussed above bring out the distinct basic mechanisms controlling the volume change behaviour of partly saturated clays during soaking, as they do for fully saturated clays. They are in qualitative conformance with the modified effective stress concept in conjunction with the mechanisms involved.

Although this investigation deals with specified systems in the laboratory, because only in that case it is possible to bring out significant parameters and understand the mechanisms involved, the inferences should be applicable to soils under natural conditions as well. The volume change behaviour of clays, in general, is influenced by both mechanisms; but one or the other usually dominates depending on the soil type.

DESICCATED SOILS

Arid and semi-arid regions cover large parts of the earth's land surface. Soils in these regions are described as desiccated soils. These soils are subjected to periodic wetting and drying as a result of climatic changes. Review of literature on the action of climatic changes and also of water on soils (Sridharan and Allam 1982 and Allam and Sridharan, 1981; Allam and Sridharan, 1984a) indicates that repeated wetting and drying

can result in the aggregation of soil particles and cementation by compounds of Ca, Mg, Al and Fe.

It has been brought out earlier that for pure clays, in the absence of cementing bonds and stress history, two mechanisms govern the volume change behaviour depending on whether the clay consists of expanding or nonexpanding lattice-structure clay minerals. As stated earlier, desiccation bonds generate larger particles or crumbs out of smaller ones due to the repeated wetting and drying process which induces these bonds. Further, the bonding between crumbs or particles imparts additional intrinsic effective stress and thus greater shearing resistance at particle contacts. The intrinsic effective stress is defined as one which is due to net attractive and repulsive forces and not borne out of external loading (Eqn. 18). In the case of desiccated soils where clay fractions consist chiefly of nonexpanding lattice-structured clay minerals, this additional shearing resistance should result in reduced compressibility as volume change in such cases will be governed by Mechanism I (wherein the shearing resistance at particle contacts govern volume change). The existence of crumbs may also result in increased permeability as the pores between the crumbs should be larger than those between the particles (without crumbs) at the same void ratio.

In the case of desiccated soil where expanding lattice-structured clay minerals dominate in their clay fraction, the formation of crumbs and the increase in intrinsic effective stress between particles and crumbs should affect their volume change behaviour and prevent the physico-chemical mechanism (Mechanism II) from independently governing their volume change behaviour. Depending on the extent of the additional intrinsic effective stress due to desiccation, the undisturbed soil may be anticipated to behave either as a swelling soil possessing lesser amount of swelling clays, or even as a non-swelling soil. It may possess a greater resistance to deformation. If the crumbs are large and prevalent, the reduced specific surface areas should result in such low mobilization of double layer repulsive forces between crumbs that the shearing resistance at the crumb contacts may control the volume change behaviour of the soil. Thus, for such soils, compression behaviour may be governed either by Mechanism II when the bonds are few, or by Mechanisms I and II successively as the load increases if bonding is large and the resulting crumbs are prevalent.

The rebound behaviour will depend on the amount of crumbs present at the end of the loading stage. If a sizeable amount is present, then the rebounds will be due not only to elastic rebound and straightening of crumbs/bent particles, but also due to formation of diffuse double layers between freed particles. That is, both Mechanisms I and II will control the rebound behaviour. If a few crumbs remain at the end of the loading stage, swelling of diffuse double layers between particles freed from the crumbs will be chiefly responsible for the rebound behaviour and the amount of rebound will be almost as large as that for the soil when desiccation bonds (and thus crumbs also) are absent. Further, the presence of crumbs should increase the permeability of the undisturbed soil. The preceeding theoretical considerations are analysed with the experimental results reported in the following sections.

This study is based on undisturbed soils found in and around Bangalore, India. Soil formation is due to the physical weakening of the parent rock caused by temperature changes which initiated the process of weathering and soil formation, accompained and supplemented by certain chemical transformations. Climate also played an important role in the weathering of rock. The soil generally met with is residual in origin and desiccated. The chemical composition varies between the following limits (percentage by dry weight): (1) Silica (SiO_2) 60-74; (2) Alumina $(Al_2 O_3)$ 14-20; (3) Ferric Oxide $(Fe_2 O_3)$ 0.5-8.0; (4) Pottasium Oxide (K_2O) 1.0-5.0; (5) Calcium Oxide (CaO) 0.5-4.5; and (6) Magnesium Oxide (MgO) 0.6-1.5.

The soils thus can be described as non-lateritic as the silica to alumina ratio exceeds 3.0. The soils studied here were sampled at depths ranging between 1 and 3 m below ground level. Their index properties vary as-liquid limit 32 to 58.5%, plastic limit, 18 to 33 %, and specific gravity, 2.59 to 2.73. They are composed of gravels, 0 to 16 %, sands, 33 to 78 %, and silts can clays, 18 to 52 %. Table 15 contians information regarding their index properties, grain size composition, and percentage free-swell exhibited by the soils studied. It also lists the various minerals detected by an X-ray diffraction analysis of their fine fractions. Conventional one-dimensional consolidation tests were conducted on the undisturbed samples saturated under field condition using a load increament ratio of unity. On completion of the consolidation tests on the undisturbed specimens, remoulded specimens were prepared from the disturbed soil. Consolidation specimens were prepared by static compaction taking care to obtain the void ratio and water content posessed initially by the undisturbed specimens. From the viod ratio-consolidation pressure curves it is seen that three distinctly different responses to consolidation are exhibited by desiccated soils when the volume change behaviour for their undisturbed and remolded states is compared. The soils have been classified into three groups and the characteristic volume change behaviour of each group is examined in the following sections.

TABLE 15

PROPERTIES OF SOIL USED

Test details	Group I Soils					Group
(1)	SM1 (2)	P5D4 (3)	P5D9 (4)	SM2 (5)	P2D4 (6)	P2D10 (7)
Specific gravity	2.60	2.70	2.70	2.60	2.71	2.70
Liquid Limit (%)	38	48	40	32.1	58.5	56
Plastic Limit (%)	18	31	25	18.7	32.5	27
Plasticity Index (%)	20	17	15	13.4	26	29
Gravel (%)	–	13	4	5	16	12
Sand (%)	70	47	78	60	49	42
Silt and clays (%)	30	40	18	35	35	46
Insitu dry density in (g cm^{-3})	1.52	1.54	1.45	1.42	1.52	1.50
Insitu moisture content (%)	26.9	27.4	31.5	31.4	28.6	29.3
Free Swell (%)	35	50	30	40	70	60
X-ray diffraction analysis	Kaolinite, Chlorite, feldspar, and quartz	Kaolinite, halloysite, calcite, chlorite, illite, hematite, feldspar and quartz	Kaolinite, halloysite, chlorite, illite, qaurtz, and feldspar	Kaolinite, illite, and quartz	Montmorillonite, illite, chlorite, feldspar, and quartz feldspar	montmorillonite, illite chlorite, halloysite, quartz and

TABLE 15 CONTD.

PROPERTIES OF SOIL USED

Test details	II Soils	Group III Soils						
	P7D4 (8)	P3D11 (9)	P7D9 (10)	NGI (11)	NG2 (12)	NG3 (13)	IT1 (14)	IT2 (15)
Specific gravity	2.72	2.73	2.71	2.6	2.62	2.59	2.63	2.59
Liquid Limit (%)	44.5	52	47	43	44.5	35.5	53	50.5
Plastic Limit (%)	28	33	30	25.8	25.4	22.5	32	31
Plasticity Index (%)	16.5	19	17	17.2	19.1	13	21	19.5
Gravel (%)	4	3	11	3	6	5	2	2.1
Sand (%)	47	65	61	49.5	33.5	48	49.9	46.2
Silt and clays (%)	49	32	28	47.5	60.5	47	48.1	51.7
Insitu dry density in (g cm^{-3})	1.64	1.52	1.64	1.64	1.53	1.61	1.54	1.46
Insitu moisture content (%)	23.7	28.7	24.4	23.0	27.0	22.9	26.1	29.0
Free Swell (%)	65	65	70	35	50	30	50	35
X-ray diffraction analysis	Montmorillonite, illite, calcite, cholrite, kaolinite, feldspar and quartz	Montmorillonite, illite, chlorite, feldspar quartz and kaolinite	Montmorillonite, illite, feldspar, and quartz	Montmorillonite, illite, calcite, and quartz	Montmorillonite, illite, hematite, and quartz	Montmorillonite, illite, chlorite, hematite, feldspar and quartz	Montmorillonite, illite, chlorite, and quartz	Montmorillonite, illite, chlorite feldspar and quartz
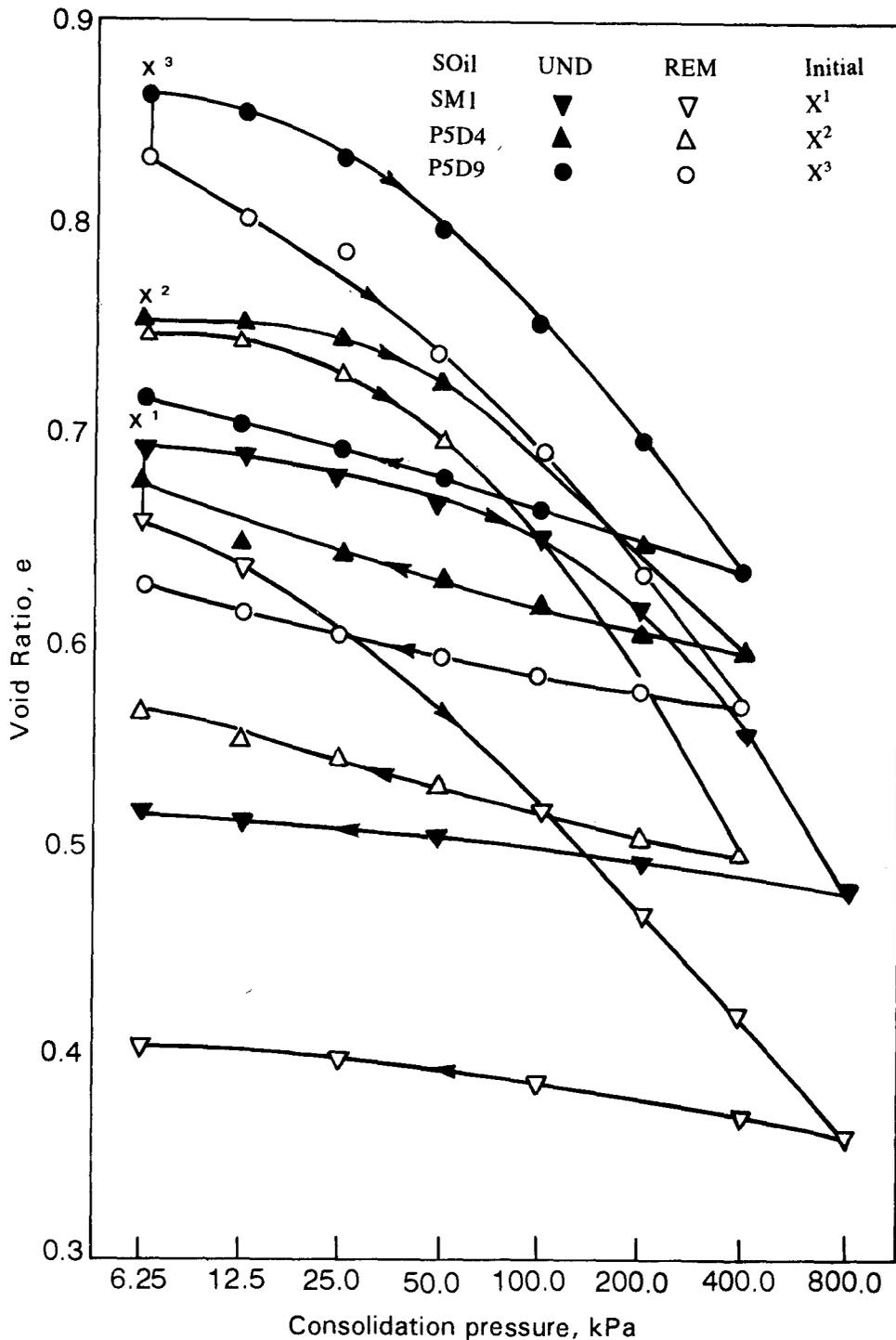
FIGURE 47 One-Dimensional Consolidation Curves for Soils SM1, P5D4 and P5D9 (98 kPa = 1 kg/cm²)

Group I soils: Soils SM1, P5D4, P5D9, and SM2 belong to this group. These soils have relatively low liquid limits (32 to 48 %) and plasticity index values (13.4 to 20%). The percentage of silts and clays present is also low (18 to 40 %). Their clay mineralogical investigation shows that they are composed of mainly non-expanding lattice structured clay minerals. Free swell test data (table 15) indicates that these soils are of the relatively non-swelling type. Figs 47 and 48 show their consolidation behaviour. The soils in both their undisturbed and remoulded conditions compress on soaking at the initial pressure of 6.25 kPa, greater compressions occuring for soils in the remoulded condition. The total compression at the end of the loading cycle of the test is also more for the remoulded condition. The rebound curves are flat and almost parallel to each other for soils in their undisturbed and remoulded states.Flat rebound curves, or in other words, small expansions on unloading, are an indication that swelling is due to elastic rebound and straightening of bent particles indicating that the soil is of a non-swelling
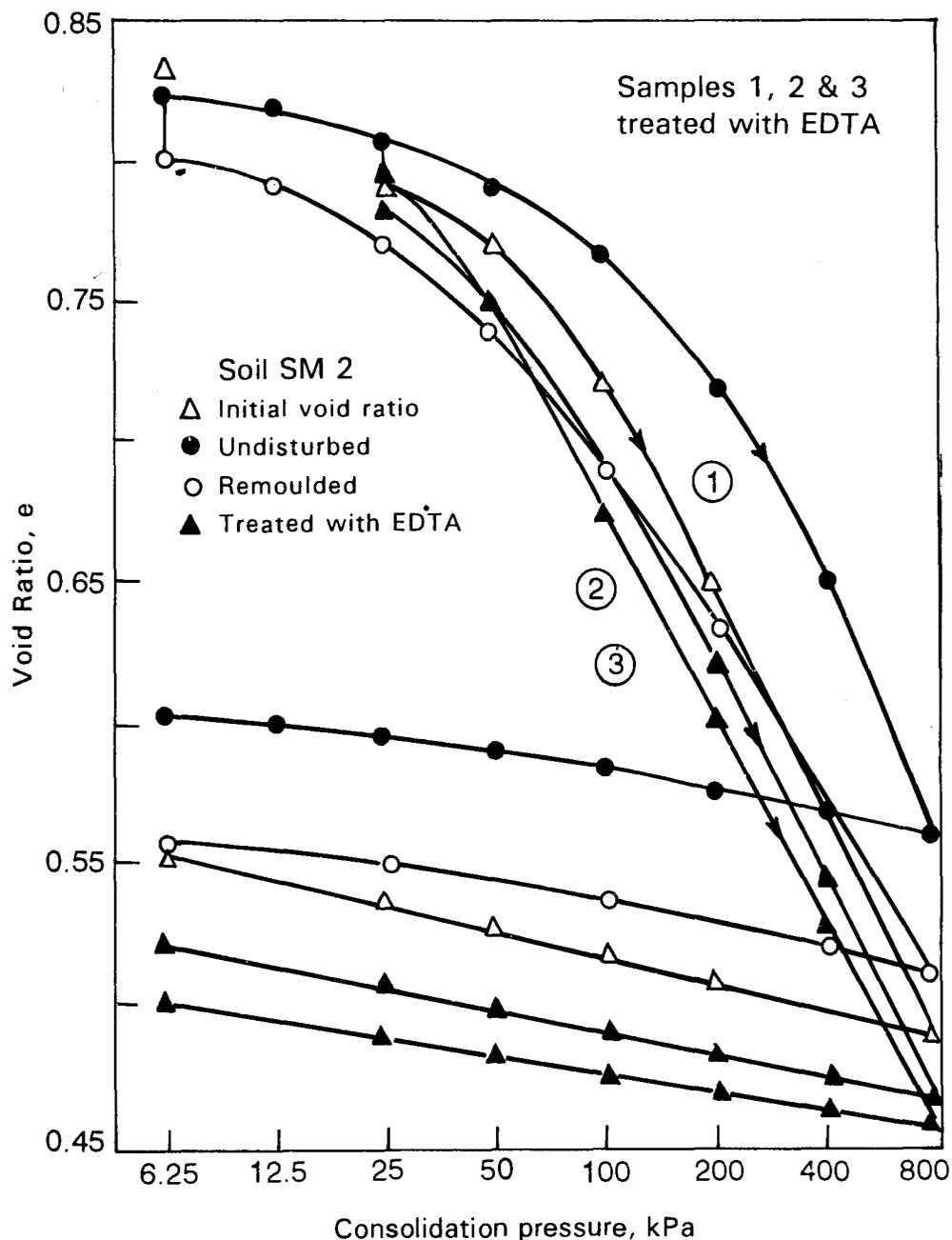
FIGURE 48 One-Dimensional Consolidation Curves for Soil SM2 (98 kPa = 1 kg/cm ^{2})

type and thus its volume change behaviour is governed by Mechanism I wherein the shearing resistance at particle contacts governs the compression behaviour (Śridharan and Rao, 1973).

For undisturbed soil, the presence of desiccation bonds between soil particles augments the intrinsic effective stress present between them and thus the shearing resistance at particle contacts. The larger shearing resistance present at the particulate level enables the undisturbed soil to support the external load at a higher void ratio compared with the remoulded soil where this intrinsic effective stress is not present as the desiccation bonds are disrupted by remoulding. Further, when the soil is soaked at the initial pressure of 0.0625 kg/cm^2 (6.25kPa), the net attractive forces between the soil particles decrease and thus compressions take place due to the reduction in shearing resistance at particle contacts. That desiccation bonds are less affected by soaking is evidenced by the smaller compressions taking place for undisturbed soils.

The parallelness of the rebound curves between undisturbed and remoulded soils indicates that either all the disiccation bonds are disrupted during the loading stage of the test, or the crumbs or grains formed by bonding expanded on unloading to the same extent as the more discrete soils particles obtained on remoulding.

The preceeding analysis shows that the behaviour of Group I soils is as analysed in the earlier sections.

