TIME DEPENDENT DEFORMATION BEHAVIOUR OF SOILS

A Thesis

by

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ABSTRACT

The stress-strain-time relationships of three cohesive soils were studied to verify some of the available models, specially the Kavazanjian-Mitchell (1980) general model under one dimensional consolidation to predict the deformation behaviour with time.

The investigations involved one dimensional consolidation test on six identical specimens of each type of soil at stress intensities of 20 to 120 per cent of unconfined compressive strength.

On the basis of the test results a model was proposed to predict the time dependent deformation behaviour of cohesive soils.

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NOTATIONS

A	=	projected value of strain rate at zero creep stress on logarithm of strain rate, vs. creep stress plot for unit time;
a	=	constant of integration in Singh-Mitchell creep function;
C_c	=	compression index;
C'_c	=	true compression index;
C_α	=	secondary compression index;
C'_α	=	true secondary compression index;
D,	=	deviatoric stress; $(\sigma_1 - \sigma_3)$;
D_f	=	strength of soil; $(\sigma_1 - \sigma_3)_f$;
$\bar{D}$	=	stress level;
e	=	void ratio;
e_o	=	initial void ratio;
H	=	solid height;
k_o	=	coefficient of earth pressure at rest;
LL	=	liquid limit;
m	=	slope of logarithm of strain rate versus logarithm time straight line;
PL	=	plastic limit;
PI	=	plasticity index, LL-PL;
q_u	=	unconfined compressive strength;
R_{do}	=	the theoretical dial reading at $t = 0$;
R_{d1}, R_{d2}	=	dial readings at time t_{d1} and t_{d2} respectively;
R_{d100}	=	dial reading to the time corresponding to the end of primary consolidation;
t	=	any time;

t_1 = unit time;

t_i = the time required for instant compression to occur;

t_{d1} = any time at which less than half the consolidation has taken place;

t_{d2} = 4 t_{d1};

α = value of the slope of mid range linear portion of the log strain rate vs. creep stress plot;

$\bar{\alpha}$ = αD_f;

e = axial strain at any time, t;

e_1 = strain at time t_1;

e_v = volumetric strain;

$(e_v)_d$ = delayed volumetric strain;

$(e_v)_i$ = immediate volumetric strain;

$\dot{e}$ = strain rate at any time t;

$\dot{e}_{(t,D_o)}$ = projected value of strain rate at D=0 on log strain rate vs. stress plot at any time, t;

$\dot{e}_{(t_1,D)}$ = value of strain rate at unit time, a function of stress intensity, D;

σ = applied pressure; normal pressure;

σ_1 = applied vertical stress;

σ_3 = lateral stress;

$\sigma_1 - \sigma_3$ = applied deviatoric stress; $(\bar{\sigma}_1 - \bar{\sigma}_3)$

$(\sigma_1 - \sigma_3)_f$ = deviatoric stress at failure;

σ_{1f} = D_f in one dimensional compression;

$\bar{\sigma}_1$ = vertical effective stress;

$\bar{\sigma}_3$ = lateral effective stress; and

σ_c = critical pressure.

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CHAPTER 1

INTRODUCTION

1.1 General

The engineers of twentieth century are finding it increasingly necessary to rely on rational, scientific methods of analysis, with correspondingly less emphasis on empirical rules or on experience. For a systematic method of analysis and for reliable results, a good knowledge of the physical properties of soil as well as its behaviour under various conditions are required. Since the physical properties of soil depend on a large variety of factors and the great complexity of the interaction of these factors, they impose a heavy burden on the investigator. The foundation engineer is concerned with the deformation behaviour of soils and has to know by how much a soil will be compressed under load and at what rate the compression will occur. This information is important in order to make reliable estimates of the settlement of foundations and the possible volume change of an earthwork.

Soil engineers have, in the past, focussed their attention mostly on the strength of soils, but very little on time dependent stress-strain behaviour. To date, no completely satisfactory stress theory has been developed by means of which the deformation behaviour of the soil underneath the structural loading can be accurately predicted.

For an improved understanding of fundamental mechanism contributing to the settlement of foundation in soils, the studies of stress-strain-time effects are required. In order to develop stress-strain-time relations for a real material, an accurate model of the material is required which will explain the physical parameters responsible for deformation behaviour. At the present time, no general constitutive model which can accurately predict the time-dependent deformation of cohesive soil is available. In fact, the development of a realistic stress-strain-time relation for soils is one of the greatest unsolved problems in soil mechanics.

1.2 Introduction to the Problem

In an element of soil which is subjected to constant external stresses, a change in the internal pore pressure will cause an alteration in the effective or intergranular stresses. We may examine the effect of such change of effective stress on void ratio.

The compressibility of the soil structure is the principal factor to be considered in the deformation of a cohesive soil. In a situation of this type, the compressibility of the soil solids is negligible and that of a pore water is very small in comparison with the compressibility of soil structure (Scott, 1963). The soil structure includes the consideration of the orientation and distribution of particles in a soil mass and the forces between adjacent particles. In general any structure with high porosity, is more compressible than a dense one. A soil in remoulded state is usually more compressible than the same soil in natural state.

The time lag during compression in sands is largely of frictional nature. Generally sands are so pervious that the time lag is about same whether the sand is saturated or dry. However, because of the low permeabilities of clay, the compression under a load does not occur immediately, since time is required for expulsion of water from soil voids. The process is, therefore, time dependent and takes place through the gradual transfer of the applied stress from the pore water to mineral skeleton, so that ultimately load is carried by the soil structure and the process is called consolidation. If the rate is controlled solely by the resistance of flow of water under the induced hydraulic gradient, the process is termed as primary compression and the decrease in volume of the soil mass that takes place after the dissipation of all excess pore water pressure is known as secondary compression.

In many soil deposits, the volume change that occurs after excess pore water pressures have dissipated are too small to be of practical importance. However, in some soil deposits, the subsequent volume changes may exceed those occurring during primary consolidation. In highly organic soils, such as peat, excess hydrostatic pressure may dissipate so rapidly that only secondary compression may be of practical interest. Generally, in most organic soils and in some inorganic silts the magnitude of secondary compression is significant.

Several case records are available which clearly show that in certain circumstances a large part of the observed settlement has occurred after full dissipation of excess pore water pressure. The three well known Chicago structures, the Masonic Temple, the Monadnock Block and the Auditorium Tower showed substantial amount of secondary settlement (noted by Simons and Menzies, 1976 from Skempton, Peck and MacDonald, 1955). More than 90 per cent of the total final settlement of Masonic Temple developed after five years. The corresponding figures for the Monadnock Block and the Auditorium Tower are 47 per cent and 62 per cent. Furthermore, after 10 years the settlement of the Masonic Temple was virtually complete, but for the other two structures, settlements were still taking place after 30 years. In Drammen, Norway, structures suffered large secondary settlements. For example, a gymnasium in Drammen continued a settlement of 23 inch in 22 years after the end of construction. From the observed settlement versus log time plot, it was found that the settlement was mainly due to the secondary time effect. This conclusion was arrived at from the measurements which showed full dissipation of excess pore pressure (Simons, 1957). Same fact was observed by Lewis (1956). He found that the soil underlying an embankment, the height of which did not exceed 4 feet, contained strata of organic clay and peat, suffered a settlement of 9 inch in $4\frac{1}{2}$ years and the settlement was continued. Bejerrum (1968) reported that the structures subjected to large variation in live load, for example, silos, storage tanks, and high-rise structures under the wind action may experience appreciably larger secondary settlements compared to the nonvarying load conditions. Satisfactory prediction of secondary settlement is, therefore, certainly a matter of practical importance.

The present available methods of settlement calculations do not include the effect of secondary consolidation. The 24-hours period observation between increments of load, for example, may result in an incomplete picture of the true settlement-time performance. Therefore consolidation observation for structures and embankments should be continued over a sufficient period of time for the long term trends to be indicated. However, it is convenient to develop a model that would provide accurate prediction of the time dependent deformation with stresses from test results that take reasonably short period of time. But the settlement of soil deposits, specially that exhibit secondary compression to an important degree, cannot be evaluated rationally because of complex nature of secondary consolidation.

Generally, secondary settlement is approximated by a straight line on a settlement versus logarithm of time plot (Buisman, 1936; Koppejan, 1948; Zeevaert, 1957). Buisman (1936) cited an example of the settlement behaviour of an embankment on peat soil in Holland which suffered a continuous settlement, linear with logarithm of time for more than eighty years. But there are many evidences to indicate that such a simple extrapolation cannot in general be expected to give reliable prediction in field problems (Leonards, 1962; Simon and Menzies, 1975).

Various theoretical models have been proposed (Wahls, 1962; Barden, 1968; Singh and Mitchell, 1968) in order to predict secondary compression with time, but there is yet no satisfactory treatment available which takes into account all the factors which are known to affect secondary compression. Therefore, a general constitutive model suitable for use for analysis of geotechnical problems that involves time dependent deformation behaviour of cohesive soils under arbitrary stress condition is an urgent requirement. However, a constitutive model to predict such behaviour has recently been suggested (Kavazanjian and Mitchell, 1980) by unifying existing phenomenological models and requires intensive experimental verification. The present research is performed with a view to investigating the various parameters of this constitutive model.

With this in mind, the constitutive model was tested for different types of soil (for purpose of this thesis, the term soil will be used for cohesive soils only) and stress-strain-time effects for soil deformation are studied on the basis of this model under restricted boundary conditions. To get mors information about the deformation behaviour of soil, it is required to check the previous phenomenological relationships used to define the general constitutive model and to suggest modifications so that improvements can be made on the general model to predict deformations more accurstely.

CHAPTER 2

LITERATURE REVIEW

2.1 Consolidation Process

At the instant of application of pressure on a saturated, fine-grained soil mass, almost all the applied pressure is transferred to the pore water, because water is virtually incompressible in comparison with the compressibility of soil structure. The excess hydrostatic pressure initiate a flow of water to drain out of the voids and the soil mass begins to compress. A portion of the applied stress is transferred to the mineral skeleton, which in turn causes a reduction in the excess pore pressure. This process involving a gradual compression occurring simultaneously with a flow of water out of the mass and with a gradual transfer of the applied pressure from the pore water to the mineral skeleton, is called consolidation. Conventionally, the process of consolidation is divided into primary consolidation and secondary compression. The reduction in volume which is solely due to the flow of water from the voids under excess hydrostatic pressure is termed primary consolidation, primary compression or primary time effect and is compatible with the Terzaghi (1943) consolidation theory. Even after the reduction of all excess hydrostatic pressure to zero, the soil mass continues to decrease in volume with time and is unaccounted by Terzaghi theory. This process of reduction in volume is referred to as secondary compression secondary consolidation or secular time effect.

2.2 Terzaghi's Consolidation Theory

The mechanics of consolidation proposed by Terzaghi (1943) are formulated from a hydrodynamic process. The concept is based on the premise that in a saturated soil for every void ratio there exists a maximum effective pressure that can be supported by the intergranular soil skeleton. If a pressure is applied in excess of the capacity of the soil skeleton at the existing void ratio, the excess pressure at the instant of application must be supported by the pore water. The development of pore water pressure in excess of hydrostatic pressure causes water to flow out from the soil and the soil mass decrease in void ratio to support the applied pressure. This decrease in void ratio increases the effective pressure of the soil skeleton and causes corresponding decrease in pore water pressure. This process continues until the excess pore water pressure is reduced and the applied pressure is entirely supported by the soil skeleton. Thus, in Terzaghi's theory, primary consolidation is related to the development and subsequent dissipation of pressure in the pore water and the rate of primary consolidation is derived from the principles of hydrodynamics.

2.3 Deviations from Terzaghi's Theory

Expression for the rate of consolidation was first developed by Terzaghi (1943) for the special case of one dimensional flow from a laterally confined soil. However, important abberations are observed between predictions made by the Terzaghi theory and observations of the time rate of consolidation, both in the field and during laboratory tests. The most notable difference in the time-deformation curves occur when the theoretical primary curve approaches its 100 per cent consolidation (see Fig. 2.1). It is found that, the

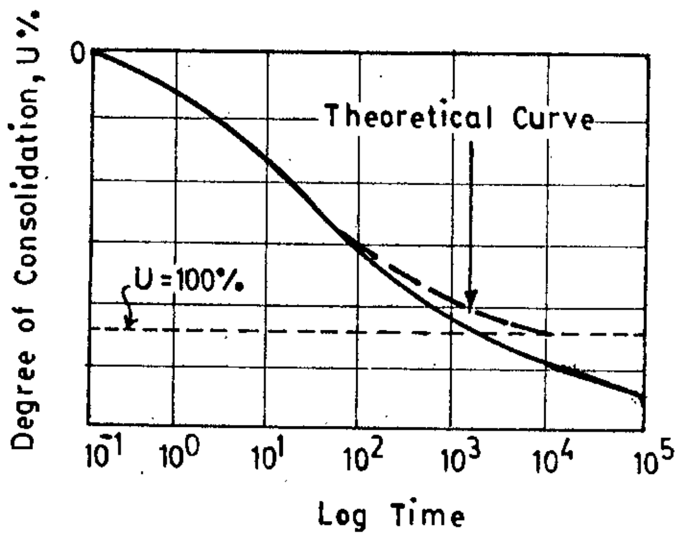

Fig. 2.1 Typical consolidation versus logarithm of time curve.

experimental time consolidation curve is in agreement with Terzagh's theory of consolidation only upto about 60 per cent consolidation. This difference between observed consolidation and theoretical primary consolidation is due to the secondary time effects and defined as secondary compression.

2.4 General Characteristics of Time-deformation Curves

The time-deformation curves for remoulded clays for secondary compression have a similar shape throughout all the load increments irrespective of the type of clay. With a few exceptions for laboratory clays, the ultimate settlement may be reached within three weeks, the time required increase slightly with the loading increments. The undisturbed samples, however, exhibit pronounced changes in the shape of secondary compression curves for different load ranges. For pressure increments below the preconsolidation load, the secondary time effect is small and the final settlement is reached within a period of less than a week. At pressures near the preconsolidation load, there is an abrupt change in the shape of compression curve and secondary compression is important (Lo, 1961).

From a study of the available literatures published on the time-deformation characteristics and experimental evidences, Lo (1961) classified time-deformation curves from their characteristic shapes in a logarithmic time plot into three catagories as shown in Fig. 2.2.

(a) Type-I curve has a gentle curvature concaving upwards, the rate of secondary consolidation decreasing with time and the curve becomes horizontal when the ultimate settlement is reached. The time required for this type of curve to reach final settlement is comparitively short being the order of 20 days or so for remoulded soils. Most of the remoulded sample of different clays fall within this group. The distinctive characteristics of this type of curve is the existence of an inflection point.

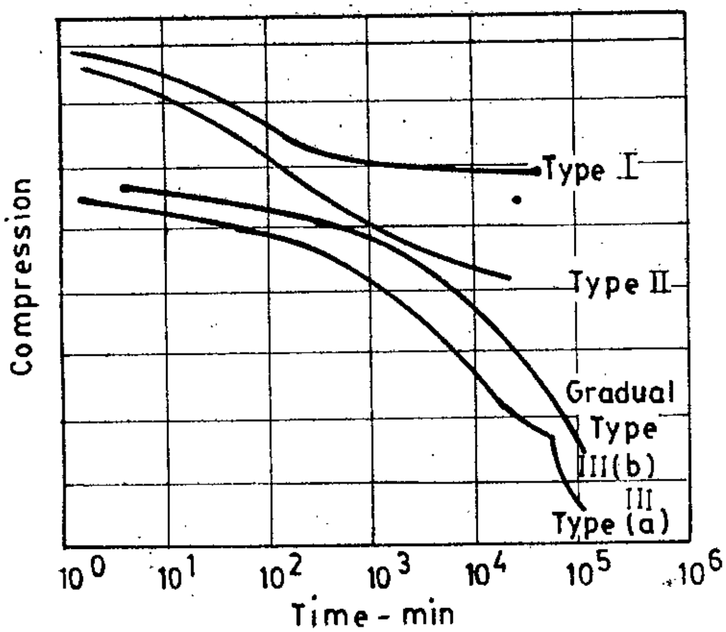

Fig. 2.2 Types of compression curves; Lo (1961).

(b) Type-II curves are characterized by the proportionality of secondary compression with the logarithm of time for an appreciable range of time. The rate of compression decreases rapidly near the ultimate stage and become zero as the final settlement is attained. This type of curve has no well marked inflection point.

(c) Type-III curve is concave downward, the rate of secondary compression increases with the logarithm of time, then slows down gradually and finally vanishes. The acceleration of compression can be either gradual as in Type III(b) or abrupt as in Type III(a). The ultimate settlement for this type of compression takes a long period to attain and tests take up to 100 days or more to reach equilibrium. Type-III curves are present only in natural deposits of a loose structure and is believed to the breakage of the bond between particles. The rate, however, decreases at large time and becomes zero at the ultimate stage.

2.5 Factors Affecting Time-deformation Behaviour of Soils

The shape of the time-deformation curve can vary over a wide range depending on the soil type, structural arrangement of soil, total pressure and void ratio, stress history, and on drainage condition.

Effect of Soil Type : The relative magnitude of primary and secondary compression vary with soil type. Wahls (1962) pointed out that for most inorganic clays, primary consolidation is much greater than secondary compression. However, in most organic soils and in some inorganic silts, the magnitude of secondary compression is significant. For some of these soils, the secondary compression is so large that the observed consolidation curve bears little resemblance to the theoretical primary consolidation curve.

Effect of Structure: The structure of a natural deposit can have a marked influence on the time-deformation characteristics of soil. Lo (1961) reported that Type-III curves are present only in natural deposits of a loose structure and most of the remoulded samples of different clay gives Type-I time-deformation curves. The later case takes much less time than the undisturbed sample of the same soil. Scott (1963) mentioned that when a body of soil is subjected to a given deformation which is thereafter held constant, it developes an initial interparticle stress which diminishes as a function of time as the molecules gradually migrate to minimum energy position. Therefore, the deformation process depends on the size of clay platelets and the structural arrangement of particles as they affect the yielding or sliding of clay platelets of their points of contact.

Effect of Total Pressure and Void Ratio: Based on the secondary compression characteristics observed in the laboratory studies Wahls (1962) found that the magnitude of secondary compression is a function of total pressure, the void ratio and time and is independent of the magnitude of the pressure increment. Vaid and Campenella (1977) observed that deformation increases under constant applied stresses as the level of creep stress increases.

Effect of Stress-History: Vaid and Campenella (1977) investigated the time dependence of undrained stress-strain and strength behaviour of a natural sensitive marine clay and found that the stress-strain and strength response of clay is a function of history of loading or deformation. They found good agreement between the observed and predicted behaviour to correlate stress-strain-time behaviour of clay by using the concept that stress is a function of current strain and strain rate only.

Mesri and Godlewski (1977) reported that every mechanism of volume change is a chain reaction process and is therefore time-dependent. The magnitude of the changes in fabric and in the forces between particles during an increment of time or an increment of effective stress should depend on the intensity of the previous changes with time and effective stress.

Effect of Drainage Condition: The relative magnitude of primary and secondary compression depend on the time required to dissipate excess pore pressure and hence on the thickness of the soil specimen. With the decrease in the length of the drainage path, the time required to dissipate pore pressure becomes shorter and the secondary compression is more pronounced. From the test results of Berre and Iversen (1972), it was found that the rate of secondary compression increases with the increase of the thickness of the specimen though more time is required for the dissipation of excess pore water pressure.

2.6 Causes of Secondary Compression

Possibly the viscous yielding of the grain structure is the cause of secondary compression. A delayed progressive slippage of grain upon grain, or plate upon plate, as the particles adjust themselves to a dense condition in the viscous state, appears to be responsible for secondary compression. Terzaghi and Peck (1948) assumed this concept that the secondary compression is due to the gradual readjustment of the soil structure to stress combined with the resistance offered by the viscosity of the adsorbed layers to a slippage between grains. Wahls (1962) also support the view of the viscous yielding of the grain structure as the cause of the secondary compression. The effective pressure that the interrgranular structure can support depends on the particle orientation as well as the void ratio. Therefore viscous reorientation of the grains gradually reduces the capacity of the intergranular skeleton and produces a tendency for a small part of the intergranular pressure to be transferred to the pore water. This tendency is relieved if the void ratio is reduced slightly. It is generally assumed that the particle orientation occurs so slowly that the pore pressure developed by the process are negligible. Thus, the secondary rate of void-ratio change is governed entirely by the rate of viscous yielding and is independent of hydrodynamics.

Barden (1968) suggested that a cause of prolonged secondary tail is due to the marked decrease of permeability during the consolidation process. According to him the possibility of micropore structure being responsible for secondary effect. He considered the soil as a domain or packet structure containing micropores, interwoven by a network of macropore. The primary stage is the usual Terzaghi process governed by the rate of dissipation of pore pressure in the network of macropores. As this pore pressure falls, load is transferred to the packets and secondary consolidation process taking place in the micropores of the packets; the rate of consolidation then being independent of the overall sample dimension.

Sowers (1979) stated that secondary compression appears due to the result of plastic readjustment of the soil grains to the new stress, of progressive fracture of the particle themselves.

2.7 General Creep Behaviour of Soil

If a given stress is applied to a body of soil a continuous deformation will take place as the soil particles migrate in the microscopic stress field, provided that, the microstress exceeds a certain value determined by the nature of the intermolecular bonds, the regularity of the structure and the thermal state of the system. The phenomenon is called plastic flow or creep. The creep behaviour is present to some extent in primary consolidation and solely in secondary compression. In the following discussions, the term 'creep' will be used for the deformation of the soil structure due to the plastic flow at constant stress in both the primary and secondary compression. Sometimes creep stress is expressed as a percentage of failure strength of the soil and is termed as stress level.

A number of different rheological models have been proposed to provide a description of the creep behaviour of soils. Some investigators (Gibson and Lo, 1961; Murayama and Shibate, 1961; Garlanger; 1972) constructed the mechanical models of clay by introducing mechanical elements like springs, dashpots and friction surface and arranged them in such a way that they provide a reasonable approximation of the behaviour of certain soils and loading conditions. Others (e.g., Singh and Mitchell, 1968) used phenomenological models. The mechanical models gave rise to differential equations which were complicated and difficult to solve even for simple boundary conditions (Scott, 1963). For the present thesis studies concerning the creep behaviour of soil were limited to the phenomenological models only.

Finne and Heller (1959) suggested that following the period of transient creep during which the strain rate continously decreases, creep continues at a constant rate for some period of time as shown in Fig. 2.3 for a soil consisting of a mixture of 40 per cent kaolinite and 60 per cent sand under two different creep stress level.

In the course of continuing study of the stress-strain-time behaviour of soils, Singh and Mitchell (1968) observed a phenomenological relationship of the influence

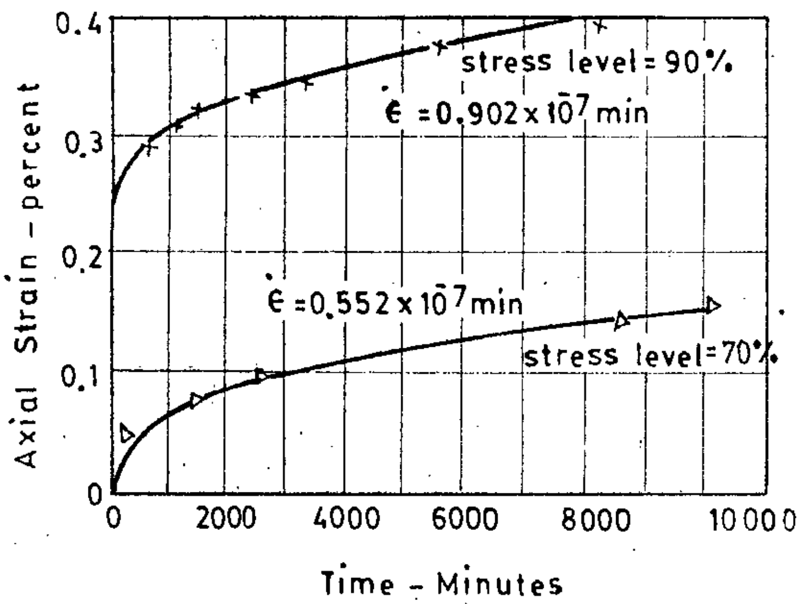

Fig. 2.3 Variation of axial strain with time during creep; Paduana (1965).

of the creep stress intensity on creep rate at some given time after the stress is applied. In the midrange of stresses which are representative of stresses used in practice, a nearly linear relationship is found between the logarithm of strain rate and stress. At low stresses creep rates are small and of little practical interest. At stresses approaching the failure stress of the material the strain rates become very large and failure occurs. Fig. 2.4 and 2.5 show this type of behaviour of soil.

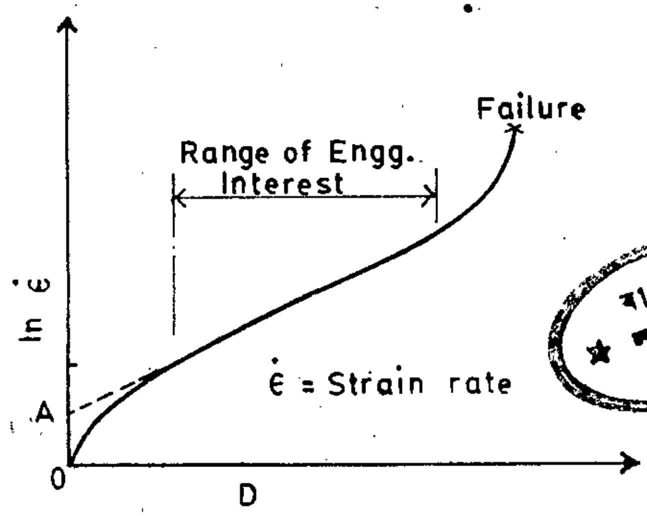

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Fig. 2.4 Influence of creep stress intensity on creep rate; Singh and Mitchell(1968).

For many clays subjected to loading within the range of engineering interest for instance 30 percent to 90 per cent of failure strength there appears to be a characteristic relationship between strain and time. This relationship is best illustrated by the form of a plot between strain rate and time as shown as an example for San Francisco bay mud in Fig. 2.6.

