CHAPTER II

FACTORS AFFECTING CONSOLIDATION OF CLAYS

2.1. Physical Nature of Clay Material

Soil is a porous composite mass constituted of weathered mineral particles of various sizes, shapes and surfaces interpersed with liquid and gas molecules held together by physico-chemical forces. A soil is described as 'clay' when it is formed from colloidal products of weathering of a size lesser than 2 micron which are of nonspherical shape having predominant specific surface and possess a definite 'lattice structure'.

In a classical sense, the behaviour of colloids is governed by surface electrical charges. Owing to these electrica charges a minimum distance between the particles is established by the joint action of mass attractive (van der woals forces) and repulsive forces. Salas and Serratosa (1953) analysed the distance between clay particles under external loadsusing the Freundlich diagram (Figure 2.1) and concluded that at the equilibrium condition of forces this distance reaches a new minimum whose value is smaller than that under no load.

The nature of mineral surfaces is influenced by the constituents of pore space and affects the processes of force transmission. In case of clay-water system, the adhesional forces are mainly of Van der Waals' nature, perhaps of ionic and covalent characters. They depend on the mutual distance between mineral surfaces and adsorbed surface cations. Van der Waals attractive forces are opposed by the tendency of the ions to distribute themselves throughout the system. As a result, a diffuse type of distribution is formed and since the negative charges of the particle are seperated from the positive charges of the counter ions, the system is described as an 'Electric Double' Layer' (Figure 2.2). Physically a clay water system has been identified by Verowey and Overbekk (1948) as an Osmotic system. Rosenqvist (1959) inferred that the outer most layer of the mineral (water film) will yield plastically and little or no elastic stresses will arise at the area of contact.

In a clay mass these colloids are visualized by most research workers - Lambe (1953), Mitchell (1956), Tan (1957)- as arranged in a 'cardhouse' manner. The formation of edge to face (card house) structure is most probable under the normal conditions in which plate shaped mineral particles carry net attractive electrical charges as argued by Lambe (1960).

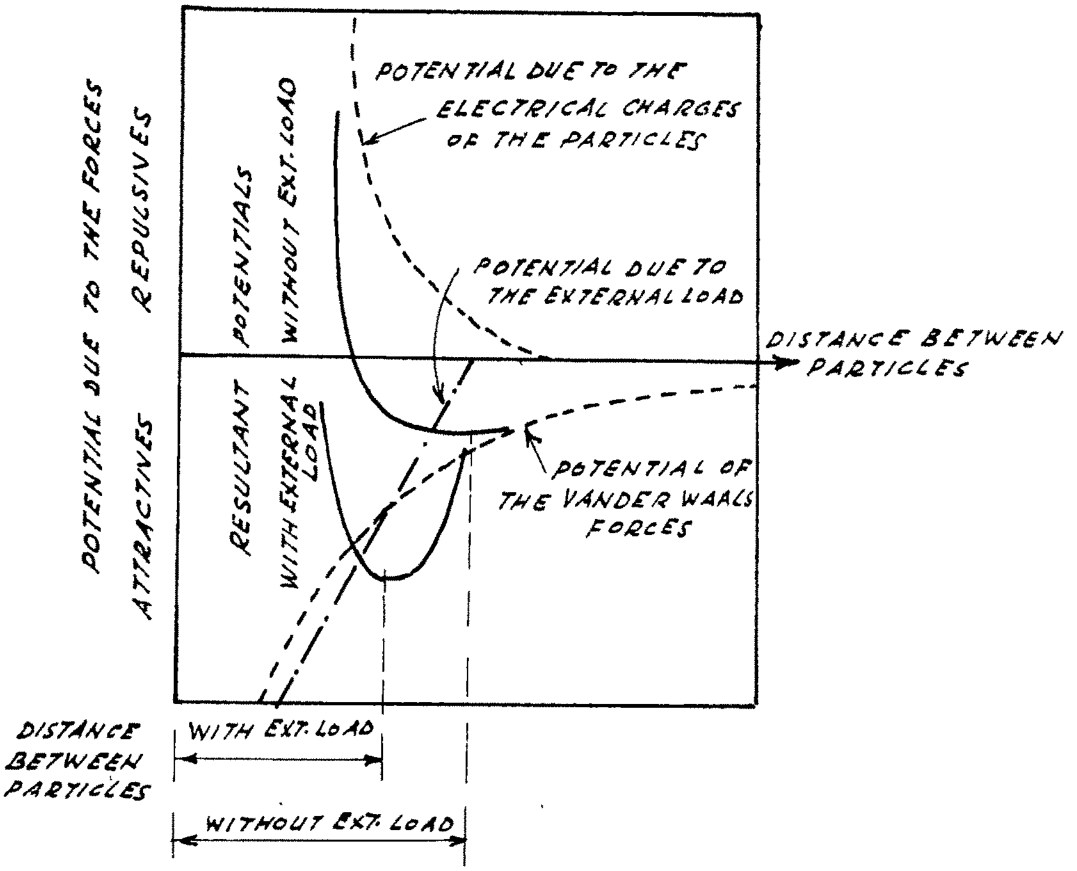


FIG. 2·1 FREUNDLICH DIAGRAM OF THE EQUILIBRIUM BETWEEN TWO PARTICLES OF CLAY

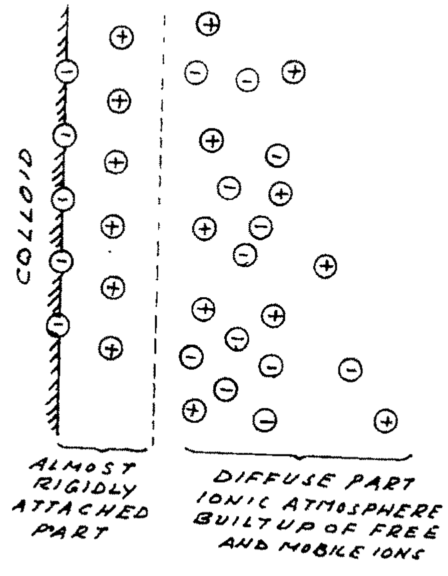


FIG. 2·2 DOUBLE LAYER

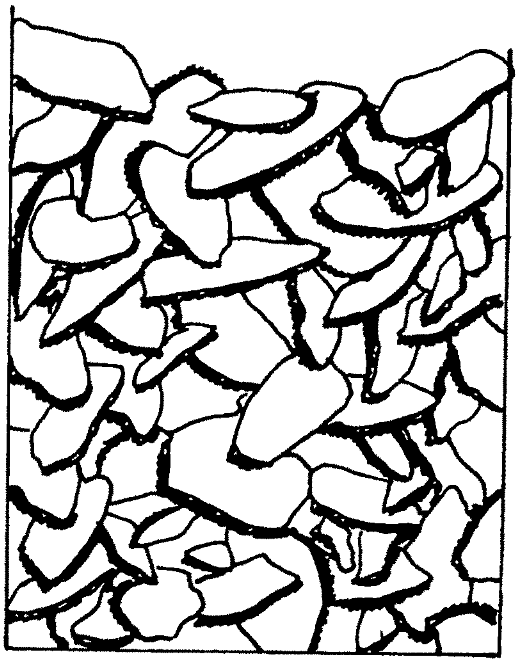


FIG. 2·3 SCHEMATIC PICTURE OF CLAY (AFTER TAN, 1957)

Tan (1957) presents a schematic picture of clay mineral net work as represented in Figure 2.3.

2.2. Consolidation Concepts

2.2.1.Mechanistic Concepts

Taylor (1942) was the first to suggest a modification to the idealized Terzaghi classical notion by invoking a concept of 'plastic structural resistance to compression' during the consolidation process. Recently, Barden (1965) interpreted this 'plastic resistance' as 'structural viscosity' of nonlinear nature. Accordingly, the total process of consolidation is seperated into primary consolidation and secondary compression. During primary consolidation, the total applied stress is shared by the pressure in the free pore fluid (water), by plastic resistance in the viscous adsorbed double layer (film) and by solid to solid contact between the particles (grain). During the secondary compression it is shared between the film and the grain bonds.

2.2.2. Physical Concepts

Distinct from the Terzaghi-Taylor mechanistic approach are those of Buisman (1936), Geuze (1948) and Bolt (1956). Buisman (1936) observed that the time delayed reaction of the soil skeleton is due to two distinct mechanisms, one referred as 'secular effect' resulting from viscous character of pore water binding the particles and the other is size difference of the pores through which water is expelled causing settlement. Buisman (1961) suggested a physical model of cavity channel net work to account for the secular effects. Geuze (1948) proposed the application of first law of Thermodynamics to evaluate the amount of consolidation from the amount of work done on the sample. The total work is spent partly in the displacement of soil particles and partly in the viscous resistance of pore fluid. While Bolt (1956) regards the clay water system as an osmotic system and indicated that a certain amount of water is pressed out as a result of imbalance in osmotic pressure of free water and adsorbed water.

2.2.3. Statistical Concepts

The statistical method of stochastic process has been also considered worth attempt for a highly complex behaviour of soil colloids under stress. Marshall (1965) put forth a model similar to that adopted for the 'Brownian' movement stating that the movements of particles are produced by unbalanced contact forces acting on them. These forces consist of a constant intergranular force and erratic contact forces whose resultaht vector changes in magnitude and direction with time. The use of stochastic process has also been employed by Josselin de Jong (1968) on a cavity model of Buisman (1941) consisting of a new net work of cavities of different compressibilities and channels of different conductivities for a consolidating soil.

2.3. Consolidation Behaviour of Clays

2.3.1. Clay Mineral Dependent

In general, clay particles are sheet mineral crystals built up from various combinations of silica, brucite and gibsite structures. Their laterial surfaces are oxides or hydroxides. The characteristics potential carried by virtually all clay minerals in aqueous media arises either due to isomorphous substitution within the crystal lattice as observed in three layered silicates of the montmorillonite and illite types or surface dissociation of hydroxyl ions as seen in two layered silicates such as kaolinites. The post effect of isomorphous substitution is expansion of lattice as seen in montmorillonite which is prevented in case of illites due to the presence of natural potassium. Investigations on clays predominantly composed of kaolinite, montmorillonite and illite minerals are worth a reference to understand their influence on the consolidation behaviour of clays.

One school notably of Samuels (1950), Cornell University (1951), Olson and Mitronovas (1960) and Crawford (1965) analysed the nature of adsorbed layer on the clay mineral and their influence on the consolidation characteristics of clays. In kaolinite and illite clays the adsorbed water molecules are weakly held on the edges and is imperfectly oriented behaving virtually like a free water so that the consolidation occurs primarily due to 'hydrodynamic lag' as imagined in the classical Terzaghi theory. But in case of montmorillonite clays the adsorbed water is strongly held due to electrostatic imbalance in the clay mineral and is perfectly oriented possessing a semi rigid structure which offers plastic resistance causing 'plastic lag' exhibiting larger secondary compression during consolidation. Experimental results of Samules (1950) are reproduced in figures 2.4., 2.5 and 2.6 in support of the arguments.