Group II soils : Soils P2D4, P2D10, P7D4, and P3D11 belong to this group. These soils have relatively

508
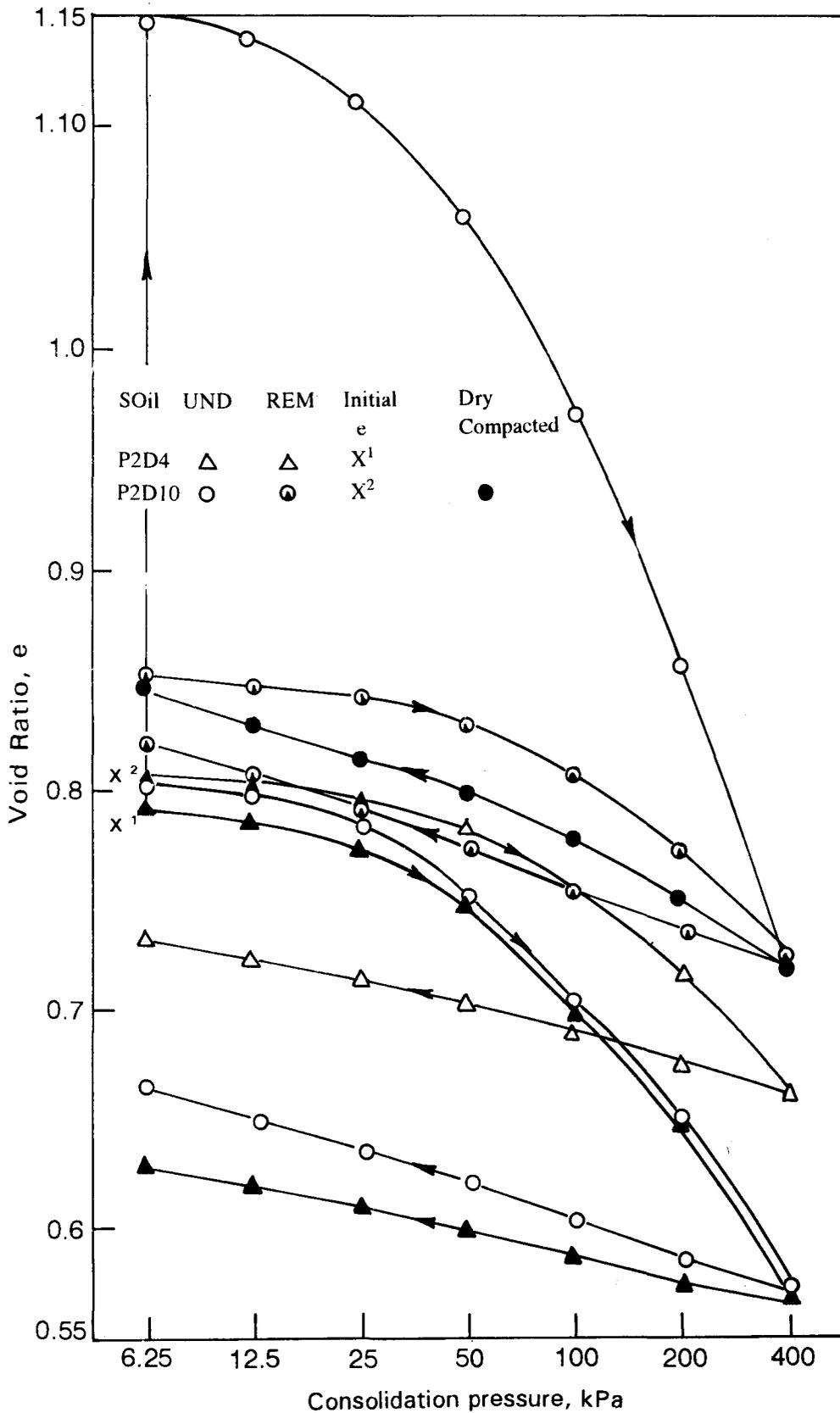
FIGURE 49 One-Dimensional Consolidation Curves for Soils P2D4 and P2D10
(98 kPa = 1 kg/cm ^{2})

large liquid limits (44.5 to 58.5%) and plasticity index values (16.5 to 29 %). The percentage of silts and clays is also relatively high (32 to 49 %). The clay mineralogical investigation shows that they are mainly composed of expanding lattice-structured clay minerals. The free swell test data (Table 15) also confirms their swelling nature. Figs 49 and 50 show that these soils in both their undisturbed and remoulded states swell on soaking at
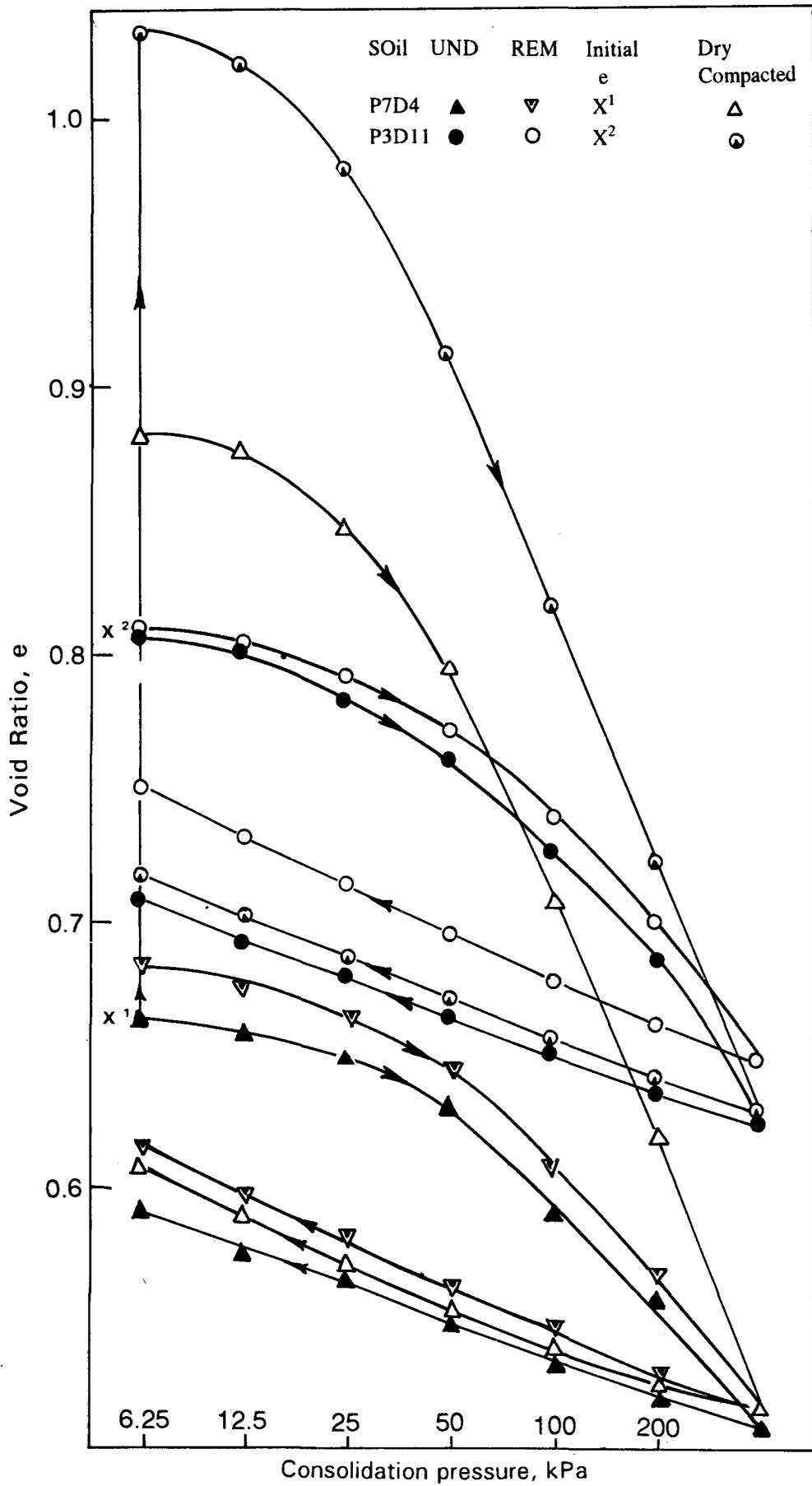
FIGURE 50 One-Dimensional Consolidation Curves for Soils P7D4 and P3D10 (98 kPa = 1 kg/cm²)
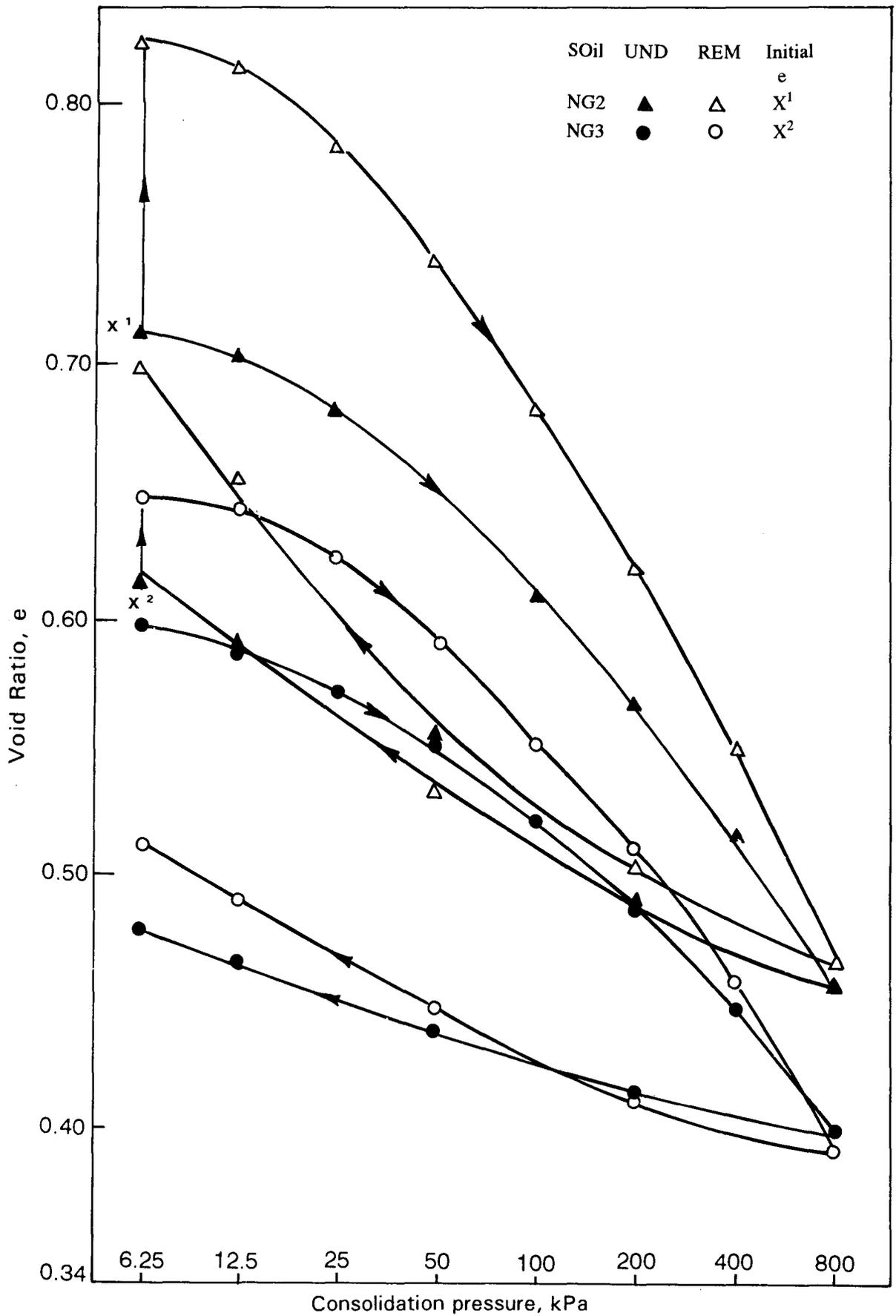
510

FIGURE 51 One-Dimensional Consolidation Curves for Soils NG2 and NG3 (98 kPa = 1 kg/cm ^{2})

the initial pressure of 6.25 kPa, the magnitude of swelling being more for the remoulded state. The swelling is noticeable, although slight, for the undisturbed state. Apart from remoulding these soils at the field water contents, consolidation tests were also carried out on dry samples of the soils P2D10, P7D4, and P3D11 (Figs. 49 & 50) compacted to the void ratios existing in the field. In these tests, the swelling observed at the initial pressure of 6.25 kPa, when the samples were allowed to imbibe water, greatly exceeded that for the remoulded samples. It is well known that as the initial water content decreases, swelling increases. The rebound curves for the soils in their undisturbed, remoulded, and dry compacted states are observed to be reasonably parallel.

The desiccation bonds present in the undisturbed soils cement soils parcicles together to form larger particles or crumbs resulting in reduced effective specific surface areas. On soaking at the initial pressure of 6.25 kPa, the crumbs are able to mobilize diffuse double layers and consequently, repulsive forces only on their surfaces, since the bonds present prevent the component particles from mobilizing diffuse double layers between themselves. While the increased repulsion between the crumbs causes the soils to swell, the swelling is small compared with that undergone by the remoulded soils with its relatively larger effective specific surface and thus repulsive force.

During the loading cycle of the consolidation test, the remoulded soils is able to mobilize the necessary interparticle repulsion to support the external pressure at a higher void ratio, owing to its larger effective specific surface than the undisturbed soil. The compression curve for the remoulded soil thus lies above that for the undisturbed soil (Figs. 49 and 50).

As unloading progresses, the net electrical repulsive forces between soil particles are greater than the external pressure, and the diffuse double layers expand in order to increase the particle spacing. This reduces the repulsive forces so that equilibrium between the intrinsic repulsive stresses and the external pressure is achieved. As the remoulded soil had supported the maximum external pressure at a higher void ratio than the undisturbed soil, it swells on unloading from a higher void ratio and, while the rebound curves are essentially parallel, the remoulded soil equilibrates at a higher void ratio than the undisturbed soil on removal of the external pressure. However, it may be seen (Figs. 49 and 50) that the amount of rebound is always larger for remoulded soil, as can be expected. The test results of Group II soils indicate that Mechanism II, wherein diffuse double layer repulsive forces govern volume changes, controls their behaviour. This is in accordance with the earlier discussions.

Group III Soils : Soils P7D9, NG1, NG3, IT1 and IT2 belong to this group and these soils essentially contian expanding lattice-structured clay minerals in their fines, as is observed from diffraction data (Table 15). As they are comprised of a large percentage of sands (Table 15) their plasticity indices are less.

On Soaking at the initail pressure of 6.25 kPa, the soils in their remoluded state swell, while in their undisturbed state, they compress (only limited results are shown in Figs. 51 and 52). Further the rebound curves are steeper and the recovery ratios are greater for the remoulded state.

As has been seen in the case of Group I soils, the undisturbed samples of Group III soils compress at the initial pressure of 6.25 kPa when allowed to imbibe water and thus it can be assumed that Mechanism I governs their compression behaviour in the initial stage, and Mechanism II begins to dominate over Mechanism I at larger pressures.

In the case of remoulded Group III soils, the soil swells on soaking indicating that repulsive forces dominate and volume changes are governed by the diffuse double layer forces viz., mechanism II. All the crumbs were not destroyed during the loading cycle. It is seen that the behaviour of Group III soils is in accordance with that suggested in the theoretical considerations for a swelling soil in which desiccation bonds and crumbs are prevalent.

TESTS WITH **EDTA**

Three undisturbed samples of Soil SM2 (Fig 48), which is quite similar to soil SM1 and contains kaolinite as the dominant clay mineral in its clay fraction, were subjected to chemical treatment during the consolidation test. After consolidating five samples of SM2 to a pressure of 25 kPA, ($\approx$0.25 kg/cm^2) two samples were adopted as control samples, while the remaining three were treated with a saturated solution of EDTA (disodium salt of ethelene-diamine tetra acetic acid) which entered the samples through the bottom under a head of 0.6m. The reacted solution was drained from the top of the samples. When no further volume changes occurred under constant load, circulation of the EDTA solution was stopped and distilled water was circulated to flush out traces of spent and unreacted EDTA from the samples. The treated as well as the control samples were then subjected
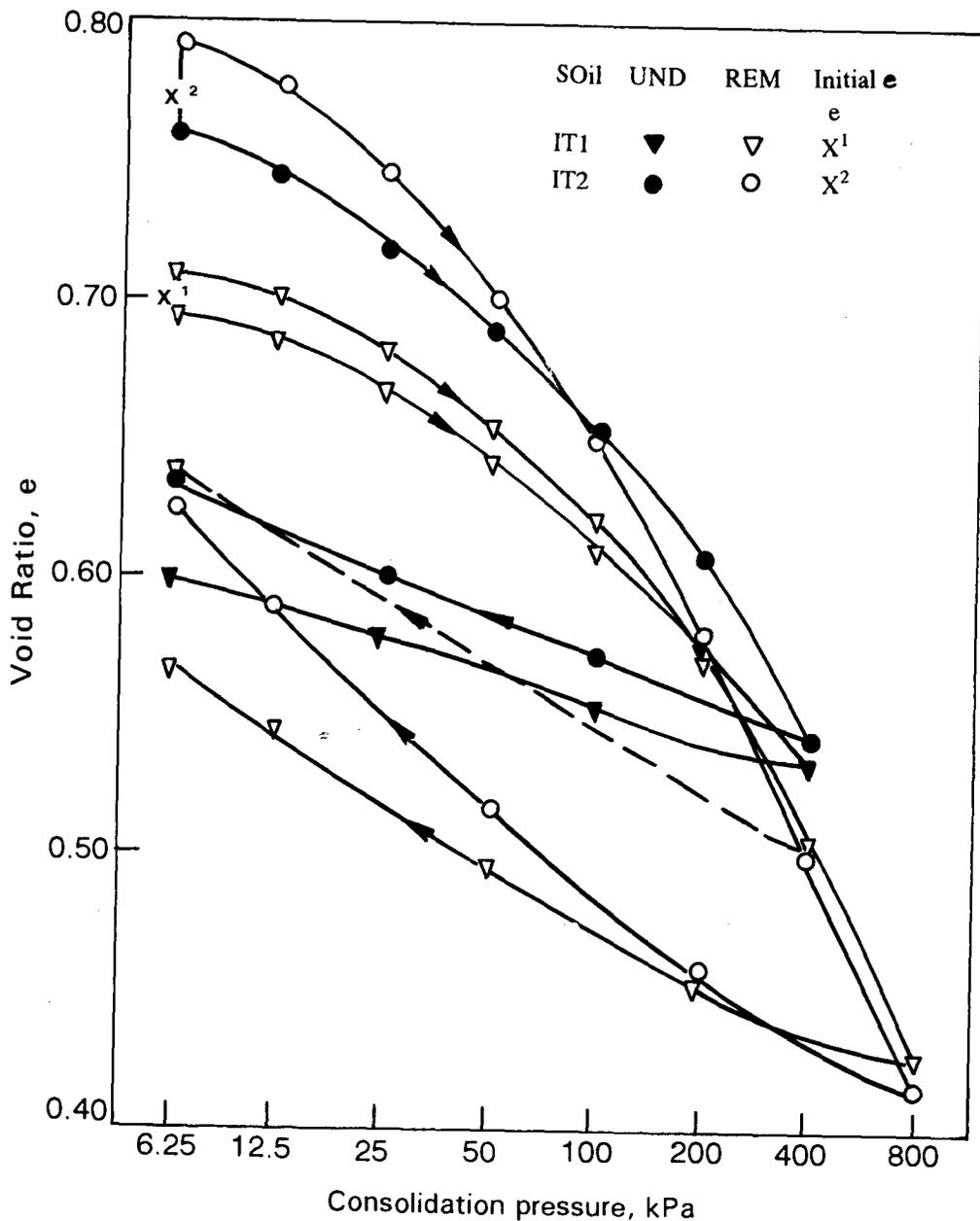
FIGURE 52 One-Dimensional Consolidation Curves for Soils IT1 and IT2
(98 kPa = 1 kg/cm²)

to the rest of the loading cycle up to 800 kPa and were then unloaded. Comparison of the compression curves of the untreated and treated samples (Fig.48) shows that while no further compression occurred in the control samples at 25 kPa during the treatment peroid, the treated samples underwent further compressions when the EDTA solution was circulated through them.

It has been reported in the literature that treatment with EDTA removes cementing agents of Ca, Mg, Al, and Fe. Further, it has been observed earlier that desiccation bonds are chemical in nature and may be comprised of Mg, Ca, Fe, and Al, among other cementing agents derived by the solution and redeposition of the mineral constituents of soils by wetting and drying. Therefore, the compressions taking place during treatment with EDTA can be attributed to the destruction of bonds and , thus, loss of intrinsic effective stress and, therefore, shearing resistance at particle contacts. It has been seen that group I soils are governed by Mechanism I in their compression behaviour.

The compression under constant load during the EDTA treatment ceases when a void ratio is attained at which the soil can support the external load in the absence of the shearing resistance contributed by the desiccation bonds. It is seen from Fig. 48 that this void ratio is somewhat larger than the one at which the remoulded soil supports the same load, and this is perhaps due to incomplete removal of desiccation bonds as well as fabric differences occurring on remoulding. The compression curves obtained on further loading of the

513

treated samples are similar in nature to those obtained for the remoulded state. The recovery on unloading for the treated and remoulded samples is of the same order and much less than that for the undisturbed state.