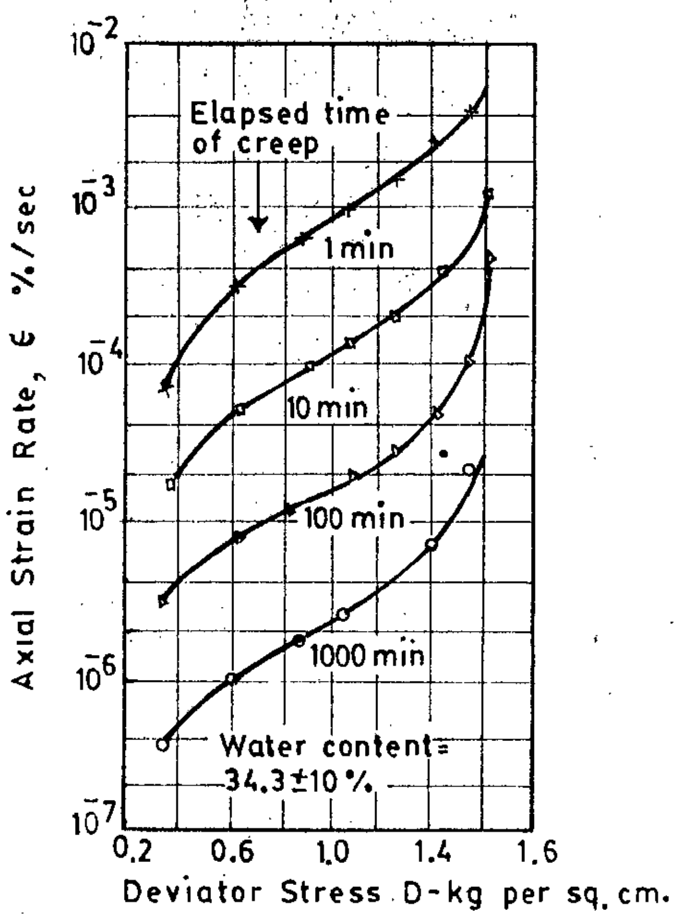

Fig. 2.5 Variation of strain rate with deviator stress for undrained creep of remoulded illite; Singh and Mitchell (1968).

2.8 Singh-Mitchell (1968) Creep-rate Functions

Based on the phenomenological relationship mentioned above Singh-Mitchell (1968) developed a creep rate function for a range of stresses where the curves are linear.

The relationship shown can be expressed by,

$$\ln\left(\frac{\dot{e}}{\dot{e}(t_1, D)}\right) = -m \ln\left(\frac{t}{t_1}\right) \qquad (1)$$

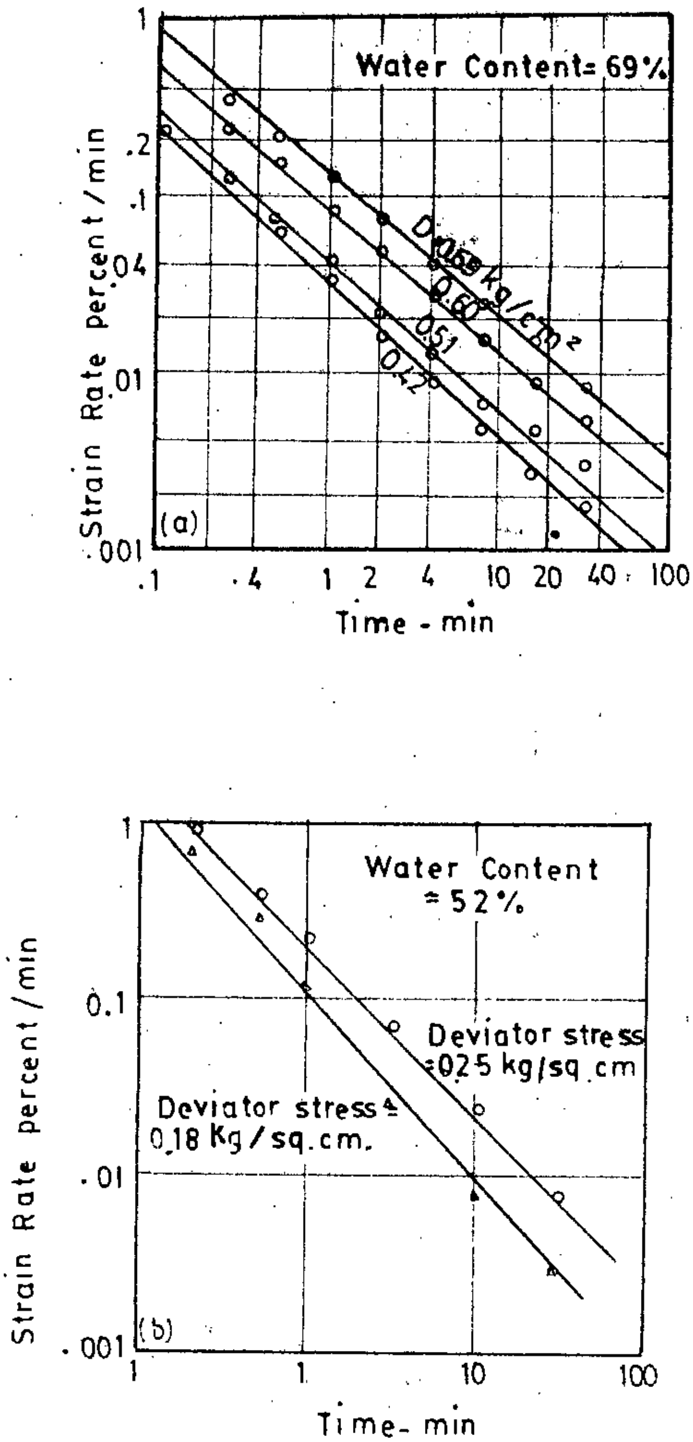

Fig.2.6 Strain rate versus time relationship during undrained creep of (a) Undisturbed (b) Remoulded San Francisco bay mud.

or, $$\ln \dot{e} = \ln \dot{e}_{(t_1,D)} - m \ln \left(\frac{t}{t_1}\right) \tag{2}$$

in which $\dot{e}$ = strain rate at any time, t;

$\dot{e}_{(t_1,D)}$ = value of strain rate at unit time, a function of stress intensity, D;

m = absolute value of slope of the straight portion of the logarithm of strain rate versus logarithm of time plot; and

t_1 = unit time, e.g., 1 min.

The same data plotted in the form of Fig. 2.5 can be expressed by

$$\ln \left(\frac{\dot{e}}{\dot{e}_{(t,D_o)}}\right) = \alpha D \tag{3}$$

or, $$\ln \dot{e} = \ln \dot{e}_{(t,D_o)} + \alpha D \tag{4}$$

in which $\dot{e}_{(t,D_o)}$ = fictitious value of strain rate at D=0, a function of time after start of creep, t ;

α = value of slope of the linear portion of the logarithm of strain-rate versus deviator-stress plot.

Eliminating $\dot{e}$ from equation (1) and (3) gives,

$$\ln \dot{e}_{(t_1,D)} - m \ln \left(\frac{t}{t_1}\right) = \ln \dot{e}_{(t,D_o)} + \alpha D \tag{5}$$

For the case of D = 0, this equation may be written as

$$\ln \dot{e}_{(t,D_o)} = \ln \dot{e}_{(t_1,D_o)} - m \ln \left(\frac{t}{t_1}\right) \tag{6}$$

in which $\dot{e}_{(t_1,D_o)}$ = value of strain rate obtained through projecting the straight portion of the relationship between log-strain rate and deviator stress at any time to a value of D = 0.

Thus eq. (4) becomes,

$$\ln \dot{e} = \ln \dot{e}_{(t_1,D_o)} + \alpha D - m \ln \left(\frac{t}{t_1} \right) \tag{7}$$

which may be written as

$$\dot{e} = \dot{e}_{(t_1,D_o)} \; e^{\alpha D} \left(\frac{t_1}{t} \right)^m \tag{8}$$

or, $\dot{e} = Ae^{\alpha D} \left(\frac{t_1}{t} \right)^m$ (9)

in which $A = \dot{e}_{(t_1,D_o)}$ (10)

Eq.(9) is a simple three-parameter relationship which appears adequate for the description of creep rate characteristics of a variety of soils.

When the creep stress is expressed as a unit of stress, the parameter α will have units of reciprocal stress, e.g., cm^2 per kg. If, on the other hand, the stress is expressed as a stress level, i.e, the ratio of creep stress to failure strength at the beginning of the creep, D/D_f then the dimensionless function αD_f is appropriate. It has been found that for a given soil type, values of αD_f do not vary greatly for different water contents. This fact was also confirmed by Campanella (1965) from creep tests on illite.

Thus eq. (9) may be further generalised to

$$\dot{\epsilon} = A e^{\bar{\alpha}\bar{D}} \left(\frac{t_1}{t} \right)^m \qquad (11)$$

in which

$\bar{\alpha} = \alpha D_f$ and $\bar{D} = D/D_f$ = stress level.

2.9 Singh-Mitchell (1968) Creep Function

A general relationship between strain and time may be obtained by integration of eq. (11).

$$\epsilon = A e^{\bar{\alpha}\bar{D}} (t_1)^m \left(\frac{1}{1-m} \right) t^{1-m} + \text{const.} \qquad (m \neq 1) \qquad (12)$$

It has been found that the value of the parameter m for practical purposes may be taken as unity for many cases. Then

$$\epsilon = A e^{\bar{\alpha}\bar{D}} t_1 \ln t + \text{const.} \qquad (m = 1) \qquad (13)$$

The constant of integration in eq. (12) and eq. (13) can be evaluated from a known value of strain at some known value of time, for instance, unity. If ϵ_1 is strain at unit time t = 1 and if $t_1 = 1$, the constant of integration, a of eq. (12) will be

$$a = \epsilon_1 - \frac{A}{1-m} e^{\bar{\alpha}\bar{D}} \qquad (14)$$

and for eq. (13) it will be $a = e_1$ (15)

Thus the solution given by eq. (12) and eq. (13) can be rewritten as

$$e = a + \frac{A}{1-m} e^{\bar{\alpha}\bar{D}} (t)^{1-m} \quad (m \neq 1) \tag{16}$$

and

$$e = e_1 + Ae^{\bar{\alpha}\bar{D}} \ln t \quad (m = 1,\ t>1) \tag{17}$$

The eqs. (16) and (17) are the derived Singh-Mitchell (1968) creep function. An examination of eqs. (16) and (17) shows that only for the case m = 1, the strain vary linearly with logarithm of time (provided $t > 1$). In general case $m \neq 1$, a nonlinear relationship holds.

2.10 Limitations of Singh-Mitchell (1968) Creep Function

Singh-Mitchell (1968) general model has been proposed to describe the axial deformation of samples in triaxial compression when subject to a constant deviator stress (creep). The rate of deformation is expressed as a function of three material constants — A, $\bar{\alpha}$ and m. These parameters describe many of the facets of the real soil behaviour. The fictitious nature of the parameter A is shown in Fig. 2.4. It is meaningful, however, it reflects the order of magnitude of the creep rate for the soil and thus in a sense of soil property that reflects composition, structure and stress history. The parameter $\bar{\alpha}$ indicates stress intensity effect on creep rate and the value of the parameter m reflects the rate at which the strain rate decreases with time. The value of m is found to lie between 0.75 and 1.0. It has been found that the value of parameter m may for practical purposes be taken as unity in many cases. Singh-Mitchell (1968) mentioned a number of such clays - Osaka alluvial clay, dry illite, London clay, remounded San Francisco bay mud.

The three parameters A, $\bar{\alpha}$ and $\bar{m}$ are calculated from two phenomenological relationships as mentioned before (Art. 2.7 and 2.8). Singh-Mitchell (1968) found these relationships valid for normally consolidated San Francisco bay mud both in undisturbed and remoulded (Fig. 2.6) conditions. Similar relationships were observed for saturated remoulded illite (Fig. 2.5; data taken from Campanella, 1965), dry illite and overconsolidated San Francisco bay mud, all tested in undrained creep. The results of drained creep tests on undisturbed London clay (Bishop, 1966), undisturbed Osaka alluvial clay (Murayama and Shibata, 1966) give the same relationship. Andersland and Akali (1967) have also observed a linear relationship between logarithm of strain rate and creep stress for frozen soils at high stresses.

Using the parameters A, α and m Singh-Mitchell (1968) developed a creep equation for a sustained deviator stress D. To compare the observed creep with creep predicted by Singh-Mitchell (1968) creep function, creep tests on illite and overconsolidated San Francisco bay mud were performed by them. The comparisons are illustrated in Fig. 2.7(a) and 2.7(b) respectively. In each case the value of 'a' or 'e_1' of eq. (16) or (17) was chosen to give the best fit between

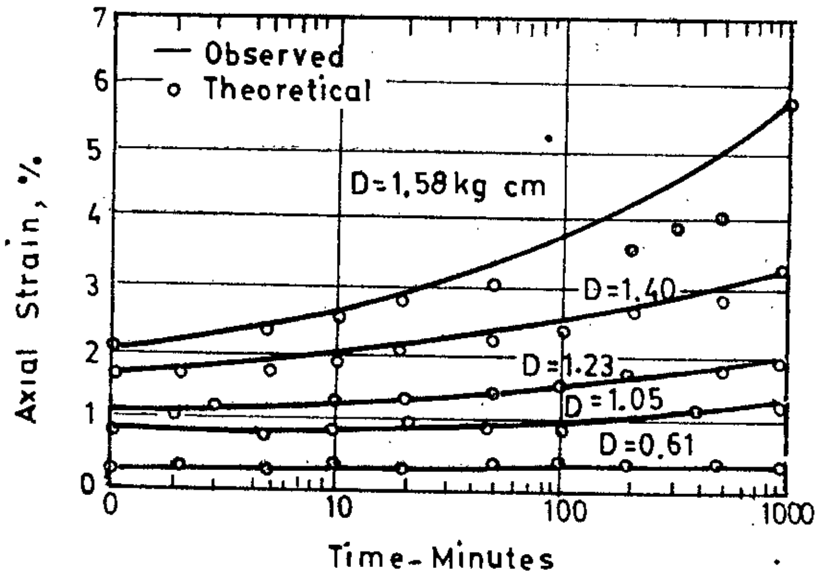

Fig. 2.7(a) Comparison between theoretical and observed creep of illite; Sing and Mitchell (1968).

observed and predicted behaviour. The agreement is observed good over a significant range of stress and time, however, it deviates as time and stress are increased. On the other hand, using the data from Bishop (1966) they found that the prediction is substantially in accordance with the real soil behaviour for any time and for any time and for the allowable stress range.

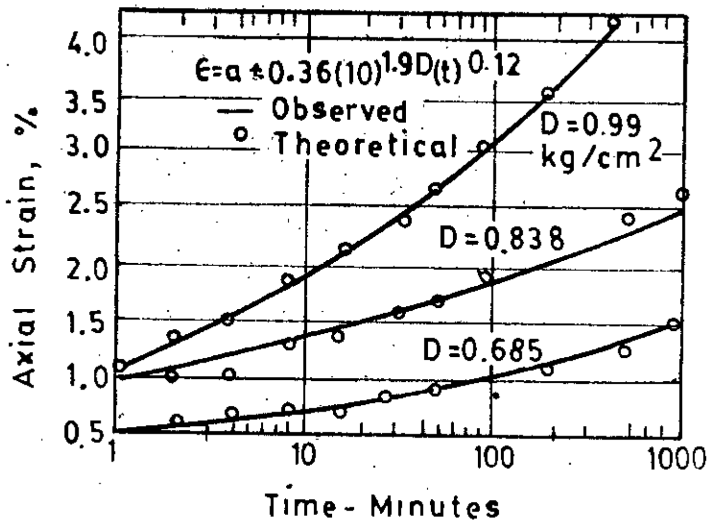

Fig. 2.7(b) Comparison between theoretical and observed creep of overconsolidated San Francisco bay mud; Singh and Mitchell (1968).

Though limited information is available, the results have proven to be valuable for further study of Singh-Mitchell (1968) creep function in order to investigate the stress-strain-time behaviour of soil.

2.11 Application of Singh-Mitchell (1968) Creep Function in one Dimensional Compression

Singh-Mitchell (1968) creep function is developed to describe the axial deformation of cylindrical samples under constant deviator stress in triaxial compression. The axial deformation is expressed as a function of three material constants (A, $\bar{\alpha}$, m), the deviator stress (D), and the instantaneous deviator stress which would cause failure in triaxial compression (D_f).

For one dimensional compression, it is necessary to predict the axial deformation. The equivalent result can be obtained by considering the influence of lateral earth pressure on time-dependent deformation of the soil.

The lateral effective stress, $\bar{\sigma}_3$ can be expressed in terms of the vertical effective stress $\bar{\sigma}_1$ and the coefficient of earth pressure at rest, k_o.

$$\bar{\sigma}_3 = k_o \ \bar{\sigma}_1 \tag{18}$$

$$D = \sigma_1 - \sigma_3 = \bar{\sigma}_1 - \bar{\sigma}_3 = \bar{\sigma}_1(1-k_o) \tag{19}$$

Unconfined compressive peak strength may be taken as the failure deviator stress, then

$$D_f = (\sigma_1 - \sigma_3)_f = \sigma_{1f} \quad (\text{Since } \sigma_3 = 0) \tag{20}$$

In such situation the stress level, ($\bar{D}$)

$$\bar{D} = D/D_f = \bar{\sigma}_1(1-k_o)/\ \sigma_{1f} \tag{21}$$

Now eqs. (16)and(17) of Singh-Mitchell creep function may be used taking the modification of $\bar{D}$ and $\bar{\alpha} = \alpha\ \sigma_{1f}$.

Again k_o is time dependent. Lacedra (1976) observed the variation of k_o with time. He found that k_o increasing from 0.53 at t = 1,000 minutes to 0.58 at t = 10,000 minutes for undisturbed San Francisco bay mud. He, however, neglected the increase of D_f due to consolidation and thus possibility of overestimating the value of k_o at larger times.

Neglecting the variation of k_o with time and pressure, its value can be calculated from the empirical relationship among k_o, overconsolidation ratio (=OCR), and plasticity index given by Brooker and Ireland (1965) shown in Fig. 2.8.

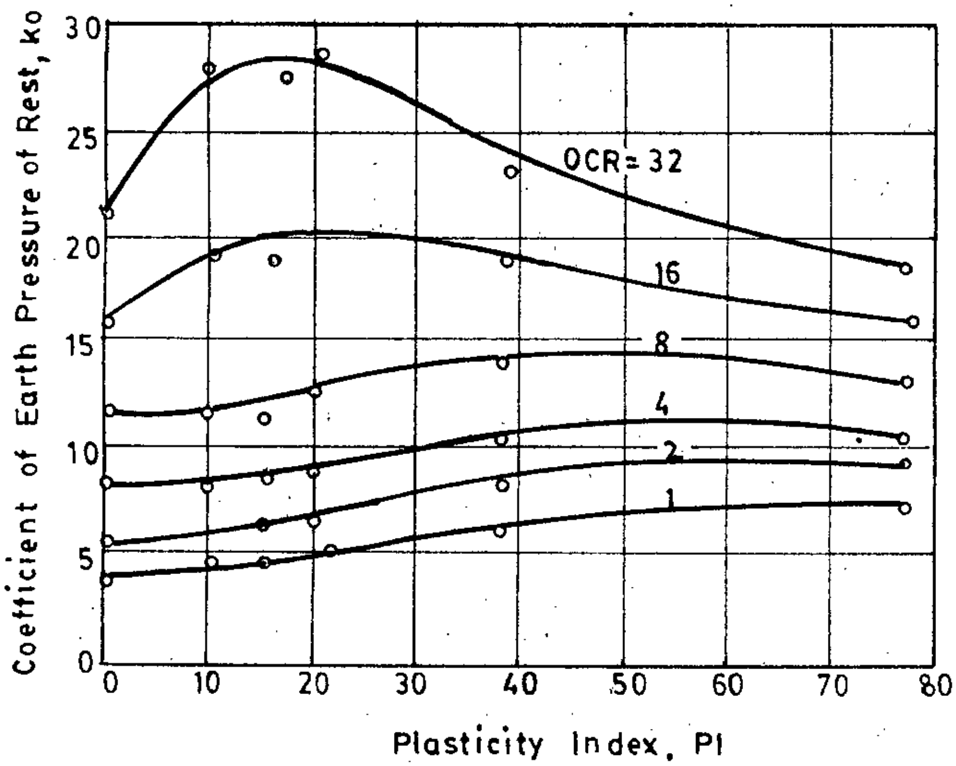

Fig. 2.8 k_o as a function of overconsolidation ratio and plasticity index, Brooker and Ireland (1965).

Over the region from PI = 0 to 40 per cent the relationship is approximately linear for nomrally consolidated clay with the following approximate equation.

$$k_o = 0.40 + 0.007\ PI \tag{22}$$

In the region PI = 40 to 80, k_o is approximately

$$k_o = 0.68 + 0.001\,(PI - 40) \qquad (23)$$

Alpan (1967) recommends

$$k_o = 0.19 + 0.233 \log PI \qquad (24)$$

for normally consolidated clay, where PI = plasticity index in percent.

Both the methods for determination of k_o were used in determining the deviator stress.

2.12 Characteristic Behaviour of Secondary Compression

To describe the secondary effects of compression of soils, C_α, the coefficient of secondary compression is used. The value C_α is defined as the slope of the void ratio versus log time curve after excess pore pressure have completely dissipated i.e, in the secondary compression range (see Fig. 2.9).

C_α in One Dimensional Compression : The most common assumption about C_α in one dimensional compression is that it is constant i.e.,secondary compression curve on a void ratio vs. log time plot is a straight line, the slope of which is independent of volumetric stress. Ladd and Preston (1965) conducted a comprehensive study one secondary compression in one dimensional consolidation tests and concluded

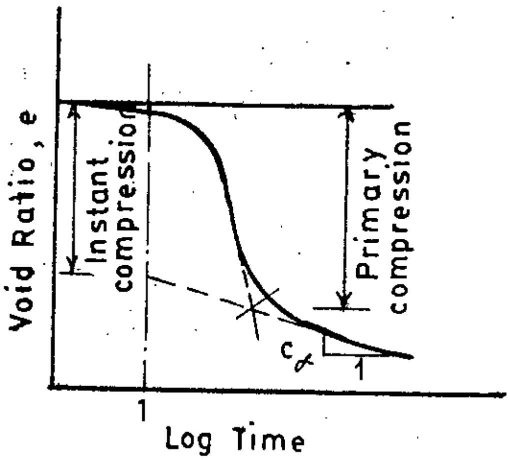

Fig. 2.9 The coefficient of secondary compression, C_α from consolidation test.

that this assumption is reasonable for most engineering problems. However, Wahls (1964) refers the experiences of Gray with New England clays where C_α atteins a maximum value in the vicinity of preconsolidation pressure and remains approximately constant for higher pressures. Sowers (1979) noted that C_α varies with effective pressure. Wahls (1962) studied a number of soils in which C_α remained practically constant beyond the preconsolidation pressure, for some soils C_α decreased with increasing pressure and for a few soils no trends were apparent. The later case involved soils for which the magnitude of secondary compression was small. Fig. 2.10 shows the variation of C_c and C_α with consolidation pressure for three clays as observed by Mesri and Godlewski (1977).

From field and laboratory measurements of secondary compression curve Mesri and Godlewski (1977) have demonstrated that in many cases the secondary compression curve

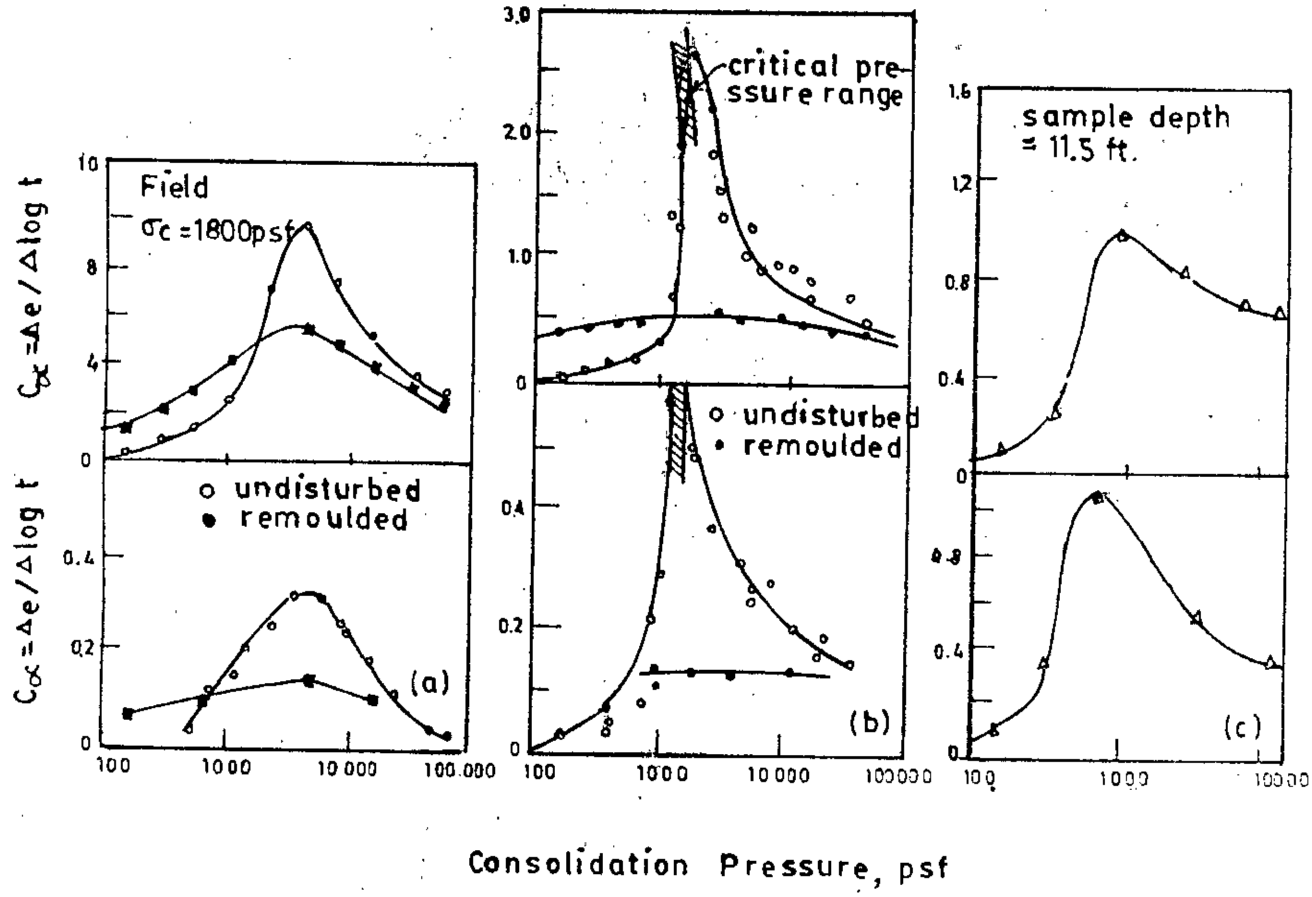

Fig. 2.10 Relationship between C_c and C_α and consolidation pressure for (a) Mexico city clay (b) Leda clay (c) Undisturbed New Haven Organic silt; Mesri and Godlewski (1977).

does not have same slope in void ratio versus logarithm of time plot. They showed that the behaviour of this type is most pronounced and often an increase in C_α for consolidation pressure in the range of 0.5 σ_c to 1.5 σ_c, in which σ_c is the critical pressure. Fig. 2.11 shows this type of behaviour of New Haven silt for consolidation pressure near critical pressure.