Another school is that of Bolt (1956), Olson and Mitronovas (1960), Bolt and Miller (1955), and Arnold (1965) who studied the behaviour on the basis of change in interparticle distance between colloids under the application of load. These studies reveal that the nature of variation in potential and consequent change in inter-particle distance is different in 'non-expanding' and 'expanding' clay minerals. In case of montmorillonite, the equilibrium is reached at smaller interparticle distance compared to that in kaolinite and illite minerals. Further it is observed from the tests on sodium montmorillonite that at interpafticle distance of the

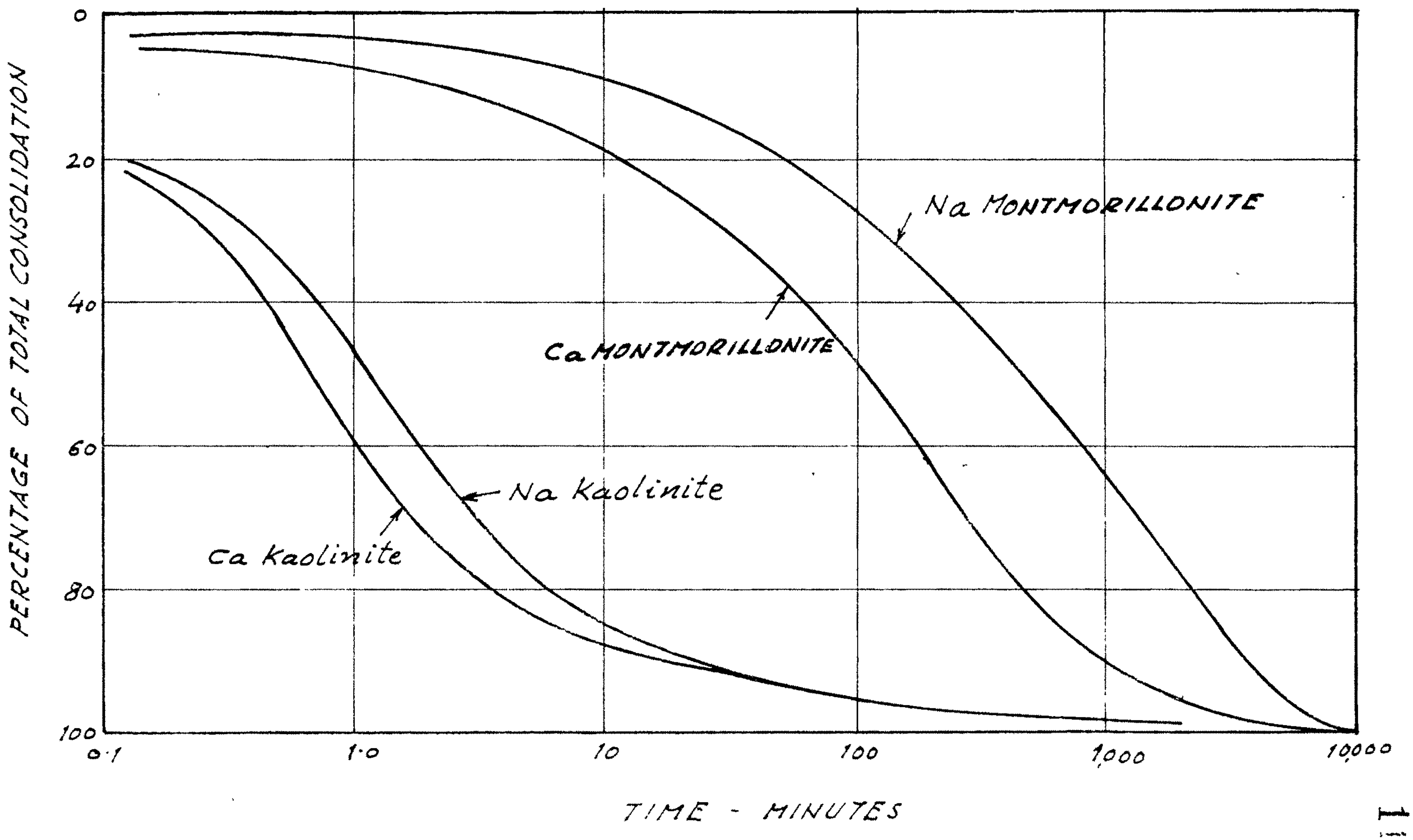


FIG. 2·4 RATE OF CONSOLIDATION OF MONTMORILLONITE AND KAOLINITE AT 4 TO 8 TONS/SFT, AFTER SAMUELS (1950)

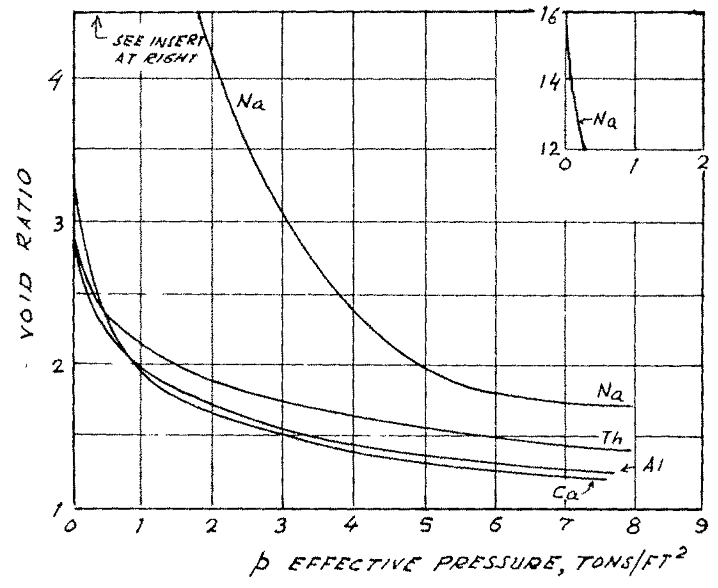


FIG. 2·5 PRESSURE-VOID RATIO CURVES FOR MONTMORILLONITE

AFTER SAMUELS (1950)

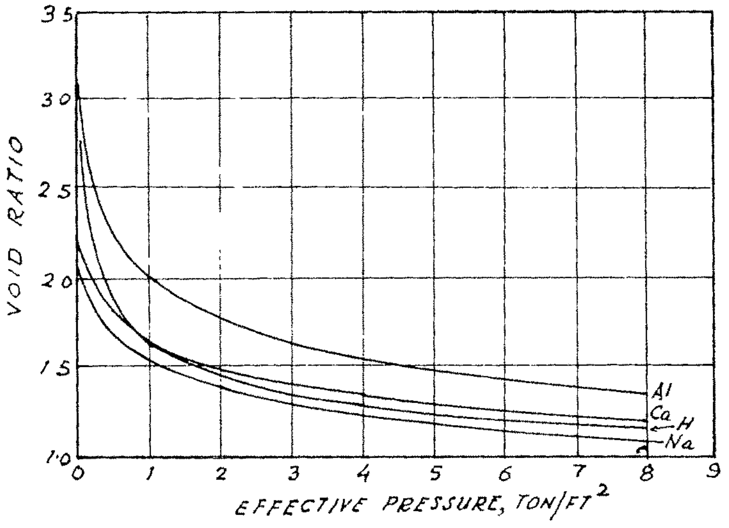


FIG. 2·6 PRESSURE-VOID RATIO CURVES OF KAOLINITE,

AFTER SAMUELS (1950)

order of $50A^{o}$, surface ion attractive forces expel water until it reaches a spacing of about $15\text{-}20A^{o}$. Under the above theoretical background Arnold(1965) presented the comparison between theory and experimental results as shown in Figures 2.7 and 2.8.

The characteristics of the pore fluid considerably change the nature of the adsorbed layer on the clay mineral and therefore many research workers investigated the effect of liquids on the consolidation characteristics of clays. Winterkorn (1941) presented a study on the influence of exchange ions on consolidation rates of Putnam clay from which it is inferred that co-efficient of consolidation is the smallest for the Na-ion and largest for the K-ion (Figure 2.9). Salas and Sarratosa (1953) working on Organic liquids of various polarity showed that secondary compression becomes prominent as the polar momentum decreases (Figures 2.10, 2.11 and 2.12) Olson and Mitronovas (1960) studied a range of electrolyte concentration from 1N to 10^{-4}N and found that the compressibil decreases as concentration diminishes (Figures 2.13 and 2.14). Bolt and Miller (1955) and Bolt (1956) produced a similar experimental work based on the measurement of interparticle distance between colloids interspersed with water mixed with various concentrations of sodium chloride (Figures 2.15 and 2.16).

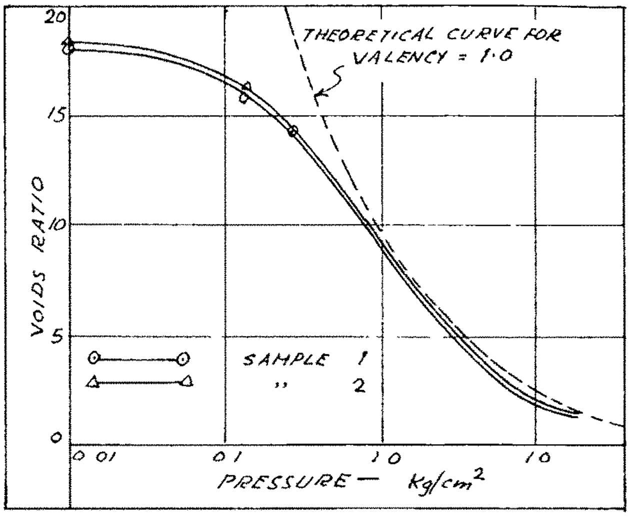


FIG. 2·7 CONSOLIDATION CHARACTERISTICS OF SODIUM BENTONITE (AFTER ARNOLD, 1965)

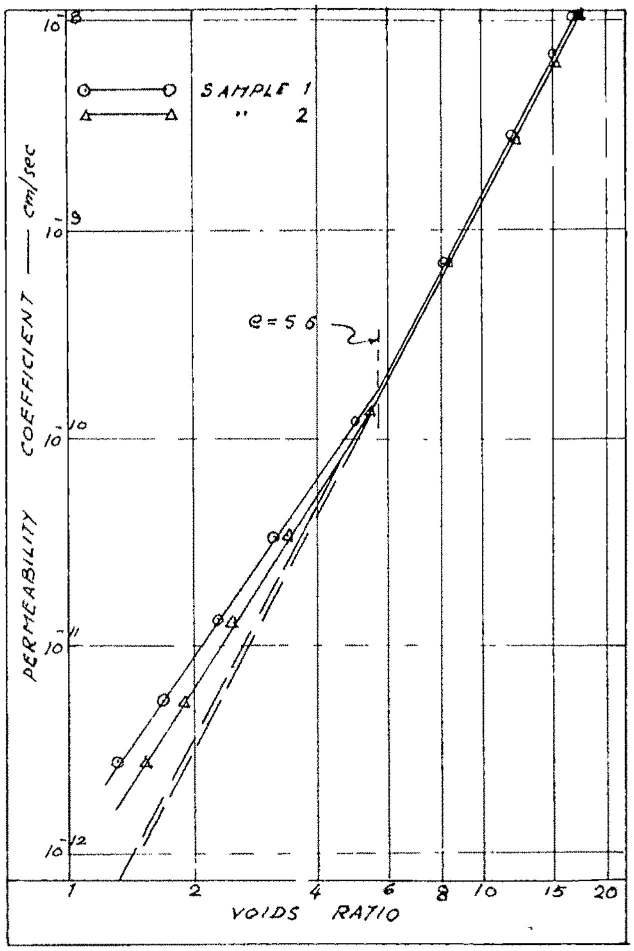


FIG. 2·8 PERMEABILITY - VOID RATIO RELATIONSHIP FOR SODIUM BENTONITE (AFTER ARNOLD, 1965)