Thus one can conclude that for desiccated soils, the two mechanisms governing the volume change behaviour of pure clays viz., Mechanism I, wherein the compressibility of a clay is primarily controlled by the shearing resistance at the near contact points, and volume changes occur by shearing displacements or sliding between particles, and Mechanism II, in which compressibility is primarily governed by the long range osmotic repulsive forces also govern that of natural desiccated saturated soils.

Desiccation bonds present in these soils, by imparting additional intrinsic effective stress to them (and thus shearing resistance at particle contacts), and by forming crumbs, affect the volume change behaviour of such soils and prevent the mechanisms independently governing their volume change behaviour.

For desiccated soils containing non-expanding lattice-structured clay minrals in their clay fraction, the desiccation bonds increase their resistance to compression by increasing the shearing resistance at particle contacts.

For desiccated soils containing non-expanding lattice-structured clay minerals in their clay fraction, the desiccation bonds, by creating crumbs with the resulting reduction in effective specific surface areas, prevent the diffuse double layer (Mechanism II) from solely deciding their responses to loading and unloading. If bonding is less, Mechanism II governs the behaviour. If bonding is large, the soil is reduced to one chiefly composed of non-expansive clays. Mechanism I will then govern the compression behaviour until bonds are disrupted. That is, dependent on the degree of bonding either Mechanism II alone or Mechanisms I and II successively will govern the compression behaviour, while the rebound behaviour is governed primarily by Mechanism II.

SECONDARY COMPRESSION BEHAVIOUR

Based on extensive investigation carried out on the Hubli black cotton soil and sodium kaolinite using different organic fluids (in order to change the electrical force system at particle level) Sridharan and Rao (1982) showed that the secondary compression coefficient (ratio of secondary compression per unit of log time to the thickness of the sample at the end of compression for that load increment) is related directly to the strength at the particle level, which is a function of the modified effective stress as given by eqn. 17. As the strength of the particle skeleton increases, the secondary compression coefficient decreases. Further, as the strength at the particle level decreases with increase in void ratio, the secondary compression coefficient increases. The level of the void ratio has significant influence on the secondary compression coefficient. Further Sridharan and Rao (1982) have showed that as the load increment increases the secondary compression coefficient increases for the same void ratio level.

A decrease in the dielectric constant of the pore medium brings about an increase in the attractive forces and a decrease in the repulsive forces and hence a net increase in the effective stress and consequent increase in the shearing resistance. This causes a significant decrease in secondary compression coefficient. Both viscosity and dipole moment of the fluid have marginal influence on the secondary compression.

SHEAR STRENGTH BEHAVIOUR

SATURATED SOILS

Knowledge of the shear strength of soils is of fundamental importance to civil engineers, as it forms the basis of the analysis of all stability problems. Unlike other materials, the strength of soil cannot be specified in terms of a few well-defined parameters. Further, the behaviour of clayey soils during shear is much more complex than those of coarse-grained or granular soils. Many factors, such as type of clay mineral, water content, nature of pore fluid, stress history, drainage during shear, stress path, rate of loading, temperature and soil fabric influence the strength behaviour of clays; hence any attempt to assign a unique value to shear strength is far from justified. Several concentrated investigations have been conducted in the past to better the understanding of the behaviour of clays as affected by the various factors mentioned above.

Realizing the importance of the better understanding of the shear strength behaviour of clay soils in soil engineering practice, Sridharan and Rao (1979) elucidated the mechanisms involved in the strength behaviour of clays with the results obtained from an experimental programme and theoretical study and by using the modified effective stress concept as given in eqn. 17. For this purpose, various organic pore fluids of different dielectric constants as given in Table 4a have been used as pore media in order that the electrical attractive and

514

TABLE 16

SOIL CHARACTERISTICS

Properties	Kaolinite	Montmorillonite
Liquid limit : %	49	305
Plastic limit : %	29	44
Plasticity index : %	20	· 261
% finer than 0.002 mm	54	100
Surface area by nitrogen adsorption method : m²/g	18.3	64.2
Specific gravity of soils	2.59	2.83
Base exchange capacity : meq/100g	8.1	74.2
% of exchangeable potassium	0	0
% of exchangeable sodium	18.2	97.8
% of exchangeable calcium	81.8	2.0
% of exchangeable magnesium	0	0.2

repulsive forces between the particle (consequently the effective stress) could be varied.

Equation 17 which defines the effective stress assumes contacts between mineral to mineral to be present. The shearing resistance of saturated clays has to date been considered to arise either from the frictional resistance at particle contacts or from the viscosity of the pore fluid in the event of particle contact being absent. Allam and Sridharan (1984b) have shown that the resistance to shear from viscosity considerations is far less than that obtained experimentally. Thus they inferred that edge-face linkage or mineral-mineral contacts are inevitable in soil systems possessing considerable shear strength.

Equation 17 has been formulated with the reasoning that the repulsive forces tend to separate the clay particles away from each other in the same way as the positive pore water pressure does, and the attractive forces tend to bring the clay particles closer together in same way as the external normal force does. It has been brought out that there are many complex factors responsible for the net attractive and repulsive forces between the clay particles. To date there is no reliable method for evaluating these forces quantitatively. However, a number of investigators have attempted to better the understanding of the nature of these forces. It has been brought out that the primary force which is responsible for repulsion between two clay particles is due to the interaction of diffuse double layers. Further, when all other factors including the distance between the particles remain constant, this repulsive force is proportional to the dielectric constant of the pore medium. Although a number of limitations exist for the application of double layer theory to soil water system, this theory is extremely useful to the soil engineer and is vital to the qualitative understanding of the fundamental behaviour of montmorillonitic soils. Further it has also been shown that the attractive forces vary inversely, and the repulsive forces vary directly, with the dielectric constant of the pore medium. Both these forces vary inversely with the distance between the particles. The dielectric constant deserves consideration as a major parameter judging by the role it plays in affecting the electrical attractive and replusive forces.

Sridharan and Rao (1979) have performed series of shear strength tests on kaolinite and montmorillonite (Table 16) varying the electrical attractive and repulsive forces using differnt organic fluids (as listed in Table 4a) of different dielectric constants.

KAOLINITE

Figure 53 presents the variation of shear strength of kaolinite for different normal pressures with variations in dielectirc constant for statically compacted samples. From this figure it is seen that the strength decreases rapidly with increasing dielectric constant for low range of dielectric constants and the decrease is small for dielectric constants higher than about 20.

It was stated earlier that c (eqn.17) the effective contact stress in the stress that controls the shearing resistacne at interparticle level. It has been shown earlier that an increase in the dielectric constant causes an increase in the electrical repulsive pressure R, and a decrease in the electrical attractive pressure A, at the particle level and/or between particles; as u_w = 0 and u_a = o this results in a decrease in the effective contact stress. In other workds, a decrease in the effective stress c (brought about by an increase in dielectric constant) lowers the

Table 17 (a)

DIRECT SHEAR TEST RESULTS FOR KAOLINTE WITH DIFFERENT PORE FLUIDS

Sample No.	Pore fluid	Dielectric constant at 25°C	Statically compacted			Hand remoulded		
			C_d: Lb/in^2	ϕ_d: degree	A-R: Lb/in^2	C_d: Lb/in^2	ϕ_d: degree	A-R Lb/in^2
1	Air	1.0	4.05	36.6	5.45	–	–	–
2	n-Hexane	1.89	3.19	35.5	4.47	–	–	–
3	n-Heptane	1.92	2.90	35.0	4.14	0.67	37.2	0.88
4	Carbon tetrachloride	2.28	2.38	35.5	3.34	—	–	—
5	Benzene	2.28	1.92	34.11	2.84	–	–	–
6	Ethyle acetate	6.02	1.22	30.6	2.00	0.50	33.2	0.76
7	Acetone	20.70	0.92	30.0	2.00	0.50	33.2	0.76
8	Ethyl alcohol	24.30	0.63	29.5	1.11	–	–	–
9	Methyl alcohol	32.63	0.29	27.5	0.56	0.50	31.9	0.80
10	water	80.40	0.00	25.4	0.00	0.00	26.0	0.00

TABLE 17(b)

DIRECT TEST RESULTS FOR MONTMORILLONITE WITH DIFFERENT PORE FLUIDS

Sample No.	Pore fluid	Dielectric constant at 25°C	Statically compacted			Hand remoulded		
			C_d: Lb/in^2	ϕ_d: degree	A-R: Lb/in^2	C_d: Lb/in^2	ϕ_d: degree	A-R Lb/in^2
1	Air	1.0	1.35	32.6	2.11	–	–	–
2	n-Hexane	1.89	1.34	32.0	1.14	–	–	–
3	n-Heptane	1.92	1.33	31.4	2.18	0.66	35.5	0.93
4	Carbon tetrachloride	2.28	1.32	28.8	2.40	–	–	–
5	Benzene	2.28	1.20	28.5	2.21	–	–	–
6	Ethyle acetate	6.02	0.95	30.5	1.61	–	–	–
7	Acetone	20.70	0.84	30.5	1.43	0.65	35.2	0.92
8	Ethyl alcohol	24.30	0.65	30.0	1.42			
9	Methyl alcohol	32.63	0.60	29.2	1.05	0.20	33.2	0.31
10	water	80.40	0.20	25.0	0.43	0.00	19.0	0.00

1 Lb/in^2 = 0.0705 kg/cm^2

shearing resistance of the system. Thus, from Fig. 53 it can be observed that the decrease in strength with an increase in dielectric constant for kaolinite and so is in qualitative conformity with the modified effective stress concept.

MONTMORILLONITE

Figure 54 presents the variation of shear strenght of montmorillonite with dielectric constant at various normal pressures when the samples are staticaly compacted. The behaviour of montmorillonite (Fig. 54) is similar in nature to that of kaolinite. Hence, it can be concluded that the strength variation of both these widely different clays with respect to dielectric constant is governed by the modified effective stess concept. It may be further observed that, compared to the variation in strength of kaolinite with dielectric constant, the variation in montmorillonite is less significant. This may possibly be due to the reason that the nature of the particle size and shape of the material like kaolinite has a tendency towards a more open structure than the fine-grained montmorillonite, and this randomness in particle orientation may result in relatively higher strength for kaolinite with fluids of low dielectric constant.

COMPONENTS OF SHEAR STRENGTH

The results presented in Figs 53 & 54 in each line refers to a particular pore fluid. Similar results have also been obtained for remoulded soils. It may be pointed out that in these figures the horizontal axis is represented by the externally applied effective normal pressure. From these figures, it is seen that for both kaolinite and montmorillonite clays the strength lines are generally placed in the order of their dielectric constant. The highest strength line is that of air, having the lowest dielectric constant, and the lowest is that of water, having the highest dielectric constant.

The measured values of C_d (cohesion intercept) and ϕ_d (angle of shearing resistance) for statically compacted and hand remoulded kaolinite and montmorillonite clays have been tabulated in Table. 17. These data show that for both statically compacted and hand-remoulded samples of kaolinite and montmorillonite, there is a definite decrease in C_d and ϕ_d with an increas in dielectric constant.

The variation in C_d and ϕ_d with dielectric constant is more significant for kaolinite when compared with that of montmorillonite. Further, it is seen that in both kaolinite and montmorillonite, C_d is generally higher for statically compacted samples than for hand-remoulded ones, especially at lower dielectric constants. For both soils, C_d tends to zero with an increase in dielectric constant.
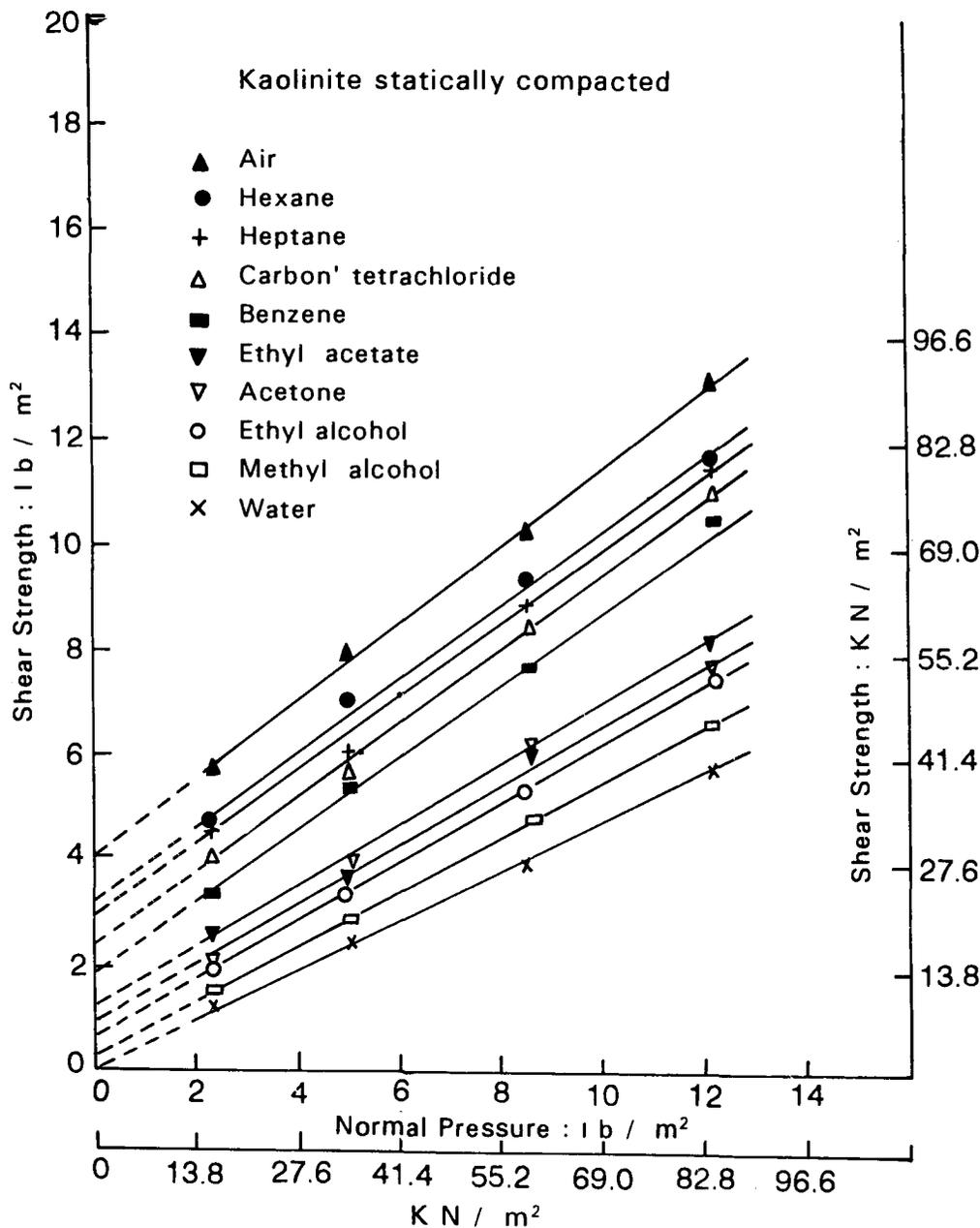
Lambe (1960a) discusses at length the components of shear strength, i.e. cohesion, friction and dilatancy. The factors responsible for cohesion and friction are complex and interrelated. Because of the complexity of the problem, these components can not be distinctly separted by any laboratory test. In other words, at present there is no standard laboratory test which can evaluate these strength components independently. What has been discribed so far is the strength components obtained by following a definite prodedure. For this reason, the term cohesion intercept has been used instead of cohesion. A highly cemented soil may show its improved strength by an increase in angle of shearing resistance rather than cohesion intecept on an effective stress-strength plot.

According to eqn. (17), the effective stress is a function of electrical attractive and replusive pressure and the effective stress increases as the net value of (A-R) increases.

There is no reliable way of determining quantitatively the value of (A-R) in a soil system. For soils in equilibrium, the values of (A-R) could be zero or positive. From Figs. 53 and 54 it has been observed that for both kaolinite and montmorillonite the strength envelopes for the fluids of lower dielectric constants are placed higher and fluids of high dielectric constants, lower; further the values of C_d and ϕ_d are found to decrease with increase in dielectric constant of the medium. It was also shown earlier that the attractive forces vary inversely and the repulsive forces vary directly with the dielectric constant of the medium. Thus it may be stated with reasonable confidence that as the dielectric constant of the pore medium increase, the net value of (A-R) decreases. If it is possible to consider quantitatively the increase in effective stress due to a postive value of (A-R) in the Mohr-Coulomb plot, then the strength envelope will move towards the right giving a reduced value or zero of cohesion intercept and possibly a greater angle of shearing resistance. This is similar to the determination of effective stress plot from total stress plots in the conventional method of obtaining the strength parameters. However, in the present context, the net value of (A-R) is always positive or zero, hence the effective stress will always increase or remain constant when values of (A-R) are taken into account. By not considering the electrical attractive and repulsive forces in the calculation of effective stress, higher values of cohesion intercepts are obtained for fluids having relatively higher values of (A-R) compared to that of the water system. This is analogous to the existence of apparent cohesion in partially saturated soils. Had the negative pore pressure been taken into consideration for calculating effective stress, the apparent cohesion might have become zero.

From the cohesion intercepts (C_d) obtained for the various systems [Table 17(a) and 17(b)] it is possible to estimate the contribution of intrinsic effective stress, ie., the net values of (A-R) to the effective stress, provided the following assumptions are made.

(a) The modified effective stress defined in eqn.(17) is valid.

(b) The effective stress-strength relationship is linear.

(C) The angle of shearing resistanced ϕ_d remains essentially the same with or without consideration of the net values of (A-R) forces.

(d) All shearing resistance in the soil is of a frictional nature. In other words, if the true effective stress is known, then the cohesion intercept should become zero.

Kaolinite statically compacted

▲ Air
● Hexane
+ Heptane
△ Carbon' tetrachloride
■ Benzene
▼ Ethyl acetate
▽ Acetone
○ Ethyl alcohol
□ Methyl alcohol
X Water

Shear Strength : l b / m²

Shear Strength : K N / m²

96.6
82.8
69.0
55.2
41.4
27.6
13.8

Normal Pressure : l b / m²

K N / m²

FIGURE 53   Effect of Dielectric Constant on Drained Shear Strength of Kaolinite

In view of these assumptions, the intrinsic effective stress σ" equals $C_d/\tan \phi_d$. The values of φ' obtained are shown in column 6 for statically compacted systems and in column 9 for hand-remoulded systems in Tables 17(a) and 17(b). A study of these tables reveals that irrespective of the type of system, there is a clear decrease in the net values of (A-R) with an increase in dielectric constant. Further, it can be observed that in both soils, hand-remoulded samples show relatively low values or (A-R) and the values of (A-R) are low in montmorillonite compared to kaolinite. This may possibly be attributed to the dominant influence of the replusive forces in montmorillonite system, relative to kaolinite systems. Considering the various limitations in the procedure adopted, the magnitude of the forces obtained is only approximate. But these computations have helped in getting an approximate range of the magnitude of these forces involved.

TESTS WITH SAND SILT

Sridharan, Rao, and Makan (1983) carried out tests with sand and silt with organic fluid CCl_4 and water in order to show that the R and A forces are insignificant. Since the electrical forces are minimal in sand and silt, shear tests with different pore mediums (water, air and carbon tetra chloride) should show minimal changes in the strength magnitude. Figs. 55 and 56 show clearly that this is indeed so.