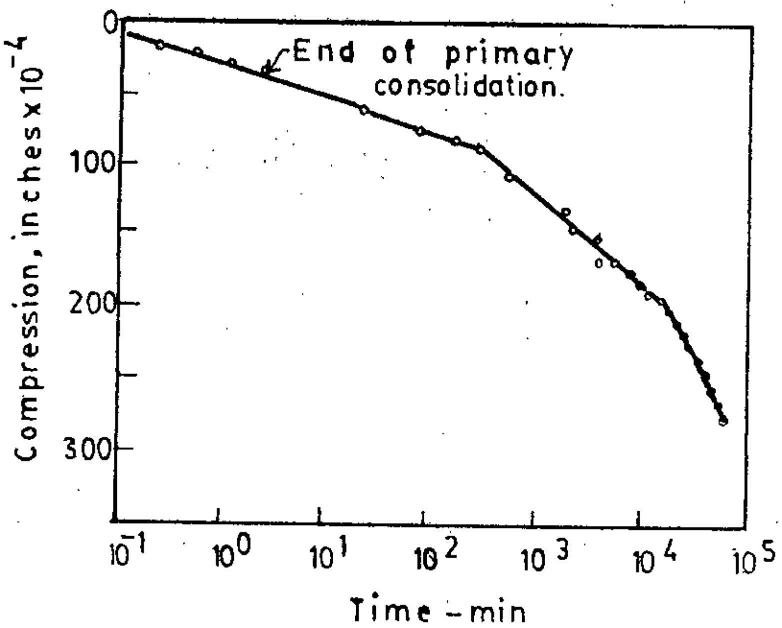

Fig. 2.11 Compression vs. log time curve for consolidation pressure near critical pressure, New Haven silt; Mesri and Godlewski(1977).

They also observed that, for consolidation pressure less than the criticel pressure, C_α increases with time whereas it decreases with time for pressure greater than the critical pressure. The critical pressure is the pressure obtained from the void ratio versus log pressure curve corresponding to the end of primary consolidation. Fig. 2.12 shows the relationship of C_α with time for two different soils. It was observed that for consolidation pressure near the critical pressure C_α initially increases with time and then decreases.

<u>C_α in Three Dimensional Compression:</u> Several investigators have measured C_α for stress states other than the

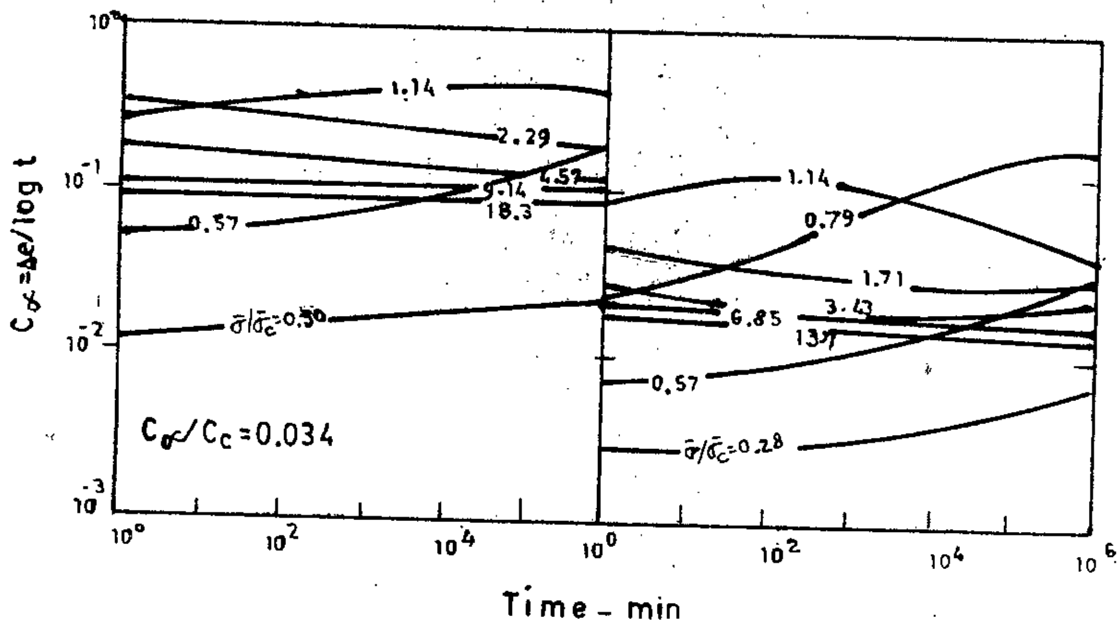

Fig. 2.12 Relationship between C_α and time for (a) Undisturbed Mexico City clay (b) Undisturbed Leda clay; Mesri and Godlewski (1977).

one dimensional case. Ladd and Preston (1965) measured C_α in triaxial compression test for different deviatoric stress level. They concluded that C_α could reasonably be assumed constant. Walker (1969) studied the volumetric creep under triaxial conditions for deviatoric stress levels for less than 10 per cent to greater than 85 per cent of the peak shear stress. He found that C_α is independent of deviatoric stress level. However, he reported a dependence on stress system, finding different values for C_α in triaxial compression. de Ambrosis (1974) in his study of the creep settlement of foundation, concluded that the effect of stress level on C_α to be insignificant.

2.13 Relationship among C_α, C_c and t

From the analysis of consolidation data on samples of three natural soil deposits, Mesri and Godlewski (1977) concluded that for any natural soil a unique relationship exists between true secondary compression index C'_α = e/ log t and true primary compression index C'_c = e/ log .. However, this result may not be true in general. This unique relationship between C'_α and C'_c holds true at any effective stress level, void ratio condition and time during the secondary compression stage. This is shown in Fig. 2.13 for two clays.

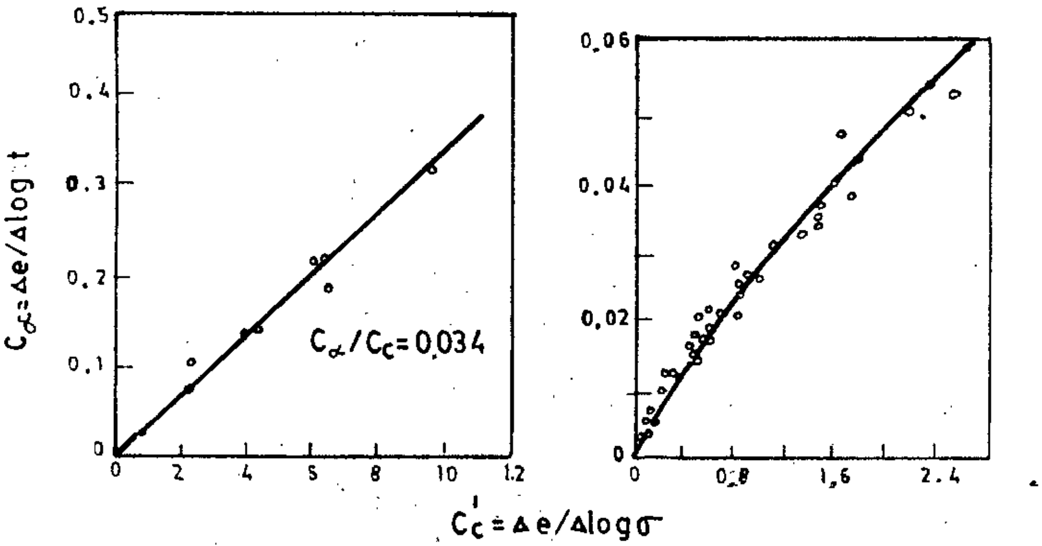

Fig. 2.13 Relationship between C'_α and C'_c for (a) Mexico City clay, (b) Lede clay; Mesri and Godlewski (1977).

They also demonstrated that for a variety of natural soils the value of C_α/C_c are in the range of 0.025 - 0.10;

the range for inorganic soil is 0.025 - 0.06. The high values of C_α/C_c correspond the high organic soils. Table 2.1 provides a list for C_α/C_c values for a number of soils.

Table 2.1 Values of C_α/C_c for Natural Soil Deposits (after Mesri and Godlewski, 1977)

Soil type	C_α/C_c
Whangamarino clay	0.03-0.04
Norfork organic silt	0.05
Calcareous organic silt	0.035-0.06
Amorphus and fibrous peat	0.035-0.083
Canadian muskeg	0.09-0.10
Leda clay	0.025-0.06
Peat	0.05-0.08
Post-glacial organic clay	0.05-0.07
Soft Blue clay	0.026
Organic clays and silts	0.04-0.06
Sensitive clay, Portland	0.025-0.055
San Francisco bay mud	0.04-0.06
New Liskeard varved clay	0.03-0.06
Silty clay, C	0.032
Nearshore clays and silts	0.055-0.075
Mexico silty clay	0.03-0.035
Hudson River silt	0.03-0.06
New Haven organic clay silt	0.04-0.075

The magnitude of C_α/C_c can be used to predict the shape of the volume change vs. logarithm of time curve in the transition from primary to secondary. This may provide a quantitative explanation for the observed shape of the settlement curve.

2.14 Kavazanjian-Mitchell (1980) Constitutive Model

A general constitutive model for the time dependent deformation behaviour of cohesive soil is developed by Kavazanjian and Mitchell (1980) by unifying existing phenomenological models for the stress-strain-time behaviour of cohesive soil under arbitrary three dimensional states of stress.

This constitutive model composed of separate, but not independent volumetric and deviatoric components. Each model component is assumed to consist of time-dependent or delayed and time-independent or immediate contributions.

Bjerrum (1967) suggested that the deformation of a soil is composed of immediate and delayed components. The immediate (or instant) component is that deformation which would result if the excess pore water pressure set up by a foundation loading could dissipate instantaneously with load application, delayed compression is the settlement developing at constant effective stress. The concept of instant and delayed compression is illustrated in Fig.2.14.

The concept of immediate compression differs from that of primary consolidation, defined as compression occuring during dissipation of excess pore pressure in a consolidation

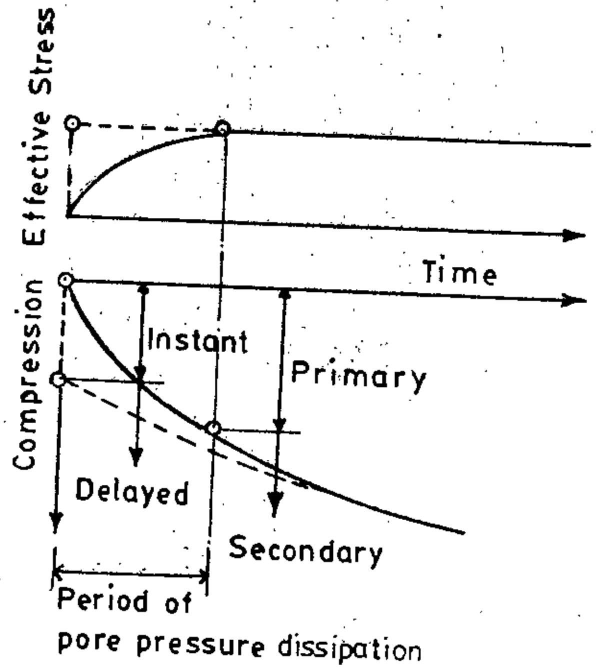

Fig. 2.14 Instant and delayed compression, after Bjerrum (1967).

test, in that primary compression can contain both immediate and delayed contributions. Secondary compression, however, consists solely of delayed deformation.

The elimination of hydrodynamic lag in the fine grained soil is not possible without altering its properties. Therefore, immediate compression cannot be measured directly. If laws governing the delayed volumetric compression of soils can be deduced from observations of secondary compression, the immediate deformation component can then be determined by subtracting the delayed contribution from primary compression.

Volumetric Model: The void ratio - log stress describe the immediate volumetric compression and C_α, the coefficient of secondary compression was used to describe the delayed volumetric compression of cohesive soils.

Based on the discussion in Art. 2.12, C_α was assumed independent of both deviatoric and volumetric stresses and assumed to be valid over a wide range of conditions encountered within normal engineering practice.

$$(e_v)_d = \frac{C_\alpha}{1+e_o} \log (t/t_i) \qquad t > t_i \tag{25}$$

where $(e_v)_d$ = the delayed volumetric strain;

e_o = the initial void ratio;

t_i = the time required for instant compression to occur;

t = any time, $t > t_i$.

The use of such a logarithmic model for delayed compression necessitates arbitrarily defining a time required for immediate compression to occur. For convenience, it is suggested that t_i is taken equal to unity. Thus volumetric strain e_v is

$$e_v = \frac{e}{1 + e_o} = (e_v)_i + (e_v)_d \tag{26}$$

in which $(e_v)_i$ = immediate volumetric strain.

The virgin compression index C_c was also assumed constant. This two assumptions create a general model. Fig. 2.15 is representation of the general volumetric

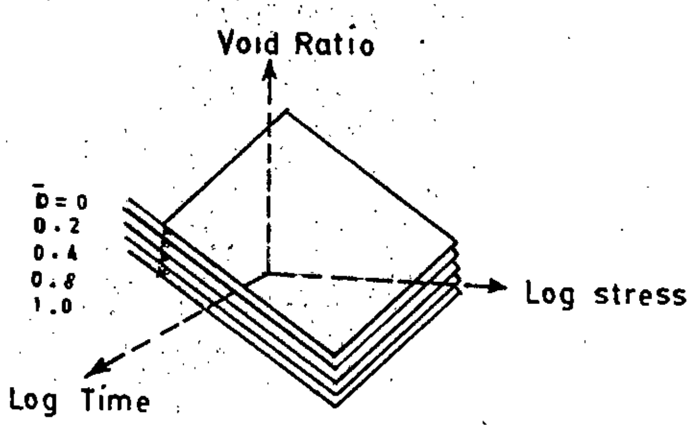

Fig. 2.15 General volumetric model, after Kavazanjian and Mitchell (1980).

model in void ratio-log stress-log time space. Deformation at constant deviatoric stress levels (or principal stress ratios) can be described by a unique relationship among void ratio, effective stress and time under sustained loading on an element of cohesive soil.

Deviatoric Model: To describe the time-independent undrained stress-strain behaviour of soils, any of the existing models can be used to describe immediate deviatoric behaviour. For reason of aimplicity and compatibility, Kavazanjian-Mitchell (1980) adopted the Kondner's (1963) hyperbolic stress-strain model in conjunction with Ladd and Foott's(1974) concept of normalized soil properties.

Kondner (1963) suggested that the stress-strain curve for a cohesive soil from a consolidated undrained triaxial test could be represented by a rectangular hyperbola. He demonstrated that the deviatoric behaviour of a cohesive soil at a specified overconsolidation ratio, rate of strain and a confining pressure can be represented by a straight line plot in which axial strain/deviatoric stress is plotted as a function of axial strain. The slope of the transformed plot is the reciprocal of the ultimate strength of the soil and the intercept is the reciprocal of the initial tangent modulus of the soil*.

Comparison of best fit hyperbolic curves with actual stress-strain curves has shown the asymptotic value of the fitted hyperbola overestimates the shear strength of the soil. For this Kavazanjian and Mitchell (1980) employed a third parameter, called R_f by Duncan and Chang (1970) which gives the ratio of the actual shear strength of the soil to the asymptote of the fitted hyperbolic curve.

To describe the stress dependence of the deviatoric stress-strain behaviour of normally consolidated cohesive soil Kavazanjian and Mitchell (1980) adopted the Ladd and Foott's (1974) suggestion and modified this model for over-consolidated soil. Since the interest is on volumetric model, no details are included in this review. However, papers by Ladd and Foott's (1974) together with Hvorslev's (1960) suggestion about equivalent consolidation pressure for overconsolidated soils may be consulted.

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* Detailed mathematical proof of the Kondner's (1963) transformed hyperbolic model is given in Appendix-I.

To describe the delayed deviatoric deformation of cohesive soils, Singh-Mitchell (1980) creep model is used. They choose this particular model because it is simple as it requires only three parameters to describe delayed deviatoric deformation and as it provides a good representation of the soil behaviour.

The general deviatoric model evaluates both immediate and delayed deformation as a function of the stress level, $(\sigma_1 - \sigma_3)/(\sigma_1 - \sigma_3)_f$, in which $(\sigma_1 - \sigma_3)$ is the applied deviator stress and $(\sigma_1 - \sigma_3)_f$ is the deviator stress at failure evaluated on the basis of the unique relationship between void ratio and deviator stress at failure postulated by the general volumetric model.

2.15 Limitations of Kavazanjian-Mitchell (1980) General Model for Predicting Stress-Strain-Time Behaviour of Soil

Kavazanjian and Mitchell (1980) developed a laboratory experiment programme to test their general model for stress-strain-time deformation of soil. They choose remoulded San Francisco bay mud, a characteristic soft clay, for their test programme.

They found that the general model did not predict well the time dependent increase in pore pressure during undrained laboratory tests. They supposed that many cohesive soil would not confirm one or more of the components of this model.

Therefore, more experimental evidence on a wide variety of cohesive soils is necessary in order to establish the validity of the model. The evidence is likely to result in some modifications of the general theory.

2.16 Summary of Main Points

The results of previous studies which have led to some conclusions and generalization concerning the time dependent deformation behaviour of soils may be summerised as follows:

1. The experimental time consolidation curve is in agreement with Terzaghi's (1943) theory of consolidation only upto 60 percent consolidation.

2. The time-deformation curves can be classified into three types according to their characteristic behaviour - type-I: the rate of secondary consolidation decreasing with time and become horizontal when the ultimate settlement is reached; type-II: the rate is proportional to logarithm of time for considerable range of time and then rapidly decreases; type-III: the rate increases with time, then slows down gradually and finally vanishes.

3. For most inorganic clays, primary consolidation is much greater than secondary compression. But in most organic soils and in some inorganic silts, the magnitude of secondary compression is significant.

4. There exists a linear relationship (a) between the logarithm of strain rate and stress at any given time and (b) between logarithm of strain rate and logarithm of time for given stress, provided the creep stress level is between the practical limits of about 30 to 90 per cent of soil strength. These findings lead to simple three parameters general stress-strain rate-time relationship. Integration of this relationship yields Singh-Mitchell (1968) creep curves which are observed to agree in form with experimental results.

5. In many cases, the secondary compression curve does not remain linear in void ratio versus logarithm of time. This type of behaviour is most pronounced with an increase in C_α for consolidation pressure in the range of 0.5 to 1.5 of critical pressure.

6. For many natural soils a unique relationship exists between $C'_\alpha = \Delta e / \Delta \log t$ and $C'_c = \Delta e / \Delta \log \sigma$.

7. The observed values of C_α / C_c for a variety of natural soils are in the range of 0.025 to 0.10. The high values of C_α / C_c correspond to the high organic soils.

8. The deformation of a soil can be divided into immediate and delayed components. The immediate (or instant) compression is the settlement which would result if the excess pore-water pressure could dissipate instantaneously with load application; delayed compression is the

settlement then developing at constant effective stress.

9. A theory for the general stress-strain-time behaviour of fine grained soils has been formulated on the basis of separate volumetric and deviatoric components. The volumetric and deviatoric models are developed from the existing knowledge of the stress-strain-time behaviour of cohesive soils. Each individual model is assumed to have an immediate and a delayed component. The volumetric model is described by a series of parallel planes in void ratio-log stress-log time space. The planes are assumed to represent a unique relationship among void ratio, effective stress and time under a given sustained loading. The deviatoric model is based on Ladd and Foott's (1974) concept in conjunction with Kondner's (1963) hyperbolic formulation and Singh-Mitchell (1968) creep function.

10. Evidence from a wide variety of soil types is necessary, before the overall validity of the Kavazanjian and Mitchell (1980) general constitutive model is established. This evidence may result in some modifications of the model.

CHAPTER 3

RESEARCH SCHEME

3.1 Statement of the Problem

It has been shown in the review that no general constitutive relationship suitable for use is yet available for analysis of geotechnical problems that involve time-dependent deformation under multi-axial states of stress. Most present models for the stress-strain-time behaviour of soils are limited to either specific phenomenological models (Singh and Mitchell, 1968) or to mechanical models of clay (Murayama and Shibata, 1961, Garlanger, 1972).

It has also been shown that a general model proposed by Kavazanjian and Mitchell, (1980) for analyzing the time dependent deformation behaviour of cohesive soil is available. This model separates the deformation as volumetric and deviatoric components. Models for each component is developed by generalizing existing hypothesis for the deviatoric and volumetric behaviour of soils. But the model has been tested for only limited types of soils and still requires to be tested for a wide variety of soil types. The present research was proposed to investigate the effect of soil type on various parameters that are involved in defining the model — specially to the volumetric component of deformation. A local soil (from Faridpur) and two reconstituted soils were used for the purpose. The specific objectives of this programme were:

(i) To evaluate the parameters (A, $\bar{\alpha}$ and m) necessary to define the Singh-Mitchell (1968) creep function and to compare the observed behaviour of deformation with the predicted creep by this formulation.

(ii) To verify the relationship between C'_α and C'_c and the range of the values of C_α/C_c as proposed by Mesri and Godlewski (1977).

(iii) To observe the effect of stress and time on C_α.

(iv) To study the time-deformation behaviour of soils with respect to its type.

As mentioned earlier, the secondary compression is very significant for organic clays and for some inorganic silts. In tropical region, organic clays occur principally in coastal areas and silts in alluvial deposits. As large soil deposits in Bangladesh are of alluvial or marine type, the secondary compression problem is also important in this country and investigation on these types of soils requires special attention. For this reason an inorganic silt was collected from Faripdur and two other soils were reconstituted at the laboratory to fulfil the requirements of the investigation.

The general constitutive model (Kavazanjian and Mitchell, 1980) was to be analyzed under three dimensional states of stress. Since the field condition resemble more closely to three dimensional states of stress, it is more justified to investigate the deformation behaviour in such condition than in one dimension. Again, the compression in the underground at shallow depths are definitely three dimensional, but that in deeply buried strata are essentially one dimensional; one dimensional analysis will, therefore, have practical applications. For this reason and also for the limitations of the laboratory facilities at BUET to perform test at three dimensional conditions of stress, a laboratory experimental programme was developed for one dimensional case only.

3.2 Laboratory Programme

A laboratory experiment programme was developed to investigate the stress-strain-time behaviour of soils for verifying the general constitutive model. The investigation of stress-strain-time relations required samples which were to be of uniform consistency, density and similar characteristic fabric. Therefore some amount of investigation was necessary to attain reproducible samples. Therefore, the overall laboratory investigation programme was divided into the following phases.

(i) Classification of soil types used in the investigation.

(ii) Evaluation of a method to provide replicate specimens.

(iii) Determination of stress-strain-time relations of soils under study.

Classification of Soils: It has been mentioned in the review that the relative magnitude of primary and secondary compression vary with soil type. In order to observe the effect of soil type on the stress-strain-time behaviour, the soils were to be classified. As the grain size distribution and Atterberg limits are useful index tests for soil classification, these along with specific gravity of the soils were determined.

Preparation of the Samples: To evaluate the Singh-Mitchell (1968) parameters the experimental programme required a number of soil samples of similar consistency. That is each sample for same batch was to be of same density and uniform structural property in order to provide reproducible results. It was necessary to develop a laboratory method that would give a soil free of random distribution of particles and uniform water content and density throughout so that the same soil would be used under different stress levels. A method for uniform sample preparation was therefore developed and detailed in Art. 4.2.

Determination of Stress-Strain-Time Relations of Soils: To observe the stress-strain-time behaviour of soils, volumetric and delayed deviatoric deformation of the soil were investigated. To find the delayed deviatoric deformation and to observe the effect of stress on C_α and to verify the relationship between C'_α and C'_c, oedometer tests were performed.

The Singh-Mitchell (1968) parameters A, $\bar{\alpha}$, m were determined from one dimensional case applying the suggestion given in Art. 2.11. At least two creep tests are required to establish the value of A, $\bar{\alpha}$ and m for any given soil. However, six identical specimens of each soil were tested using different creep stress intensities between 20 to 120 per cent of unconfined compressive strength.

To observe the effect of time on C_α, long term consolidation test was performed for a soil specimen under a stress of 80 per cent of unconfined compressive strength of the soil; a stress which is within normal engineering application range.

CHAPTER 4

LABORATORY INVESTIGATIONS

The investigations in the laboratory were conducted according to the programme outlined in Art. 3.2. The details of apparatus, experimental procedure and soil materials used will be discussed in this chapter.

4.1 Test Methods for Classifying the Soils

Physical properties of soils required for soil classification, that is liquid limit, plastic limit, specific gravity and grain size distribution were performed in accordance with the procedure specified by the American Society for Testing and Materials (ASTM). ASTM standard D423-66 (1972) was followed for liquid limit, D424-59 (1971) for plastic limit, D854-58 (1972) for specific gravity and D422-63 (1972) for grain size distribution. The soils were classified according to ASTM standard D2487-69 (1975) based on Unified Soil Classification System.

4.2 Replicate Sample Preparations

A technique was developed for preparing replicate soil specimens - that is soil specimens having identical density, fabric, strength and stress-strain characteristics. Many processes were tried in achieving uniformity in the consistency and fabric of the soil, but none of them proved entirely satisfactory. However, the method developed by

Leonards' (1955) was followed with some modifications. A general description of the apparatus and procedures followed by Leonards (1955) is given below.

Leonards' (1955) Method: In this method, the disturbed soil samples were to be sealed in metal containers and stored in a constant humidity room (having a relative humidity of 98 per cent) for not less than one month in order to obtain a reasonably constant initial water content. All processings were to be done in the humid room.

The process involved compaction of disturbed soils in a specified manner. Before compaction, the soils must be mixed with water, if water was to be added. The soil, thus processed, was then to be force-sieved with a large rubber stopper, through a No. 20 sieve which was to be backed by a coarse sieve, say No. 3 sieve, for support. All vissible particles were to be removed, and the product, expected to be a homogeneous material was to be sealed again in metal containers and to be stored in the humid room. Leonards (1955) found that the maximum variation in water content between specimens processed in this manner was 0.2 per cent.

At first, Leonards (1955) attempted to compact the individual specimen using static pressure in a 3 in. diameter mould. But he found variation in structure caused by placing the soil in the mould and variations in unit weight at the top, centre, bottom of the specimen because of side friction. He, therefore, used 10 in. diameter moulds to minimize the effects of side friction and to permit simultaneous compaction of all the specimens to be used in a particular test series.