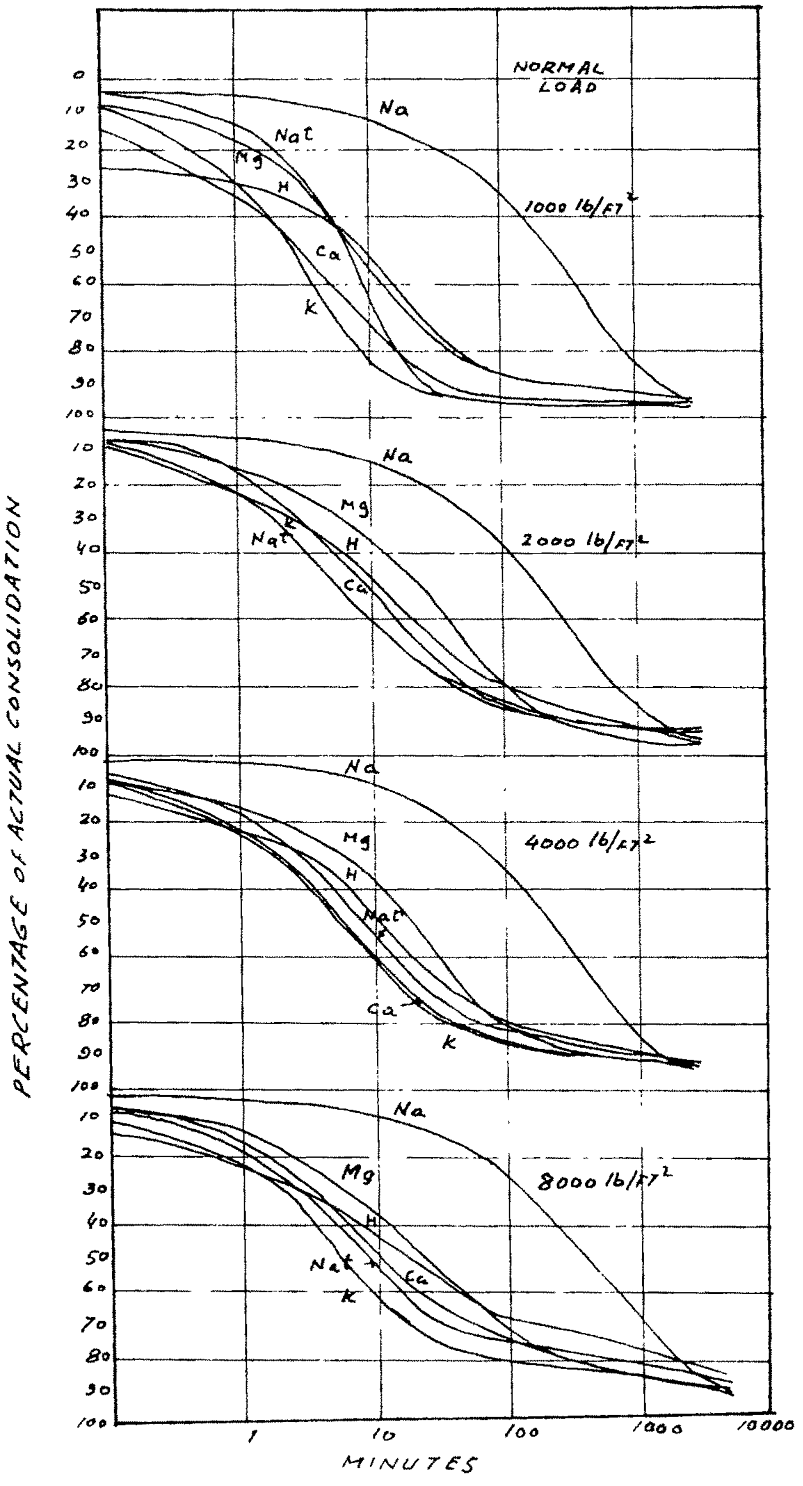


FIG 2·9 INFLUENCE OF EXCHANGE IONS ON CONSOLIDATION RATES (AFTER WINTERKORN, 1941)

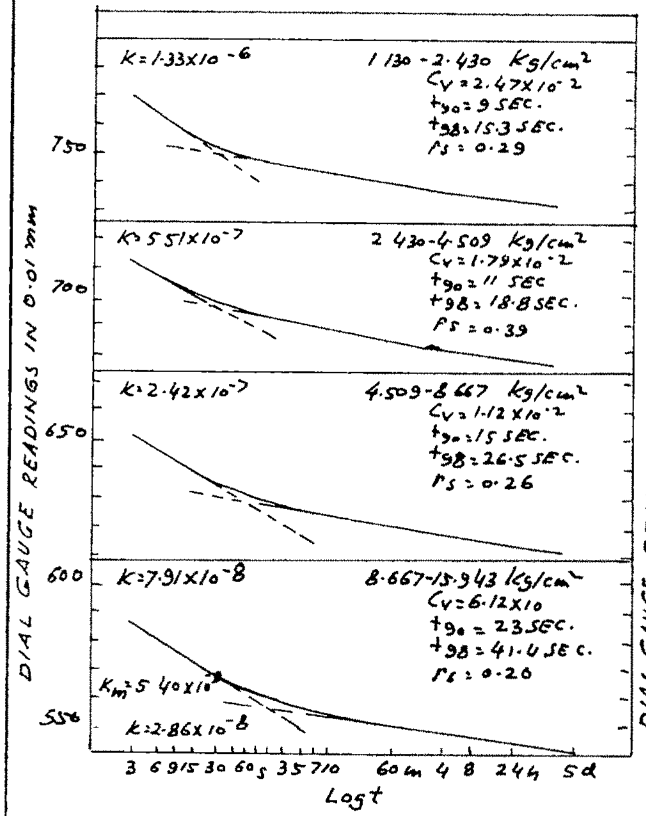


FIG. 2·10 CONSOLIDATION CURVES OF BENTONITE WITH METHYL ALCOHOL. K DETERMINED DIRECTLY.

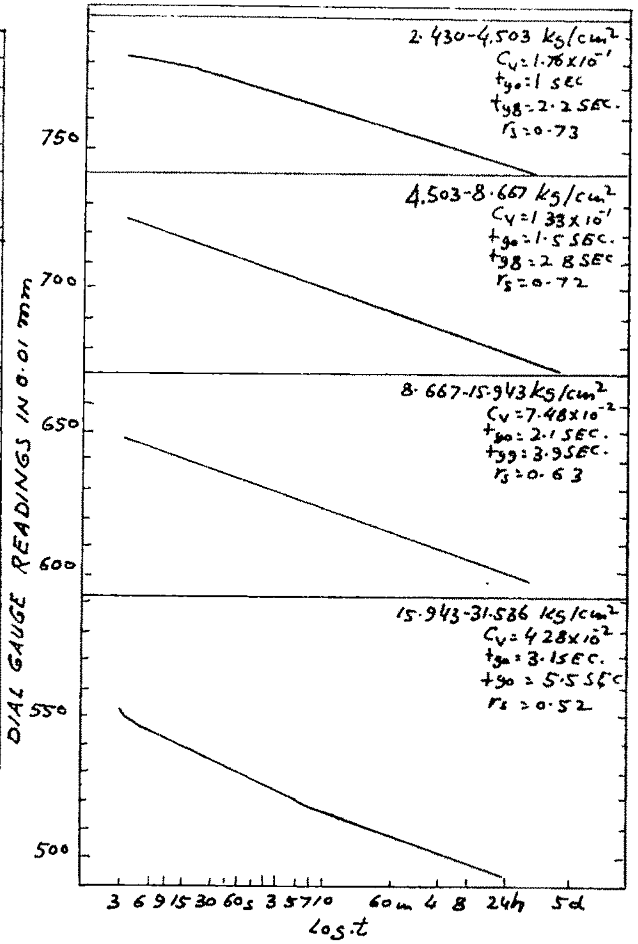


FIG. 2·11 CONSOLIDATION CURVES OF BENTONITE WITH ETHYL ALCOHOL.

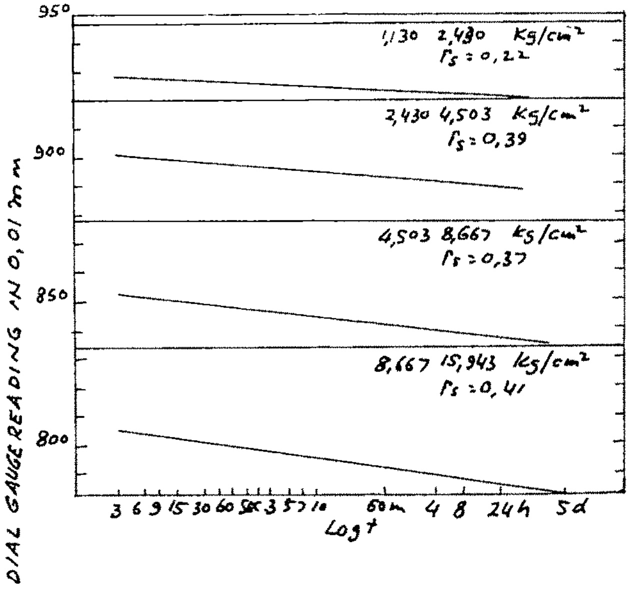


FIG. 2·12 CONSOLIDATION CURVES OF BENTONITE WITH CARBON TETRACHLORIDE

(———— AFTER SALAS-SERRATOSSA - 1953 ————)

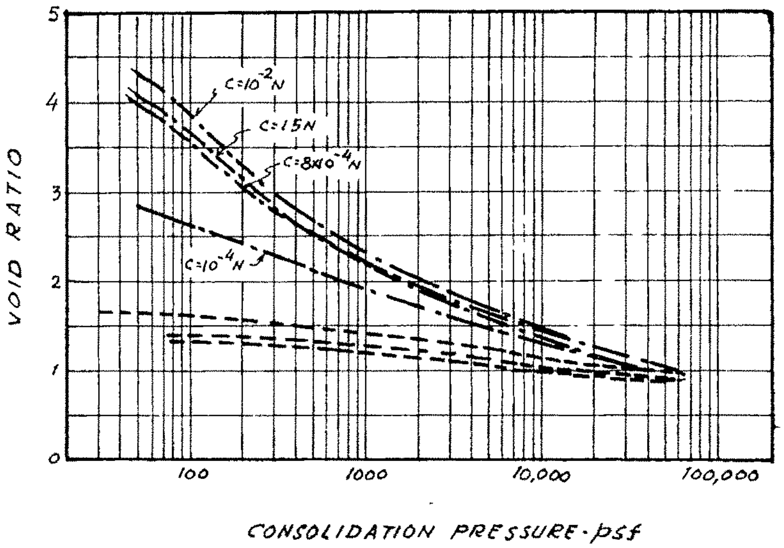


FIG. 2·13 COMPRESSION CURVES OF Ca-ILLITE.
ALL SAMPLES SEDIMENTED FROM DILUTE SUSPENSION
(AFTER OLSON et al, 1960)

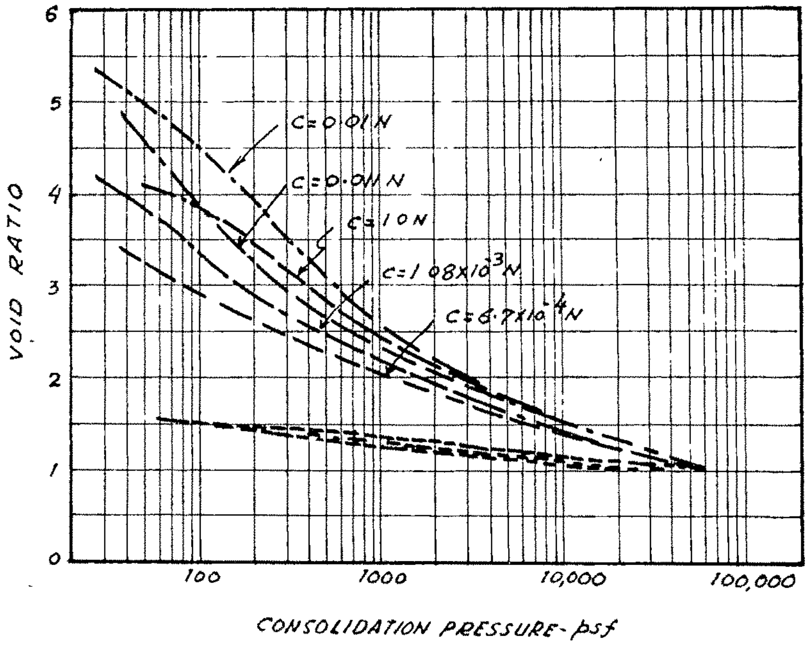


FIG. 2·14 COMPRESSION CURVES OF Mg-ILLITE.
ALL SAMPLES SEDIMENTED FROM DILUTE SUSPENSION
(AFTER OLSON et al, 1960)