518
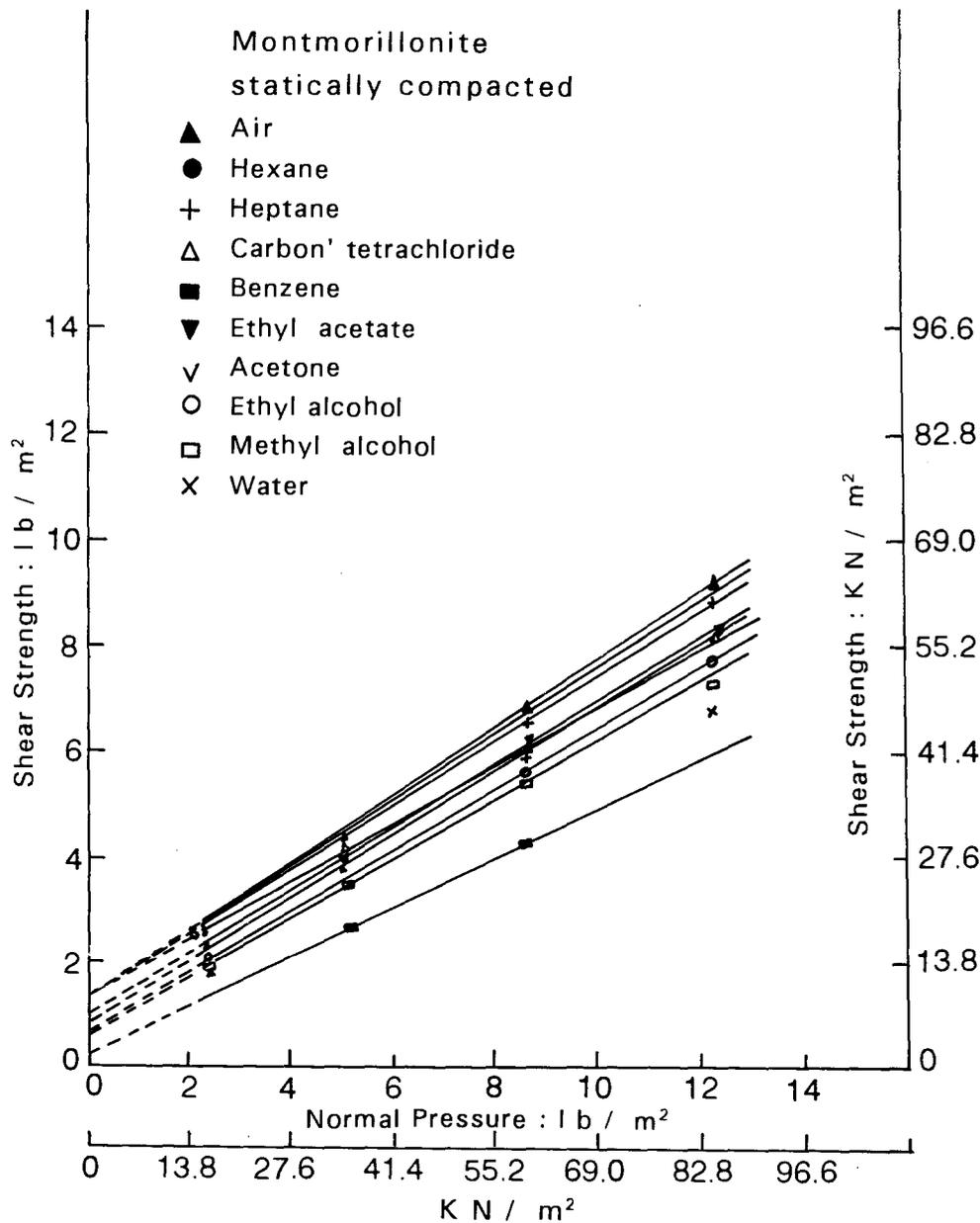
FIGURE 54 Effect of Dielectric constant on Drained Shear Strength of Montmorillonite

In contrast to the kaolinite and montmorillonite soil behaviour, both fine sand and silt have shown marginal influence due to the changes in the pore medium. In both the soils, for all the three pore mediums, the cohesion intercept is zero though the soils, have been compacted. This has been attributed to the presence of negligible electrical attractive forces in silt and fine sand. Further, for sand the angle of shearing resistance obtained for both air and CCl^4 as pore medium is one and the same. This is as ought to be because, sand being an inert material, CCl^4 as pore medium did not affect. Since both sand and silt are essentially not charged particles unlike the clay particle in kaolinite and montmorillonite, the repulsive pressure developed is negligible. Thus the result brings out the dominant influence of repulsive pressure in reducing the shear strength of clayey soils with water as pore medium.

PARTLY SATURATED SOILS

In arid and semi-arid areas, fine grained soils are often encountered in a partly saturated state to a considerable depth. Compacted soils are also usually unsaturated. Krishnamurthy, Sridharan and Nagaraj (1987) carried out comprehensive studies on the shear behaviour of partly saturated soils and brought out the role of the modified effective stress equation (eqn. 17). Since fabric can affect the strength behaviour, samples
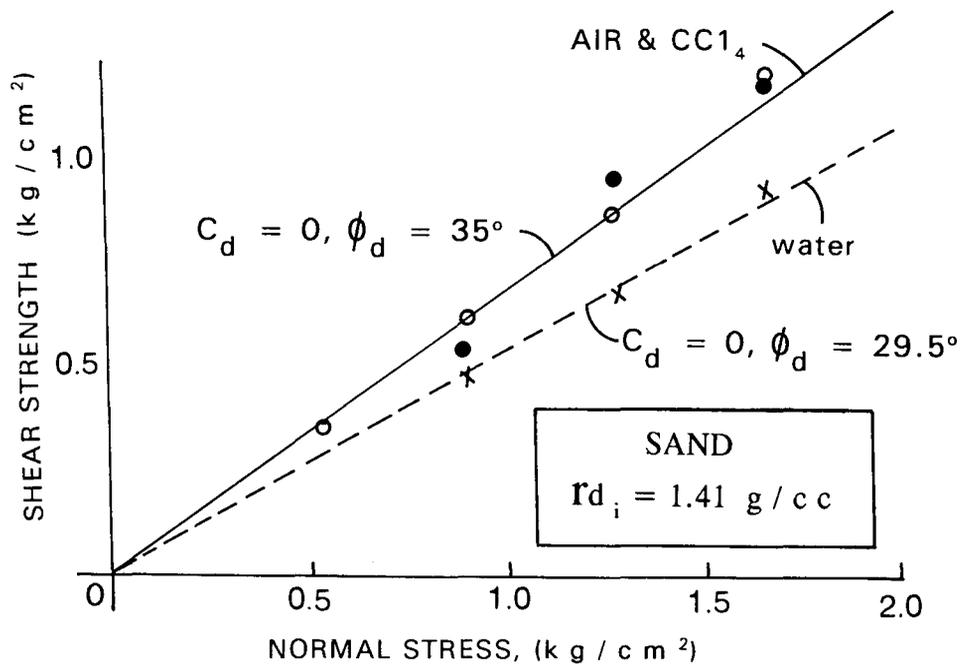
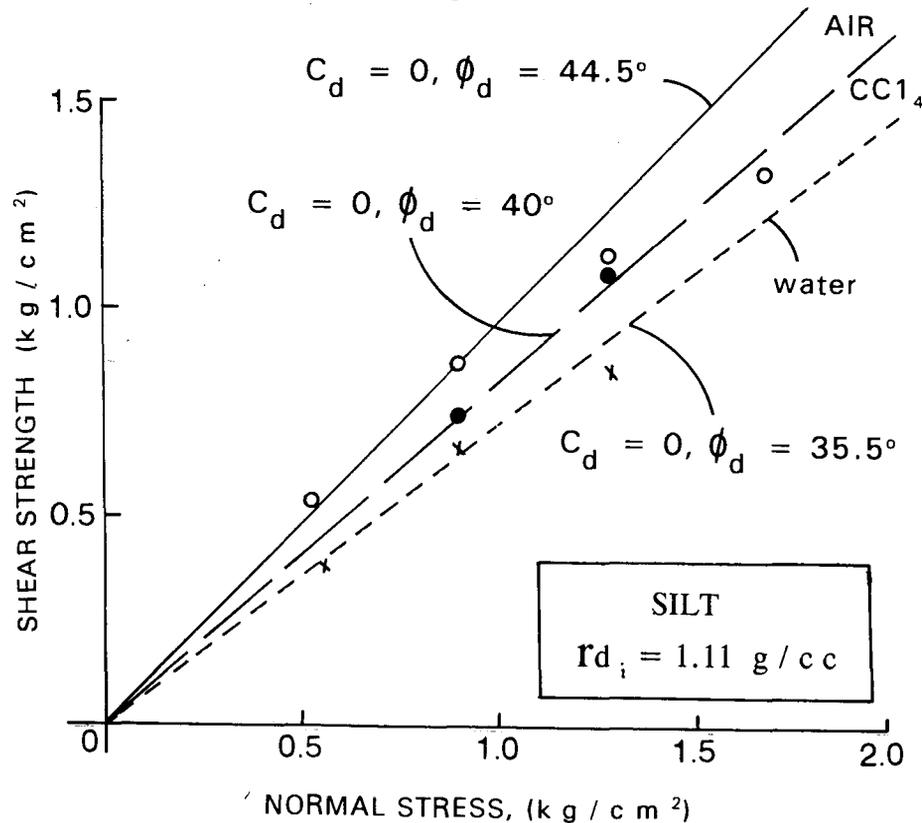**FIGURE 55** Strength Envelope for Sand with Different Fluids

FIGURE 56 Strength Envelope for Silt with Different Fluids

with same fabric have been studied with variation in degree of saturation. In order to obtain this, samples compacted at the same water content were cured under different relative humidities.

With the strength defined at maximum principal stress difference, Figs 57 and 58 show the results of drained compression test in the modified Mohr-Coulomb diagrams. The strength parameters, c and ϕ determined from the plotted envelopes are to be considered in the context of total stresses rather than the true effective stresses since the effective negative pore water pressure, $\bar{u}_w$ and R and A forces are not considered. The angle of shearing resistance remained approximately constant for different degrees of saturation except for S_r=99 per cent. Lower angle of ϕ for S_r=99% may be attributed to the slight increase in the initial void ratio during resaturation. However, the cohesion intercept, c varied with the degrees of saturation. This is as ought to be under the conditions in which the tests were carried out in this investigation. For the specimens having constant dry density
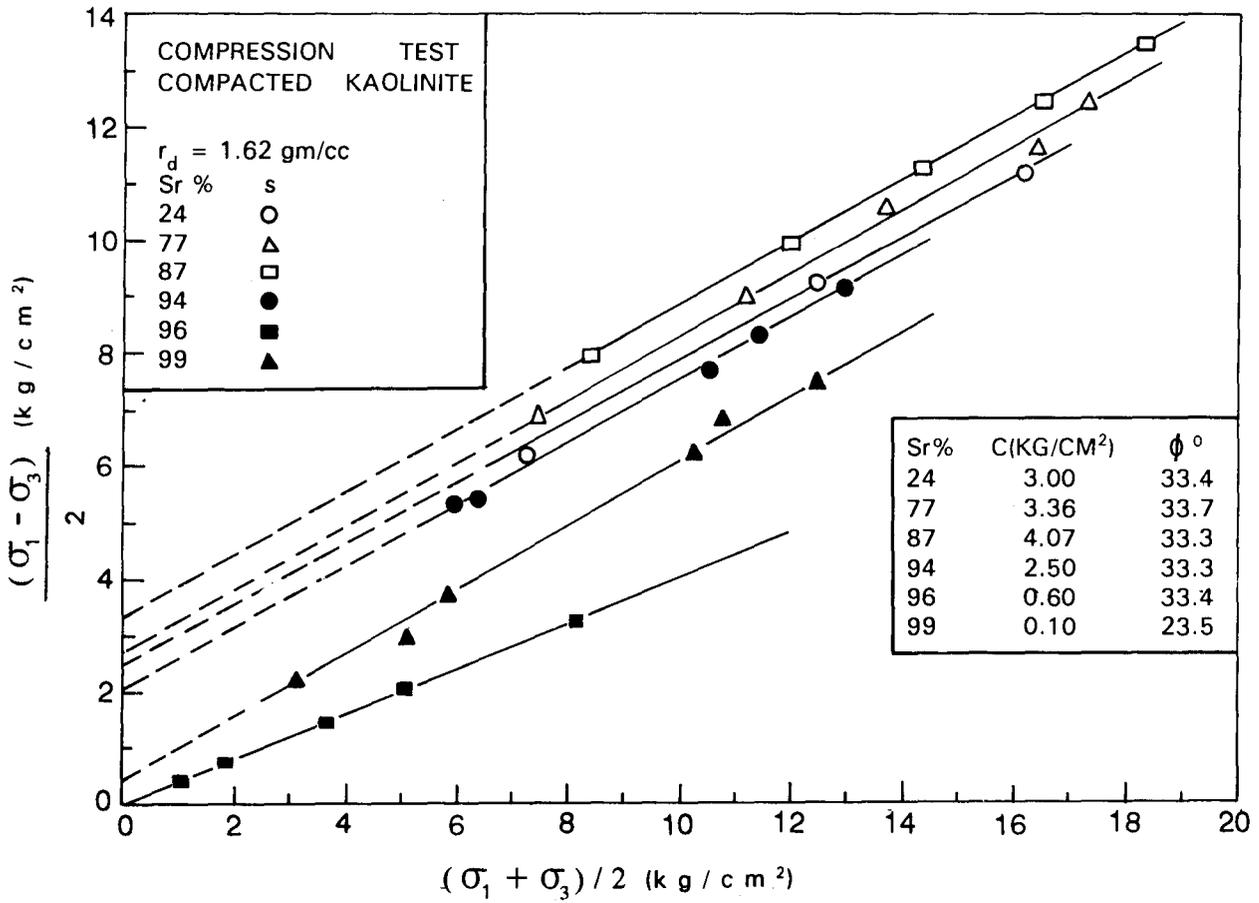
520

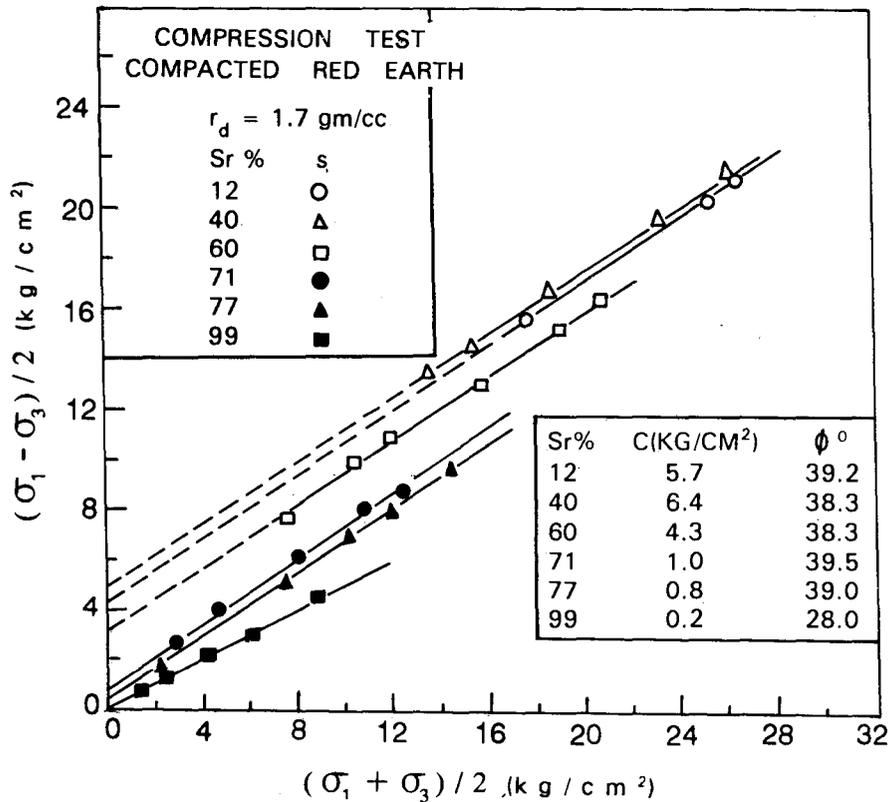**FIGURE 57 Modified Mohr-Coulomb Strength Envelopes for Kaolinite**

Sr%	C(KG/CM²)	φ°
24	3.00	33.4
77	3.36	33.7
87	4.07	33.3
94	2.50	33.3
96	0.60	33.4
99	0.10	23.5

FIGURE 58 Modified Mohr-Coulomb Strength Envelopes for Red Earth

Sr%	C(KG/CM²)	φ°
12	5.7	39.2
40	6.4	38.3
60	4.3	38.3
71	1.0	39.5
77	0.8	39.0
99	0.2	28.0

and fabric (in this investigation both have been maintained approximately constant), the angle of shearing resistance need not be affected by the changes in the degree of saturation. A change in the degree of saturation produces necessary change in the effective negative pore water pressure, $\bar{u}_w$ and R and A forces which have been reflected as change in apparent cohesion. Thus, the apparent cohesion may vanish if the results are plotted truly on the basis of effective stress. For Figs 57 and 58, it is further noticed that, starting from a low degree of saturation, the cohesion intercept increased with the degree of saturation upto a certain optimum value and thereafter decreased rapidly with the degree of saturation.

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

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

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

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

(iii) For the range of degree of saturaion used, variation in strength is primarily influenced by variation in pore water tension and the area over which $\bar{u}_w$ acts rather than R and A forces (Sridharan, 1968).

With these assumptions, the cohesion intercept reflects nothing but the effect of negative pore water pressure at failure for partially saturated specimens for a fully saturated specimen, the Mohr-Coulomb envelope passes through the origin owing to the absence of any negative pore water pressure. But in an unsaturated condition, the negative pore water pressure is a function of the degree of saturation, thus producting a cohesion intercept in the Mohr-Coulomb diagram if the effective stress is not considered. So, the effective negative pore water pressure could be computed by shifting the shear stress axis laterally so that the cohesion intercept is zero. The distance, $c \cot \phi$ through which the axis is shifted provides an estimate of the effective negative pore water pressure at failure at any degree of saturation. Table 18 presents the computed values of $c \cot \phi$ $(=\bar{u}_{wf})$ for different degrees of saturation. The effective negative pore water pressure as estimated by $c \cot \phi$ increased upto a certain optimum degree of saturation and therafter decreased. The maximum estimated negative pore water pressures are 6.2 kg/cm² and 8.1 kg/cm² respectively in kaolinite and red earth.