According to the method, the 10 in. diameter mould consisting of three rings and two collars was to be used for checking uniformity in dry unit weight, water content and soil structure. After compacting the sample by application of static pressure, the unit weight of the entire sample was to be determined. Leonards (1955) used a hydraulic testing machine of 100,000 lb capacity for compacting the soil. Variations in horizontal direction were to be determined using a proctor penetration needle attached to a sensitive proving frame. Water contents were to be determined at each penetration point. After removing the top ring, the protruding soil was to be cut off with a wire saw and the soil surface screeded with a straight edge. Again the unit weight, penetration resistance and water contents were to be determined. The entire procedures were to be repeated after the removal of second ring. By successive repetitions of this process a compaction procedure would be developed that would result in uniform condition of water content, void ratio and soil fabric.

A second 10 in. diameter mould was to be used to yield an "undisturbed" sample approximately 3½ in. high from which a number of replicate specimens could be cut. This mould would consist of a split ring section and two collars. One 10 in. diameter piston and spacer block to permit compaction from top and bottom simultaneously, were to be fabricated to fit this mould with just enough space for movement but not enough to permit the soil to enter the clearance spaces.

The compaction process would yield a cake of soil 10 in. in diameter and $3\frac{1}{2}$ in. high. The cake was then to be centred in the base of a mitered box, the collar put in place and the soil cut along the mitered grooves. Leonards (1955) suggested that a thin, continuous, high-speed band saw was the only satisfactory tool for this purpose and found that cutting with piano wire or with coping saw was not entirely satisfactory. He found that the maximum variation in unit weight of a given series was less than 0.5 per cent and that was roughly equal to the precision of the measurements used to determine the unit weight.

<u>The procedures Followed</u>: Since Leonards (1955) method is tedious and time consuming, a technique, suitable for present use giving satisfactory results, was developed and is described below.

About 15 lb of soil enough to prepare a number of specimens at Single water content was taken. Water was added and thoroughly mixed by hand kneading. All visible coarse particles were removed and the product was kept in a metal container for one day.

The remoulded soil was then placed in the mould. The mould consisted of a ring 6 in. in diameter and 6 in. height and a collar of 2½ in. high. The innerface of the mould was lubricated with grease for reducing side friction. Two filter papers, one at top and the other at bottom of the soil were placed to permit drainage. A spacer block was placed at the top filter paper and used as loading disc for compaction of the soil. The spacer block fitted the mould with a clearance to eliminate friction but not permitting the soil to enter the clearance spaces. The arrangement for placement of soil, filter papers and spacer block in the mould is shown in Fig. 4.1

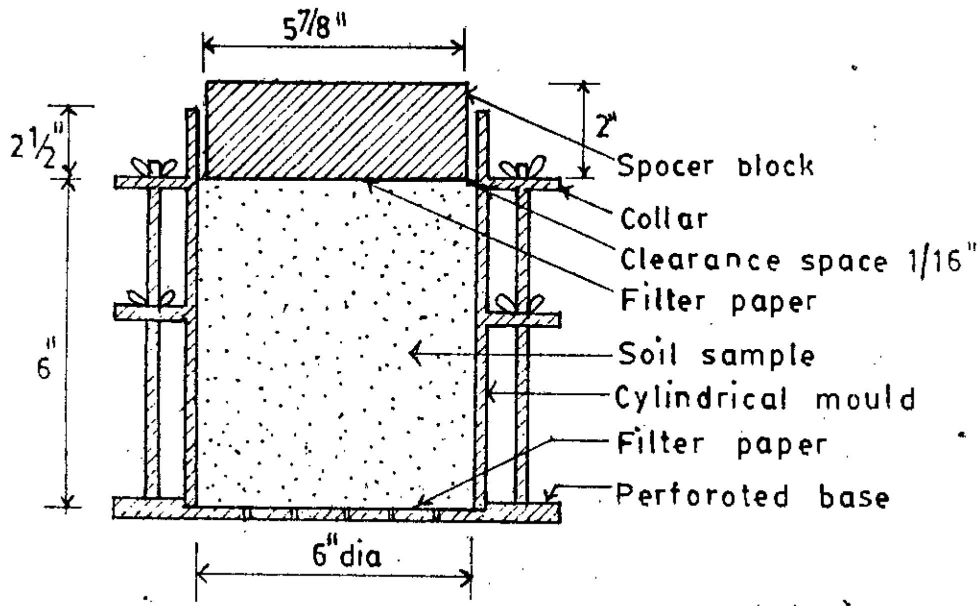

Fig. 4.1 The placement of soil sample in the mould.

The sample was compressed under a static pressure provided through a loading frame. The arrangement of the mould in the loading frame is shown in Fig. 4.2. The load was applied slowly and the soil was allowed to dissipate

Fig. 4.2 The arrangement of the mould in the loading frame.

excess hydrostatic pressure through the drainage at top and bottom of the mould. After the compaction of compression which was indicated by the reversal of the load dial to initial reading, further load was applied and time was allowed for further compression. The process was repeated until the load dial reading and strain dial reading were no longer moved. Six to nine days were required to achieve this equilibrium condition.

The process of compression yielded a cake of soil 6 in. in diameter and approximately 6 in. high. The soil cake was extruded from the mould using a hydraulic jack. The water contents and unit weights from different portions of the cake were measured to check the uniformity of the specimens. For the purpose of checking, small grooves were cut, approximately rectangular in size on top face and sides of the soil cake. Four to six grooves on top face and three to four grooves on sides were cut. The unit weights and water contents of the soils from these grooves were measured. To determine the unit weight, the volume of the soil was measured by mercury displacement. It was found that the variation in unit weights and water content in these measurements were limited to 1 per cent and 0.5 per cent respectively.

4.3 Test Procedures for Determination of Stress-Strain-Time Behaviour of Soils

After checking for uniformity, the samples were trimmed to the required sizes for unconfined compression test and for consolidation test. Six samples were prepared for consolidation test and one for unconfined compression test for each type of the soil. Detailed dascription of the test procedures in performing the consolidation and unconfined compression test are given below.

Determination of Strength of Soils: To determine the strength of soil, unconfined compression test were performed on soil cylinder of 1.4 in.diameter and 2.8 in. high. The test method followed to determine unconfined compressive strength was ASTM standard D2166-66 (1972).

Concolidation Tests : To investigate the time deformation behaviour at different stresses, consolidation tests were performed on six identical specimens of each type of soils at stress intensities of 20 to 120 per cent of unconfined compressive strength. The dimension of each specimen before the test was 2.5 inch in diameter and 1 in. thick. The consolidometer rings were lubricated with grease to reduce side friction. Filter papers were placed at top and bottom of the sample to prevent the soil from being forced into the porous stone. In all cases, drainage was permitted at both ends of the sample. Before applying the load, the samples within the ring were kept in water for one day for saturating the soil but not allowing the soil to expand. Each specimen was tested under a given sustained load.

Arrangements were made so that the soil specimen always resisted the same vertical stress. The time of readings after the application of stress were $\frac{1}{4}$, $\frac{1}{2}$, 1,2,4,8,15,30, 60,120,240,480,1440,2880,5760,10080 minutes, that is approximately doubling the previous time of reading. Consolidation tests were performed according to ASTM standard D2435-70. Fig. 4.3 shows a view of the consolidation test machines used for the determination of stress-strain-time behaviour of soil.

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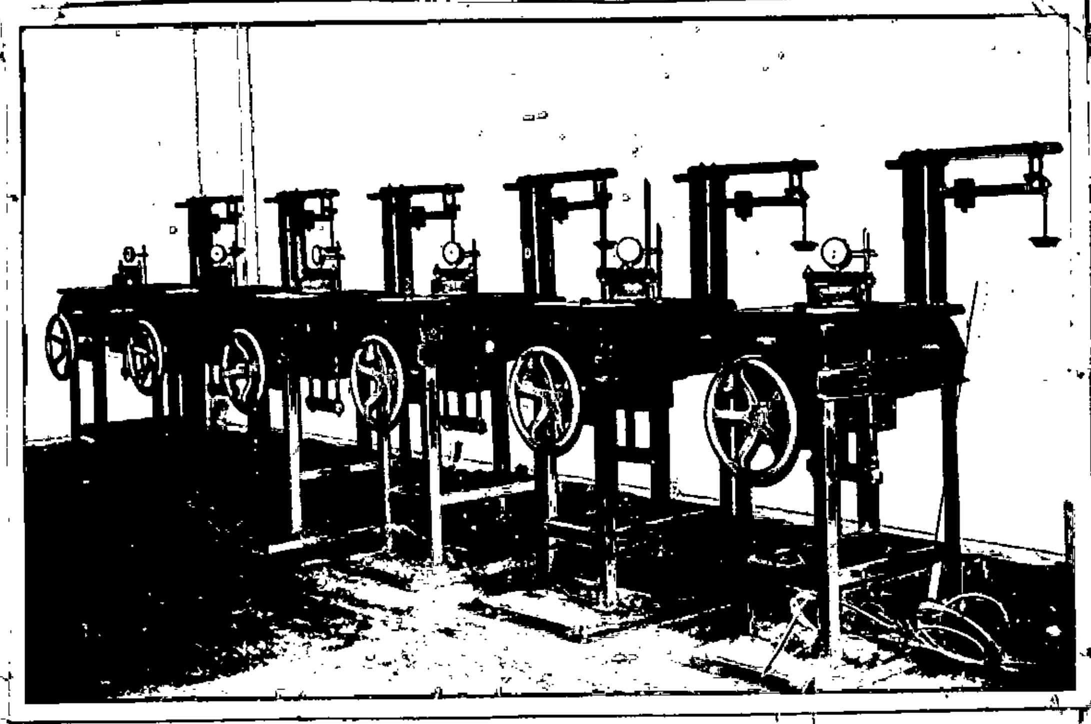

Fig. 4.3 A view of the consolidation test machines used in the experiment.

Test Procedure to Observe the Effect of Time on C_α:

To observe the effect of time on C_α only one sample of a natural soil was used. The soil was prepared according to the procedure developed and described in Art. 4.2. Its strength was determined by unconfined compression test. Consolidation test was performed on the sample under a stress of 80 per cent of the unconfined compressive strength.

4.4 Soil Materials used

Three soils were selected for the present study. Of them, one was a natural soil and the other two were reconstituted at the laboratory. The natural soil, an inorganic silt, was collected from Faridpur*. The reconstituted soils were prepared by mixing two different type of soils — one of which had a substantial percentage of clay particles and the other one had large percentage of silts, so that the soils resulted were of clayey-silt type. Before mixing the soils, the coarse particles were removed by sieving with a No. 30 sieve and the two soils were mixed at different proportions.

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* The soil sample was in disturbed state and supplied by M/S Foundation Consultant, Dhaka.

The three soils used for present investigation differ in their physical properties. Their classification and particle size distribution are given in Table 4.1 and in Fig. 4.4 and 4.5.

These three soils were prepared to investigate their stress-strain-time behaviour. The characteristics of the soil specimens prepared for the study are given in Table 4.2. The natural soil collected from Faridpur was remoulded and used to observe the effect of time on C_α.

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Table 4.1 Classification of the Soil Used in the Investigation

Soil No.	Location	Grain Size Distribution			Atterberg Limits			Specific gravity	Unified soil classification
		% Sand	% Silt	% Clay	LL (%)	PL (%)	PI (%)		
1	Faridpur	3	79	18	40	31	9	2.61	ML
2	Reconstituted	19	54	27	29	19	10	2.77	CL
3	Reconstituted	14	50	36	37	23	14	2.66	CL

4.5 Analysis of Test Results

The unconfined compression test data were used to plot the stress-strain curve. The peak of the curve was taken as the strength of the soil. The stress-strain curves of three soils are given in Fig. 4.6 and their strengths are given in Table 4.3.

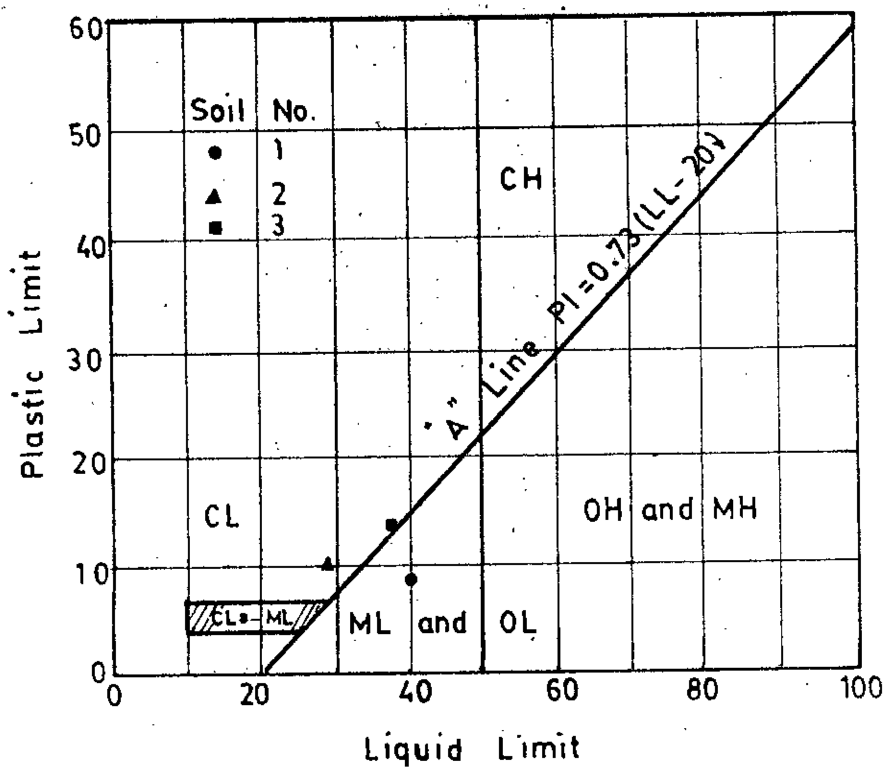
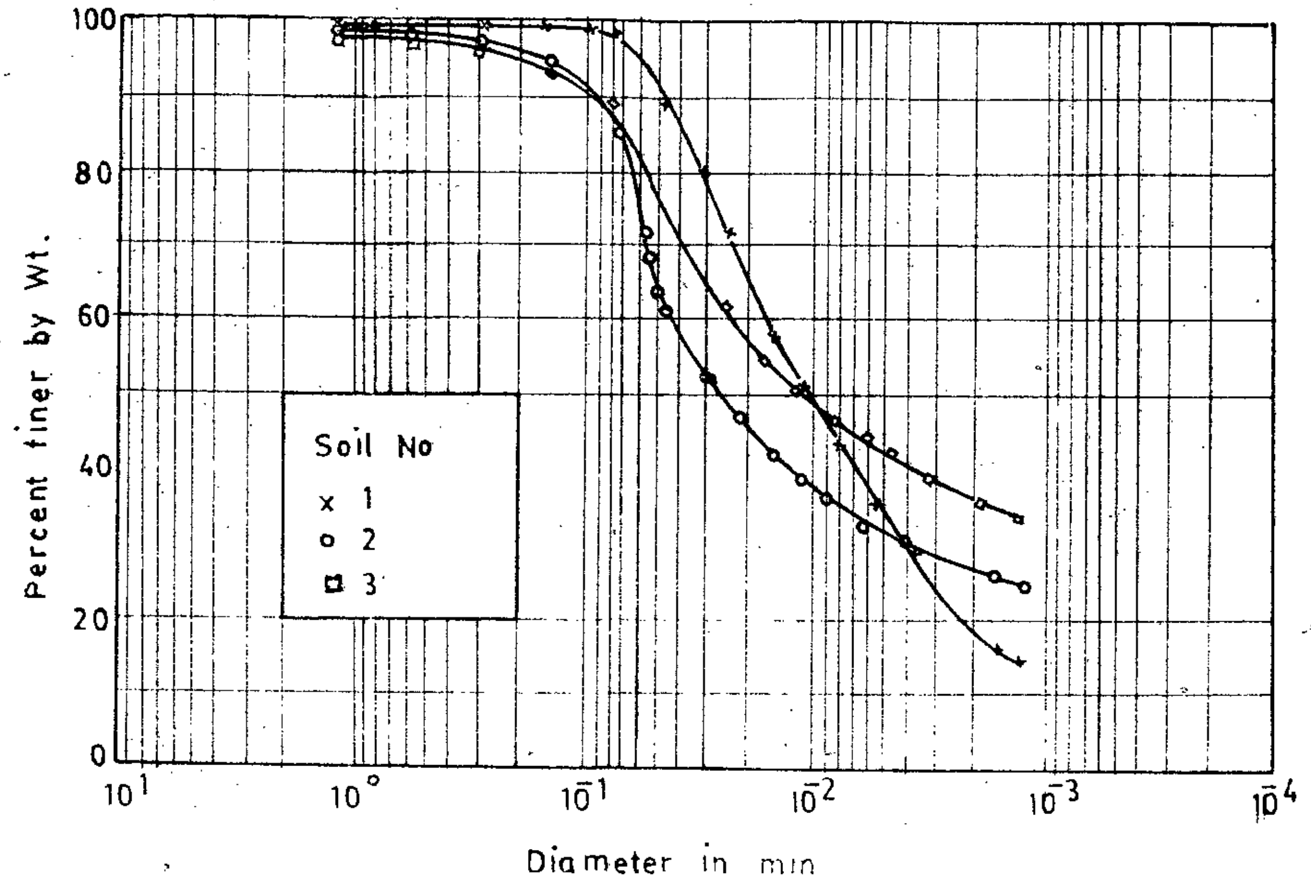


Fig. 4.4 Classification of soils invetigated.

Table 4.2 Characteristics of Soil Samples Used in Consolidation Test to observe the Singh-Mitchell Creep Function

Soil No.	Type of soil	Water content (%)	Wet density (pcf)	Dry density (pcf)	Degree of saturation (%)
1	ML	33	116	87	97.80
2	CL	25	121	97	88.51
3	CL	21	123	102	89.09


100
80
60
40
20
0
Percent finer by Wt.
10 1
10°
10̄1
10̄2
10̄3
10̄4
Diameter in mm
Soil No
x 1
o 2
□ 3

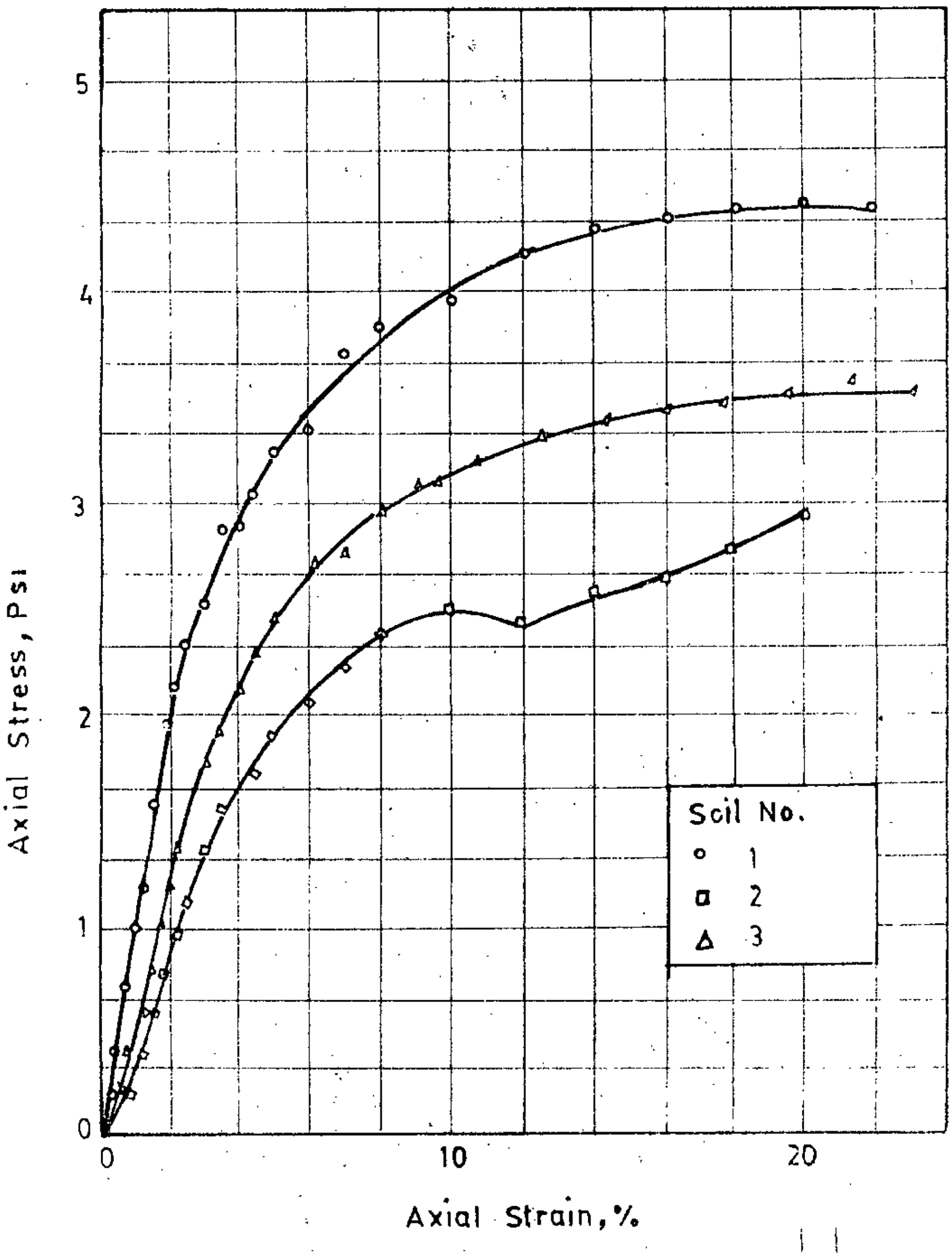


Fig. 4.6 Stress strain curves for the soils tested in unconfined compression.

Table 4.3 Unconfined Compressive Strengths

Soil No.	Unconfined compressive strength (tsf)	Corresponding strain (%)	Water content (%)	Density γ_d (pcf)
1	0.27	20	33	87
2	0.17	22	25	97
3	0.23	10	21	102

The consolidation test data were used to observe time-deformation behaviour at different stress intensities and shown in Fig. 4.7(a), 4.7(b), 4.7(c). The corresponding readings and calculations are given in Appendix-III, from Table I to III. These curves were also used in calculating the strain rate and the magnitude of C_α.

To find the strain rate it was necessary to obtain R_{do}, - the theoretical dial reading at t = 0. To obtain R_{do} on the semi-logarithmic plot, a time t_{d1} and a time $t_{d2} = 4t_{d1}$ were selected. The ordinate from t_{d1} to t_{d2} on the curve was then laid off vertically above the ordinate at t_{d1}. The intercept of the horizontal line through this point on the dial reading ordinate is R_{do}. Refinement was made for R_{do} by using other points along the curve for t_{d1} and t_{d2} and the average value of R_{do} was taken.

To plot the void ratio versus log pressure curve R_{d100}, the dial reading to the time corresponding to the end of

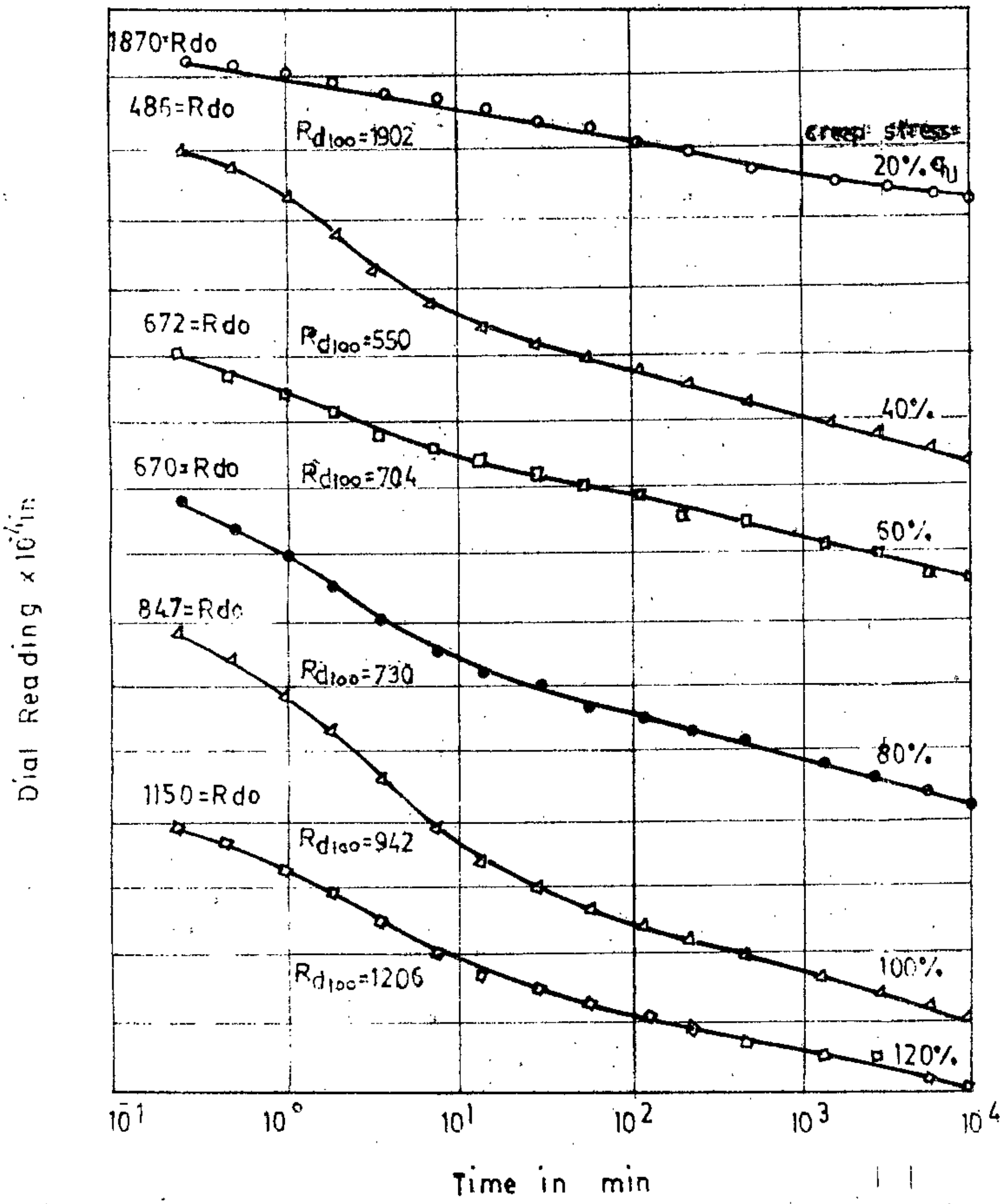


Fig. 4.7(a) Dial reading versus log time curves for soil type 1.