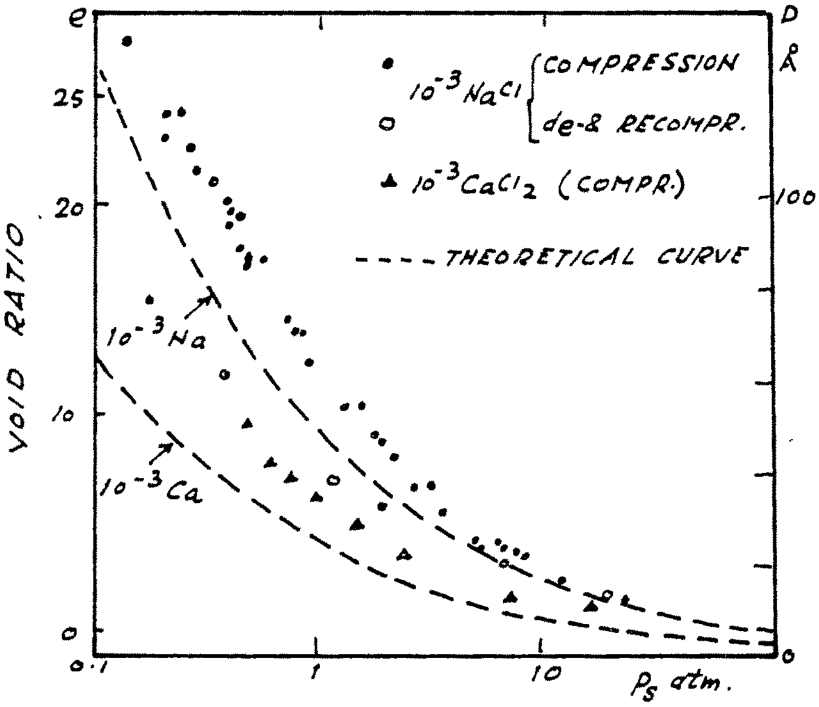


FIG. 2.15 COMPARISSION CURVE OF Na-MONTMORILLONITE AND Ca-MONTMORILLONITE, FRACTION < 0.2 μ, IN EQUILIBRIUM WITH 10^{-3} MOLAR NaCl AND $CaCl_2$, RESPECTIVELY. (BOLT - 1956)

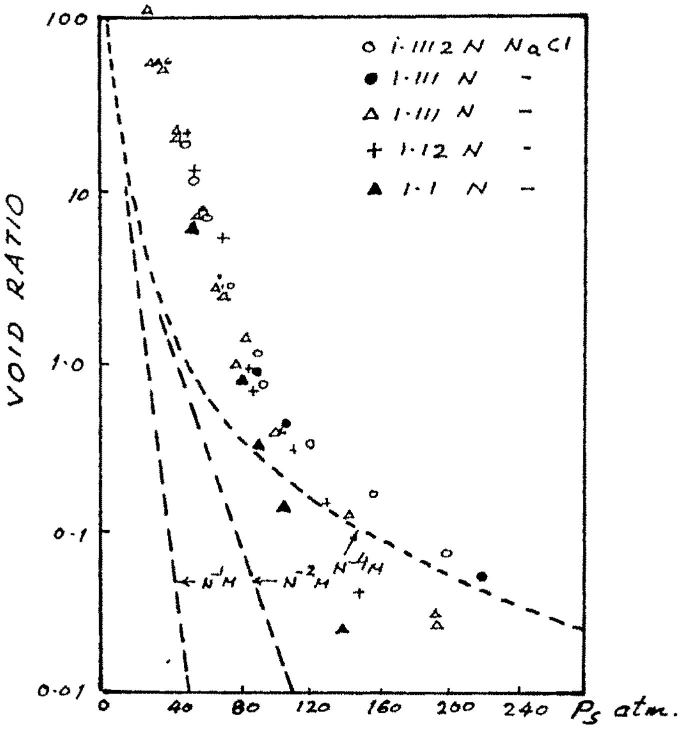


FIG. 2.16 PLATE SEPARATION AND SWELLING PRESSURE OF Na-ILLITE SUSPENSIONS IN EQUILIBRIUM WITH SOLUTIONS OF NaCl. (BOLT et al, 1955)

2.3.2. Fabric Structure Dependent

The fabric structure in a clay mass is identified mainly into two classes, namely, 'edge to face' termed flocculated and 'face to face' termed dispersed. Generally when the net electrical forces between adjacent particles are attractive it forms a flocculated structure whereas under repulsive forces a disperse structure results. The contribution of Yong and Warkentin (1966) is of useful value in visualizing the picture of variation of fabric structure of clay undef the consolidating pressures. In the undisturbed clay primary consolidation is initially due to the breakdown of edge to surface or edge to edge bonds and as time proceeds further breakdown occurs as the soil particles tend to achieve positions of equilibrium finally resulting into a semioriented structure. The secondary compression is because of reduction in particle spacing of this semi-oriented structure. In the remoulded clay whose structure is already semi-oriented, to start with, the deformation is due to decrease in interparticle spacing accompanied by some reorientation of particles. The reduction in interparticle spacing is resisted by repulsive forces between particles which becomes equal to that applied at the secondary compression stage. Since the bonds in dispersed clays are weaker than in flocculated clays, it is argued that the total consolidation is reached relatively earlier in the former as shown in the time deformation curve in Figure 2.17. The characteristics hump, marked C, in the stress deformation curve (Fig. 2.18) seen for flocculent clay is markedly absent in case of dispersed clays. It is further explained that particles in a randomly oriented sample are rotated into more parallel orientation and pushed closer together. These actions may be occuring simultaneously. This process is quite distinct from the complete structural breakdown that occurs under a consolidation pressure larger than the bond strength in a flocculated structure.

X-ray diffraction analysis and electron microphotography are employed to examine the validity of the hypothetical picture. Quigley (unpublished) presented X-ray diffraction studies for two soil structures under consolidation by a plot of X-ray peak intensity in terms of counts per second against the applied pressure, tons per sq.ft. The characteristics of flocculated undisturbed clay as represented by curve (a) in Figure 2.19 is that the counts are lower in the beginning indicative of edge to face configuration and as the load increases the arrangement became face to face as reflected by the higher number of counts per second. In the curve (b) in Fig. 2.19, the characteristics of remoulded clay is evident by higher counts in the starting itself which gradually increase

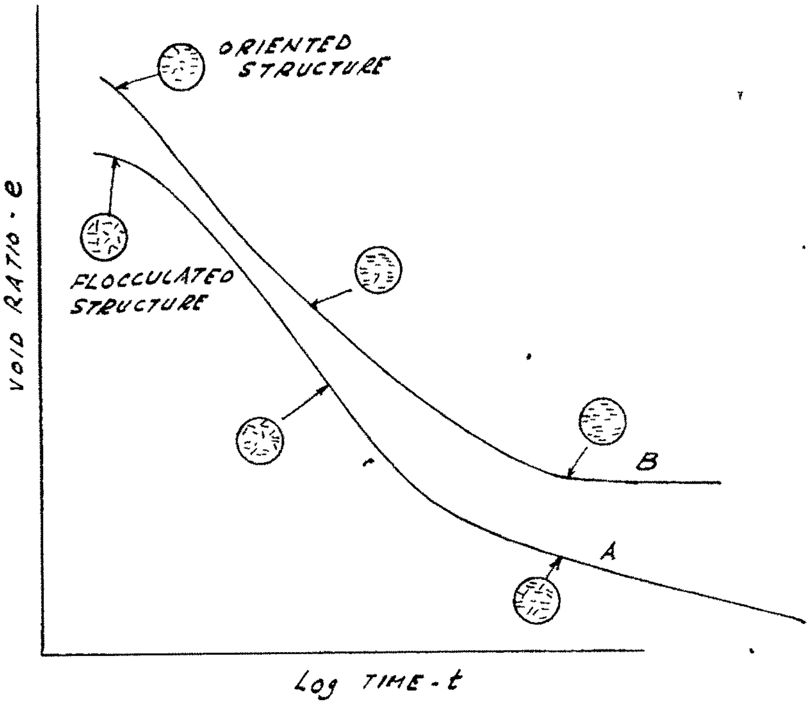


FIG. 2·17 INFLUENCE OF INITIAL STRUCTURE ON TIME - DEFORMATION CURVES. (AFTER YOUNG AND WARKENTIN, 1966)

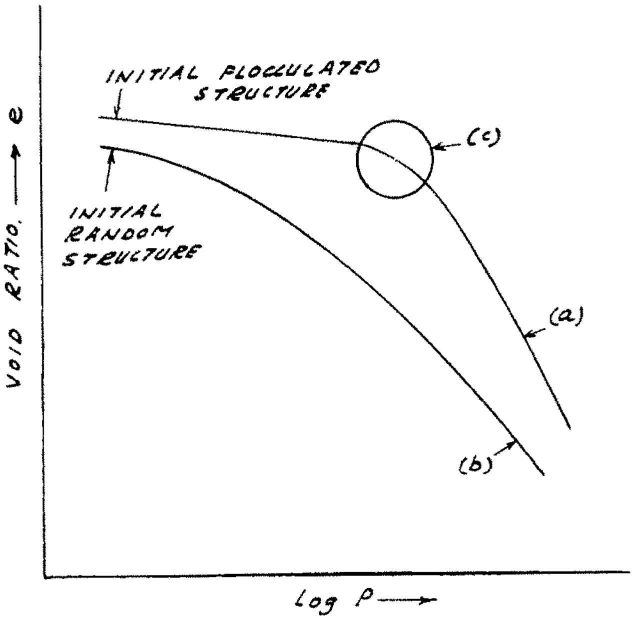


FIG. 2·18 CHANGE IN SOIL STRUCTURE UNDER LOAD; FINAL EQUILIBRIUM STRUCTURE DEPENDS ON INITIAL STRUCTURAL AND APPLIED LOAD