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

The high range of negative pore water pressure obtained by Sridharan (1968) and Rao and Sridharan (1978) are primarily due to the assumptions made in their calculations that the capillary equation is valid in the estimation of negative pore water pressure and for any particular S_r, the average pore size filled with water determines the magnitude of the negative pore water pressure. But in an experimental investigation like this, the effective negative pore water pressure at failure is primarily due to the pore water tension in the pores through which the failure plane passes. (This implies that the effective negative pore water pressure need not be same throughout the specimen for the experimental conditions obtained in this study). Obviously the failure plane passes through the path of least resistance and so probably through the pores of large size. The corresponding
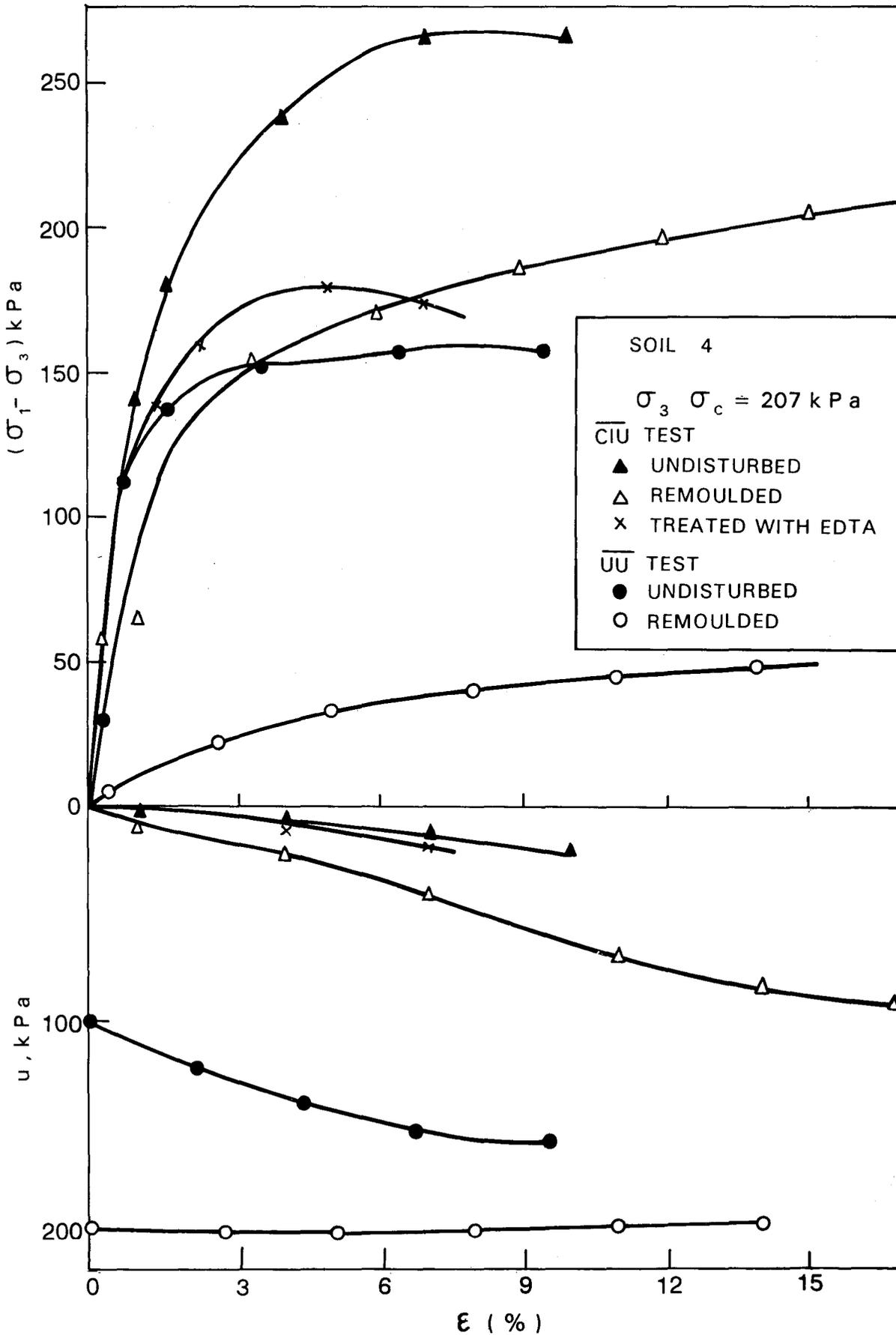
FIGURE 59 Typical Stress-Strain and Pore Water Pressure-Strain Curves for Soil 4
(98 kPa = 1 kg/cm²)

TABLE 18

ESTIMATED EFFECTIVE NEGATIVE PORE WATER PRESSURE AT FAILURE

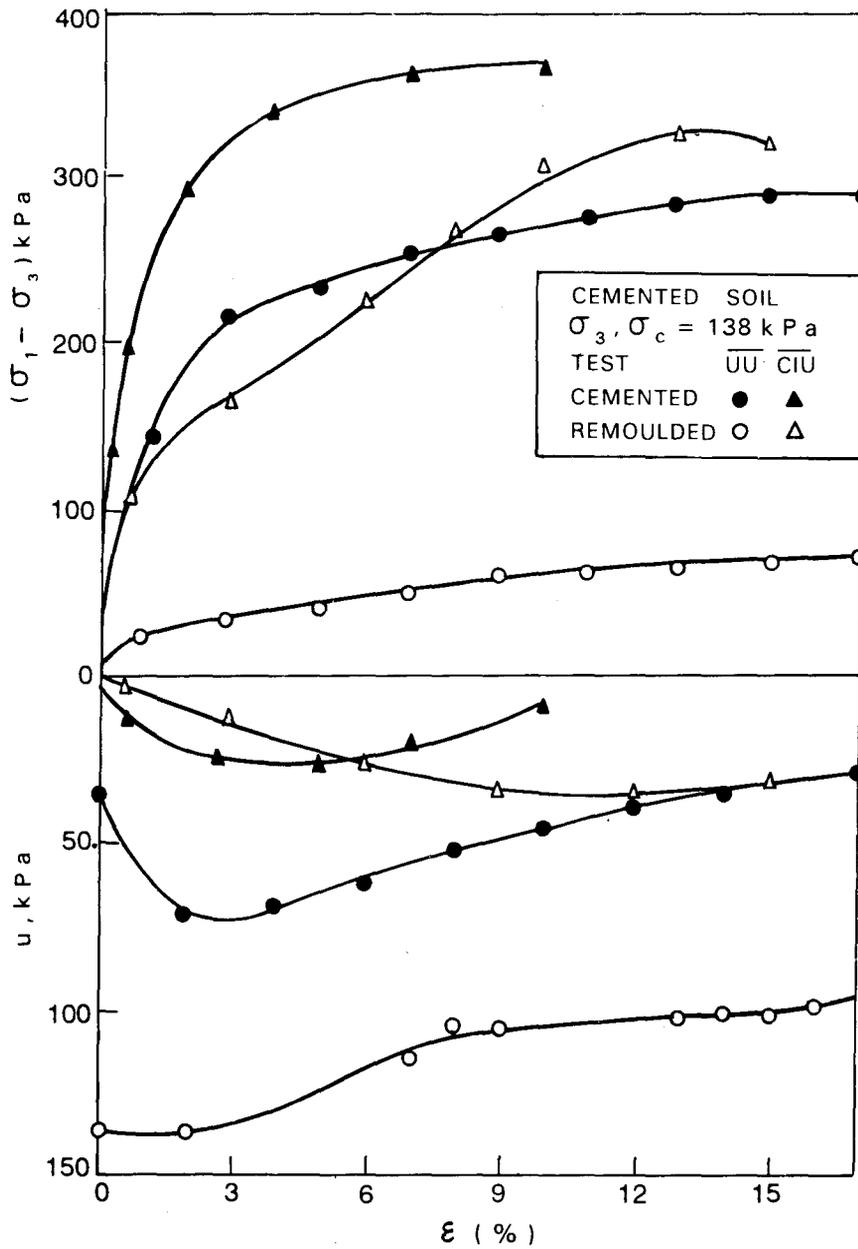| Kaolinite		Red Earth	
Sr (%)	c cot ϕ or $\overline{u}_{wf}$, kg/cm^2	Sr (%)	c cot ϕ or $\overline{u}_{wf}$ kg/cm^2
24	4.55	12	7.0
77	5.05	40	8.0
87	6.2	60	5.4
94	3.8	71	1.2
96	0.9	77	1.0
99	0.23	99	0.38

FIGURE 60 Typical Stress-Strain and Pore Water Pressure-Strain Curves for Cemented Soil (98 kPa = 1 kg/cm ^{2})

effective negative pore water pressure is likely to be much less than that obtained by theoretical calculation. The theoretical calculation assumes that for any particular degree of saturation, the pores are filled with water uniformly to a particular level of pore size (average pore size) which is always less than the pore size that is actually filled in the test specimen. Therefore, the theoretical calculations have serious limitations in that they always estimate the negative pore water pressure on the higher side. The results obtained in this study cannot be compared directly with those of Sridharan (1968) and Rao and Sridharan (1978) as they were theoretically over estimated. However, the range of negative pore water pressures measured by Olson and Langfelder (1965) compare favourably with the results obtained in the present investigation.

DESICCATED SOILS

Allam and Sridharan (1984a) have made a detailed study of the strength behaviour of desiccated soils. Their study brought out that the shear strength of desiccated soils are significantly by chemical bonding. Some typical results are presented below.

Fig. 59 shows typical results obtained from UU and CIU shear tests on desiccated soils obtained in and around Bangalore. It is observed that the undisturbded soil exhibits larger shear strength and a stiffer stress-strain response. Further, the strain at failure is more for remoulded soil, and this is more pronounced in the UU tests,. The average strain at failure for this is more for remoulded soil, and this is more pronounced in the UU test. The average strain at failure for UU and CIU tests are given in Table 19. The reduction in shear strength and stiffness on remoulding is greater in case of UU as compared to CIU tests. Fig. 60 presents the stress-strain response of the cemented soil in the laboratory from which it is seen that identical changes in strength, stiffness and failure strain (Table 20) occur on remoulding. Since these changes can be attributed to the iron bonds introduced in the cemented soil it can, therefore, be concluded that similar bonds exist in the desiccated soil. The shear strength of the cemented soil is greater than that of the desiccated soil (Figs. 59 and 60). This can be attributed to the presence of more and/or better bonds in the cemented soils as they were generated under controlled conditions. Fig. 59 also shows the stress-strain behaviour of desiccated soil wherein EDTA solution was circulted to remove any chemical bonds present. The noticeable reduction in shear strength occurring on treatment indicates that the bonds present in desiccated soils are possibly chemical in nature as an analysis of the spent EDTA solution showed that quantities of Ca, Mg, Al, and Fe had been removed from the soil.

For disturbed natural soils subjected to repeated wetting and drying, it was reported by Allam and Sridharan (1981) that the soil gained progressively in strength and stiffness (Fig.61). Significant reductions in shear strength and noticeable reductions in stiffness were also reported when, after having undergone many cycles of wetting and drying, EDTA solution was circulated prior to consolidating and shearing. This was attributed to fabric changes and bonding brought about by the process of repeated wetting and drying, and these bonds were considered to be chemical in nature. The similarity of the stress-strain behaviour of desiccated soil to the reported behaviour of soil subjected to wetting and drying (Allam and Sridharan, 1981) suggests that perhaps desiccation bonds are induced by repeated wetting and drying.

In all cases it was found that the deviator stress-strain curves obtained could be transformed into rectangular hyperbolae (Sridharan and Narsimha Rao, 1972).

The shear parameters obtained for desiccated soils in their undisturbed and remoulded states are presented in Table 19, while Table 20 presents the values obtained for the cemented soil. It is observed that the undisturbed desiccated soil possesses greater cohesion (both in terms of total and effective stresses) than the remoulded soil. The same is observed for cemented soil. This is attributed to that part of the intrinsic effective stress which is imparted to the soil by desiccation bonds and by the iron hydroxide bonds in the cemented soil.

The data in Table 19 and 20 also show that the UU shear tests on the soils (particularly undisturbed and cemented soil) yield angles of shearing resistance both in terms of total and effective stresses. This is because the intrinsic effective stress imparted by the bonds increases the incompressibility of the soil so that the soils supports significant proportions of the applied all round stress under no volume change conditions. consequently the shear strength varies linearly and significantly with change in confining pressure so that the failure envelope possesses a non-zero slope. For the remouled soil, the absence of bonds implies reduced effective stress inspite of the lower void ratios and hence a more compressible fabric which supports only a small proportion of the aplied confining pressure. For this reason the change in shear strength with respect to change in σ_3 is small and the failure envelope is, as a result, flatter.

It is also observed that the desiccated soils possess larger effctive stress angle of shearing resistance from both CIU and UU shear tests in their remoulded state as compared to the undisturbed state. This is as
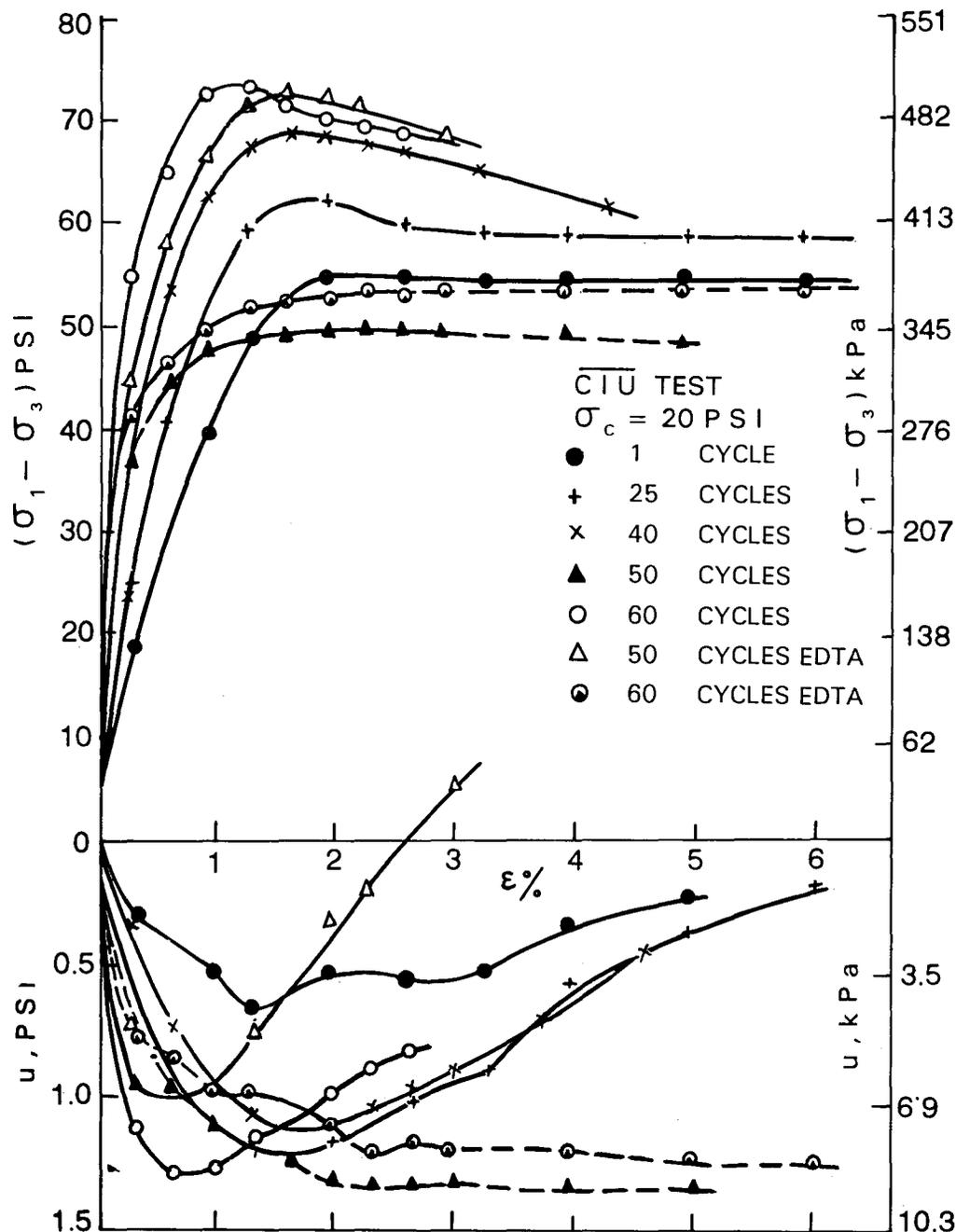
FIGURE 61.Typical Deviator Stress-Axial Strain and Pore Pressure Curves for Wetting and Drying Specimens (98 kPa = 1 kg/cm^2)

anticipated and can be attributed to the more homogenous and structural defect free specimens obtained by remoulding. This is confirmed by the value obtained from EDTA treated soil 4 specimens (Table 21) which yielded a ϕ'_{cu} value closer to the undisturbed value than the remoulded value. The EDTA treatment removes bonds but leaves the soil fabric and structural defects unaltered. A similar finding has been reported for soil subjected to repeated wetting and drying by Allam and Sridharan (1981).

In the case of the cemented soil it is observed that the effective angles of shearing resistance are the same for cemented and remoulded states (Table 20) indicating that the specimen preparation technique yielded similar fabric in both states.

The UU shear tests are found to yield larger effective stress angles of shearing resistance as compared to CIU shear tests (Table 19). This is because, while changes in effective stress occur in UU shear tests without significant volume changes, this is not the case in CIU shear tests. The volume changes occurinng during consolidation bring about a re-orientation of soil particles and hence lower the effective angles of shearing resistance.

526
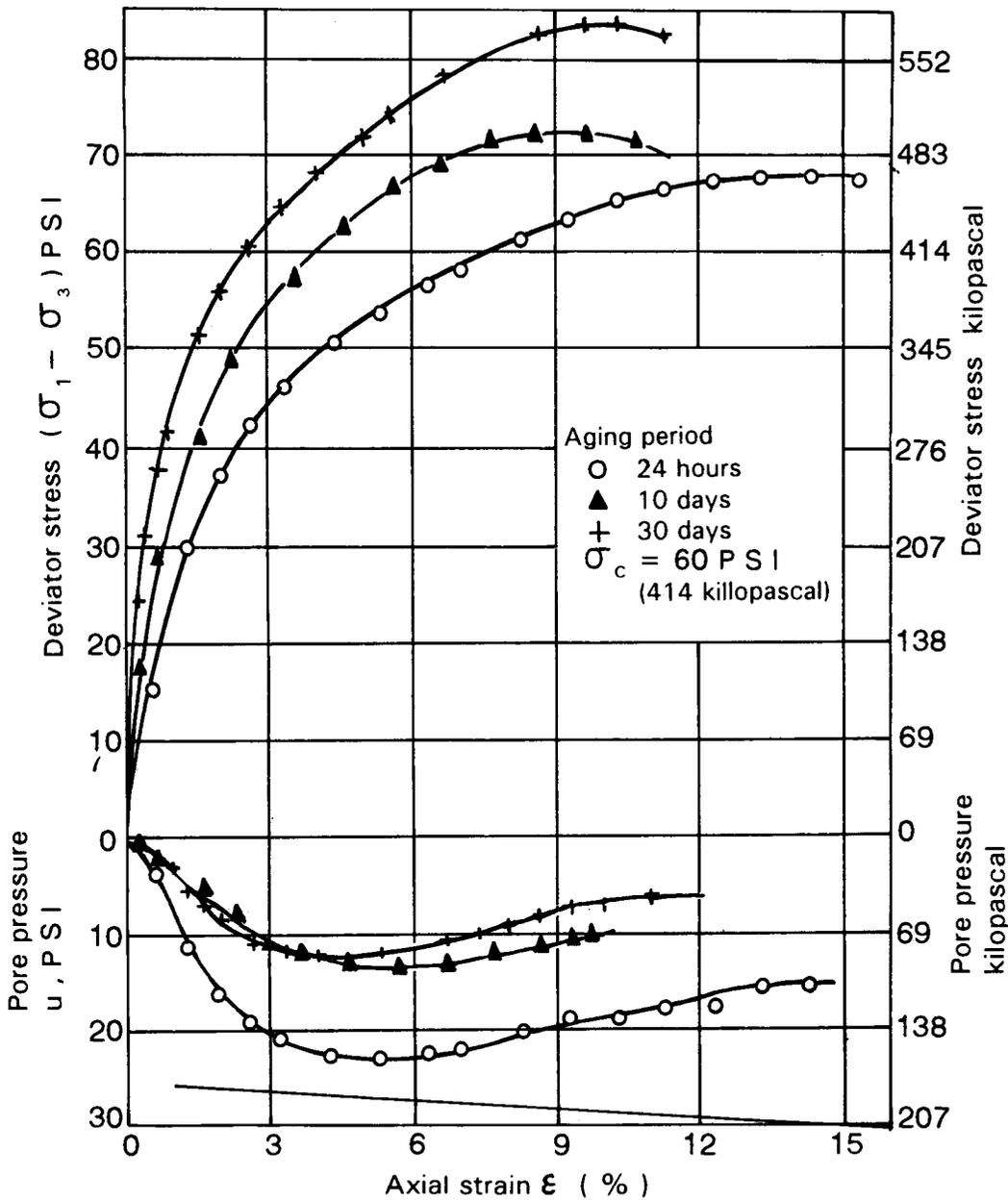
FIGURE 62 Typical Stress-Strain and Pore Water Pressure-Strain Curves for Kaolinite (14 psi = 1kg/cm²)

From Tables 19 and 20 it is also seen that UU shear tests generally yield smaller cohesion values than CIU shear tests. This may be because the greater particle spacing in the former gives lesser net attractive forces between particles as the intensity of interparticle attraction is inversely proportional to the spacing between particles.