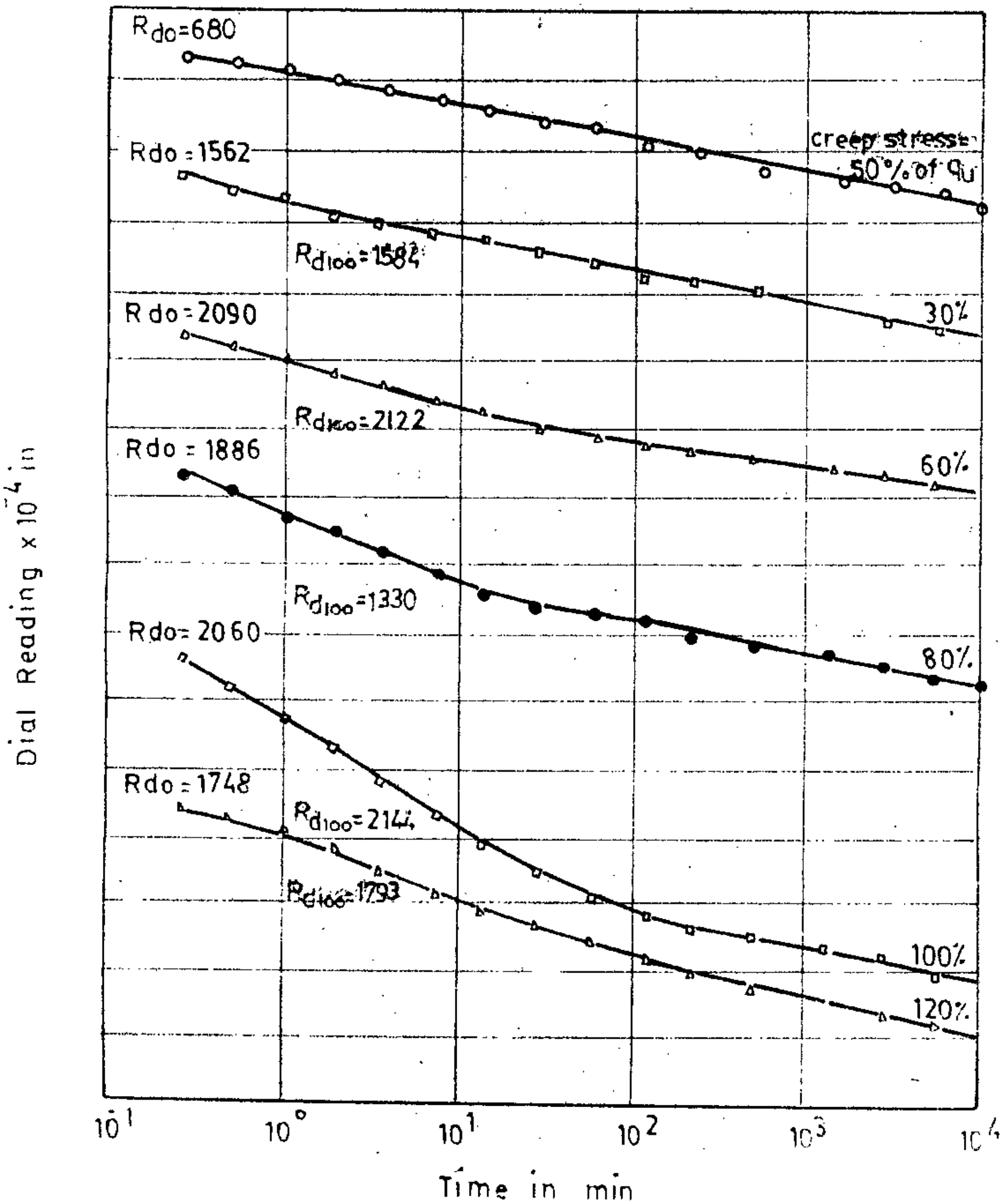


Fig. 4.7(b) Dial reading versus log time curves for soil type 2.

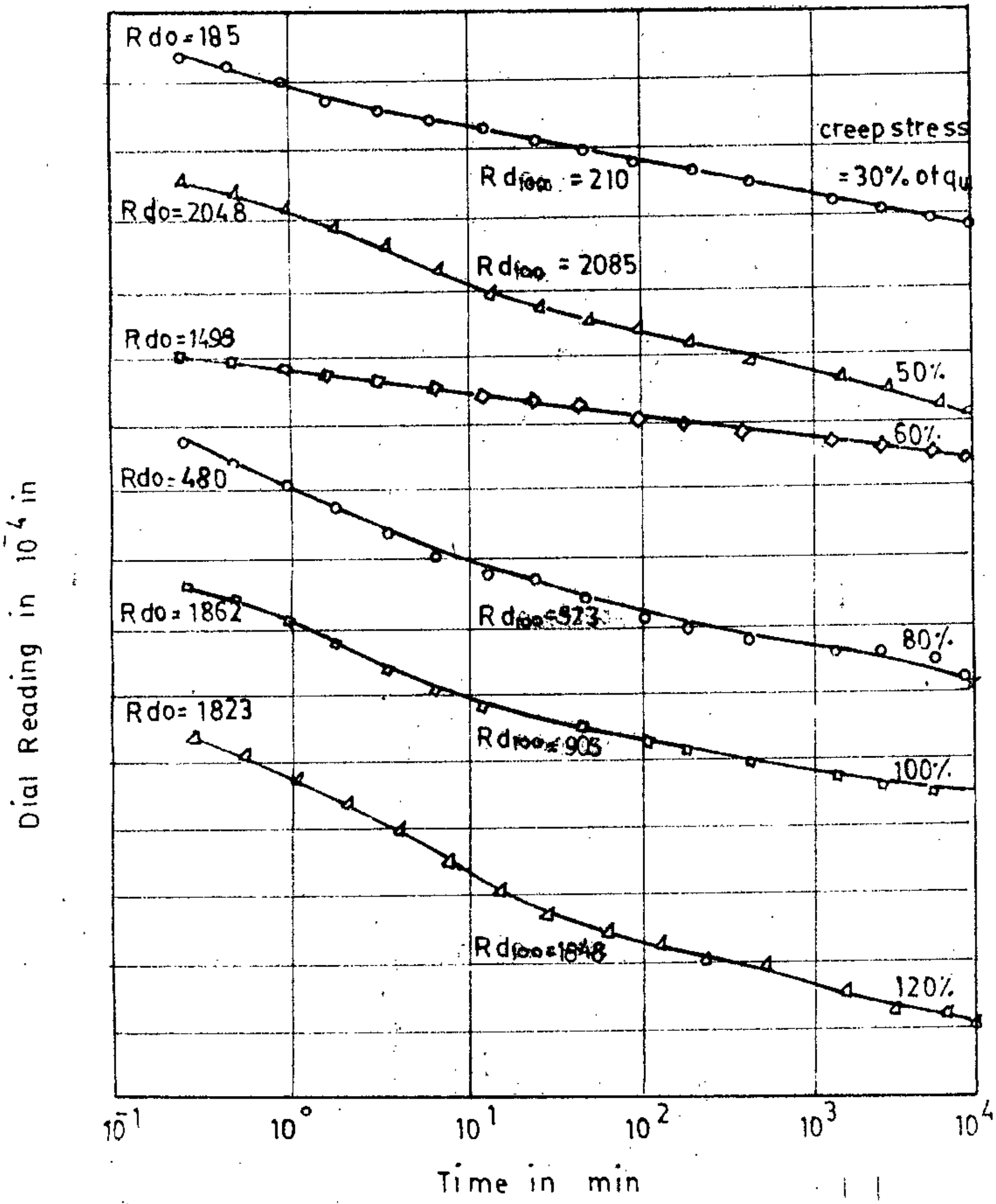


Fig. 4.7(c) Dial reading versus log time curves for soil type 3.

primary consolidation was used. Since C_c, the slope of this plot is the measure of primary consolidation, the change in void ratio from R_{do} to R_{d100} was used in obtaining this plot. To obtain R_{d100}, from the dial reading versus log time curve, the straight line portion of the primary and secondary range of the curve were extended. The intercept of the horizontal line through their point of intersection on the dial reading ordinate is R_{d100}.

The slope of the straight line portion of the curve in the secondary range was used to compute the secondary compression index, C_α and

$$C_\alpha = \frac{\Delta e}{\log t_{d2}/t_{d1}} = \frac{(R_{d2} - R_{d1})/H}{\log t_{d2}/t_{d1}} \tag{27}$$

where, R_{d1} and R_{d2} are dial readings at time t_{d1} and t_{d2} respectively and H is the solid height. It is convenient to use $t_{d2} = 10\ t_{d1}$, so that Δe, the change in the void ratio between time t_{d1} and t_{d2} is equal to C_α.

The strength of the soil used to observe the effect of time on C_α was determined from stress-strain plot given in Fig. 4.8. The dial reading versus log time plot is given in Fig. 4.9 and the corresponding data are given in Appendix-III, Table IV.

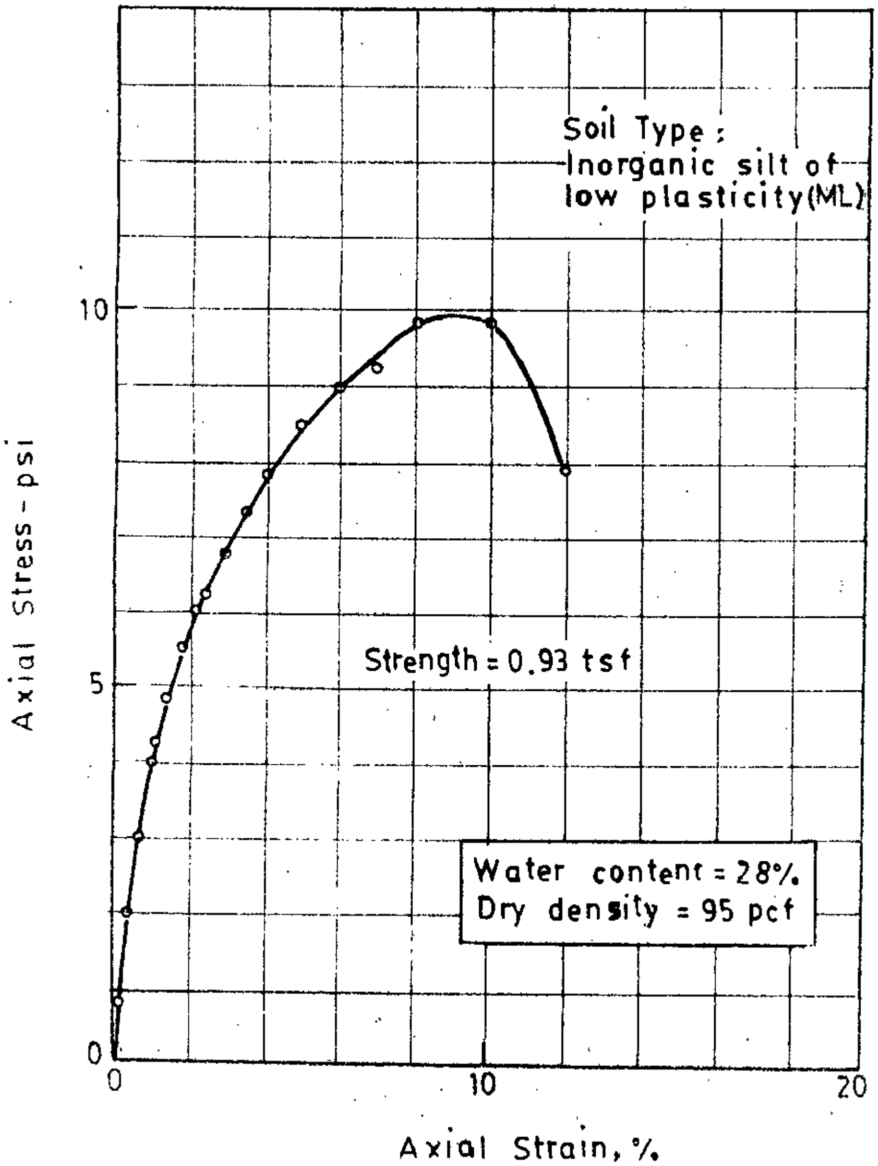


Fig. 4.8 Stress vs. strain curve in unconfined compression for the soil used to observe the effect of time on C_α.

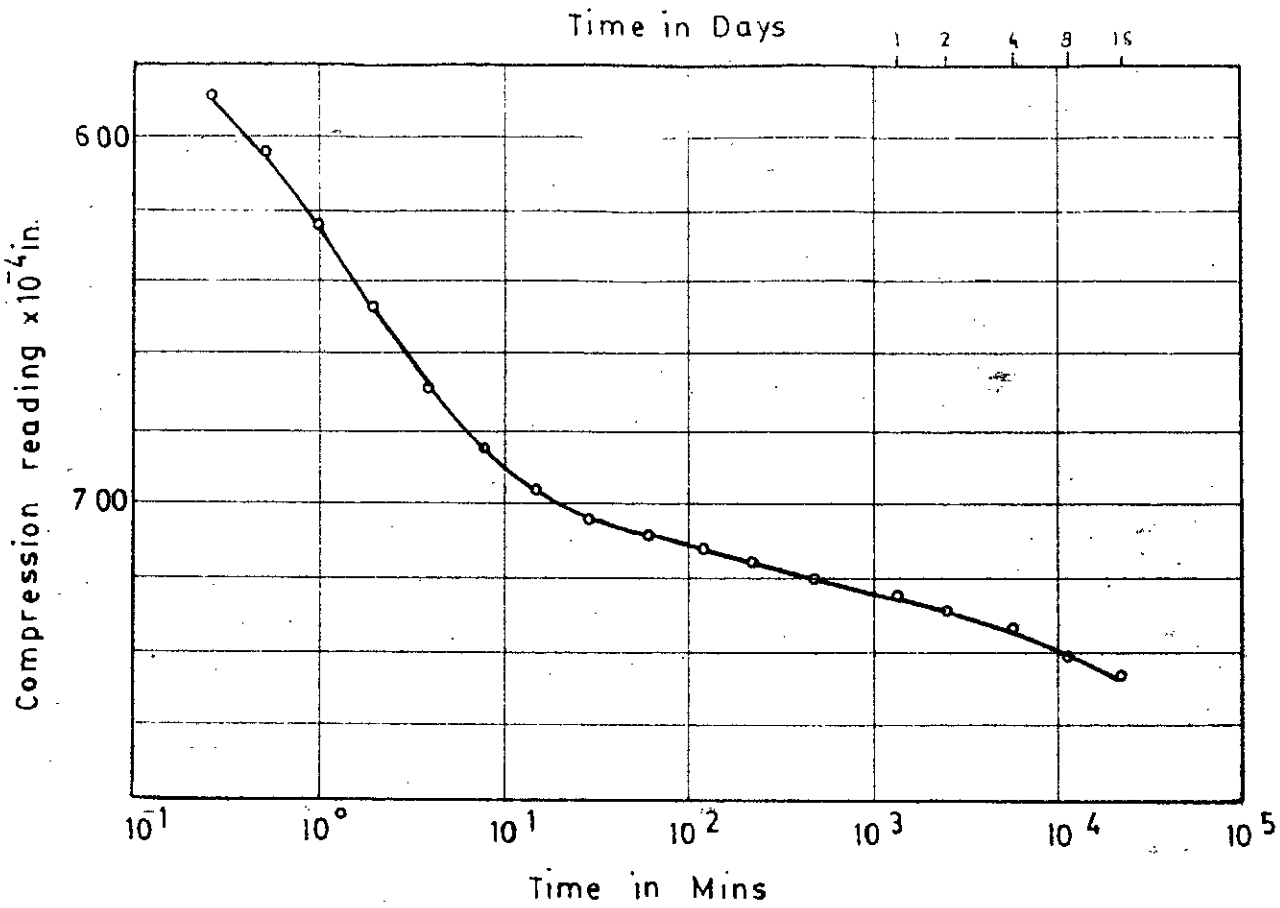


Fig. 4.9 Dial reading versus log time curve for the soil used to observe the effect of time on C_{α}.

CHAPTER 5

ANALYSIS AND DISCUSSION

5.1 Time-deformation Characteristics of Soils

a) General Characteristics of Time-eformation Curves

The general characteristics of time deformation curve in confined compression for three different soils investigated are shown in Fig. 5.1(a), (b), (c) at the creep stresses of 50 per cent to 100 per cent of unconfined compressive strength, q_u. The Figures show that after a period of transient creep during which the strain rate decreased continuously, the creep continued at a constant rate for some period of time. Approximately after one day, the rate became linear. Thus the results confirm to the suggestion of Finne and Heller (1959) that following the period of transient creep during which the strain rate continuously decreases creep continues for some period of time as mentioned in Art. 2.7.

The figures also show that the constant strain rate is approximately independent of the applied stress. It may be mentioned that the soils were investigated within the range of practical interest, i.e.40 per cent to 100 per cent of q_u. Since the engineers are generally interested in stresses within this range, the dependence of the creep rates of the soil with creep stresses outside this range was not investigated. The increase of strain rate with increasing stress was negligible. For example, for soil 1, the rate increased

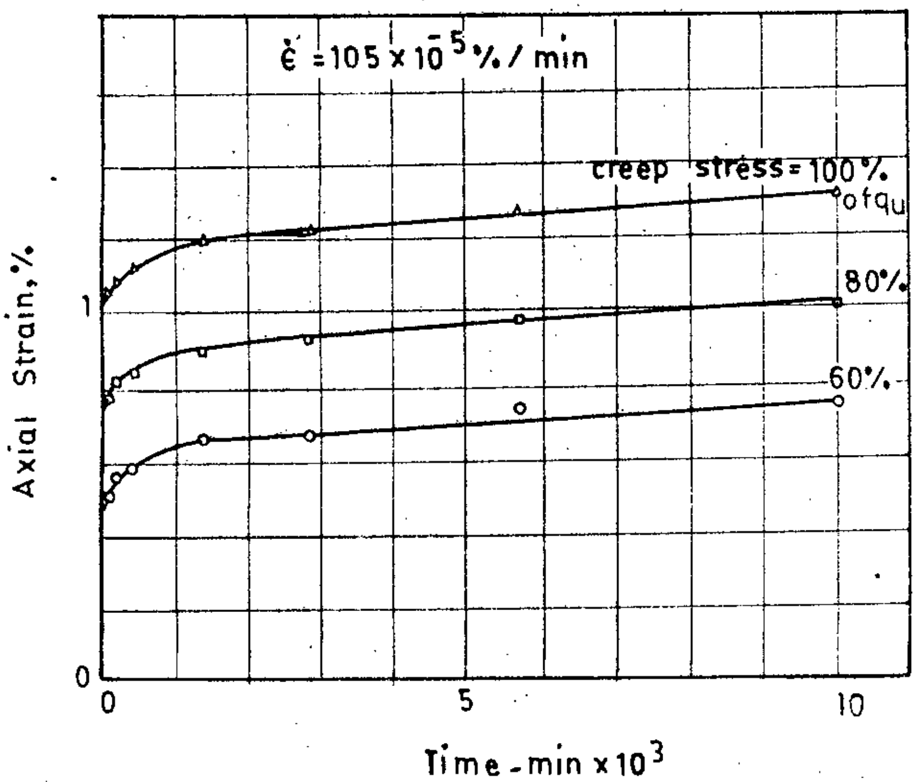


Fig. 5.1(a) Variation of axial strain with time during creep of soil 1 in confined compression.

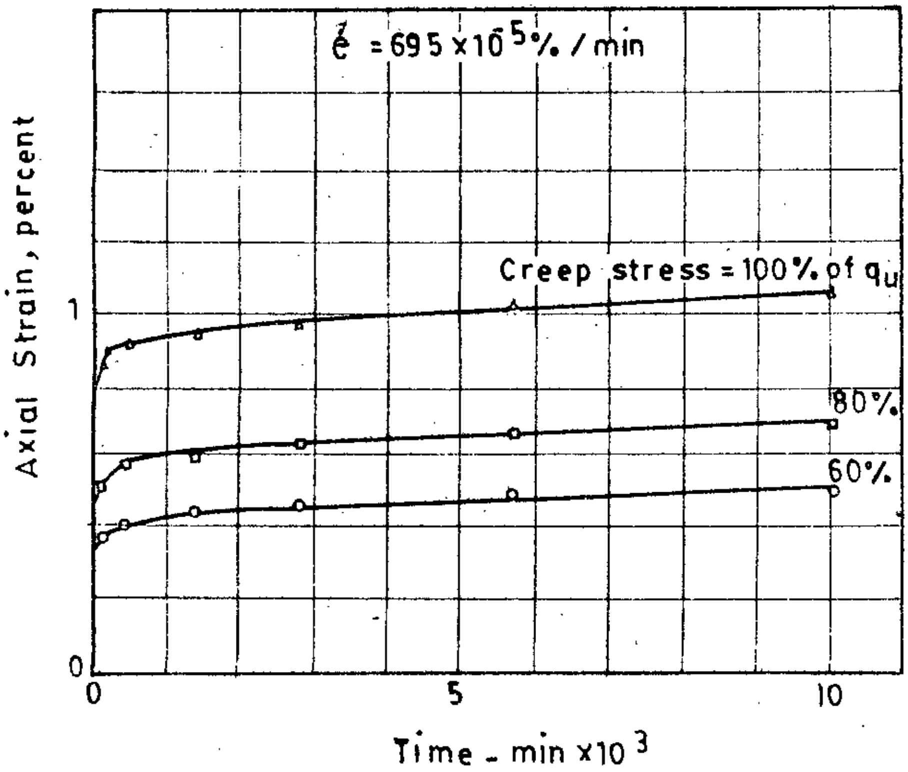


Fig. 5.1(b) Variation of axial strain with time during creep of soil 2 in confined compression.

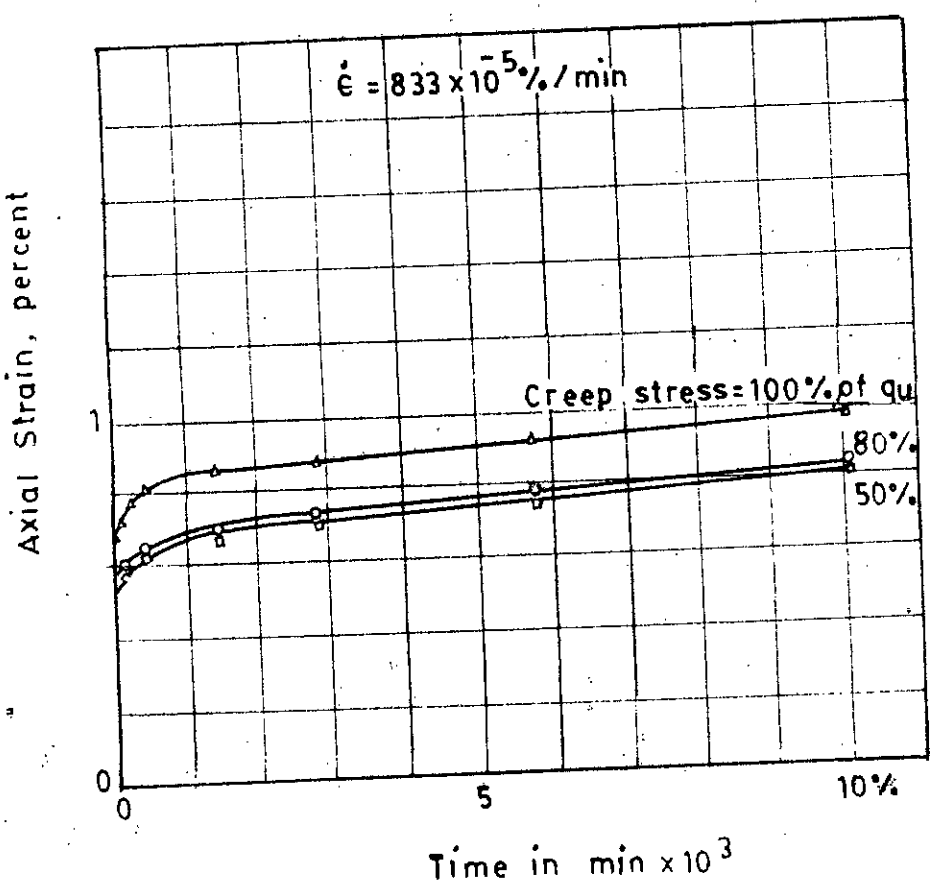


Fig. 5.1(c) Variation of axial strain with time during creep of soil 3 in confined compression.

from 0.972×10^{-5} per cent per minute to 1.11×10^{-5} per cent per minute as stress increased from 60 to 100 per cent of q_u. Therefore it can be inferred that, the strain rate after the period of transient creep is independent of stress if the stress is within the practical range mentioned above.

b) Strain Rate-Stress-Time Relationship

The relationship between stress intensity and log of strain rate* for three different soils is shown in Fig.5.2 (a), (b), (c). For soil 1, the relationship was found to be linear for stress of 30 to 110 per cent of unconfined compressive strength. The corresponding limits for soil 2 and 3 were approximately between 40 to 100 per cent, and 40 to 120 per cent of q_u respectively.

Some anomalies can be observed in these figures, the strain rates corresponding to the stresses of 40 and 120 per cent of q_u for soil 1, 120 per cent of q_u for soil 2 and 30 per cent and 60 per cent of q_u for soil 3 were not considered in obtaining this relationship. Because the strain for a given period of time cannot be less for higher stress, the influence of these strain rates were omitted. However, these strain

* The methods followed to calculate the strain rate and the parameters m, $\bar{\alpha}$, and A are given in Appendix-II. The strain rate for different stresses at different time are given in Table V to X in Appendix-IV.