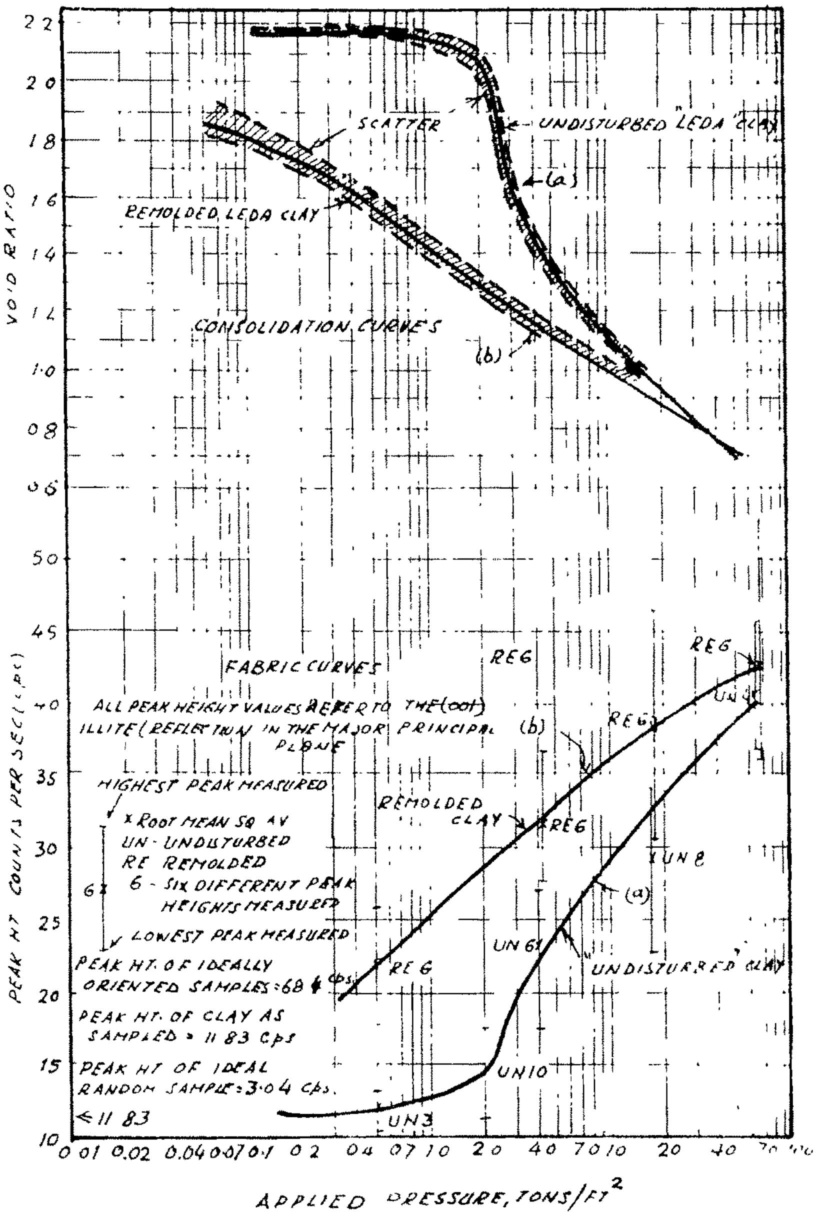


FIG 2·19 ORIENTATION OF PARALLEL CLAY PARTICLES PRODUCED BY ONE-DIMENSIONAL CONSOLIDATION OF UNDISTURBED AND REMOLDED SAMPLES OF LEDA CLAY

(UNPUBLISHED RESULTS COURTESY OF PROF. R.M. QUIGLEY, UNI. OF WESTERN ONTARIO) as soil particles become more and more oriented under the higher applied load. The absence of structural collapse in dispersed clay structure produces a smooth curve without the characteristic hump as seen in flocculated clay structure. Mitchell (1956) produced photomicrographs of powder Boston Blue clay consolidated one dimensionally to 4 kg/cm^2 as shown in Fig. 2.20. The parallel orientation of clay particles is indicated through the large difference in intensity between extinction and illumination. The greater the parallel orientation caused by remoulding, the steeper is the slope of the straight line portion of the compression curve as seen from the void ratio versus log pressure relationships of Boston Blue clay (Figure 2.21). The fabric structure of the clay material cannot be regarded as isotropic and consequently the flow of water under the applied stress cannot be identical in all directions. It is well known that in most clays the horizontal permeability is higher than the vertical permeability. Mitchell (1956) illustrated schematically (Figure 2.22) the flow of water through a clay material which clearly indicates that water flowing vertically must follow a much more tortuous path than water flowing horizontally. Ward et al (1959) experimentally investigated the vertically and the horizontally laminated samples of clay and found that their coefficients of consolidation are much

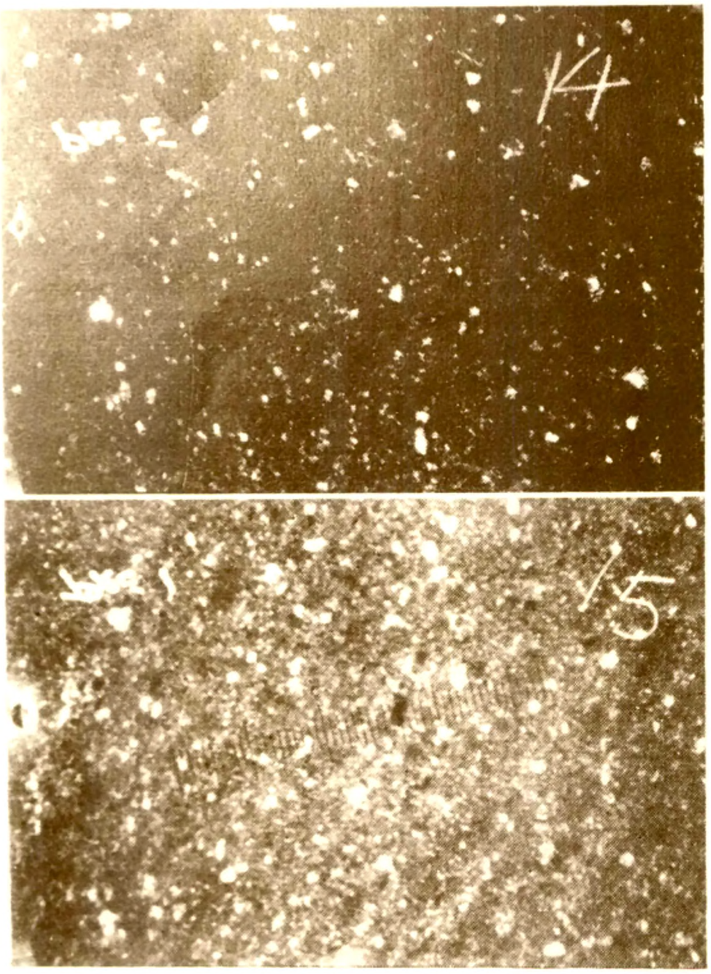


Figure 20. Photomicrographs of powdered Boston blue clay consolidated one-dimensionally to 4 kg/cm^2 in sea water. Above: Extinction. Below: Illumination. Large difference in intensity between extinction and illumination indicates parallel orientation of clay particles.

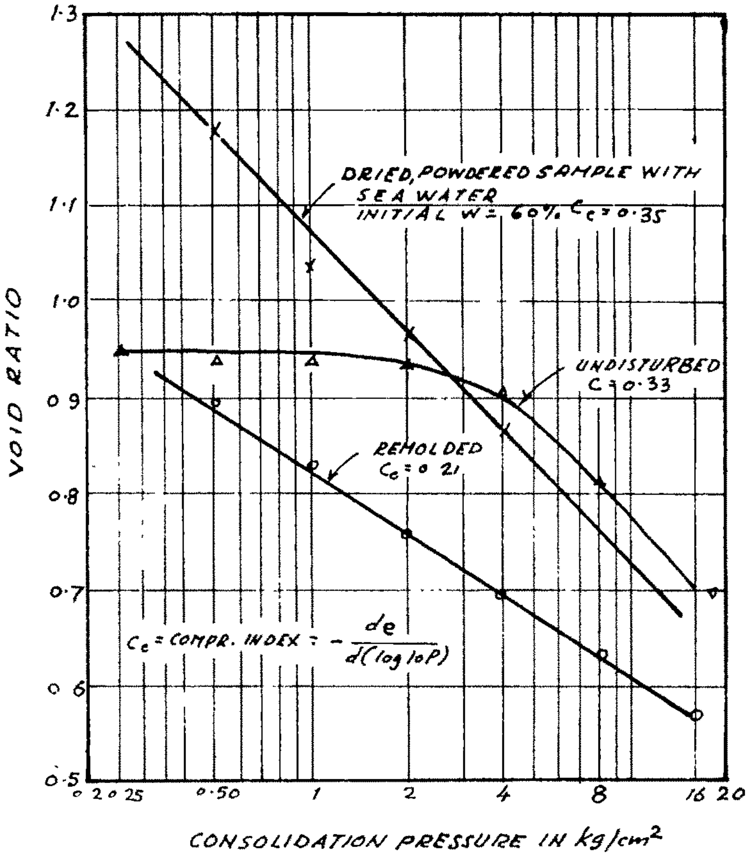


FIG. 2·21 CONSOLIDATION OF DIFFERENT SAMPLES OF BOSTON BLUE CLAY (MITCHELL - 1956)

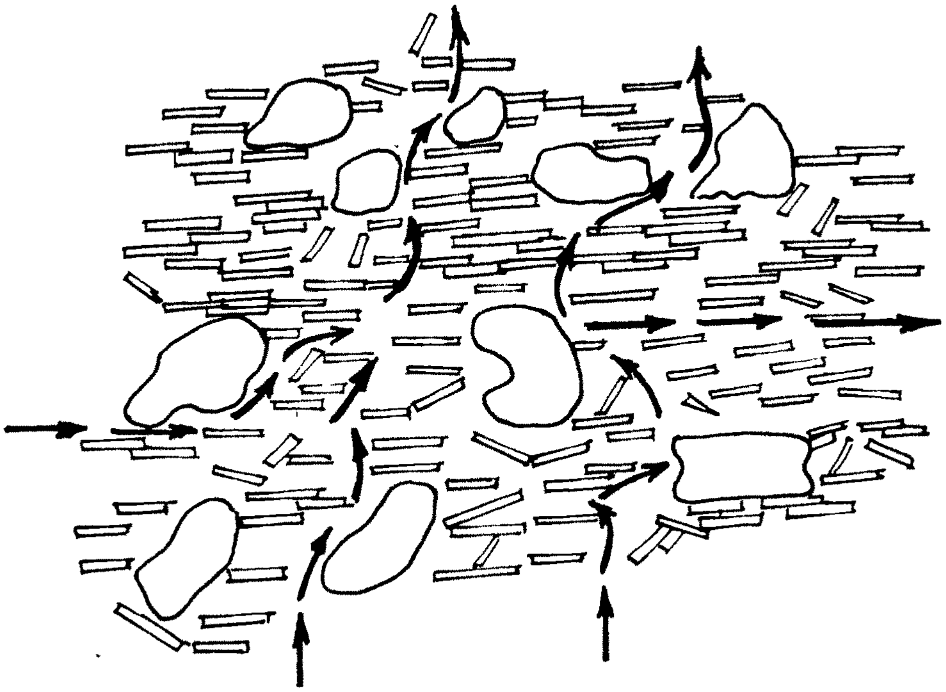


FIG. 2·22 SCHEMATIC DIAGRAM OF WATER FLOW THROUGH ORIENTED CLAY (MITCHELL, 1956) different as shown in Figures 2.23 and 2.24.

The formation of a particular structure in a clay material is also dependent on the water content at that instant. Using the compaction curve (Fig. 2.25) Lambe (1960) discussed the influence of the moulding water content on the structure of clay material. At A, insufficient water causes very high concentration of electrolyte which depresses the electric double layer and as a result reduces the interparticle repulsion, thereby creating a situation towards flocculation of colloids. From A to B, with increase in water content the double layer expands, concentration of electrolyte reduces and consequently, tendency will be to form lesser flocculation of colloids which permits orderly arrangement of particles. With further increase of water the structure will become more and more oriented. Figure 2.26 illustrates the difference in compression characteristics between the saturated clay samples at the same density, one compacted on the dry side of optimum and other on the wet side. Since the flocculated particles have more resistance to compression the wet side compacted sample is more compressible than the dry side sample at low pressures but at high pressure even the dry side compacted show greater compressibility because of the possibility of better orientation of particles.