EFFECT OF AGING

Similar to what has been noticed for desiccated soils, has significant effect on the shear strength behaviour (Allam and Sridharan, 1979). The deviator stress vs. axial strain curves (Figs. 62 and 63) for hand remoulded, saturated black cotton soil and kaolinite indicate that on aging, an increase in shear strength and a stiffer stress-strain response occur along with a decrease in failure strain; these are more pronounced for the black cotton soil. Both soils display an increasing fabric rigidity with aging in their deviator stress-strain curves indicating that the initial tangent modulus increases with aging. Further, a hyperbolic deviator stress vs. strain response is in evidence (Sridharan and Narsimha Rao, 1972). The initial tangent moduli computed from test data (assuming the deviator stress vs. strain relationship to be a rectangular hyperbola) have shown that for both soil,

527
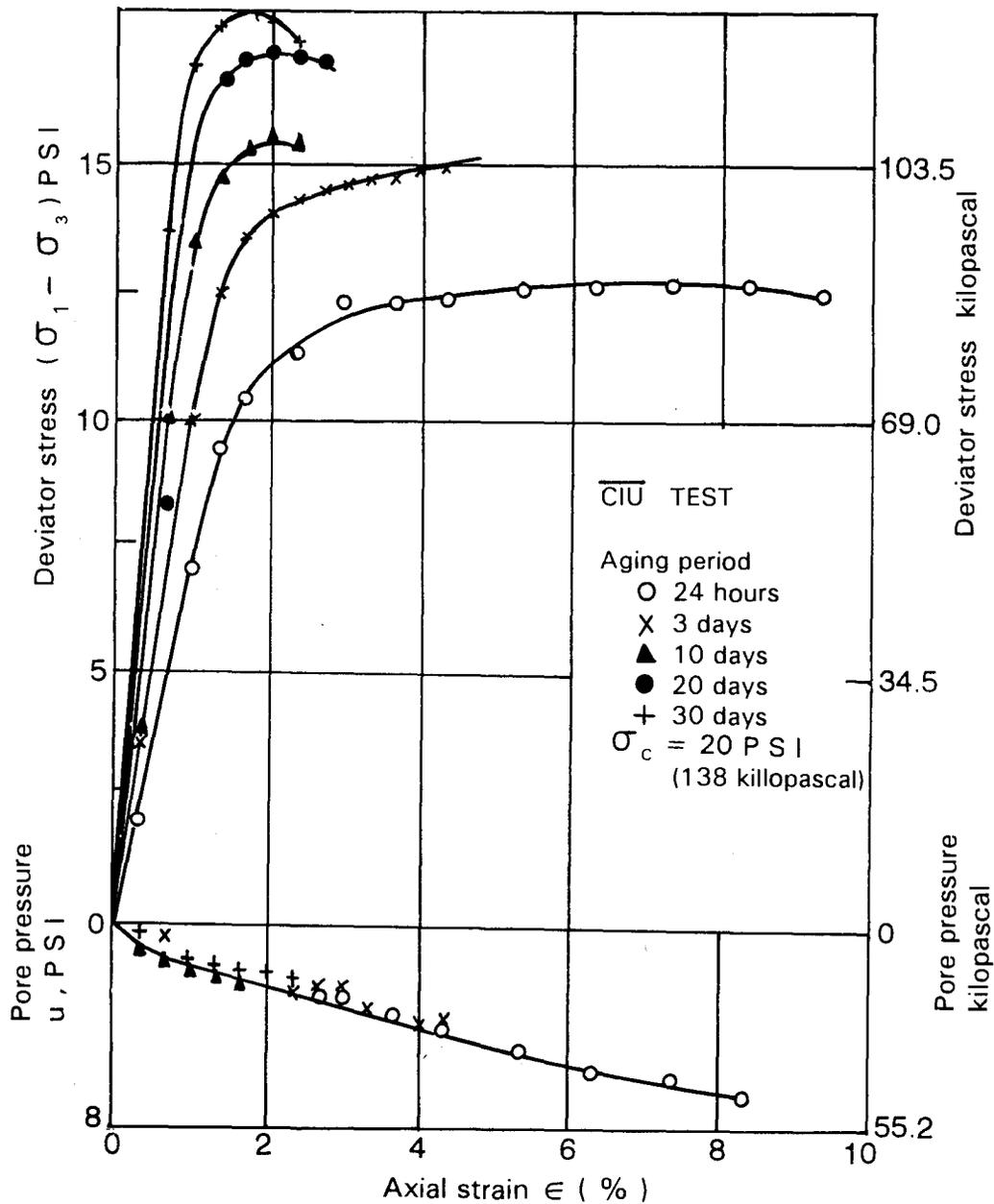
FIGURE 63 Typical Stress-Strain and Pore Water Pressure-Strain Curves for Black Cotton Soil (14 psi = 1kg/cm²)

the initial tangent modulus increases with confining pressure lilnearly for any given period of aging. Also, the initial tangent modulus increases with the period of aging and this increase is greater in the early stages of aging. Whereas a kaolinite spcimen at a consolidation pressure of 414 kPa (60 psi) registers an 85% increase over its 1 day value on aging for 30 days, a black cotton soil under similar conditions exhibits a 1000% increase, indicating that the influence of aging on the stress-strain response is greater for black cotton soil, which exhibits greater secondary time effects.

The increase in the initial tangent modulus on aging may be due to the increase in intrinsic effective stress occuring because of the developement of bonds at particle contact during aging as well as fabric changes (Allam and Sridharan, 1979).

The total and effective stress shear strength parameters obtained from CIU tests on black cotton soil and kaolinite after aging them for different periods are presented in Table 22. The total and effective stress angles of shearing resistance are found to increase on aging. This rate of increase decrease with age and tends to become asymptotic in the case of black cotton soil and kaolinite.

TABLE 19

SUMMARY OF TEST RESULTS FOR DESICCATED SOILS

Soil	1		2		3		4	
Soil State	UND	REM	UND	REM	UND	REM	UND	REM
Sand (per cent)	53.0		51.0		48.3		70.0	
Clay and Silt (per cent)	47.0		48.1		51.7		30.0	
Liquid Limit (per cent)	49.0		53.2		50.6		38.0	
Plastic limit (per cent)	25.0		32.6		31.0		18.0	
Specific Gravity, G	2.63		2.63		2.68		2.60	
Void Ratio,e	0.80	0.75	0.63	0.65	0.68	0.62	0.69	0.62
Pore Pressure Parameter B	0.63	0.93	0.72	0.85	0.78	0.82	0.60	0.91
UU shear Tests :								
C_{uu} (kPa)	38.9	15.5	39.8	28.3	32.8	27.6	24.5	17.2
ϕ_{uu} (deg)	13.4	1.7	5.5	2.5	4.5	3.02	11.0	2.5
C'_{uu} (kPa)	8.0	0.0	0.0	0.0	8.3	0.0	22.8	3.9
ϕ_{uu} (deg)	30.6	35.2	31.4	31.4	25.8	35.2	25.1	29.9
CIU Shear Tests :								
C_{uu} (kPa)	36.9	28.6	57.9	28.52	39.3	31.8	32.8	14.5
ϕ_{uu} (deg)	21.0	15.5	17.8	15.3	15.0	13.4	19.3	16.7
C'_{uu} (kPa)	23.4	0.0	47.5	0.0	32.8	0.0	22.8	3.9
ϕ'_{uu} (deg)	27.8	31.3	19.6	27.8	19.0	23.8	25.1	27.5
Average Failure UU	11.7	15.6	13.2	18.3	13.6	18.0	11.3	23.5
Strain (per cent) CIU	11.8	14.8	11.8	14.8	13.1	16.0	14.6	16.3

UND=Undisturbed   REM=Remoulded

TABLE 20

SUMMARY OF TEST RESULTS FOR CEMENTED SOIL

Soil State	Cemented	Remoulded
Void Ratio	0.470	0.468
Degree of Saturation Sr, (percent)	100	100
Pore pressure Parameter-B	0.37	0.96
$\overline{UU}$ Shear Test		
C_{uu} (kPa)	71.2	24.1
ϕ_{uu} (deg)	14.4	3.0
C'_{uu} (kPa)	16.5	4.2
ϕ'_{uu} (deg)	33.7	35.2
CIU Shear Test		
C_{cu} (kPa)	38.9	26.7
ϕ_{cu} (deg)	27.8	25.8
C'_{cu} (kPa)	16.5	8.3
ϕ'_{cu} (deg)	33.7	33.7
Average Failure		
Strain (per cent) $\overline{UU}$	14.4	17.3
$\overline{CIU}$	13.7	15.5

TABLE 21

CIU SHEAR TEST RESULTS FOR UNDISTURBED REMOULDED AND EDTA TREATED CONDITIONS

Soil State	UND	REM	Treated with EDTA
c'_{cu} (kPa)	32.8	14.3	5.4
ϕ_{cu} (deg)	19.3	16.7	16.7
c'_{cu} (kPa)	22.8	3.9	0.0
ϕ'_{cu} (deg)	25.1	27.5	24.5

The increase in ϕ_{cu} and ϕ' values over their 1 day value on aging for 30 days are about 40% and 8% for black cotton soil and kaolinite respectively. The much larger increases in the angles of shearing resistance displayed by black cotton soil as compared to kaolinite are attributable to the former possessing greater secondary time effects. The increase in the angle of shearing resistance is attributable to the better packing of particles at the microlevel due to aging, thus leading to greater interference effects.

It is seen from Table 22 that the cohesion intercepts C_{cu} and C' in general decrease on aging. This implies that the net intrinsic electrical attractive force at failure decreases on aging.

Similar to what has been observed for hand remoulded saturated soils (as above), Krishnamurthy, Sridharan and Nagraj (1980) reported results for over consolidated soils also.

TABLE 22

SHEAR STRENGTH PARAMETERS

Age (days)	Black CottonSoil					Kaolinite				
	C_{cu} (kPa)	ϕ_{cu} (deg)	C' (kPa)	ϕ'_{cu} (deg)	$\in_f$ (%)	C_{cu} (kPa)	ϕ_{cu} (deg)	C' (kPa)	ϕ' (deg)	$\in_f$ (%)
1	19.2 (2.78)	8.1	21.0 (3.05)	9.9	6.3	14.7 (2.13)	19.9	3.8 (0.55)	24.5	12.4
2	20.9 (3.03)	8.6	21.0 (3.05)	10.2	4.5	–	–	–	–	–
3	21.0 (3.05)	10.2	24.5 (3.56)	10.7	4.1	12.8 (1.86)	20.1	3.8 (0.55)	24.8	11.3
4	21.0 (3.05)	10.0	24.6 (3.57)	11.2	3.8	–	–	–	–	–
5	11.9 (1.60)	20.7	3.8 (0.55)	25.1	11.2	–	–	–	–	–
6	22.9 (3.32)	11.2	21.2 (3.07)	12.0	2.3	–	–	–	–	–
10	21.1 (3.06)	11.7	17.7 (2.56)	12.5	2.0	11.1 (1.61)	21.1	3.9 (0.56)	25.8	11.0
20	17.7 (2.56)	12.8	15.9 (2.31)	13.4	1.9	5.6 (0.81)	22.0	1.9 (0.28)	25.8	10.9
30	15.9 (2.31)	13.4	17.8 (2.58)	14.4	1.5	3.7 (0.54)	23.2	0.0 (0.00)	26.4	11.3

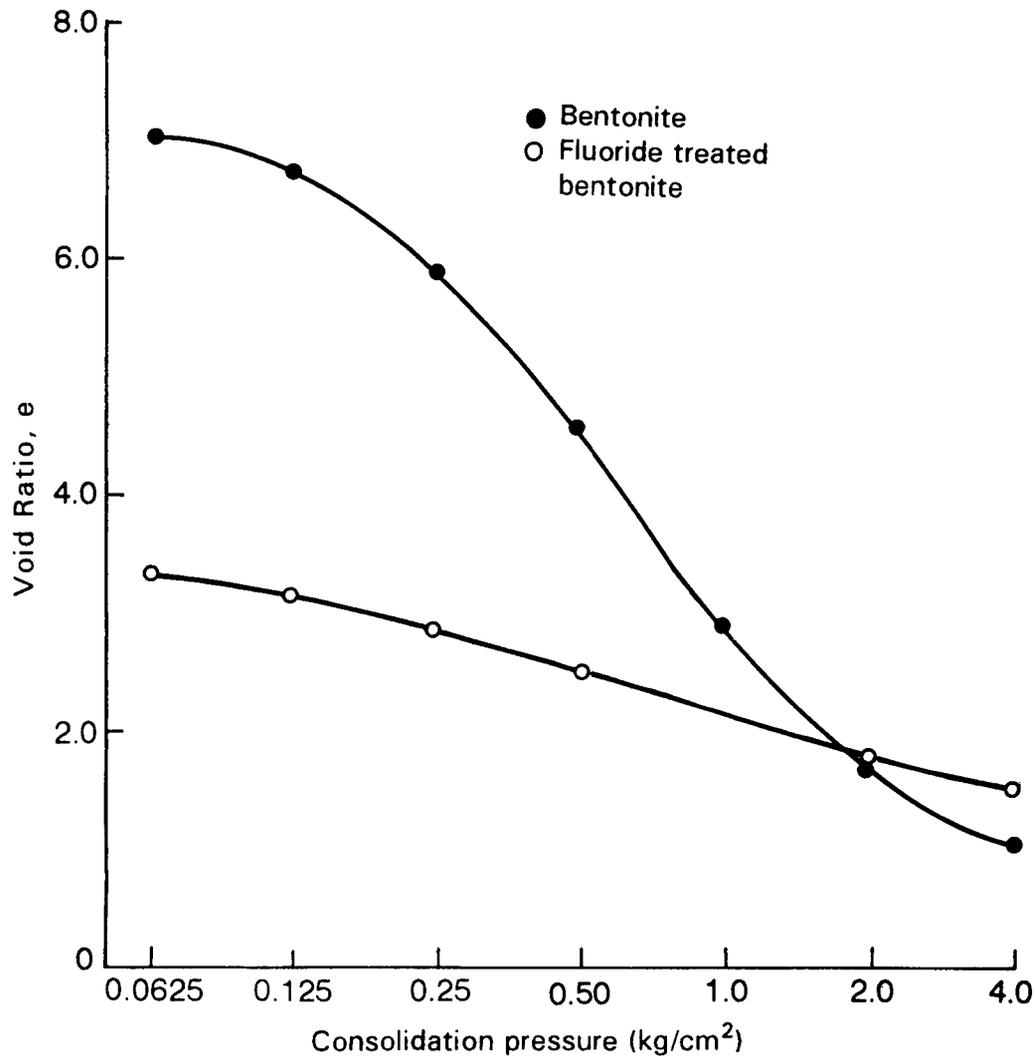Note : Figures in Parenthesis are in psi

FIGURE 64 Effect of Fluoride Anion on Compression Behaviour of Bentonite

COEFFICIENT OF PERMEABILITY

The factors affecting saturated flow through clay soils may be classified under :

- Factors associated with permeant : viscosity, density and hydraulic gradient.

- Soil properties : void ratio, thickness of water layer held to the soil particles, clay fabric and tortuosity

It is difficult to assess the relative importance of each of the factors, since many of them are interdependent. Rao and Sridharan (1987) have highlighted the role of various factors (essentially brought about by changes in chemical environment) in influencing the hydraulic conductivity of montmorillonites (Table 23.)

Replacement of exchangeable sodium by calcium ions resulted in almost 10 fold increase in the permeability coefficient of montmorillonite at a given void ratio. Investigations have brought out that the diffuse double layer associated with clay particle surfaces acts to decrease the permeability of montmorillonite by constricting the flow channels thus reducing the amount of effective pore space for water flow and by mobilizing a dispersed clay fabric which increases the tortuosity factor.

Replacement of monovalent soidum by calcium ions leads to a marked reduction in diffuse double layer thickness. Concomitantly condensation of lamellae (single elementary silicate sheets) occur giving thicker clay particles. The dual effects of reduction in diffuse double layer thickness and formation of quasicrystals increase the effective pore space for water flow, yielding high k values.

Replacement of monovalent sodium by potasium ions leads to approximatelly 5 fold increase in the permeability coefficient of montmorillonite at a given void ratio. The influence of cation size on the permeabillity of montmorillonite is attributed by Sridharan, Rao and Murthy (1986) to the higher absorption of monovalent

potassium ions in the Stern layer and partial fixation of the cation in the hexagonal oxygen holes in the surface slicate layer which leads to greater reduction in diffuse double layer thickness (in comparison to sodium bentonite), increasing the effective void space for water flow and leading to higher permeability coefficient.

In comparison to the effect of exchangeable cations, increasing the pore salt concentration from 0.001N to 0.1N (salt solution = sodium chloride) leads to a marginal 1.25 times increase in the permeability coefficeint of the montmorillonite clay specimen, evidently brought about by the suppression of diffuse double layer thickness by the higher pore salt concentration.

When compared to the action of exchangeable cations and pore salt concentration, the effect of dielectric constant of the pore medium on the permeability coefficient of montmorillonite is several folds pronounced. For example, at a given void ratio, montmorillonite specimens containing ethanol (dielectric constant = 25.0) and carbon tetrachloride (dielectric constant -= 2.24) as the pore fluids exhibit permeability coefficients 10^4 to 10^6 times higher than clay specimens having water (dielectric constant = 80.4) as the pore fluid. A decrease in dielectric constant of the pore fluid acts to contract the diffuse double layer and flocculate the clay fabric with creation of large interped macropores, responsible for the dramatic increase in the permeability coefficient of the clay specimen.

ENVIRONMENTAL GEOTECHNIQUE

A logical extension to the studies on fundamental mechanisms governing the engineering behaviour of clay soils is their application to environmental geotechnical aspects of soil-inorganic contaminant interactions. This is essentially so as an understanding of the modification of soil mechanical properties on chemical contamination calls for an in -depth elucidation of the soil-contaminant interaction process.

Modification of soil mechanical properties of soils from chemical contamination form an important aspect of environmental problems relating to geotechnical engineering. Case histories of foundation failures, structural damage to light industrial buildings on contamination of subsoils by chemicals pollutants (Lukas *et al.* 1972, Sridharan *et al.* 1981, Kumapley and Ishola, 1985) emphasize the importance which needs to be attached to the consideration of the above problem.