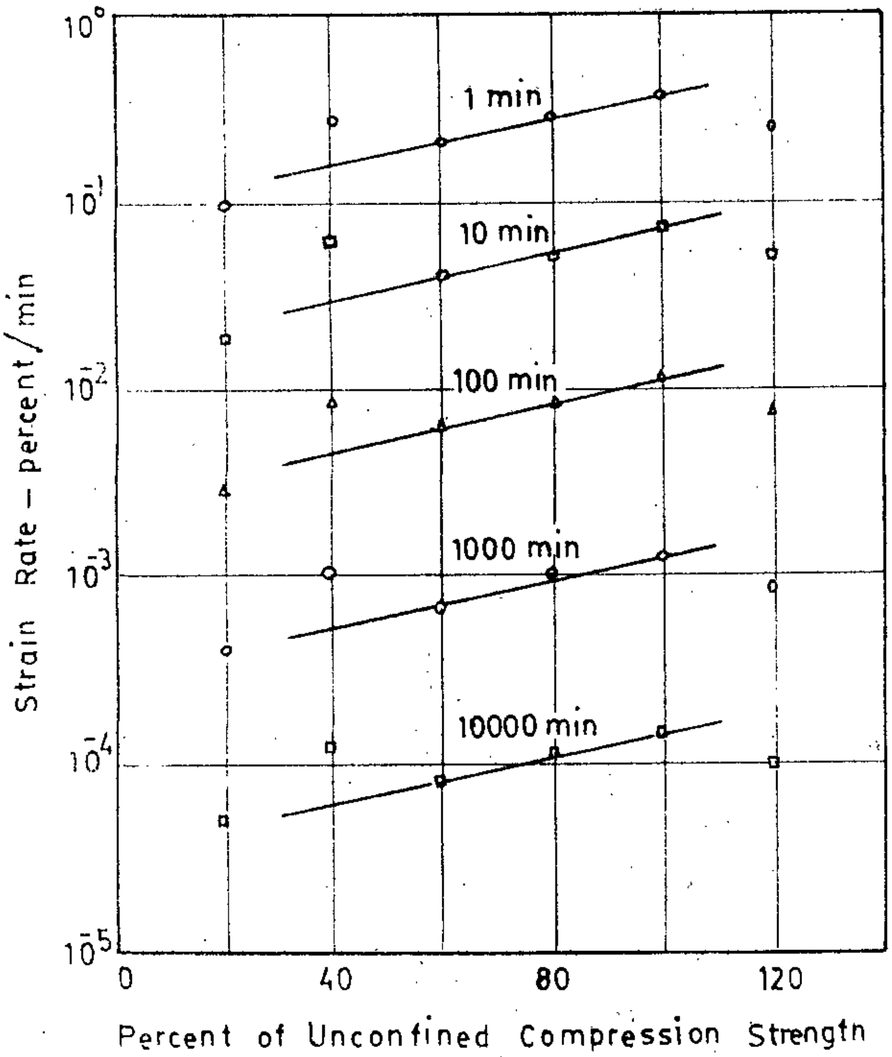


Fig. 5.2(a) Variation of strain rate with stress for creep of soil type I.

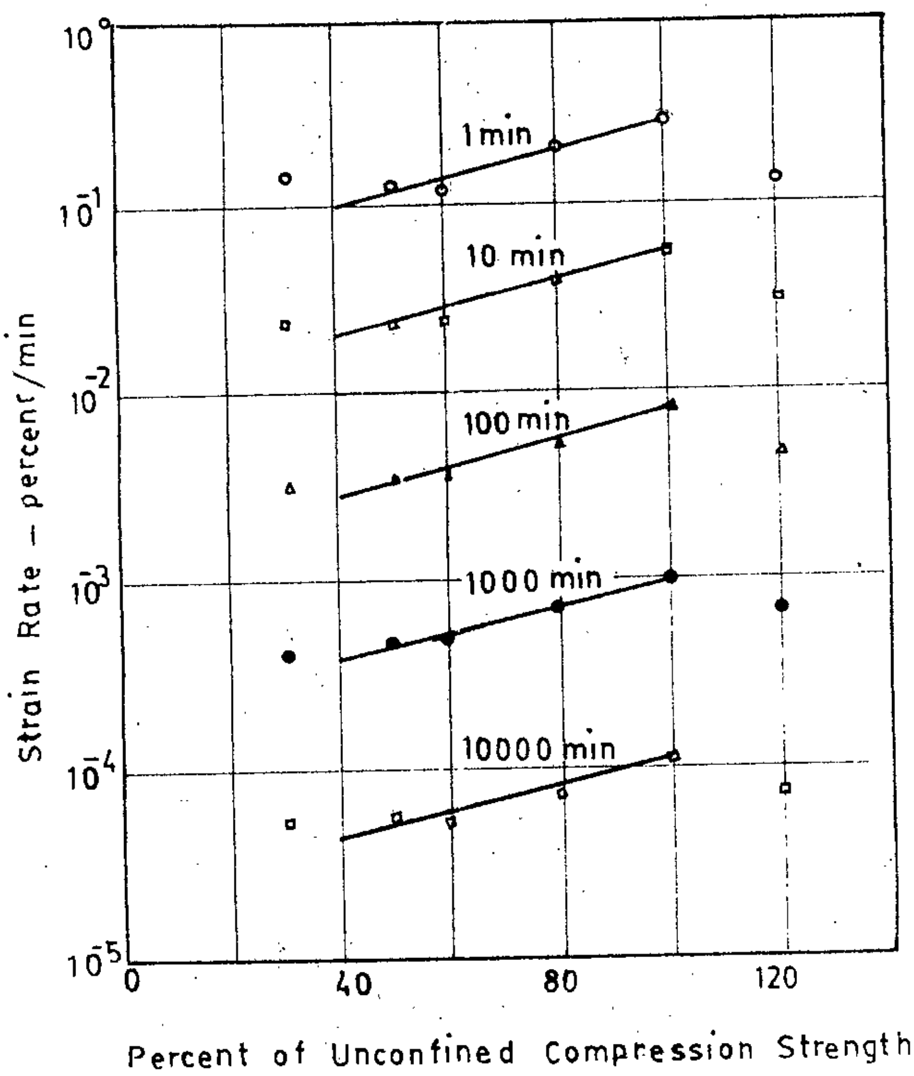


Fig. 5.2(b) Variation of strain rate with stress for creep of soil type 2.

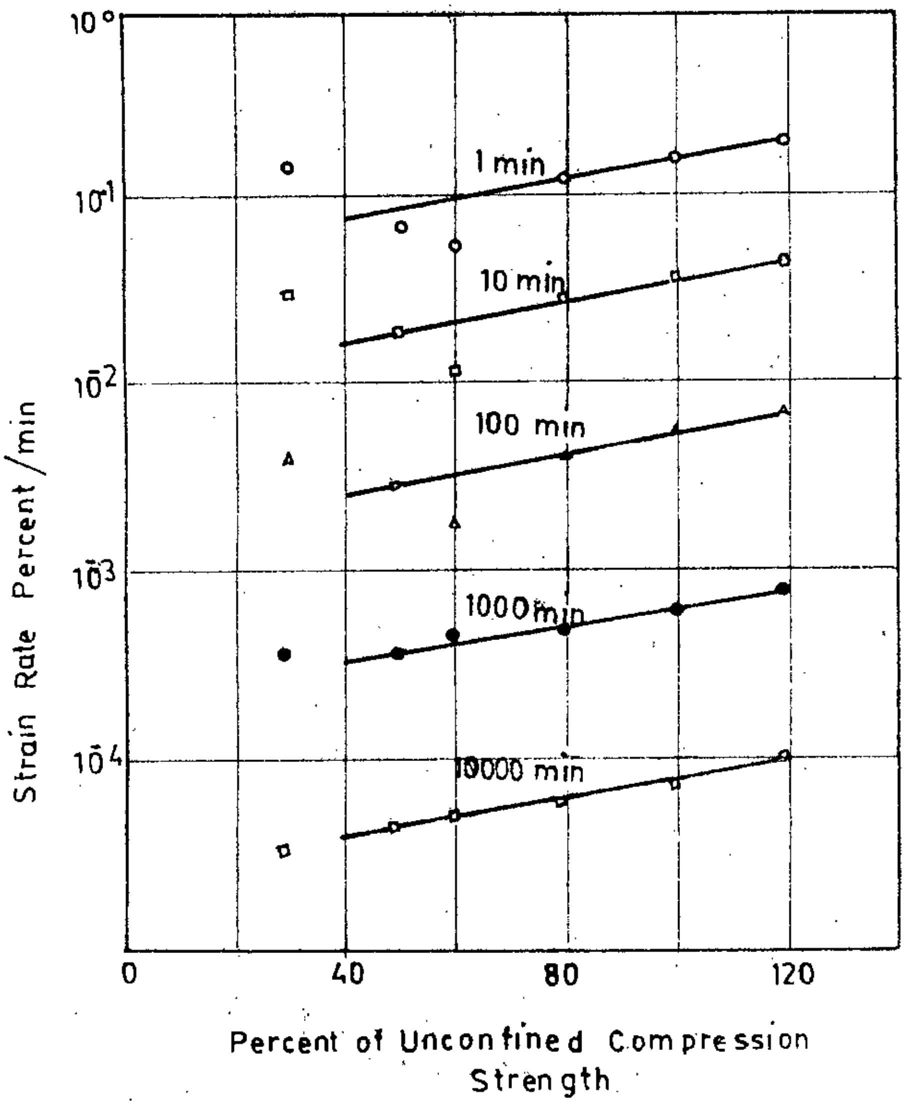


Fig. 5.2(c) Variation of strain rate with stress for creep of soil type 3.

rates are shown on the plot of log of strain rate versus stress intensity. These anomalies were possibly due to the variation in structure between the samples of each soil tested or due to the initial error in setting the load dial of the samples in the consolidation machines.

Another relationship was found between logarithm of strain rate and logarithm of time. It was found that log of strain rate decreased linearly with log of time. Furthermore, the slope of the curve was found independent of the creep stress and the increase in stress only shifted the line vertically upwards, that is, the initial creep was higher for higher stress. The relationships are shown in Fig. 5.3 (a), (b), (c).

These findings are similar to those obtained by the previous investigators from three dimensional consolidation test results described in Art. 2.7. The value of m for soil 1, 2 and 3 were respectively 0.85, 0.86 and 0.86. These values are within the range suggested by Singh and Mitchell (1968) for three dimensional consolidation.

c) Variation of C_α with Pressure

From the result of present investigation, it is found that C_α varied linearly with logarithm of stress. The relationship is shown in Fig. 5.4 for three soil types tested. The variation of C_α was so small that it may be assumed

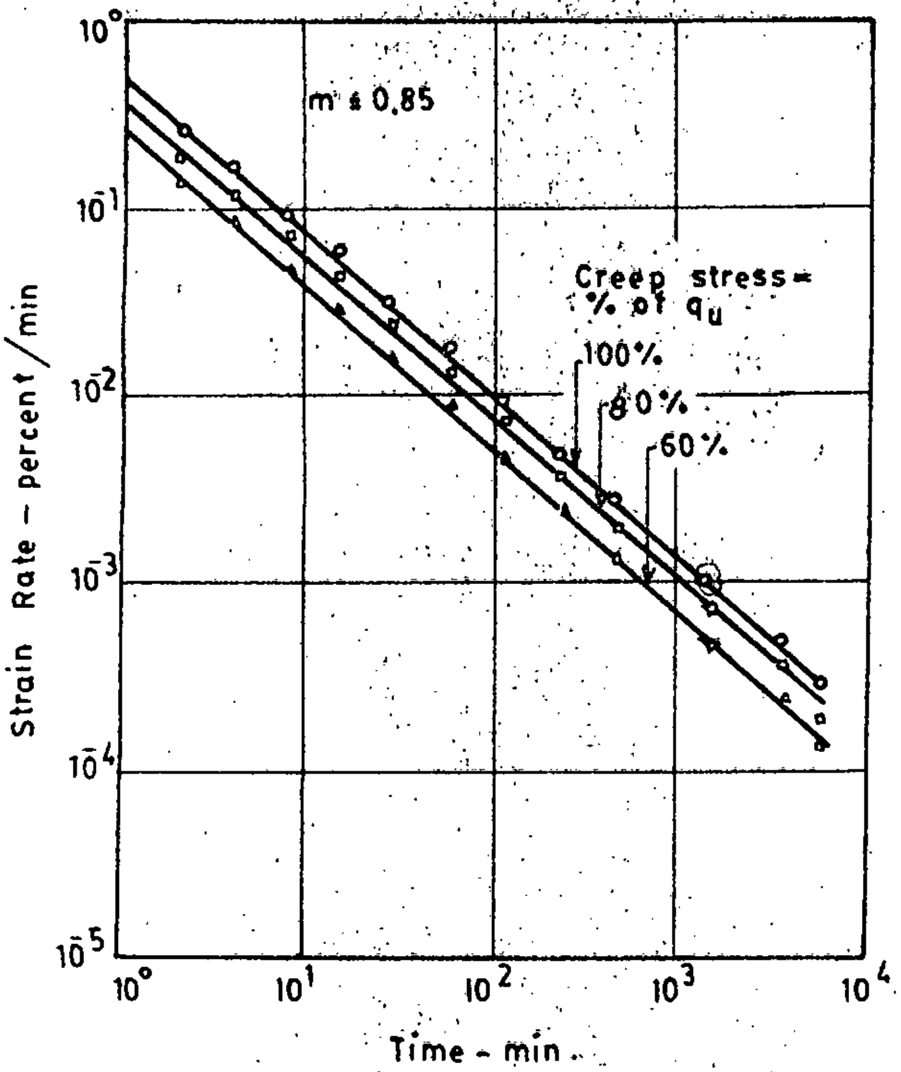


Fig. 5.3(a) Strain rate versus time relationships during creep of soil type 1.

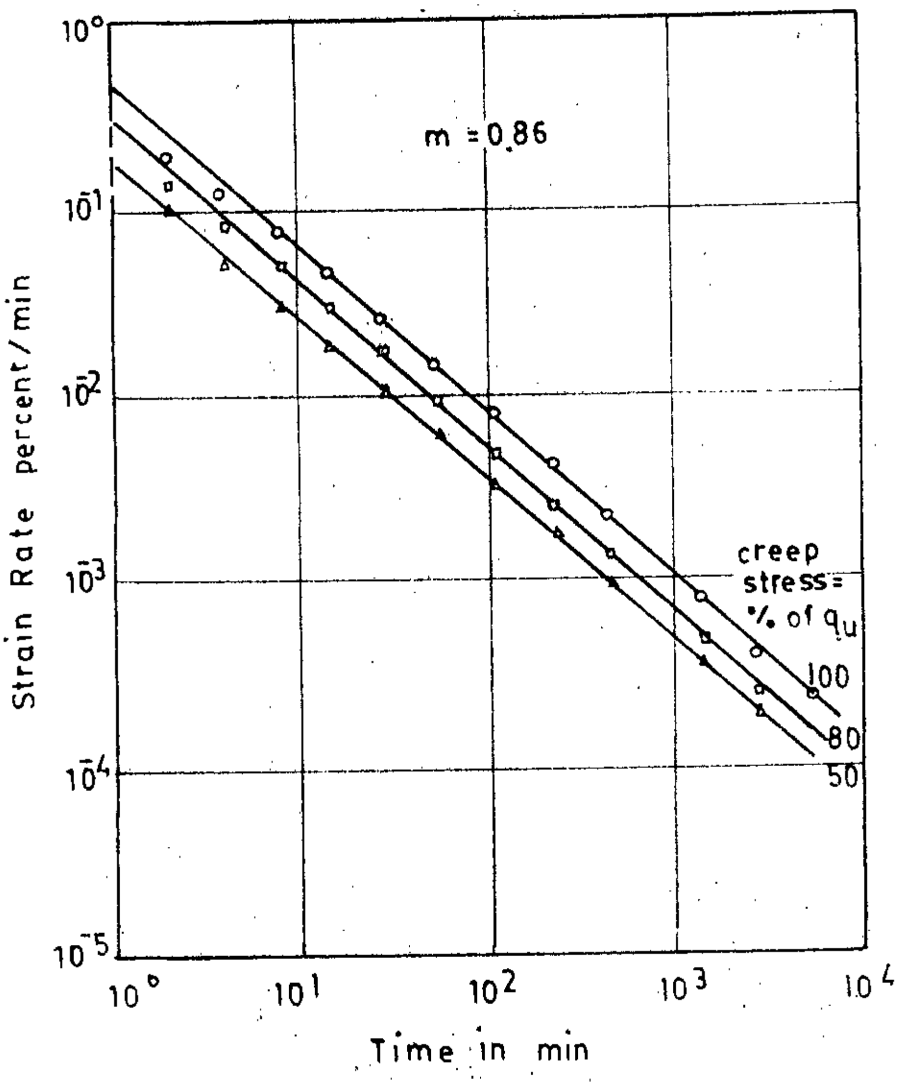


Fig. 5.3(b) Strain rate versus time relationship during creep of soil type 2.

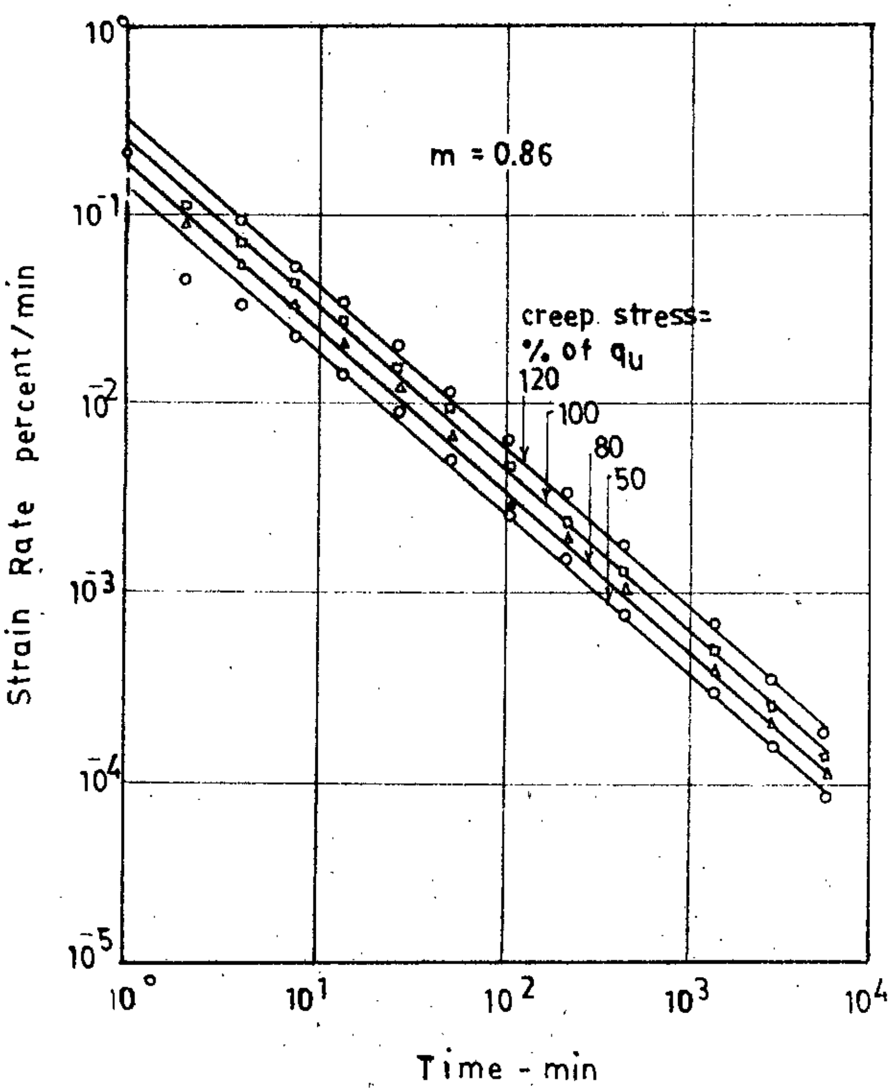


Fig.5.3(c) Strain rate versus time relationship during creep of soil type 3.

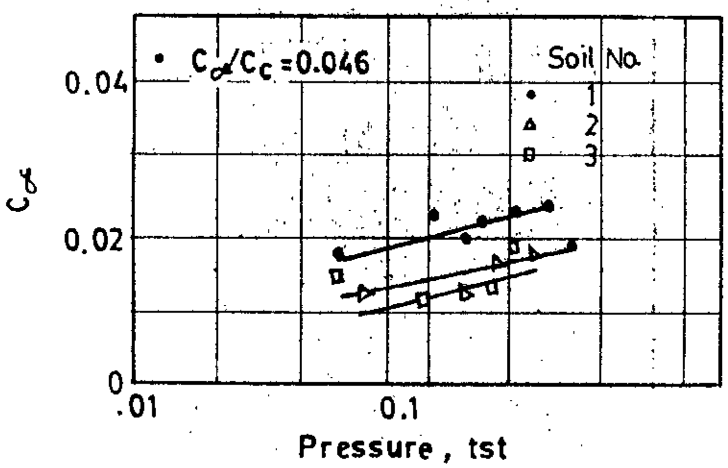


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Fig. 5.4 Variation of C_α with pressure.

constant within the practical range of interest. For example, for soil 1, the magnitude of C_α increased from 0.0226 to 0.0245 for increase in creep stress from 60 to 100 per cent of unconfined compressive strength. The order of variation was same for other soils investigated. Therefore, it may be assumed that C_α is independent of pressure, provided the pressure in within the limit of normal engineering practice.

d) Variation of C_α with Time

It was found that C_α did not remain constant with time after the end of primary consolidation. Mesri and Godlewski (1977) first reported this behaviour. The result of long term consolidation test shown in Fig. 4.9 for the soil types investigated give more proof about this behaviour. After the end of primary consolidation C_α remained constant to a value of 0.00237 for 4 days, then increased to a new value of 0.00386. This later constant value was maintained within the remaining period of test. However, this relationship may not be valid for the soil types different from that presently used. Whether similar behaviour will occur or not will depend upon the type of soil and therefore, subject to confirmation from experimental results.

5.2 Relationship Between C_α and C_c

A linear relationship was observed between true secondary compression index, C'_α and true primary compression index C'_c. Soil 2 was only used for this investigation. To obtain this relationship, the soil deformation was observed for 24 hours period between increments of load with load increment ratio of 1 between 1/4 tsf to 2 tsf. The dial reading versus log time curve during the observation is illustrated in Fig. 5.5. To plot the curve between C'_α and C'_c, the true compression index may be considered as $C'_c = \Delta e/\Delta \log\sigma$ at a particular pressure and these values of C'_c were taken at different pressures and the corresponding values of $C'_\alpha = \Delta e/\Delta \log t$ at was used. The relationship thus found, is shown in Fig. 5.6. The equation of the linear relationship between C'_α and C'_c is found to be $C'_\alpha = 0.000862 + 0.0172\, C'_c$.

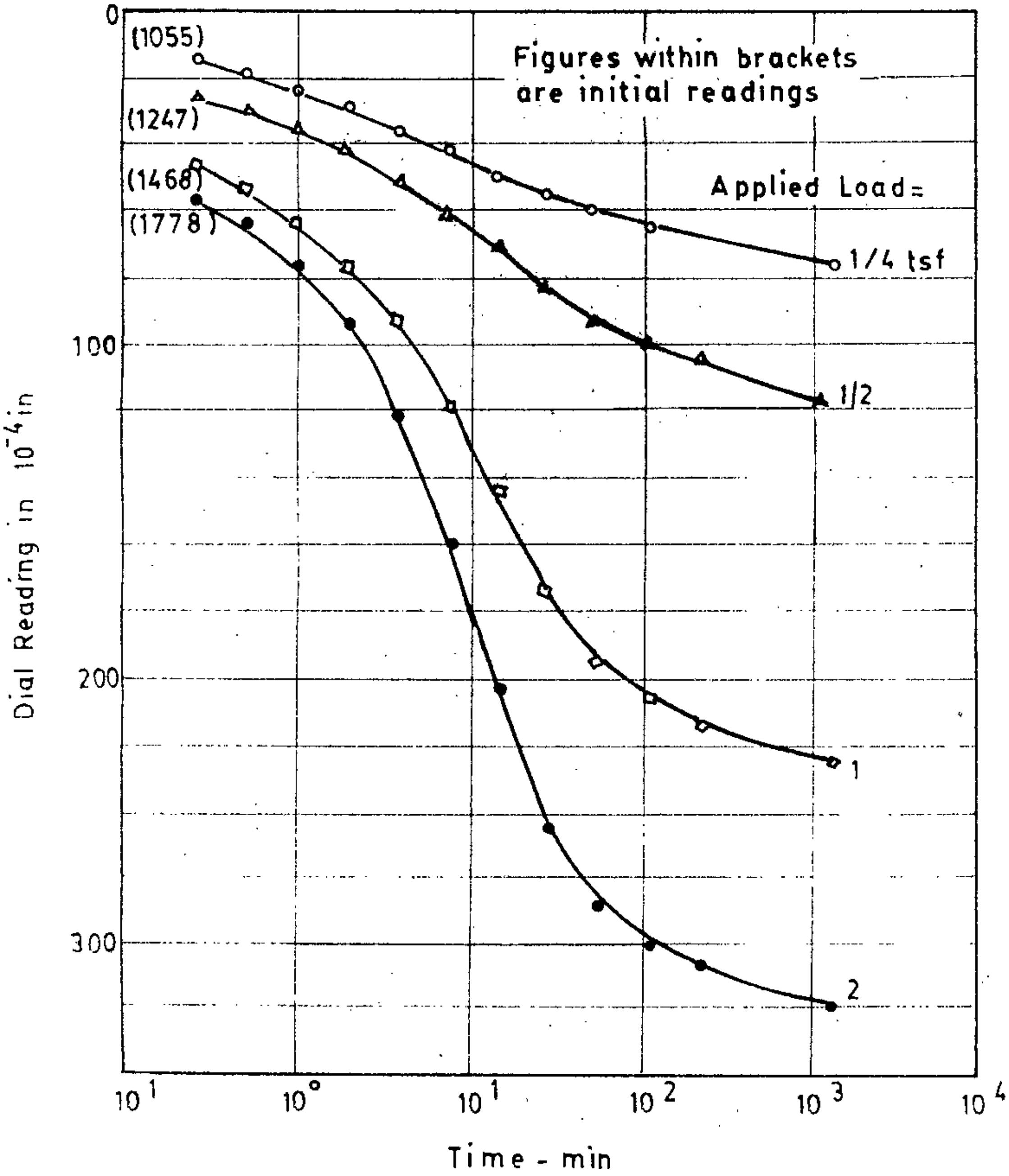

Fig. 5.5 Dial reading versus log time curve for the soil sample used to observe the relationship between C'_{α} and C'_{c}.