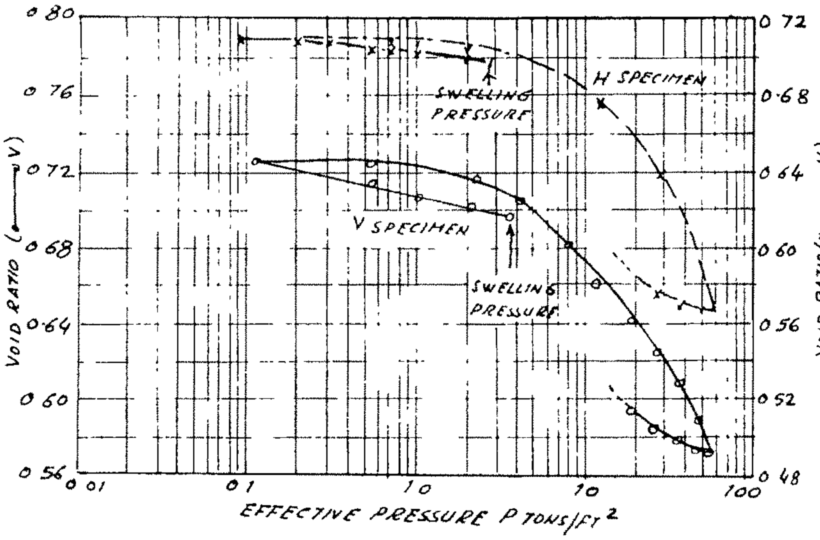


FIG 2·23 TYPICAL VOID RATIO-PRESSURE CURVES FOR HORIZONTAL AND VERTICAL SPECIMENS OF LONDON CLAY (WARD et al, 1959)

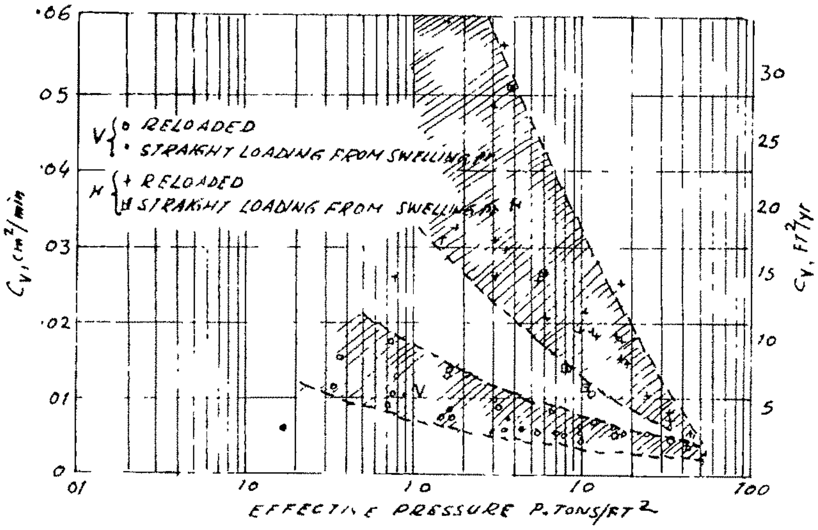


FIG 2·24 VARIATION OF THE CONSOLIDATION COEFFICIENT WITH EFFECTIVE PRESSURE FOR HORIZONTAL AND VERTICAL SPECIMENS OF LONDON CLAY (WARD et al., 1959)

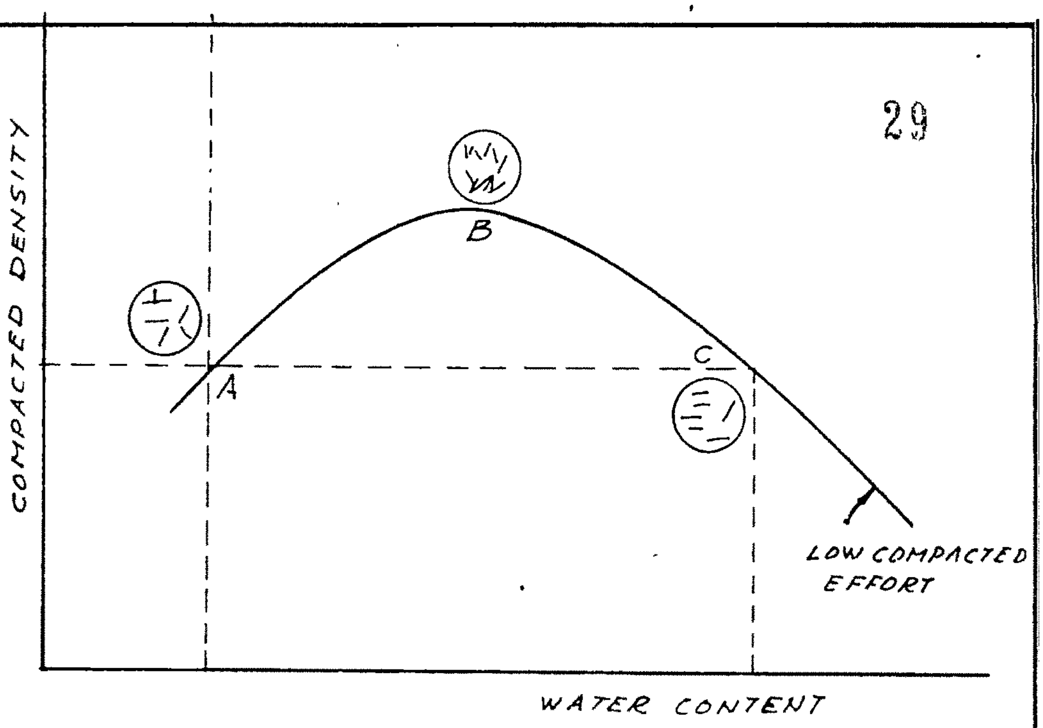
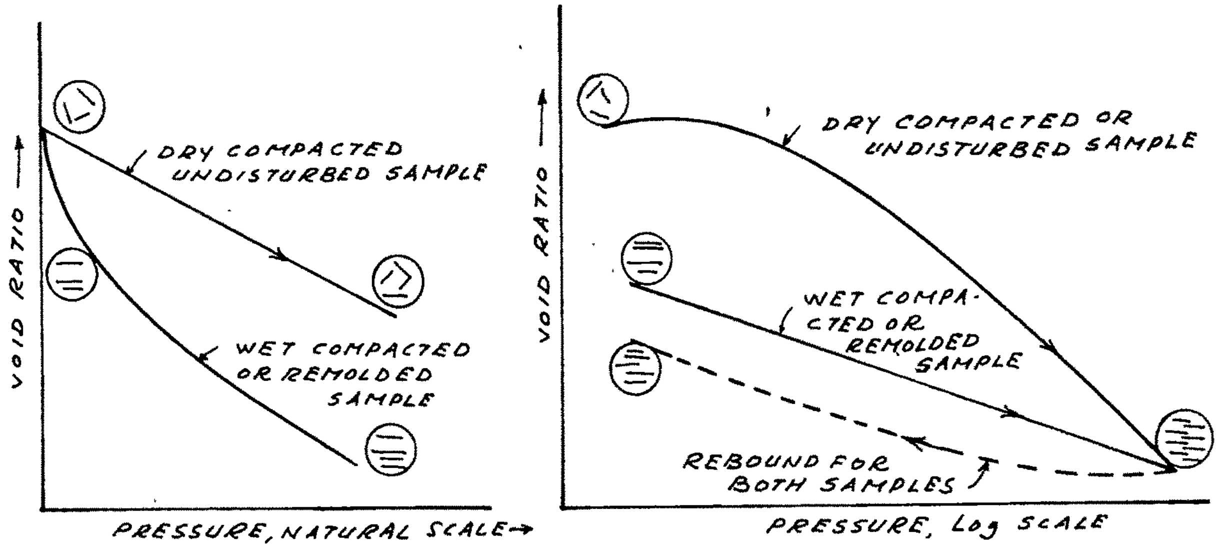


FIG. 2·25 INFLUENCE OF MOULDING WATER CONTENT ON STRUCTURE

(AFTER LAMBE, 1960)



(a) LOW PRESSURE CONSOLIDATION (b) HIGH PRESSURE CONSOLIDATION

FIG. 2·26 EFFECT OF ONE-DIMENSIONAL COMPRESSION ON STRUCTURE

(AFTER LAMBE, 1960)

2.3.3. Stress History Dependent

The mode of deformation must be irreversible, inelastic and time dependent for a material in which particles possess anisotropic shape, water molecules exhibit viscous characteristics, particles bounded weakly and the structural pattern not particularly ordered. In a disperse colloidal system, such as clay water, the physico-chemical factors play a vital role but while assessing its mechanical behaviour the physical factors, such as mode of deformability and state of packing cannot be ignored. The loading as a result of deposition and unloading due to erosion from time to time alter the state of the packing of soil which produces stress history dependent characteristics in the material. From the stand point of energy considerations, in a multiphase system like soil, the energy is derived in two ways, one from mechanical forces and other from physico-chemical forces. The non-conservative characteristics of clay material has been corroborated from the test results of clays subjected to various cycles of loading and unloading reported by various workers. The typical test results from the work of Hafeli(1948) and Sohaad (1948) as represented in Figures 2.27 and 2.28 point towards the characteristics hysteresis loops which signify mode of deformation. Terzaghi (1931) explained that the relative displacement of particles cannot be possibly reversible

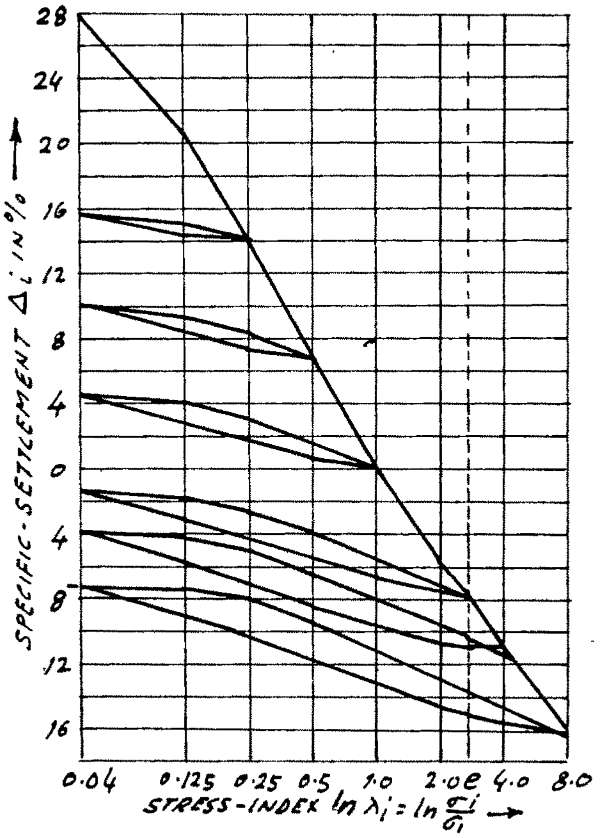


FIG. 2·27· HYSTERESIS LOOPS OF LOAD SETTLEMENT DIAGRAM. (HAEFELI, 1948)