Environmental geotechnics is realized to be more of a planning and decision tool to be used for forecasting geotechnical problems that may ultimately arise rather than solving after the event has occurred. Such a requirement calls for a fore-knowledge of the effects of industrial contaminants on the geotechnical properties

TABLE 23

INFLUENCE OF EXCHANGEABLE CATION TYPE AND PORE FLUID CHARACTERSTICS
ON PERMEABILITY COEFFICIENT OF MONTMORILLONITE

| Inorganic cations			
(a) Valency effect	Montmorillonite	$\dfrac{K_{ca}^{2+}}{KNa+}$	=28 at e = 2.5
	Bentonite	$\dfrac{K_{ca}2+}{KNa+}$	=8 at e = 2.0
		$\dfrac{KFe^{3+}}{KNa+}$	=33 at e = 2.0
(b) Cation size effect	Bentonite	$\dfrac{KK+}{KNa+}$	= 5 at 3 = 2.0
Concentrated salt		$\dfrac{K0.1N\ NaCl}{k0.001N\ NaCl}$	$=10^4$ at e = 2.0
Solution	Montmorillonite		
Organic Solvents	Montmorillonite	$\dfrac{Kethyl\ alcohol.}{K\ water}$	$=10^6$ at e = 2.0
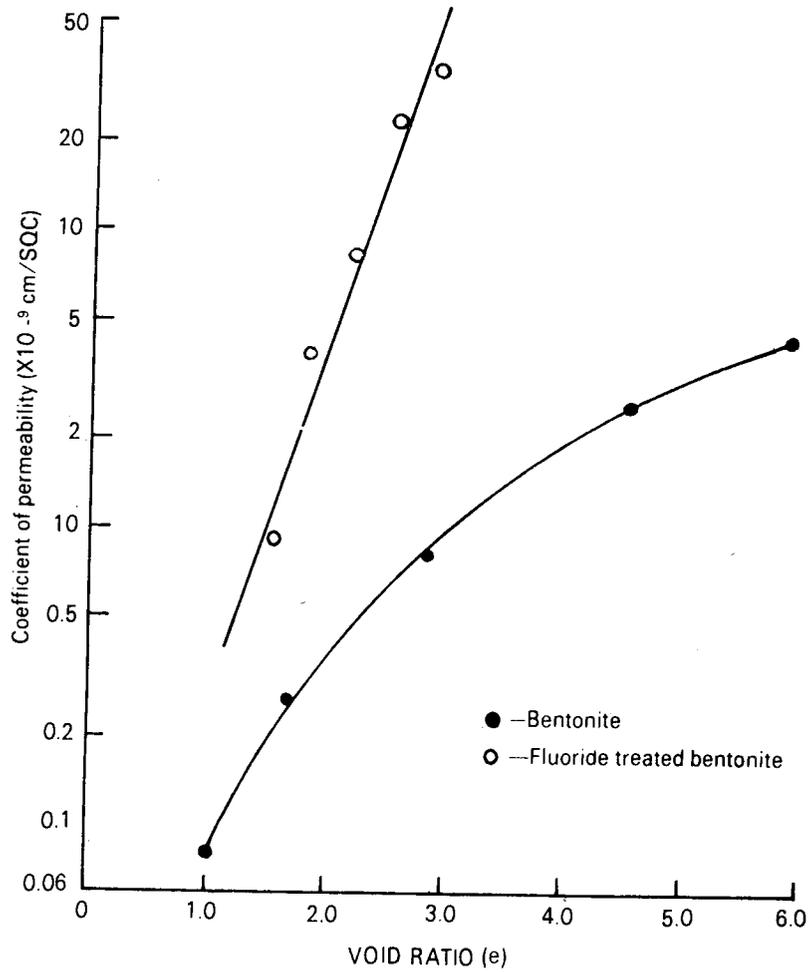
FIGURE 65 Effect of Fluoride on Coefficient of Permaability of Bentonite
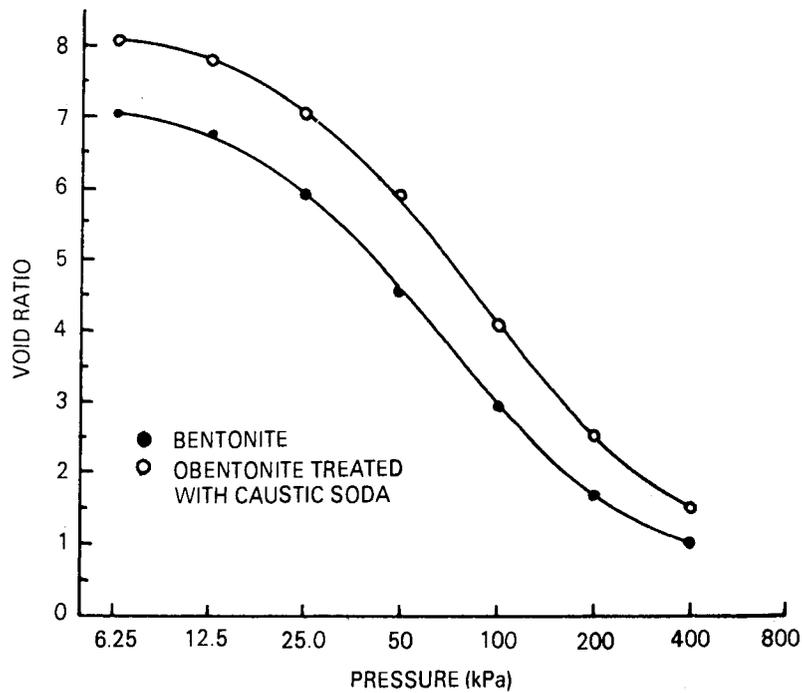
FIGURE 66 Void Ratio—Pressure Relations for Bentonite and Caustic Soda Treated Bentonite Specimens

of soil materials; the contaminants include inorganic and organic solutes in aqueous solutions and organic fluids. Although the role of inorganic cations and organic fluids in influencing geotechnical properties of clay soils has been the consideration of several workers (for example, Salas and Serratosa, 1953; Mesri and Olson, 1971 Sridharan and Rao 1973, Fernandez and Quigley, 1985; Sridharan, Rao and Murthy, 1986) insufficient information exists on the action of inorganic anions on mechanical properties of soil. Sustained research efforts have, therefore, been directed towards highlighting the impact of inorganic anions on the engineering behaviour of clay soils.
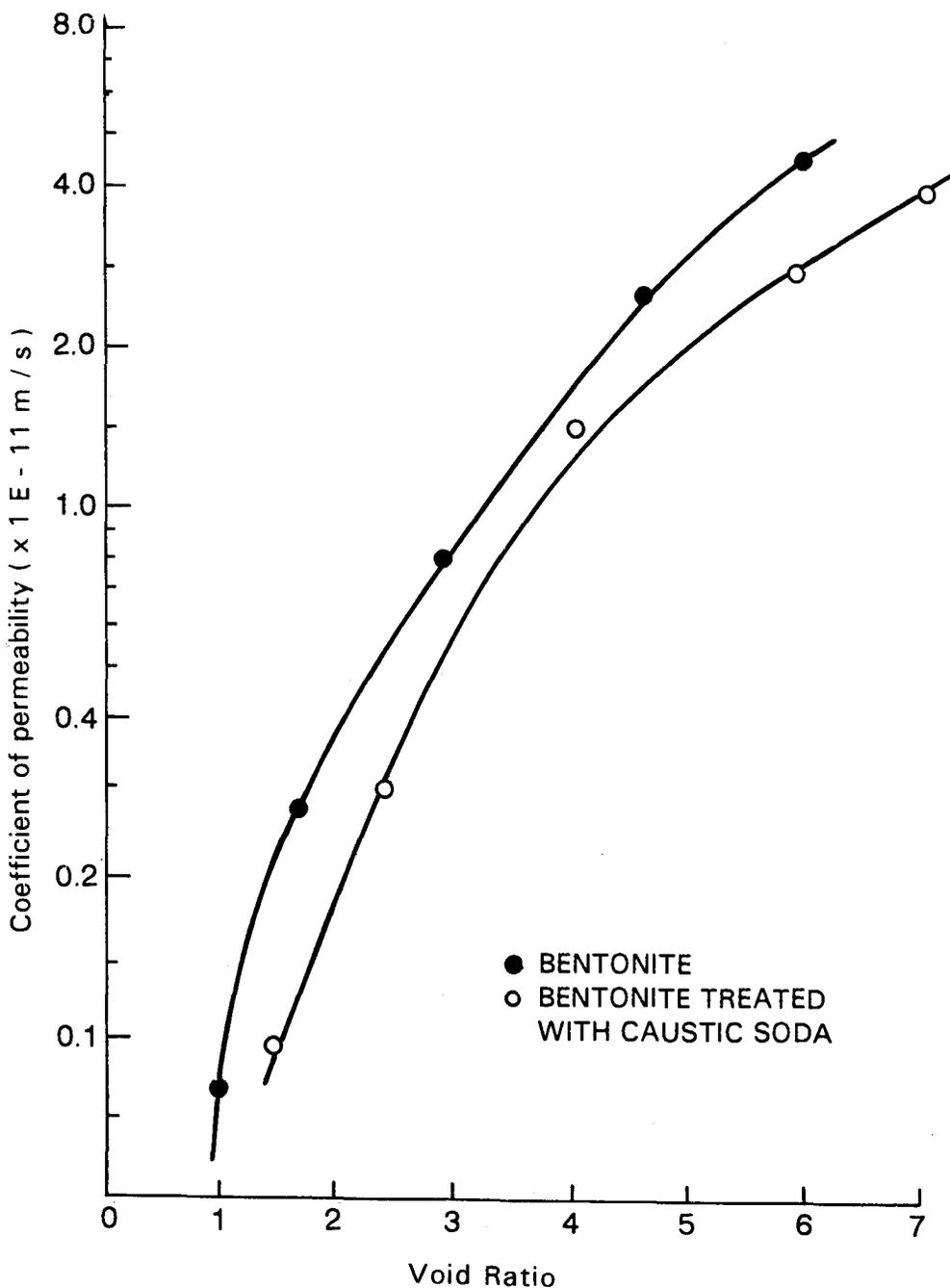
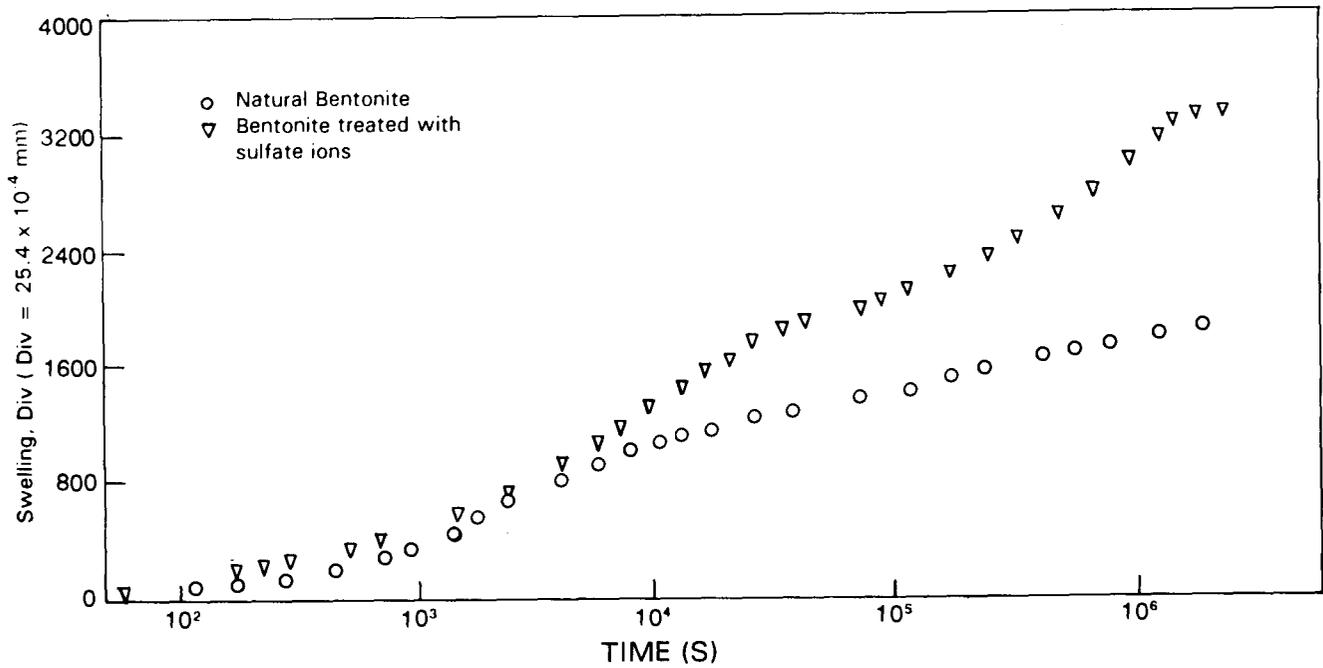FIGURE 67 Coefficient of Permeability versus Void Ratio Plots for Caustic Soda Treated Bentonite and Kaolinite

FIGURE 68 Effect of Sulfate Anion on Time-Swelling Curves of Bentonite
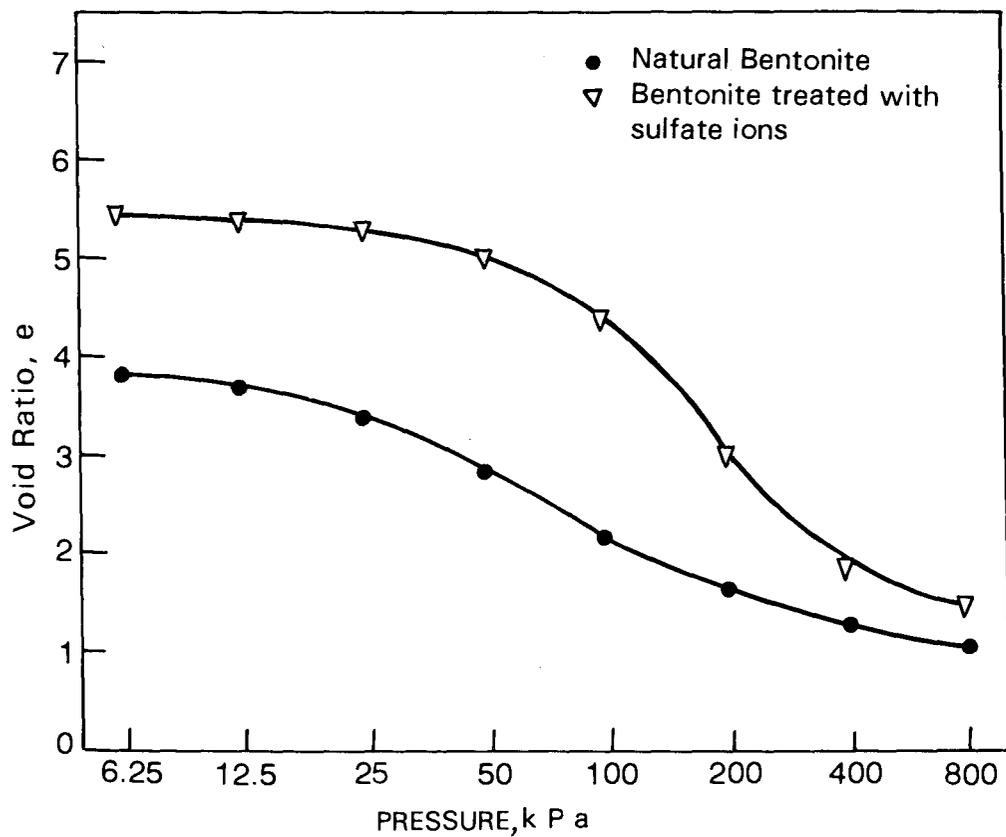

FIGURE 69 Effect of Sulfate Anion on Compression Behaviour

ROLE OF INORGANIC ANIONS ON THE ENGINEERING BEHAVIOUR OF CLAY SOILS -

Soils can be contaminated by inorganic anions from the following sources. Significant amounts of sulfate ions infiltrate into the soil as a result Sulfate polluted acid rain precipitation. Flouride ion is a potentially hazaradous element that can be depostied on a soil from such sources as phosphate fertilizers, sewage sludge and air-borne emissions from aluminium smelters, phosphate processing plants and enamel factories. Hydroxyl anion (in caustic soda form) is a common reagent used in several industrial processes and can contaminate the soil through accidental spillage and leakages.

Detailed investigations have been carried with regard to the impact of flouride, hydroxyl and sulfate anions on the geotechnical properties clay of soils (Sridharan, Rao and Gajarajan 1987a, b, 1990) and the salient findings are presented.

Treatment of a montmorillonite clay soils specimen with flouride anions results in a reduction in the diffuse double layer thickness and the associated repulsion forces leading to a lower void ratio at any pressure increment (Fig. 64) as well as an average four fold decrease in the compression index values. Treatment with fluoride significantly increased the permeability of the montmorillonites (Fig. 65). Flouride treatment decreases the diffuse double layer thickness and facilitates the formation of larger pores. As these pores are relatively less constricted by the contracted diffuse double layers they are more open for water flow and so result in higher permeability coefficients.

Conventional oedometer test results showed that on hydroxyl anion treatment, the bentonite clay sustains the external load at higher equilibrium void ratio (Fig. 66) as a consequence of enhanced double layer repulsion. Results of the 'free-swell oedometer test results' showed a marked increase in the magnitude of maximum swelling and swelling pressure caused by the increase in diffuse double layer replusion forces. At a given void ratio (Fig. 67), the hydroxyl anion treated specimen exhibits a lower permeability coefficient than untreated bentonite. Hydroxyl anion treatment increases the net negative charges of clay particles enhancing particle dispersion, resulting in narrower pores between particles and a mure torutous flow path. Further, the available channels would be relatively more constricted by the expanded diffuse double layers (in comparision to the untreated specimen), reducing the effective void space for water flow. Apparently, the increased tortousity factor and greater constriction of flow channels cause lower k values for the hydroxyl anion treated bentonite specimen.

The swelling magnitude and swelling pressure of bentonite clay markedly increase on sulfate anion adsorption (Fig. 68). It is likely that, for the anion treated clay, besides the growth of diffuse double layer, water is imbibed by entrapment in the large void spaces of its flocculated structure, thus explaining the higher swelling magnitude of the anion treated clay. Further, it is considered that the higher degree of particle interference associated with the flocculated fabric of the anion treated clay resists particle re-arrangement under external stresses more effectively than the oriented array of the natural clay, accounting for the larger swelling pressure of the former specimen. The greater particle interference associated with the sulfate clay presumably is responsible for its consolidation curve being postioned above that of the natural clay (Fig. 6).

The de/d (log p) versus pressure plots indicated higher compressions for the anion-treated clay at any given pressure increment. Possibly the anion treated clay with a flocculated fabric undergoes greater particle re-orientation under a given load, explaining its high compressibility. The similar ranges of permeability coefficients observed for the natural and anion treated clays suggested that the changes in clay fabric brought about by sulfate adsoption are not significant enough to influence the permeability of bentonite clay. Thus the result presented bring out that an understanding of the fundamental mechanisms governing the engineering behaviour of soils is essential for solving problems related to environmental geotechnical aspects.

CONCLUDING REMARKS

The great variety and complexity observed in the engineering behaviour of clay soils is attributable to their clay mineral constituents. The clay mineral particles are characterized by diffuse double layer induced repulsive forces, R and van der Waals and Coulombic attractive forces, A. the diffuse double layer repulsive forces are strongly dependent on the surface area of the clay mineral, type and concentration of exchangeable cations, dissolved ion concentration and dielectric constant of the pore medium and inter-particle separation. The Gouy-Chapman theory of electrical diffuse double layer enables qualitative prediction of repulsive pressure between clay platelets and in some instances quantitatively. The attractive forces are also strongly dependent on inter-particle separation, dielectric constant of the pore medium and the type of exchangeable cations present. The attractive and repulsive forces determine the geometric arrangement of clay particles, termed as clay fabric. The inter-particle forces and clay fabric (termed as soil structure) have a profound influence on the engineering

properties of clay soils.

The classical effective stress concept does not consider the electrical attractive and repulusive forces and hence cannot describe the engineering behaviour of clay soils correctly. The modified effective stress eqn., $\sigma' = \sigma - u - R + A$ which accounts for the A and R forces is successful in explaining the behaviour of clay soils, which is verified experimentally.