(c)

(b)

(a)

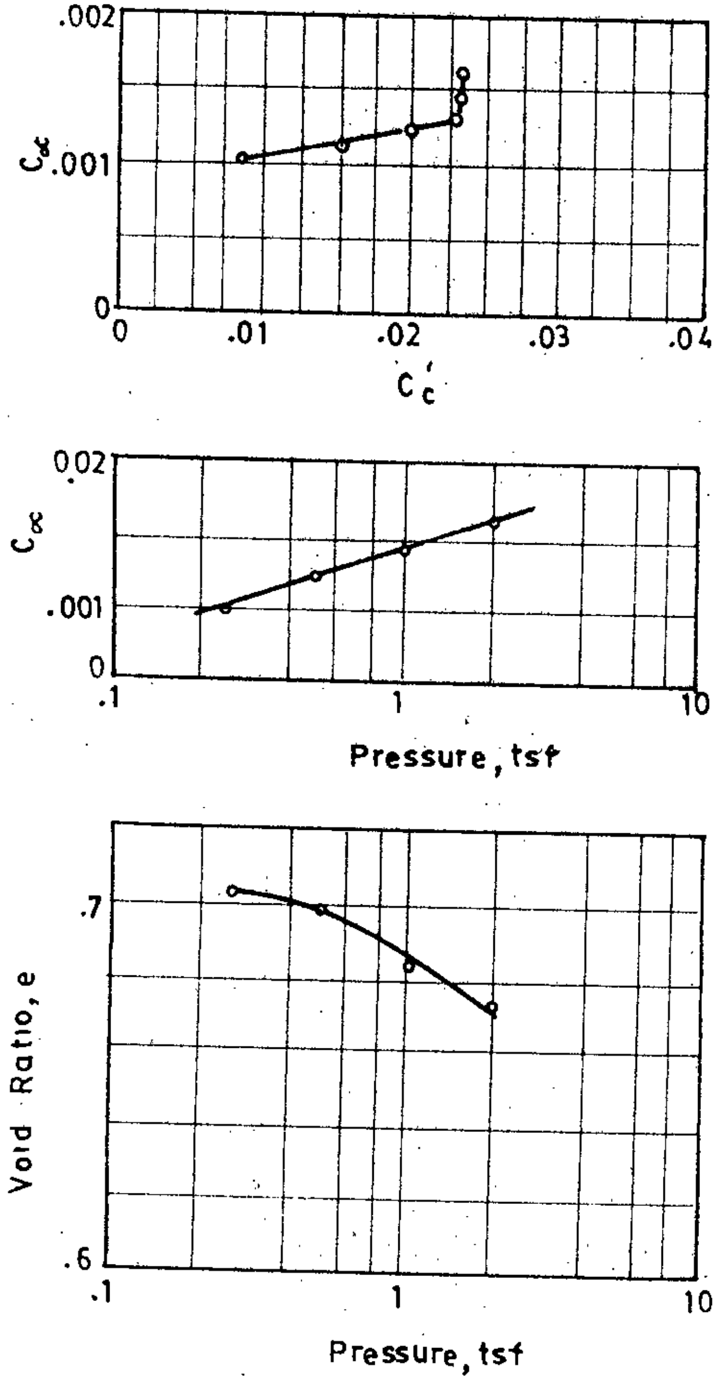

Fig. 5.6 (a) Void ratio versus log pressure curve (b) Variation of C_α with pressure and (c) Relationship between C_α and C_c'.

Since it is a general practice to assume a constant C_c value for a soil and also that C_α is generally assumed constant, therefore C_α/C_c value may be assumed constant for a particular soil. This may vary with soil type. It is mentioned earlier in Art. 2.13 that Mesri and Godlewski (1977) found the value of C_α/C_c for a variety of natural soils to lie within the range of 0.025 to 0.10. Only one natural soil was investigated for the purpose and the magnitude of C_α/C_c for it in the remoulded state was found to be 0.046. To find the value of C_α/C_c, the slope of the straight line portion of the void ratio versus log pressure curve was taken as C_c and the average value of C_α within the pressure whether e-log curve is straight was used. The magnitude of C_α/C_c for soil 1, thus gave further confirmation to the demonstration of Mesri and Godlewski (1977).

5.3 Verification of Singh-Mitchell (1968) Creep Function in One Dimensional Case

It is mentioned earlier in Art. 2.13 that more experimental evidence for different types of soils were necessary in order to establish the overall validity of the creep function. This function was tested using triaxial test results only. The applicability of the results for confined compression from the results of one dimensional consolidation tests has not been investigated. Further confirmation in both three dimensional compression and uni-axial compression were necessary for various types of soils. In the present investigation, time deformation results of one dimensional consolidation tests on three different types of soils have been analyzed.

Since Singh-Mitchell (1968) creep function was developed for predicting the axial deformation from the results of three dimensional consolidation, it was necessary to modify the creep parameters for predicting deformation in one dimensional strain condition. Such modifications have been presented in Art. 2.11 and discussed in detail. On the basis of these modifications, the Singh-Mitchell parameters A, $\bar{\alpha}$, m were calculated from the relationship of log strain rate versus stress and log strain rate versus log time.

To calculate $\bar{\alpha}$, it was necessary to use deviatoric stress, D. The value of the lateral pressure $\bar{\sigma}_3 = k_o \bar{\sigma}_1$ was used in the calculation of above deviatoric stress value. Two methods were available for calculation of k_o value and they yielded two values of D as well as two different values of $\bar{\alpha}$. Again to predict the creep at any time the creep function required the use of $\bar{\alpha}$ and $\bar{D}$. The values of the deviatoric stress, D and the corresponding stress level $\bar{D}$ for the creep stress at 100 per cent of the unconfined compressive strength (q_u) for different soils are given in Table 5.1. For other stresses, D and $\bar{D}$ can be calculated by multiplying D and $\bar{D}$ values in Table 5.1 by the corresponding

Table 5.1 Calculation of Stress Level

Soil No.	Plasticity index (PI)	Creep stress (tsf)	(a)			(b)		
			k_o	$D = \sigma_1' - k_o\sigma_1$ (tsf)	$\bar{D} = D/q_u$	k_o	$D = \sigma_1 - k_o\sigma_1$ (tsf)	$\bar{D} = D/q_u$
1	9	0.27	0.46	0.145	0.54	0.42	0.159	0.59
2	10	0.17	0.47	0.090	0.53	0.42	0.098	0.58
3	14	0.23	0.50	0.115	0.50	0.47	0.125	0.54

(a) Brooker and Ireland's method was used in finding k_o (Art.2.11).
(b) Alapn's method was used in calculating k_o (Art. 2.11).

percentage of the q_u value. In the present investigation, the creep function was examined within the range of practical interest. The creep function was analyzed at three creep stresses for each type of soil, that is, at 60, 80 and 100 per cent of unconfined compressive strength in order to verify its applicability in one dimensional compression. To verify the parameters A, $\bar{\alpha}$ and m were calculated by the methods mentioned in Appendix-II and the values listed in Table 5.2.

The Singh-Mitchell (1968) creep function used for calculating the axial strain e is

$$e = a + \frac{A}{1-m} e^{\bar{\alpha}\bar{D}} t_1^m (t)^{1-m} \quad m \neq 1, \; t_1 = 1 \qquad (16)$$

k_o was calculated by two methods, that is, Brooker and Ireland (1965) and Alpan (1967) would lead to two different values of

Table 5.2 List of the Singh-Mitchell Creep Parameters

Soil No.	m	(a)		(b)		A (%/min)
		α (1/tsf)	$\bar{\alpha} = \alpha . q_u$	α (1/tsf)	$\bar{\alpha} = \alpha . q_u$	
1	0.85	10.01	2.70	9.13	2.47	0.092
2	0.86	18.38	3.12	16.87	2.87	0.053
3	0.86	9.89	2.28	9.50	2.13	0.050

(a) α and $\bar{\alpha}$ were calculated by use of the value of k_o that was found by applying Broker and Ireland's method (Art. 2.11).

(b) α and $\bar{\alpha}$ were found using the value of k_o, which was found by applying Alpan's method (Art. 2.11).

$\bar{\alpha}$ and $\bar{D}$. However it has been observed that the strain rate remained same irrespective of whichever value of $\bar{\alpha}$ and $\bar{D}$ was used. The strain thus found, was more than three times higher than the observed strain. An empirical approach was therefore made so that the predicted strain was in good agreement with the measured value. The empirical approach involved the division of the predicted strain by a factor of 3. After division, a value of the constant 'a' for the creep function was assumed, the strain thus obtained coincided approximately with the measured strain over a range of time. On the basis of these observations, a modifed creep function may be suggested as follows:

$$e = a + \frac{1}{3} \left(\frac{A}{1-m} e^{\bar{\alpha}\bar{D}} t^{1-m} \right) \quad m \neq 1, \; t_1 = 1 \qquad (28)$$

The modified strain was compared with the observed strain and the comparison is shown in Figs. 5.7(a) to 5.7(i). From the figures it can be seen that the agreement is good over a range of time for the soils investigated. These can be a good agreement between observed and predicted strains upto a time of 120 minutes for the soils investigated at creep stress of 100 percent of q_u. The predicted strain differ widely after this period.

The reasons for variation of calculated strain by Singh-Mitchell (1968) creep function and the modified function would probably be due to the different testing methods through which these parameters were obtained. For example in defining $\bar{\bar{\alpha}}$ and $\bar{\bar{D}}$ for one dimensional compression, modifications were made in computing the value of k_o and instead of considering $(\sigma_1 - \sigma_3)$ at failure, the q_u value was used. Therefore, there will certainly be a difference in the magnitude of the parameters, for a particular soil, if they are determined from uniaxial consolidation test instead of triaxial test. Therefore an attempt may be made to correlate the axial strains determined by two methods. This may be done either by critically examining the assumptions made in Art. 2.11 or by experimentally from the test results of the same soil both in the triaxial and in one dimensional consolidation test.

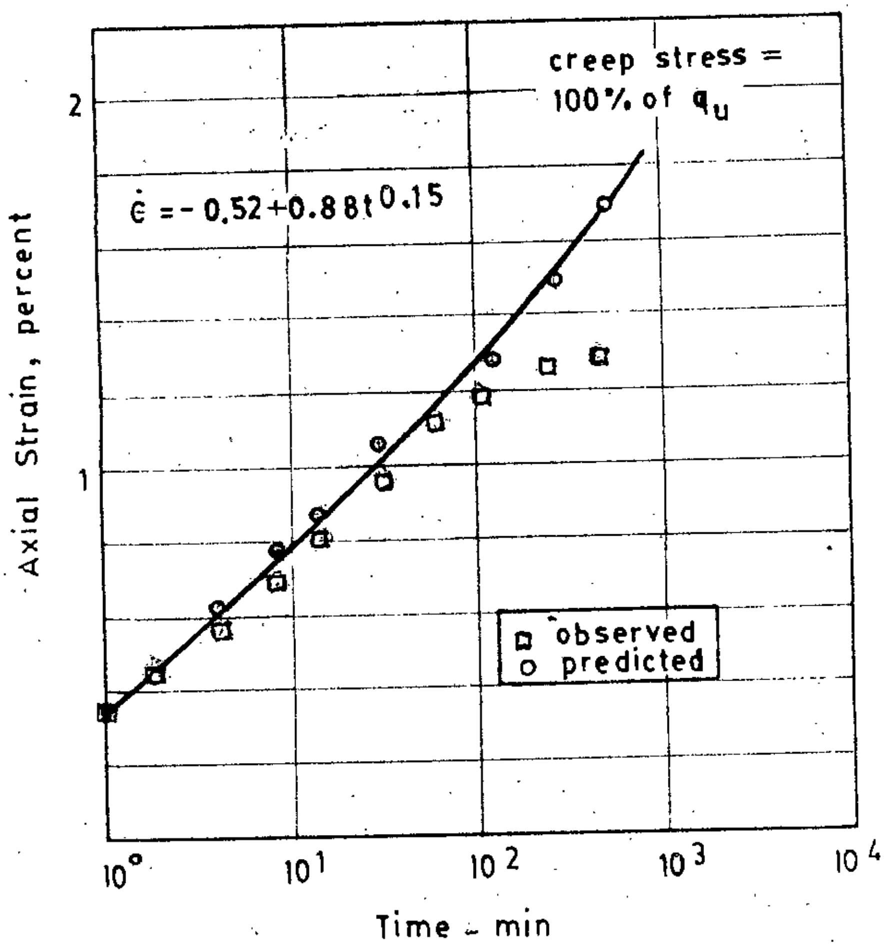

Fig. 5.7(a) Comparison between observed and predicted behaviour of creep by modified Singh-Mitchell creep function for soil type 1.

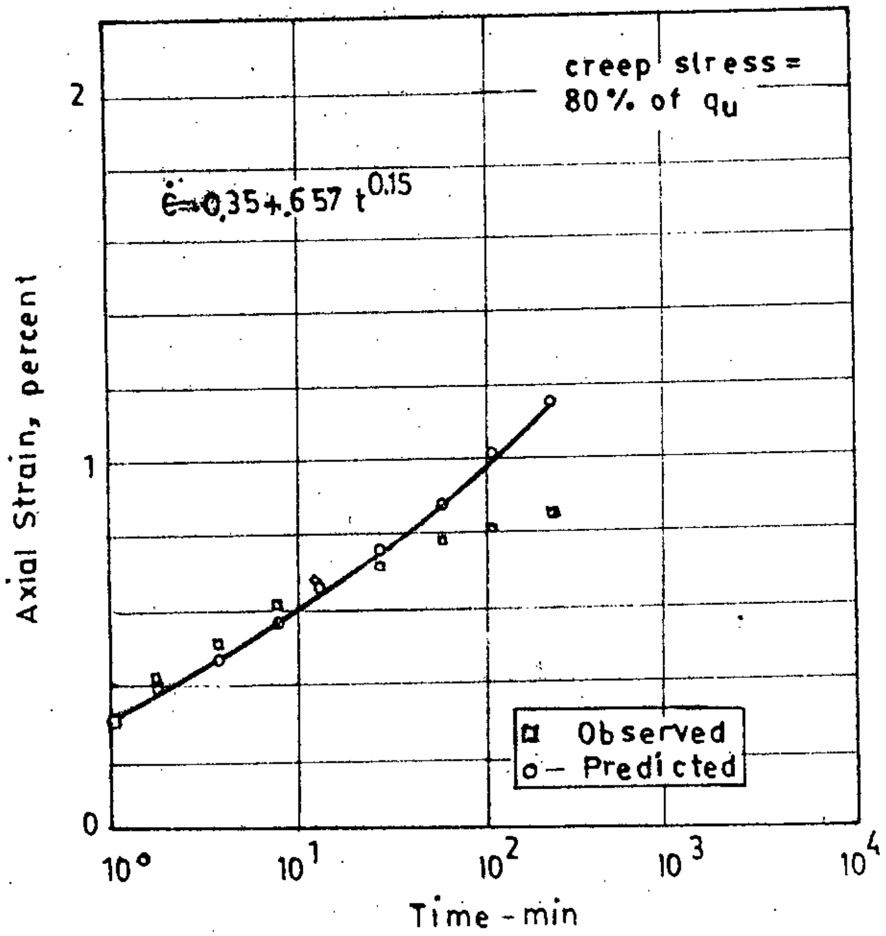

Fig.5.7(b) Comparison between observed and predicted behaviour of creep by modified Singh-Mitchell creep function for soil type 1.

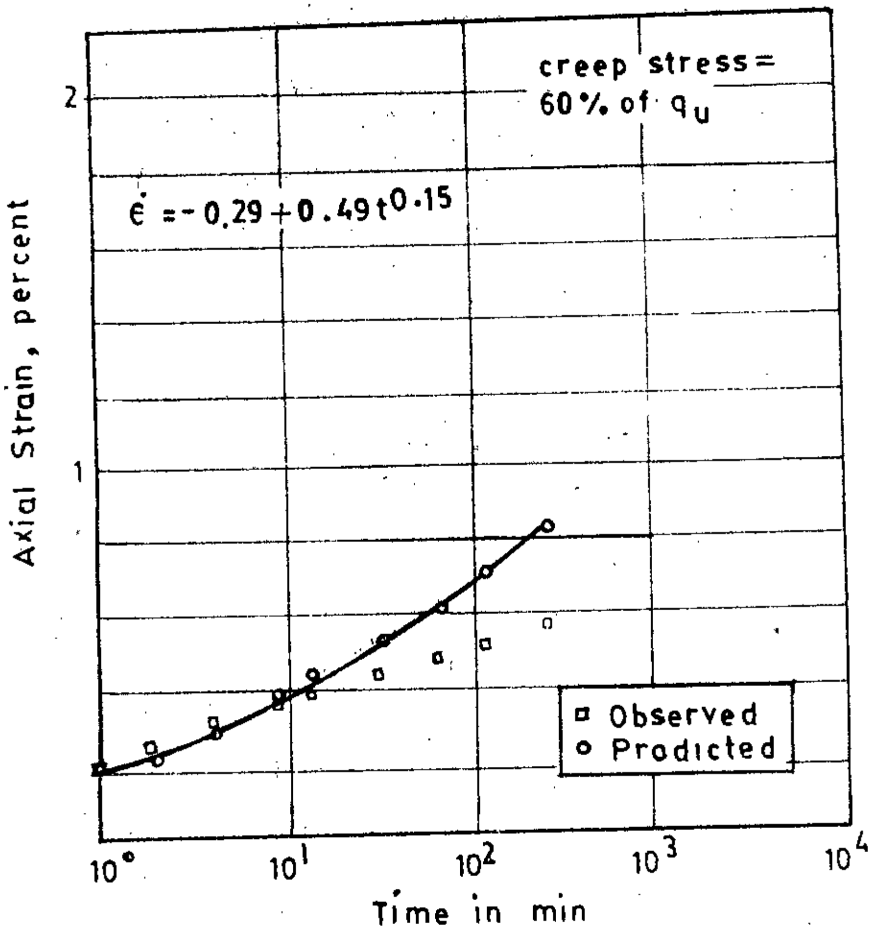

Fig. 5.7(c) Comparison between observed and predicted behaviour of creep by modified Singh-Mitchell creep function for soil type 1.

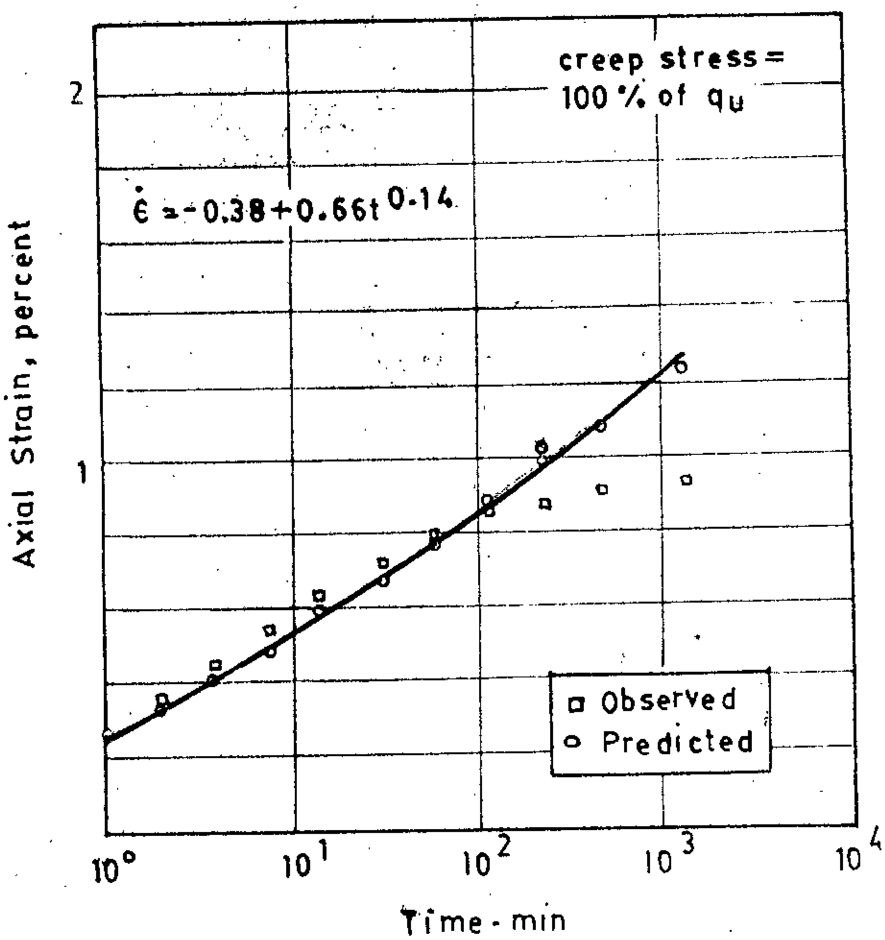

Fig. 5.7(d) Comparison between observed and predicted behavour of creep by modified Singh-Mitchell creep function for soil type 2.

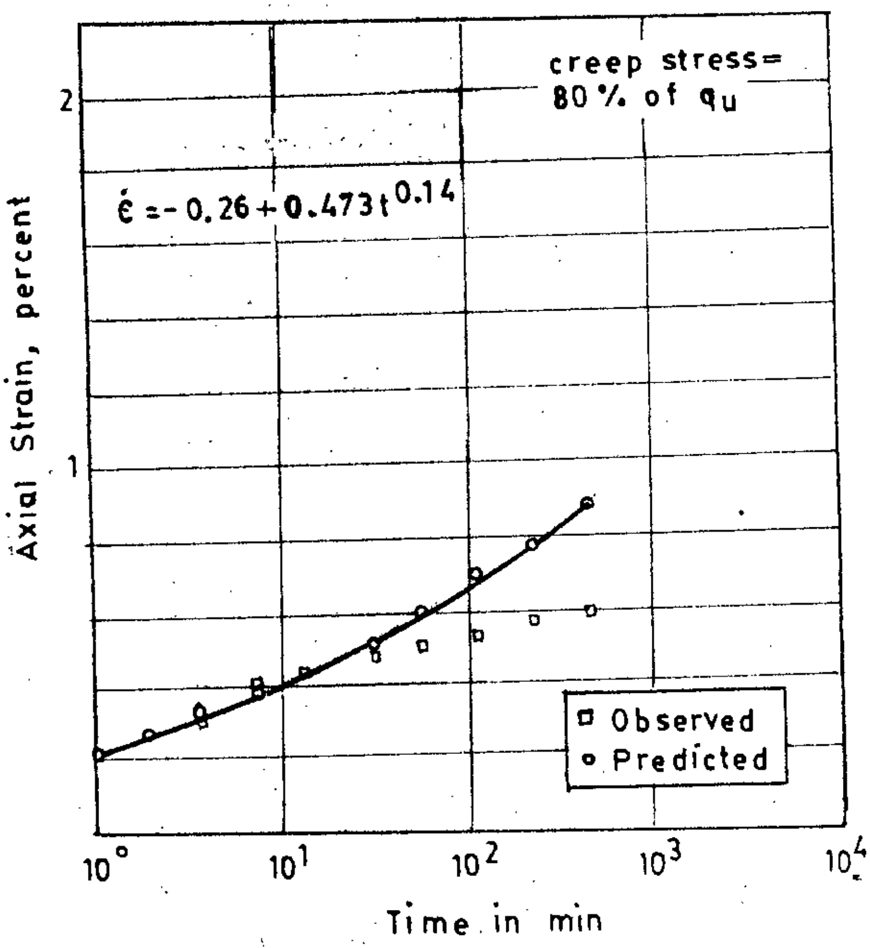

Fig. 5.7(e) Comparison between observed and predicted behaviour of creep by modified Singh-Mitchell creep function for soil type 2.

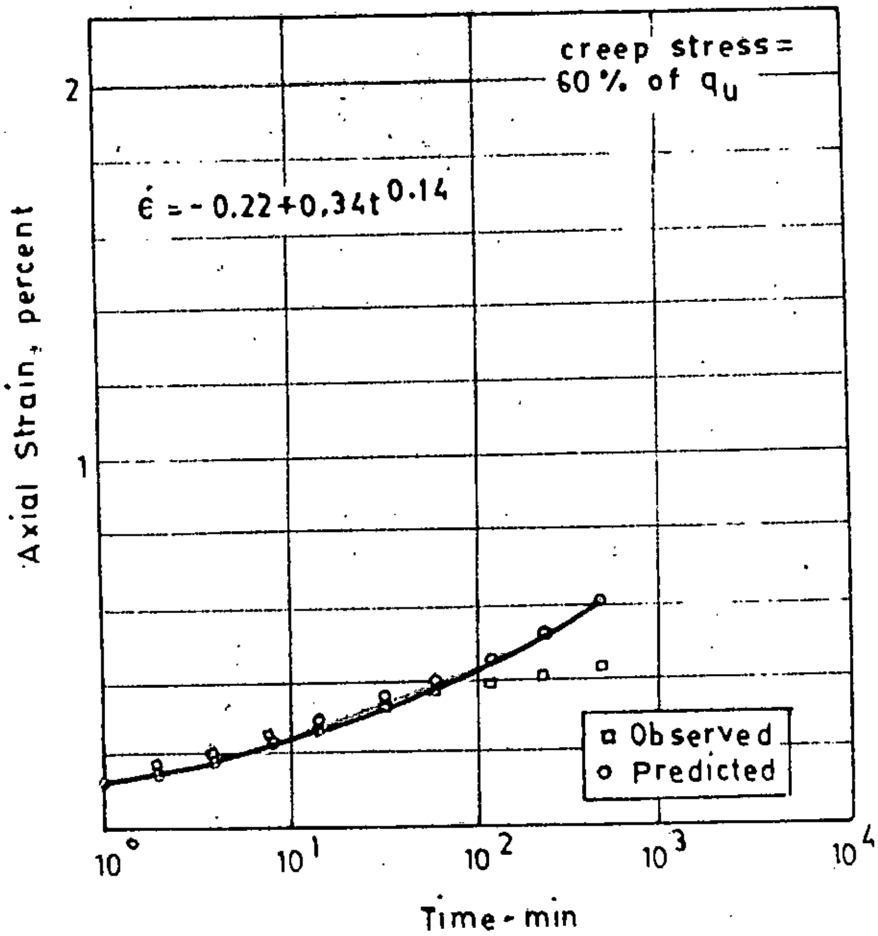

Fig. 5.7(f) Comparison between observed and predicted behaviour of creep by modified Singh-Mitchell creep function for soil type 2.

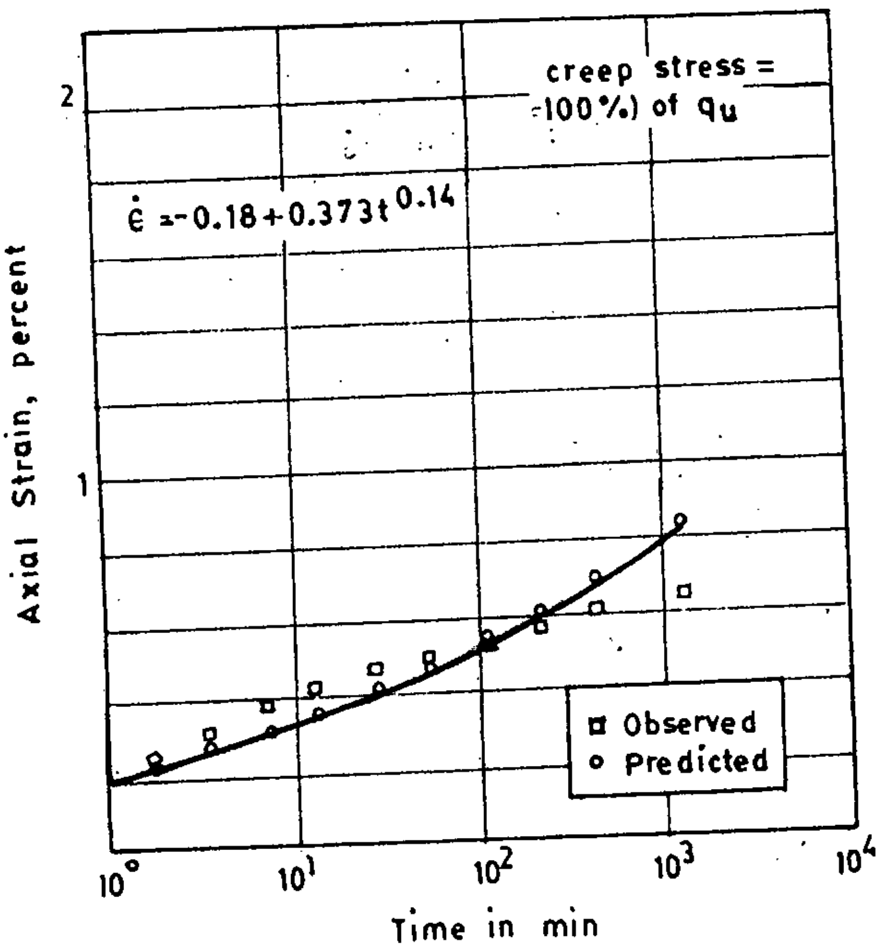

Fig. 5.7(g) Comparison between observed and predicted behaviour of creep by modified Singh-Mitchell creep function for soil type 3.