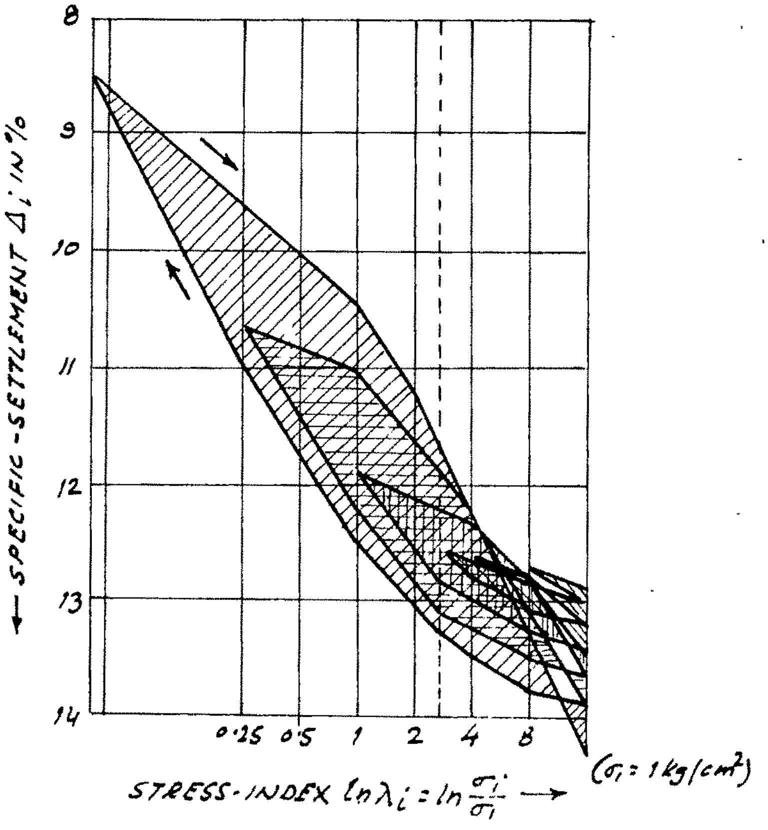
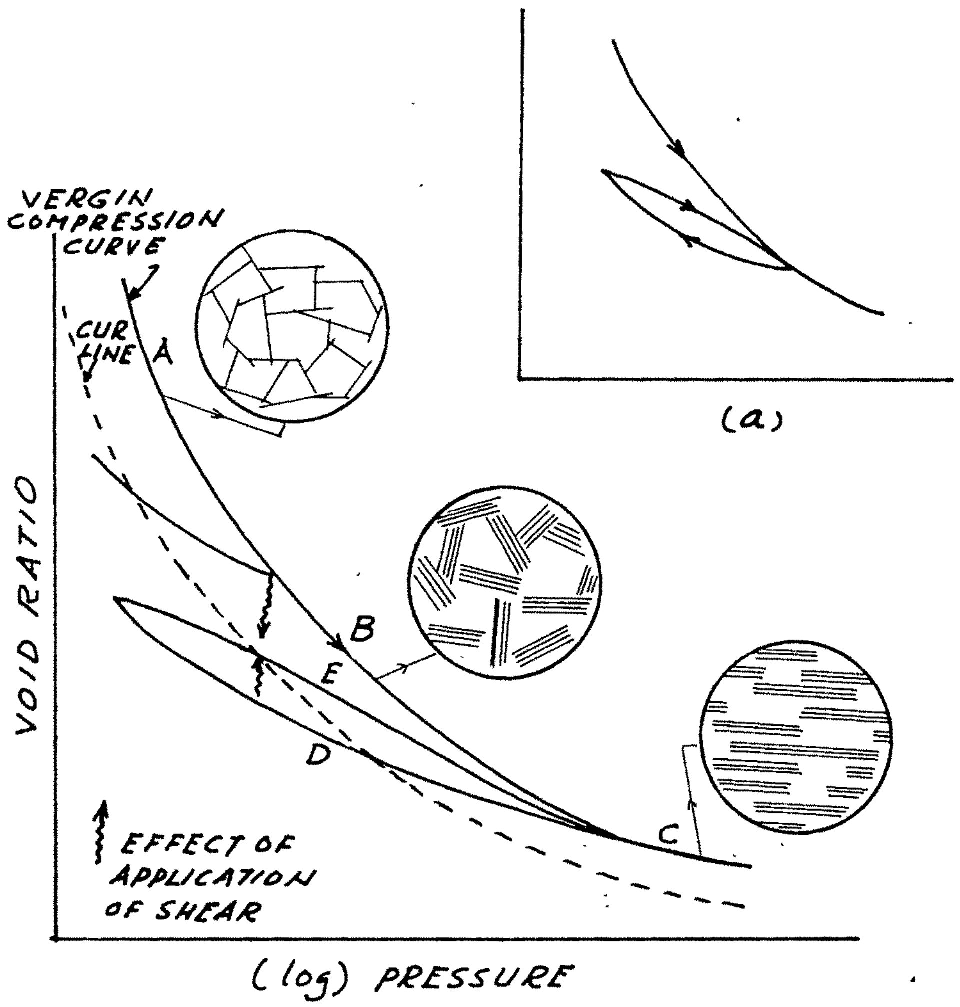


FIG. 2·28 HYSTERESIS LOOPS OF LOAD SETTLEMENT DIAGRAM (HAEFELI, 1948) (LOAD INCREMENT CONSTANT DURING 24 HOURS) since every particle tends to move towards a more stable position. The work of Lee (1968) presents a schematic diagram (Fig. 2.29) to explain the structural changes during consolidation and rebound in a clay. Under the application of load, the initial structure at A, of either edge to face or edge to edge type compresses to develope internal aggregation of face to face arrangement as shown at B, which under increasing load get wholly oriented face to face as represented at C;from this point onwards the compression occurs only against the repulsive forces on the clay minerals. Once the aggregation of crystals has occured reduction of pressure does not lead to reversible swelling but rather to a rebound curve such as D since there is no mechanism except the phenomenon of complete remoulding of the structure.

In a stress history dependent material like the soil the magnitude of surface forces and body forces must influence the rate as well as its total compression. Under the smaller increments of applied stress the compression must be smaller as the readjustment of particles to newer positions of equilibrium occurs with almost no pore pressure generations but when the stress increment is large enough to generate pore pressure, particles cannot achieve stable positions without additional movements, thus producing higher compression. The influence of self weight becomes effective with larger depths



(a)

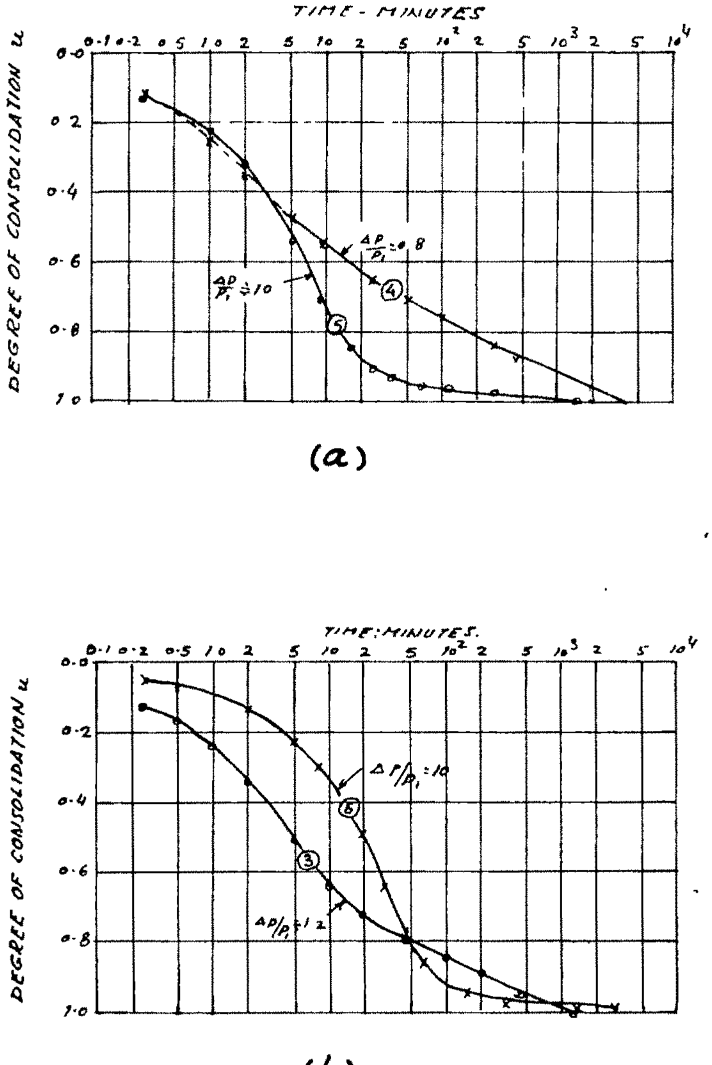
(b)

FIG. 2·29 STRUCTURAL CHANGES DURING CONSOLIDATION AND REBOUND IN A CLAY

(AFTER I. K. LEE, 1968)

as it also contributes significantly in developing the pore pressure development and thereby affecting the process of compression. The publications of Leonard and Ramiah (1959), Newland and Alley (1960), Leonard and Girault (1961), Barden and Berry (1965) on this aspect are noteworthy. Representative experimental results are reproduced in figures from 2.30 to 2.35. In general, the findings are that primary consolidation predominates in case of thicker clay samples under higher stress increments while secondary compression becomes significant in case of thinner samples under lower stress increments. It could be argued that under lower stress increments physico-chemical factors dominate the consolidation process and obscure the physical factors which becomes prominent at higher stresses minimising the effect of physico-chemical factors on the total process.

The foregoing discussion brings out various physico-chemical and mechanical factors that influence the consolidation characteristics of clays. The physico-chemical factors are the characteristics of clay mineral, the interacting forces in clay water system and the structural arrangement of particles. The mechanical factors are the magnitude of stress, its mode of application and the drainage conditions.



(a)

(b)

FIG. 2·30 COMPARISON BETWEEN LARGE AND SMALL PRESSURE - INCREMENT RATIOS

(AFTER NEWLAND AND ALLEY 1960)

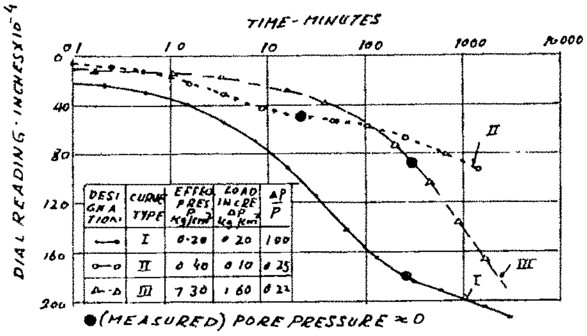


FIG. 2·31 EFFECT OF LOAD INCREMENT RATIO ON SHAPE OF DIAL READING-TIME CURVES. UNDISTURBED MEXICO-CITY CLAY (AFTER LEONARD AND GIRAULT, 1965)