With the swelling smectites characterized by strong R and weak A forces, the liquid limit, swell potential, swelling pressure and permeability characterstics are influenced by the electrical diffuse double layer forces. Thus a decrease in exchangeable cation valency, pore salt concentration or an increase in dielecctric constant of the pore medium expands the diffuse double layer thickness resulting in increase in consistency limits, swell potential and swelling pressure of the montmorillonite clay. Concomitantly, the decrease in 'effective' stress on enhanced inter-particle repulsion leads to higher compressibility and lower drained shear strength of the swelling montmorillonitic clay. The increase in diffuse double layer repulsion also mobilizes an oriented fabric that generates greater capillary stresses and provides for relatively easier movement of particles during drying, leading to lower shrinkage limits. The diffuse double layer associated with the clay surfaces also constricts the void space thus reducing the effective area for fluid-flow and leading to low permeability coefficients. Besides, the dispersed particle arrangement mobilized by diffuse double layer repulsion increases the tortuosity of flow path and also provides for low permeability coefficients.

With the non-swelling kaolinite, characterized by strong A and weak R forces, the clay fabric as determined by inter-particle forces and manifested as shearing resistance at inter-particle contacts have a great bearing on the consistency limits, shrinkage, compressibility and shear strength characteristics. A decrease in dielectric constant of the pore medium enhances the inter-particle attraction; the concomitant increase in the extent of particle flocculation provides greater void space for fluid entrapment leading to higher liquid limits. An increase in the extent of particle flocculation leads to lower shrinkage as the random fabric mobilizes lower capillary stresses and provides greater interference to particle re-arrangement (from increase in effective stress) during drying. The increase in effective stress on enhanced inter-particle attraction (brought about by decrease in dielectric constant of the pore medium) also leads to lower compressibility and higher drained strength of the non-swelling clay.

The basic concepts developed in explaining the engineering behavior of clay soils proved immensely useful in explaining the modification of soil mechanical properties on soil-inorganic contaminants interactions.

ACKNOWLEDGEMENT

REFERENCES

ALLAM, M.M. and SRIDHARAN, A. (1979) : The Influence of Aging on the Shear Strength Behaviour of two Fine Grained Soils" Canadian Geotech. Jl., 16 : 2.

ALLAM, M.M. and SRIDHARAN, A. (1981): "Effect of Wetting and Drying of Shear Strength"Jl. of the Geotech. Engg. Divn., Proc. ASCE, 107:GT4:421-438.

ALLAM, M.M. and SRIDHARAN, A. (1984a) : "Shear Strength Behaviour of Desiccated Soils", Indian Geotech.Jl., 14 : 1:40-66.

ALLAM, M.M. and SRIDHARAN, A.,(1984b) : "The Shearing Resistance of Saturated Clays", Geotechnique, 34:1: 119-122.

ALLAM, M.M. and SRIDHARAN, A. (1987a) : "Effective Stress in Clays", Indian Geotech. Jl., 17:3: 259-267.

ALLAM, M.M. and SRIDHARAN, A. (1987b) · "Stresses present in Unsaturated Soils" Geotech' Engg. Divn., proc. ASCE, 113:11: 1395-1399.

ATKINSON, G.E and McBRYDE, W.A.E., (1961) : "Graphical Representation of Hydrolysis of Ferric Ions ' Jl Chem Educ, 48: 127-128.

BLACK MORE, A.V. (1973) : "Aggregation of Clay by the Products of Iron (III) Hydrolysis", Aust. J. Soil Res.11.75-82

BOLT, G.H.,(1955) : "Analysis of the Validity of the Guoy-Chapman Theory of the Electric Double Layer". jl. Coll. Sci., 10: 206-218.

BOLT, G.H. (1956) :" Physico-Chemical Analysis of the Compressibility of Pure Clays." Geotechnique, 6:2: 86-93.

BOLT G.H. (1976): "Surface Interactions between the Soil Solid Phase and the Soil Solution" Soil Chemistry A. Basic Elements, G.H. BOLT, and M.G.M. BRUGGENWERT Eds., Elsevier, New York, p 281.

FERNANDEZ, F. and QUIGLEY, R.M., (1985) : "Hydraulic Conductivity of Natural Clays Permeated with Simple Liquid Hydrocarbons", Canadian Geotech. Jl 22: 205-214.

FOWKES, F.M. (1964) : "Attractive Forces at Interfaces", Industrial and Engineering Chemistry, 56 : 12 : 40-52.

GARICIA-BENGOCHEA, I. LOVELL, C.W.. and ALTSCHAFFL, A.G. (1979) : "Pore Distribution and Permeability of Silty Clays". ASCE Jl. of the Geotech. Engg. Divn., 105:7: 839-856.

GIBBS, H.J. (1969) : "Discussion, Proc. of the Speciality Session No.3 on Expansive Soils and Moisture Movement in Partially Saturated Soils" Proc. Seventh Int. Conf. Soil Mech. and Foudn. Engg., Mexico City, 179-494.

HAMAKER, H.C., (1937) : "The London Vander-Waal, Attraction between Spherical Particles", Physica, 4, 1058-1072.

JAYADEVA, M.S. and SRIDHARAN, A. (1981): "Prediction of Void Ratio-Osmotic Pressure Relationship of Clays" Jl. of Indian Institute of Science, India, 63 (A) : 109-122.

JAYADEVA, M.S. and SRIDHARAN, A. (1982) : "A Study on Potential-Distance Relationship of Clays", Indian Geotech. Jl., 12 : 1 : 83-97.

JENNINGS, J.E. and BURLAND, J.B. (1962) : "Limitations of the Use of Effective Stress in Partly Saturated Soils" Geotechnique, 12 : 2 : 125-144.

KARDOS, L.T. (1964): "Soil Fixatioh of Plant Nutrient", Chemistry of the Soil (ed F.E. BEAR), New York : Reinhold.

KENNY, T.C., MOUM, J. and BETTE, T. (1967) : "An Experimental Study of Bonds in a Natural Clay", Proc. Geotechnical Conf., Oslo, 65-69.

KRISHNA MURTHY, M. SRIDHARAN, A. and NAGARAJ, T.S. (1980): "Effect of Aging on Shear Behaviour of Overconsolidated Clays", Indian Geotech. Jl., 10 : 4 : 368-373

KRISHNA MURTHY, M. SRIDHARAN, A. and NAGARAJ, T.S. (1987) : "Shear Behaviour of Partially Saturated Soils," Indian Geotech. Jl., 17:2:143-158,

KUMAPLEY, N.K. and ISHOLA, A., (1985) : "The Effect of Chemical Contamination on Soil Strength," Prop. Eleventh Int. Conf. Soil Mech. Found. Engg., San Franscisco, C, 1199-1201.

LAMBE, T.W. and WHITMAN, R.V. (1969) : "Soil Mechanics", John Wiley, New York, p 554.

LAMBE, T.W., (1953) : "The Structure of Inorganic Soil", ASCE, 316.

LAMBE, T.W. (1958) : "The Structure of Compacted Clay", Jl. Soil mech. and Foundn. Divn., ASCE, 84 :SM2:1654.

LAMBE , T.W.(1960a) : "A Mechanistic Picutre of "Shear Strength in Clays", Proc. Research Conf. Shear Strength of cohesive soils, ASCE, Boulder, Colorad, 555-580.

LAMBE, T.W. (1960b) : "Discussion on factors Controlling the Strength of Partly Saturated Cohesive soils", Proc. of the Research Conf. of Shear strength of Cohesive soils, ASCE Boulder, Colorado, 1094.

LEONARDS, G.A. and ALTSCHAEFFL, A.G.(1964) : "Compressibility of Clays", Jl. Soil Mech. and Foundn. Engg. Divn., ASCE, 90 : SM5: 133-155.

LONDON, V.E. (1930) : "Zur theorie Und Systematic der Molekularkrafte", Zeitschrift fur Physik, 63: 245-279.

LOW, P.F. (1980) : "The Swelling of clay. II. Montmorillonites", Jl. Soil. Sci. Soc. Am, 44: 4: 667-676.

LUKAS, R.G. and GNAEDINGER. JR. R.J. (1972) : "Settlement due to Chemical Attack of Soils", Proc. ASCE Speciality Conf. on the Performance of Earth and Earth-supported Structures, Purdue Univ., Lafayette, Indiana, 1:1087-1104.

MESRI, G. (1973) : "Coefficient of Secondary Compression", Jl. Soil Mech. and Foundn. Engg. Divn; ASCE, 99 : SM1: 123-137.

MESRI, G. and OLSON, R.E. (1971) : "Consolidation Characterstics of Montmorillonite", Geotechnique,21 : 4 : 341-352.

MITCHELL, J.K. (1960) : "The Application of colloidal Theory to the Compressibility of Clay. Interparticle Forces in Clay-Water-Electrolyte Systems". Melbourne, CSIRO: 2:92-2.98.

MITCHELL, J.K. (1956) : "On the Fabric of Natural Clays". Proceedings of H.R.B. 35.

MITCHELL, J.K. (1976): "Fundamentals of Soil Behaviour", John Wiley, New York, p 422.

NAGARAJ, T.S. and SRINIVASA MURTHY, B.R.(1983a) : "A Critical Reappraisal of Compression Index-Equations", Geotechnique, 36 : 1 : 27-32.

NAGARAJ, T.S. and SRINIVASA MURTHY. B.R. (1983b) : "Rationalization of Skemption's Compressibility Equation" Geotechnique, 33 : 4 : 433-443.

OLSON, R.E. and LANGFELDER, L.J. (1965) : "Pore Water Pressure in Unsaturated Soils", Jl. soil Mech. and Foundn. Engg. Divn., Proc. ASCE, 91 : SM 4:127-150.

OLSON, R.E. and MESRI, G. (1970) : "Mechanisms controlling Compressibility of Clay". Jl. Soil Mech. Fdns. Divn; ASCE, 96 : SM 6: 1863-1878.

QUIRK, J.P. (1960) : "The Role of Interparticle Forces in Soil Structure", Conf. Interparticle Forces in Clay-Water-Electrolyte System, CSIR, Melborne, 2/1-2/8.

RAO, G.V. and SRIDHARAN, A.(1978) : "Strength Behaviour of Partially Saturated Clays", Proc. of GEO-CON, IGS, Conf. on Geotechnical Engg., New Delhi, Dec. 20-22, 1978.

RAO, S.M. and SRIDHARAN, A.(1985) :."Mechanism Controlling the Volume Change Behaviour of Kaolinite". Clays and Clay Minerals 33 : 4,323-328.

RAO, S.M. and SRIDHARAN, A.(1987): "The Performance of Clay Liners on Chemical Permeation". Proceedings, Indain Geotechnical Conferences, 1 : 405-408.

RAO, S.M. SRIDHARAN, A. and CHANDRAKARAN, S.(1989): "Influence of Drying on the Liquid Limit Behaviour of Marine Clay". Geotechnique, 39 : 4 : 715-719.

RENDEN-HERRERO, O. (1980) : "Universal Compression Index Equation". Jl. Geotech. Engg. Divn. ASCE, 106 GT11: 1179-1199.

ROSENQVIST, I,Th. (1955) : "Investigations in the Clay-Electrolyte-Water System". Publication No. 9, Norwegian Geotech. Society, Oslo.

SALES, J.A.J. and SERRATOSA,J.M. (1953) : "Compressibility of Clays", Proc. Third. Int. Conf. on Soil Mech. Foundn. Engg., Zurich, 1: 192-198.

SCHOFIELD, R.K. (1946) : "Ionic Forces in Thick Films of Liquid between charged Surfaces". Trans. Faraday Society, 42b : 219.

SEED, H.B., MITCHELL, J.K. and CHAN, C.K.(1960) : "The Strength of Compacted Cohesive Soils". ASCE Proc. Res. Conf. Shear Strength of Cohesive Soils, Boulder, Colorado, 877-961.

SEED, H.B., WOODWARD, R.J. and LUNDGREN,R. (1964) : "Clay Mineralogical Aspects of the Atterberg Limits".Jl. Soil Mech. and Foundn. Engg. Divn., Proc., ASCE, 90 : SM4 : 107-131.

SHAINBERG, I. and KEMPER, W.D.(1966) : "Hydration Status of Adsorbed Cations", Proc. Am. Soil Sci. Soc., 30: 707-713.

SIVAPULLAIAH, P.V. and SRIDHARAN, A.(1985) : "Liquid Limit of Soil Mixtures", Geotech. Testing Jl. ASTM, 8 :3:111-124.

SKEMPTON A.W. (1953) : "The Colloidal Activity of Clays", Proc., Third Int. Conf. Soil Mech. and Foundn. Engg., 1: 57-67.

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

SRIDHARAN, A. and ALLAM, M.M. (1982): "Volume Change Behaviour of Dessicated soils". Jl. of the Geotech. Engg. Divn., Proc. of the ASCE 108, GT 8 : 1057-1071.

SRIDHARAN, A. ALTSCHAEFTL, A.G. and DIAMOND, S. (1971) : "Pore Size Distribution Studies". Jl. of the ASCE, SM Divin. 97:5:771-787.

SRIDHARAN, A. and JAYADEVA, M.S.(1980) : "Approximate Potential Distance Relationship". Australian Jl. of Soil Research,18 : 461-466.

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

SRIDHARAN, A. and NARASIMHA RAO, S. (1972) : "A New Approach to Multistage Triaxial Tests". Proc. ASCE Jl. of SM and FE,98 :11: 1279-1286.

SRIDHARAN, A. and NARSIMHA RAO, S.(1972) :"Hyperbolic Representation of Strength, Pore Pressure and Volume Changes with Axial Strain in Triaxial Tests".Proc., Symp. on Strength and Deformation Behaviour of Soils, Bangalore, Vol. 1.

SRIDHARAN, A. and RAO A.S.(1982): "Mechanisms controlling the Secondary Compression of Clays". Geotechnique, 32 : 2:249-260.

SRIDHARAN, A. and RAO G.V.(1971): "Effective Stress Theory of Shrinkage Phenomena". Canadian Geotech. Jl. 8 : 4 : 503-513.

SRIDHARAN, A. and RAO G.V.(1973): "Mechamisms Controlling the volume Change Behaviour of Saturated clays and the Role of the Effective Stress Concept".Geotechnique, 23 : 3 : 359-382.

SRIDHARAN, A. and RAO G.V.(1975) : "Mechanisms Controlling the Liquid Limit of Clays". Proc of Istanbul conf. on Soil Mech. and Foundn. Engg., 1: 75-84.

539

SRIDHARAN, A. and RAO G.V.(1979) : "Shear Strength Behaviour of Saturated clays and the Role of the Effective Stress Concept", Geotechnique, 29 :2. 177-193.

SRIDHARAN, A. RAO, M.S. and MURTHY, N.S.(1985) : "Free Swell Index of Soils-A Need for Redefinition". Indian Geotechnical Jl., 15 : 2: 94-99.

SRIDHARAN, A. and RAO S.M. (1988) : "A Scientific Basis for the use of Index Tests in Indentification of Expansive soils", ASTM, Geotech. Testing Jl., 11 : 3 : 208-212.

SRIDHARAN, A, NAGARAJ, T.S. and SIVAPULLAIAH, P.V. (1981): "Heaving of Soil due to Acid Contamination". Proc., Tenth Int. Conf. Soil. Mech. and Foundn. Engg. Stockholm. 383-386.

SRIDHARAN , A, NARSIMHA RAO, S. and RAO, G.V.(1971): "Shear Strength charicterstics of Saturated Montmoril-lonite and Kaolinite Clays". Soils and Foundations, 11 : 3 : 1-22.

SRIDAHARN, A, RAO, A.S. and MAKAN, S.K. (1983) : "Shear Strength Behaviour of Expansive soils". Seventh Asian Reg. Conf. on Soil Mech. and Foundation Engineering, Technion, Israel Institute of Technology, Haifa, Israel, 81-83.

SRIDHARAN, A,, RAO, A.S. and SIVAPULLAIAH, P.V.(1986) : "Swelling Pressure of Clays", Geotech. Testing Jl., ASTM, 9 : 1,24-31.

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

SRIDHARAN, A, RAO, S.M. and CHANDRAKARAN, S. (1989) : Influence of Drying on the Liquid Limit behaviour of a Marine Clay, Geotechnique vol 39, No 4, pp 715-719.

SRIDHARAN, A, RAO, S.M. and GAJARAJAN, V.S.(1987) : "Effect of Caustic Soda Contamination on Volume Change Characteristics of Bentonite". Proc Int. Sym. on Prediction and Performance in Geotechnical Engineering, Calgary, 347-353.

SRIDHARAN, A,RAO, S.M. and GAJARAJAN, V.S.(1987): "Influence of Fluoride on the Compressibility of Montmoril-lonites". Geotechnique, 37 : 2 : 197-206.

SRIDHARAN, A, RAO, S.M. and GAJARAJAN, V.S.(1990) : "The Effect of Sulphate Contamination on the Volume Change Behaviour of Bentonite". Physico-Chemical Aspects of Soil and Related Materials, ASTM STP 1095, K.B. HODDINOTT AND R. P. LAMB, Eds. ASTM, Philadelphia, 60-68.

SRIDHARAN, A,RAO, S.M. and MURTHY, N.S.(1986a) : "A Rapid Method to Identify Clay Type in Soils by the Free-Swell Technique", Goetech. Testing Jl., 9 : 4: 198-203.

SRIDHARAN, A, RAO, S.M. and MURTHY, N.S.(1986b): "Compressiblity Behaviour of Homoionized Bentonites" Geotechnique, 36 : 551-564.

SRIDHARAN, A, RAO, S.M. and MURTHY, N.S.(1986c) : "Liquid Limit of montmorillonite soils", ASTM, Geotech. Test. Jl., 9 : 3 :156-159.

SRIDHARAN, A, RAO, S.M. and MURTHY, N.S.(1988): "Liquid Limit of Kaolinitic Soils", Geotechnique, 38: 2: 191-198.

STERN, O. (1924): "On the Theory of Electric Double Layer", Z. Elektrochem. 30: 509-527.

TAYLOR, A.W. (1959) : "Physico-Chemical Properties of Soils: Ion Exchange Phenomena". Jl. Soil Mech. Foundn. Divn. Proc. ASCE, 85 : 2 : 19-30.

TERZAGHI, K. (1923) : "Die Berechnung der Durchlassigkeitsziffer des Tones aus dem verlauf der Hydrodynamisthen" Spannungser-Scheinumgen Sitz, Akad, Wissen, Wien Mathnaturw Kl. Abt. IIa, 132 : 105-124.

TERZAGHI, K. (1929) : "Technisch-geologische Beschreibung der Bodtenbeschaffenheit fur bautechnische Zweche", Chapter 9, Part'A, Ingenieurgeologie by K. A. REDKICH, K. TERZAGHI and R. KAMPE, WIEN and BERLIN, Springer.

VAN DER MERWE, D.H. (1964) : "The Prediction of Heave from Plasticity Index and the Percentage Clay Fraction of the Soils", The Civil Engineer, South African Institution of Civil Engineering, 6: 103-107.

VAN OLPHE, H. (1963): An Introduction to Clay Colloid Chemistry. Wiley, New York, p. 346.

VERWEY, E.J.W. and OVERBEEK, J.T. (1948) : Theory of the Stability of Lyophobic Colloids, Elsevier, New York.

YONG, R.H. and WARKENTIN, B.P. (1966) : "Introduction to Soil Behaviour". Macmillan, New York, p. 451.

YONG, R.N., CHANG, R.K. and WARKENTIN, B.P. (1969) : "Temperature Effect on Water Retention and Swelling Pressure of Clay Soils, Effect of Temperature and Heat on Engineering Behaviour of Soils". Spec. Rep. High. Res. Bd No. 103.

YONG, R.N. and WARKENTIN, B.P. (1975): "Soil Properties and Behaviour". Elsevier Scientific Publishing Co., Amsterdam, P. 450.