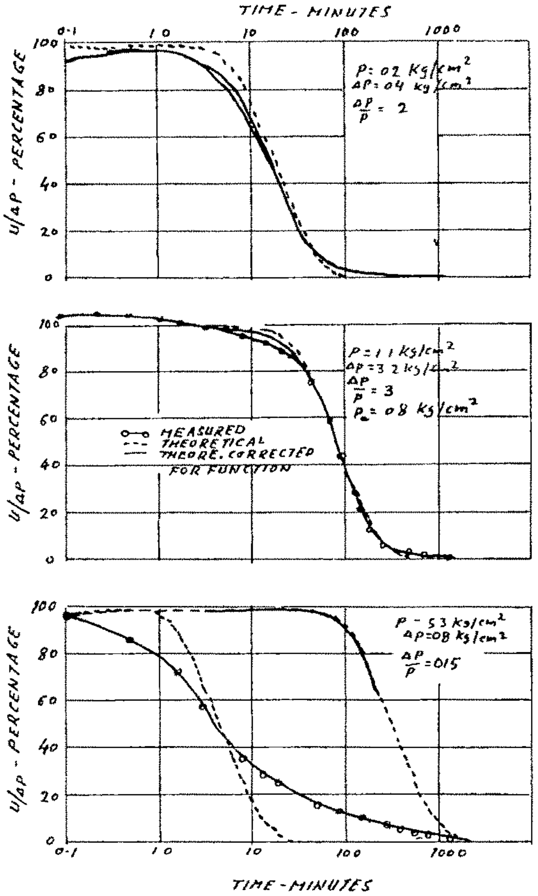
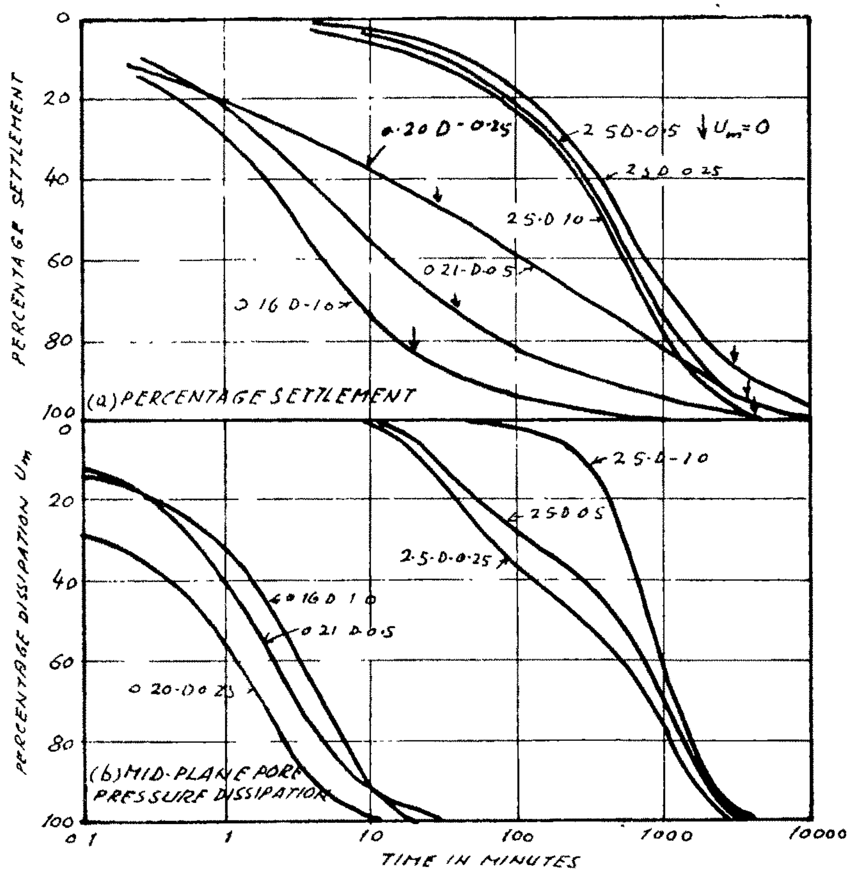
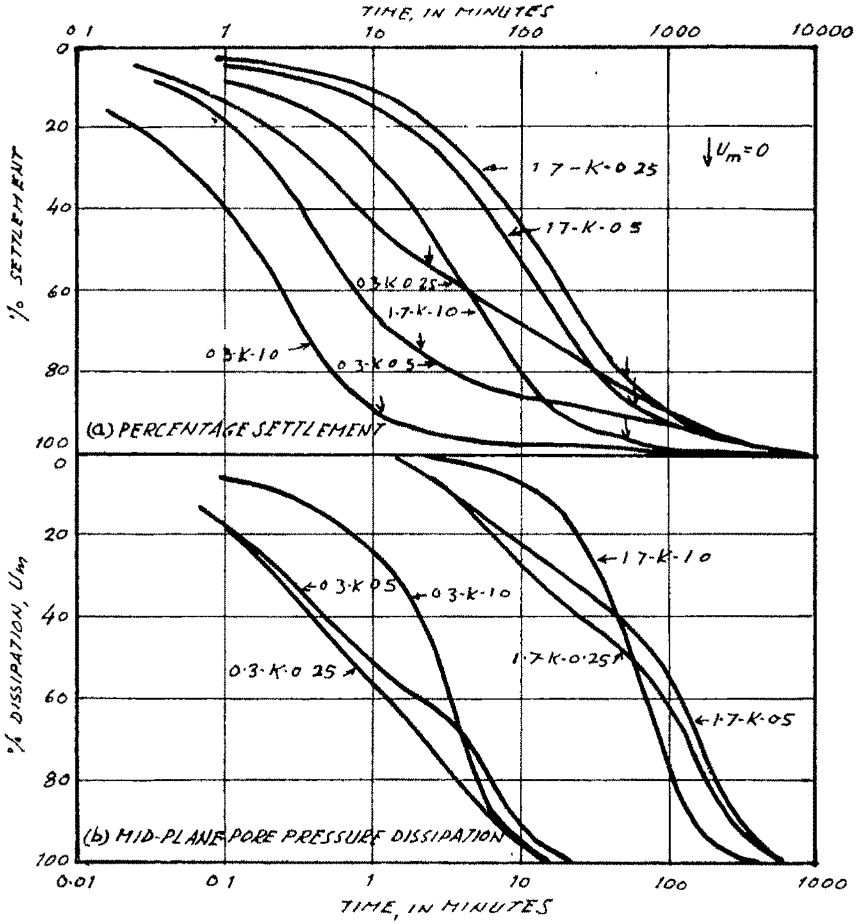


FIG. 2·32 INFLUENCE OF LOADING INCREMENT RATIO ON PORE PRESSURE DISSIPATION RATES. UNDISTURBED MEXICO CITY CLAY (AFTER LEONARD AND GIRAULT, 1965)





(AFTER BARDEN AND BERRY, 1965)

FIG 2·33 EXPERIMENTAL RESULTS FOR DERWENT CLAY.

FIG 2·34 EXPERIMENTAL RESULTS FOR KAOLINITE

CONVENTION : ·16 - D - 1·0 REFERS TO A ·16 INCH THICK SAMPLE
OF DERWENT CLAY WITH $\frac{\Delta P}{P} = 1$
1·7 - K - 1·0 REFERS TO A 1·7 INCH THICK SAMPLE
OF KAOLINITE CLAY WITH $\frac{\Delta P}{P} = 1$

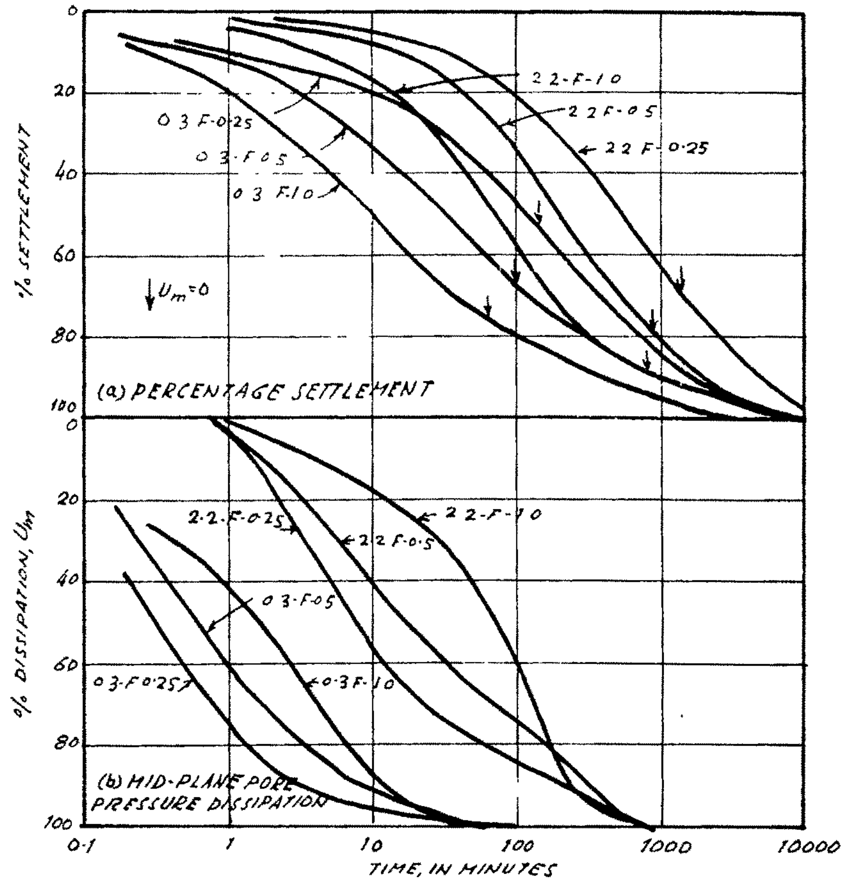


FIG 2·35 EXPERIMENTAL RESULTS FOR FRODSHAM CLAY

(AFTER BARDEN AND BERRY, 1965)

CONVENTION: 2·2-F-1·0 REFERS TO A 2·2 INCH THICK SAMPLE OF FRODSHAM CLAY WITH $\frac{\Delta P}{P} = 1$

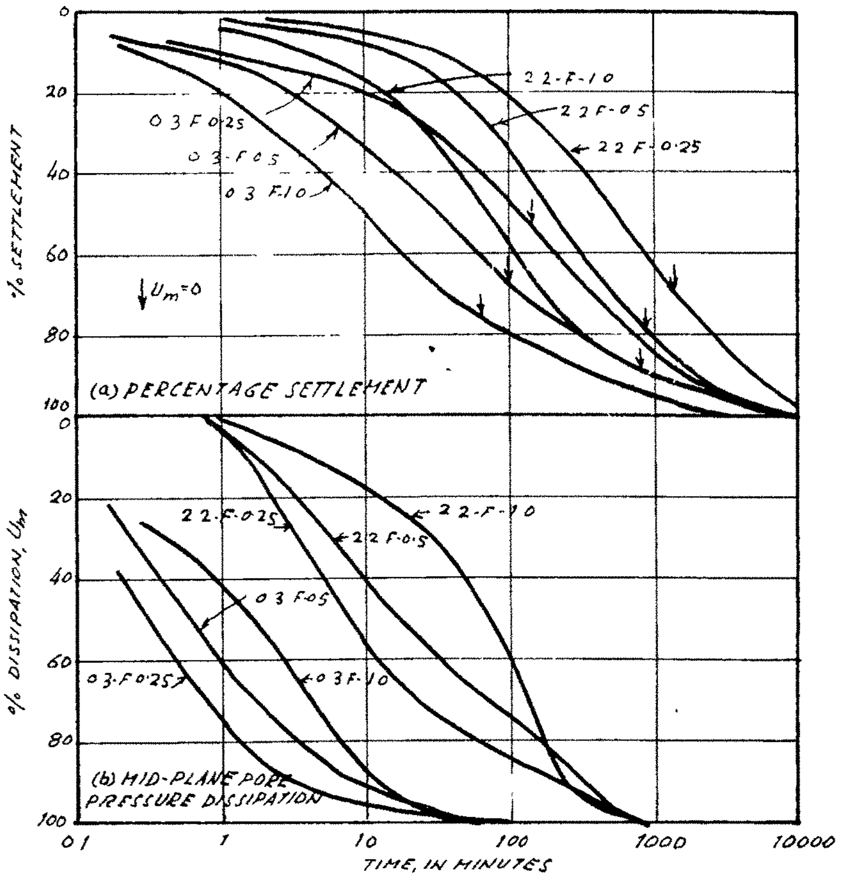


FIG 2·35 EXPERIMENTAL RESULTS FOR FRODSHAM CLAY

(AFTER BARDEN AND BERRY, 1965)

CONVENTION: 2·2-F-1·0 REFERS TO A 2·2 INCH THICK SAMPLE OF FRODSHAM CLAY WITH $\frac{\Delta P}{P} = 1$