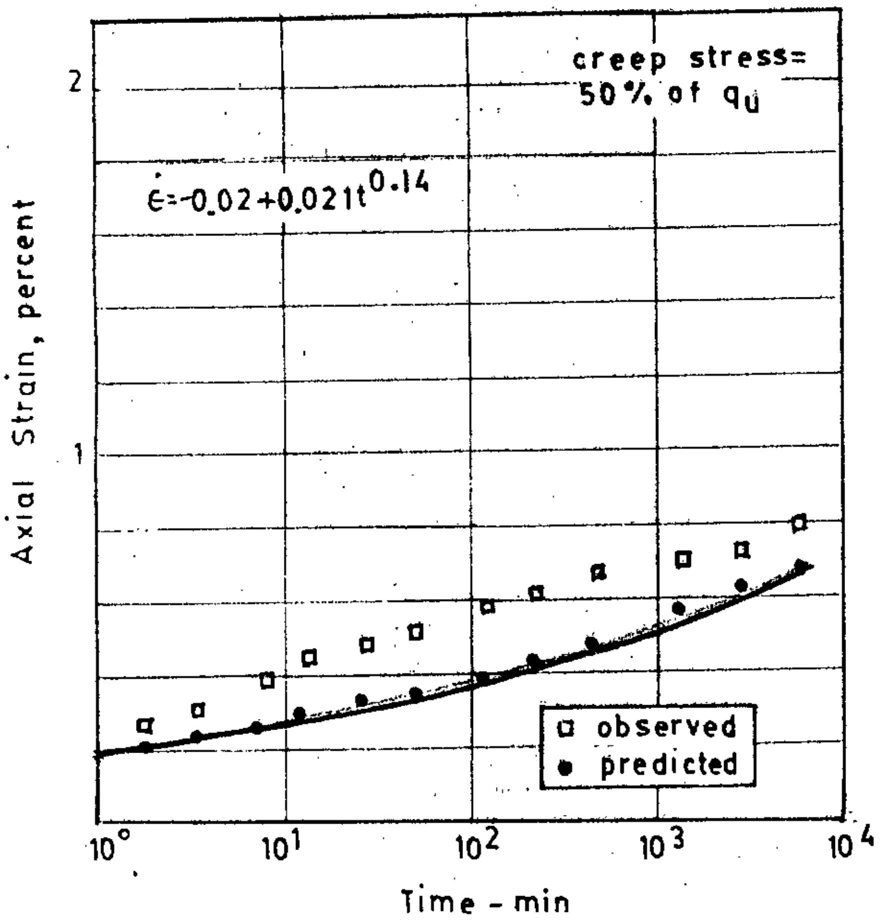

Fig. 5.7(i) Comparison between observed and predicted behaviour of creep by modified Singh-Mitchell creep function for soil type 3.

5.4 Discussion on Kavazanjian-Mitchell (1980) General Model

It was discussed in Art. 2.14 that the Kavazanjian-Mitchell (1980) general model is based on Bjerrum's (1967) concept of immediate and delayed components of deformation. In this article, each of the component of deformation is examined critically to check the model and to suggest modification, if necessary, so that the time deformational behaviour can be predicted from the result of one dimensional compression.

Volumetric Model: Because of hydrodynamic lag, immediate volumetric deformations cannot be determined directly. Rules for delayed volumetric deformations must be developed first and immediate deformation must be back calculated from primary deformations. For simplicity, deformation after unit time is taken as delayed component of deformation. To describe the delayed volumetric compression, it is assumed that C_α in one dimensional compression is constant, that is, the secondary compression curve on a void ratio-log time plot is a straight line, the slope of which is independent of both volumetric and deviatoric stress. This assumption may be taken correct as described in Art. 5.1(c).

In describing volumetric model it is also assumed that as C_c and C_α are constants, deformation at constant deviatoric stress levels can be described by a series of parallel planes, as shown in Fig. 2.17. In this assumption, however, they omit the variation of C_α with time. But the variation of C_α with time must be taken into consideration as reported by Mesri and Godlewski (1977) and also observed in the present investigation. The result of this variation is described in Art. 5.1(d). In this connection, it may be mentioned that, some investigators found constant C_α for a wide range of time (Art. 1.2) and for this constant C_α, the assumption of parallel planes may be taken correct. But in general, the assumption of unique relationship among void ratio, effective stress and time under sustained loading is subject to limitations by the assumption of constant C_α with time and therefore the soil which shows different values of C_α with time can not be correlated with this assumption.

Deviatoric Model: Only the delayed deviatoric deformation is of present interest. To describe the delayed deviatoric deformation, Kavazanjian and Mitchell (1980) adopted the Singh-Mitchell (1968) creep model. To use this particular model in one dimensional case, the deviatoric stress level is defined as $(\sigma_1 - k_o\, \sigma_1)/q_u$ (Art. 2.14). But it is shown in Art. 5.3 that there is a discrepency between observed deformation and predicted by Singh-Mitchell creep function. The reasons for this deviation may be due to the difference between the testing methods, in sample thickness and between the definition of soil strength. However, other reasons may affect this variation. Therefore another model as suggested in Art. 5.3 by eq. (28) may be used to provide a good representation of the real soil behaviour.

5.5 Deformation Behaviour of Soil with its Type

Two different soils were investigated to observe the deformation behaviour of soil with its type. The two soils, soil 2 and 3, reconstituted from the mixture of same type of silt and clay, are analyzed to observe the effect of finer particles on the nature of the time deformational behaviour. However, no general conclusion could be made from this analysis. It was observed that soil 3 has milder slope than the soil 2 in the log strain rate versus creep stress curve and also the 'A' value of soil 3 was found lower than that of soil 2. Though the m values for two soils were found same, the strains in unit time were found lower for soil 3 than soil 2. This fact was also observed by comparison of Fig. 5.1(b) and 5.1(c), that the axial strain of soil 3 was lower than that of the soil 2 in the same length of time under a particular creep stress. Since the soil 3 had higher percentage of clay it may be suggested that the lower strain of soil 3 was due to this higher clay contents.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE RESEARCH

6.1 Conclusions

The stress-strain-time relationships of cohesive soils have been investigated in the present study with a view to check the previous phenomenological models used to predict the deformational behaviour with time. Schemes were undertaken also, to observe time-dependent deformational behaviour of two soils. The important findings of the research and conclusions drawn on various aspects of the research can now be summarised as follows:

1. The general time deformation curves showed that the initial creep rate was higher and then decreased continuously for a period of time, after this period the creep rate attained a constant value. The period after which the creep rate become constant for the soils invastigated was found to be one day.

2. A linear relationships was obtained between (a) logarithm of strain rate and logarithm of time for a given creep stress and (b) between logarithm of strain rate and creep stress at any given time provided the creep stress was within the practical range of interest, that is, 40 to 100 per cent of unconfined compressive strength.

3. The secondary compression index, C_{α} can be assumed independent of creep stress, provided the stress is within the practical range of unconfined compressive strength.

4. C_α did not remain constant with time for some soils.

5. A linear relationship has been found to exist between true secondary compression index, C'_α and true primary compression index, C'_c for the soils investigated. However, this may not be true for all soil types.

6. The range of C_α/C_c values for a variety of natural soils were found to lie between 0.025 and 0.10.

7. The Singh-Mitchell (1968) creep function has been tested by the data obtained from consolidation test results of three different soils and it has been observed that the predicted strain was greater than three times than the observed strain. The following empirically modified function has been proposed:

$$e = a + \frac{1}{3}\left(Ae^{\bar{\alpha}\bar{D}}\, t^{1-m}\right) \qquad m \neq 1,\ t_1 = 1 \qquad (28)$$

8. The unique relationship among void ratio, creep stress and time under sustained loading suggested by Kavazanjian and Mitchell (1980) is subject to limitation of assumption of constant C_α with time and it has been shown that C_α did not remain constant with time for some soils.

6.2 Recommendations for Future Research

Several aspects of the work presented in this thesis require further study and have been described in the relevant chapters. Some of the important areas for further research may be listed as follows:

1. Only the deformational character of one inorganic silt and two clayey silts have been investigated in the remoulded state. To observe this behaviour on other types of soils and to observe the influence of composition of the soil on the stress-strain-time behaviour, the same investigation may be carried out for undisturbed samples of other types of soils, specially organic clays.

2. In plotting the log strain rate versus creep stress some inconsistency in the results have been observed. That is, the axial strain after a period of time is higher for a lower creep stress. This inconsistency may result from the variation in the fabric of the soil. As there was no available methods to study the fabric details of each individual samples the assumption of similar fabric condition could not be ensured. Further research is necessary to develop a method by which samples with identical soil state and fabric may be made.

3. The Singh-Mitchell (1968) creep function has been investigated on limited types of soils only. The function should be checked on a wide variety of soil in order to establish its validity. Three dimensional consolidation tests may be performed on a number of different types of soils to verify the consistency of the predicted result with the observed behaviour.

4. In the present research, the Singh-Mitchell (1980) creep function is examined with respect to prediction of axial deformation in confined compression from the results of one dimensional consolidation tests. The predicted strain by this function is found more than that observed experimentally. A modified function is therefore suggested to predict axial strain. However after a certain period of time, a variation between predicted and observed strain was noted. The reason for such discrepency is not known. An attempt may be made to relate predicted and observed axial deformations by relating the axial deformation in three dimensional with one dimensional consolidation.

5. The relationship between $C'_{\alpha} = \Delta e / \Delta \log t$ and $C'_{c} = \Delta e / \Delta \log \sigma$ and the variation of C_{α} with time have been observed for a very limited types of soil. Therefore, more experimental evidence on a wide variety of cohesive soils is necessary in order to confirm this behaviour.

APPENDIX-I

KONDNER'S (1963) HYPERBOLIC STRESS-STRAIN RELATION

Kondner (1963) suggested that the stress-strain curve for a cohesive soil from a consolidated undrained triaxial test could be represented by a rectangular hyperbola. Fig. I-1 shows a rectangular hyperbola passing the origin in a two dimensional stress-strain plane and having as asymptotes the lines

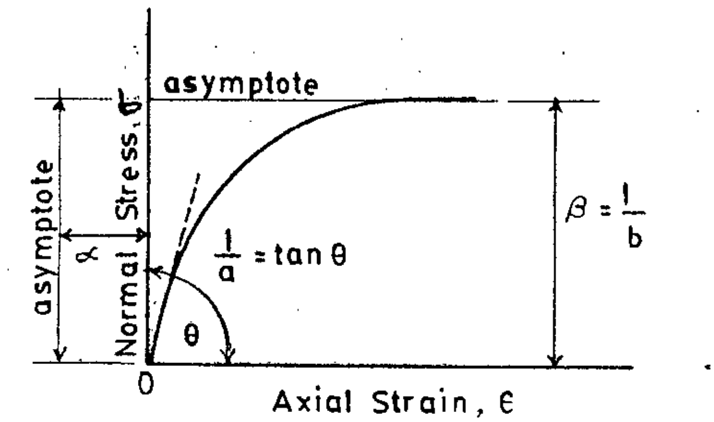

Fig. I-1 Rectangular hyperbolic representation of stress-strain (Kondner, 1963).

$$e + \alpha = 0 \quad (29)$$

$$\sigma - \beta = 0 \quad (3)$$

in which σ is the deviator stress, $\sigma_1 - \sigma_3$.

The equation of the hyperbola can be written as

$$e\sigma - \beta e + \alpha = 0 \quad (31)$$

Dividing by σ and letting $k = \frac{e}{\sigma}$ (32)

Eq. (31) becomes

$$e - \beta k + \alpha = 0 \qquad (33)$$

$$\text{or, } k = \frac{1}{\beta} e + \frac{\alpha}{\beta} \qquad (34)$$

which is the form of a straight line, if k is plotted as a function of e. The straight line thus obtained intersects the strain axis at a point $(-\alpha, 0)$ on the vertical asymptote of the original hyperbola. The inverse of the slope of the line (de/dk) is a measure of the height, β; of the horizontal asymptote.

Rearranging the terms eq. (34) yields,

$$\frac{e}{\sigma} = a + be \qquad (35)$$

in which

$$a = \frac{\alpha}{\beta} \qquad (36)$$

$$b = \frac{1}{\beta} \qquad (37)$$

Fig. I-2 is a plot of straight line of the form of eq. (35) in terms of e/σ and e.

Solving for stress eq. (35) yields

$$\sigma = \frac{e}{a + be} \qquad (38)$$

The ultimate value of the stress can be obtained by taking the limit of eq. (38) as e becomes very large, or

$$\sigma_{ult.} = \lim_{e \to \infty} \frac{e}{a + be} = \frac{1}{b} \qquad (39)$$

Thus, the ultimate strength is measured by the inverse of the slope of the straight line as given in Fig. I-2.

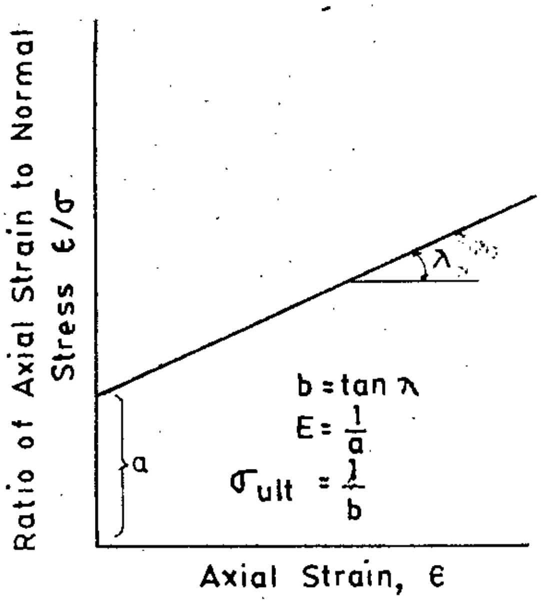

Fig. I-2 Transformed hyperbolic representation of stress-strain (Kondner, 1963).

Differentiating eq. (38) with respect to strain and evaluating the derivative at e equal zero yields

$$\left(\frac{d\sigma}{de}\right)_{e=0} = \frac{1}{a} \qquad (40)$$

Thus, the inverse of the intercept of the straight line form on the e/σ axis of Fig. I-2 is a measure of the initial tangent modulus of the material tested.

Therefore, the deviatoric behaviour of a cohesive soil at a specified overconsolidation ratio and a specified confining pressure can be described on the basis of this plot by two parameters, a slope and an intercept. The intercept is the reciprocal of the initial tangent modulus of the hyperbola and the slope is the reciprocal of the asymptote of the hyperbolic curve.

If the initial portion of the original stress-strain curve is linear, the initial part of the hyperbolic plot of e/σ versus e will be horizontal. For such a situation, the initial tangent modulus would still be $\frac{1}{a}$, in which 'a' is the intercept of the horizontal line AB, and $\frac{1}{b}$ would still be a measure of the ultimate strength of the material, in which b is the slope of the inclined straight line portion of the plot given as BC of Fig. I-3.

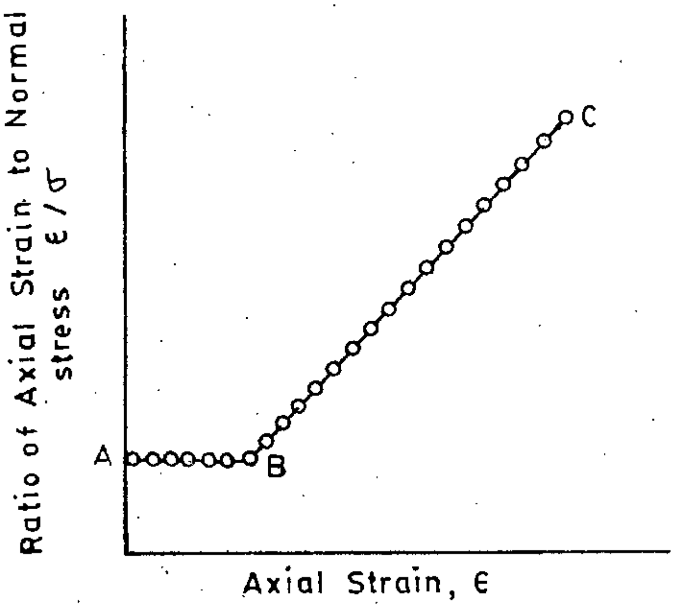

Fig. I-3 Composits response in transformed hperbolic form; (Kondner,1963).

APPENDIX-II

TEST PROCEDURE TO MEASURE THE PARAMETERS m, $\bar{\alpha}$, A

At least two identical samples, at the same water content and the same initial stress conditions, are necessary to measure the creep parameters of a given soil. The two samples are subjected to creep tests under two different stresses, say D_1 and D_2, such that the stress in either case is within the practical range of 40 to 100 per cent of unconfined compressive strength. This range may be different for different soils. Under these sustained loads, strain is observed against time.

The values of parameters m, $\bar{\alpha}$ and A can be found as follows:

Step 1: Strains are to be plotted against logarithm of time and a smooth curve is to be passed through the data points.

Step 2: For a particular applied stress, from the plot of strain versus logarithm of time, the corrected zero reading is to be determined.

Step 3: Strain rates at various values of time are to be computed mathematically or graphically. The best fit linear curve is to be passed upto the desired time, from which the strain at that time is to be calculated. By subtracting the corrected zero reading from the strain reading and dividing this by time, the strain rate can be found*.

Step 4: Strain rates are then to be plotted against time on log-log graph paper. Log strain rate versus log time plots for different stress values are served to be parallel straight lines. The slope of these straight lines gives the value of m.

Step 5: Values of strain rates $\dot{e}_1$ and $\dot{e}_2$ at some known time, for instance t = 1, are to be read off for the two deviatoric stresses D_1 and D_2. For t = 1, eq. 2 gives $\dot{e}_1 = Ae^{\alpha D_1}$ and $\dot{e}_2 = Ae^{\alpha D_2}$. Solving the two simultaneous equations gives

$$\alpha = \frac{1}{D_2 - D_1} \ln \frac{\dot{e}_2}{\dot{e}_1} \tag{41}$$

$$A = (\dot{e}_2 - \dot{e}_1) / (e^{\alpha D_2} - e^{\alpha D_1}) \tag{42}$$

These parameters can also be determined by plotting log $\dot{e}$ vs. D for a given value of time. The slope of this plot gives the value of α and its intercept for D = 0 and t = 1 gives the value of A.

Now $\bar{\alpha} = \alpha D_f$ where D_f is assumed unconfined compressive strength for one dimensional case..

* A computer programme was run to obtain strain rate for the determination of m, α and A. The listing of the programme is given in Appendix-VI.

APPENDIX-III

CONSOLIDATION READINGS

Table -I Consolidation Readings for Soil Type 1

Time (min.)	Dial Reading, in 10^{-4} in.					
	Percent of Unconfined Compressive Strength					
	20	40	60	80	100	120
$\frac{1}{4}$	1075	500	680	685	864	1162
$\frac{1}{2}$	1077	505	687	693	872	1167
1	1079	513	692	701	883	1175
2	1082	525	698	710	894	1183
4	1085	537	704	720	908	1192
8	1087	545	708	730	922	1200
15	1090	552	711	736	932	1206
30	1093	556	716	740	941	1211
60	1095	562	720	747	947	1215
120	1099	565	723	750	952	1220
240	1101	570	729	754	955	1222
480	1108	576	730	756	959	1226
1440	1111	582	737	763	966	1230
2880	1113	585	739	766	971	1231
5760	1115	589	745	771	974	1237
10080	1116	591	747	775	978	1239

Table-II Consolidation Readings for Soil Type 2

Time (min.)	Dial Reading, in 10^{-4} in.					
	Percent of Unconfined Compressive Strength					
	30	50	60	80	100	120
$\frac{1}{8}$	1568	2683	2095	1895	2070	1755
$\frac{1}{2}$	1573	2689	2099	1900	2079	1758
1	1575	2692	2102	1907	2088	1762
2	1579	2695	2106	1912	2097	1767
4	1582	2699	2110	1918	2107	1773
8	1584	2702	2114	1925	2116	1780
15	1586	2706	2117	1929	2125	1785
30	1590	2710	2122	1933	2132	1790
60	1592	2714	2125	1935	2140	1794
120	1594	2717	2127	1938	2145	1798
240	1598	2720	2129	1941	2149	1802
480	1600	2723	2131	1944	2151	1806
1440	1601	2726	2134	1946	2154	1810
2880	1610	2729	2136	1949	2157	1813
5760	1612	2732	2138	1952	2161	1816
10080	1616	2735	2140	1954	2164	1821

Table-III Concolidation Readings for Soil Type 3

Time (min.)	Dial Reading, in 10^{-4} in.					
	Percent of Unconfined Compressive Strength					
	30	50	60	80	100	120
¼	192	2058	1501	486	1870	1834
½	196	2062	1502	492	1874	1839
1	200	2067	1504	498	1880	1845
2	205	2072	1506	505	1886	1851
4	208	2077	1508	512	1893	1860
8	211	2087	1510	519	1899	1870
15	214	2092	1511	525	1904	1878
30	217	2095	1514	529	1908	1885
60	221	2100	1516	533	1910	1889
120	224	2105	1520	538	1914	1894
240	228	2109	1521	542	1918	1897
480	231	2114	1524	545	1922	1900
1440	236	2117	1527	547	1926	1909
2880	238	2119	1529	549	1928	1914
5760	241	2127	1531	551	1930	1917
10080	244	2129	1533	555	1934	1920

Table-IV Consolidation Reading to Observe the Effect of Time on C_α.

Creep Stress: 80 per cent of Unconfined compressive Strength.

Time (min.)	Dial Reading in 10^{-4} in.
$\frac{1}{4}$	590
$\frac{1}{2}$	606
1	625
2	648
4	670
8	687
15	696
30	705
60	709
120	712
240	716
480	720
1440	725
2880	729
5760	733
11520	740
23040	746

APPENDIX-IV

TABLES OF STRAIN RATES

Table-V Strain Rates to Plot log Strain Rate Versus Log Time Curve for Soil 1.

Time (min.)	Strain rate, percent per minute Creep stress, per cent of unconfined compressive strength					
	20	40	60	80	100	120
1	0.090000	0.264997	0.203332	0.280002	0.355000	0.244997
2	0.058594	0.185998	0.130500	0.183498	0.231997	0.161995
4	0.036500	0.122000	0.080000	0.115500	0.147999	0.102500
8	0.021309	0.073273	0.046130	0.069999	0.090892	0.061666
15	0.013078	0.044454	0.027096	0.042423	0.056051	0.037481
30	0.007505	0.024520	0.015048	0.023412	0.031713	0.020915
60	0.004181	0.013334	0.008238	0.012816	0.017377	0.011378
120	0.002346	0.007098	0.004436	0.006852	0.009322	0.006128
240	0.001287	0.003757	0.002404	0.003627	0.004912	0.003238
480	0.000727	0.001987	0.001275	0.001900	0.002576	0.001703
1440	0.000277	0.000718	0.000468	0.000683	0.000921	0.000611
2880	0.000148	0.000372	0.000244	0.000352	0.000474	0.000313
5760	0.000079	0.000192	0.000129	0.000182	0.000243	0.000162
10080	0.000047	0.000112	0.000076	0.000107	0.000141	0.000094

Table-VI Strain Rates to Plot log Strain Rate Versus log Time Curve for Soil 2.

Time (min.)	Strain rate, percent per minute Creep stress, per cent of unconfined compressive strength					
	30	50	60	80	100	120
1	0.134995	0.124999	0.121667	0.206667	0.280000	0.138334
2	0.084995	0.077996	0.079494	0.130996	0.184998	0.092498
4	0.050499	0.047998	0.049500	0.080000	0.116499	0.059997
8	0.028451	0.028095	0.029702	0.048035	0.069880	0.037916
15	0.016596	0.017256	0.018065	0.028974	0.043067	0.023890
30	0.009311	0.009914	0.010441	0.016124	0.024442	0.013816
60	0.005097	0.005610	0.005851	0.008692	0.013594	0.007745
120	0.002749	0.003105	0.003189	0.004632	0.007402	0.004260
240	0.001469	0.001695	0.001710	0.002458	0.003951	0.002314
480	0.000779	0.000915	0.000907	0.001293	0.002077	0.001246
1440	0.000283	0.000337	0.000329	0.000461	0.000740	0.000458
2880	0.000155	0.000177	0.000172	0.000238	0.000380	0.000240
5760	0.000083	0.000093	0.000089	0.000123	0.000194	0.000125
10080	0.000051	0.000056	0.000059	0.000072	0.000113	0.000075

Table-VII Strain Rates to Plot log Strain Rate Versus log Time Curve for Soil 3

Time (min.)	Strain rate, percent per minute Creep stress, per cent of unconfined compressive strength					
	30	50	60	80	100	120
1	0.149999	0.069997	0.058335	0.128327	0.176665	0.218327
2	0.098499	0.048494	0.038999	0.087498	0.117996	0.138994
4	0.061500	0.034497	0.024498	0.055249	0.075498	0.088996
8	0.036309	0.023035	0.014761	0.034642	0.045654	0.055773
15	0.021739	0.015083	0.008846	0.021438	0.027968	0.035084
30	0.012053	0.008786	0.005151	0.012020	0.015706	0.020295
60	0.006578	0.004876	0.002931	0.006568	0.008509	0.011262
120	0.003524	0.002651	0.001697	0.003535	0.004579	0.006137
240	0.001867	0.001423	0.000942	0.001905	0.002451	0.003278
480	0.000988	0.000761	0.000522	0.001018	0.001306	0.001730
1440	0.000359	0.000278	0.000199	0.000373	0.000473	0.000633
2880	0.000188	0.000146	0.000107	0.000197	0.000246	0.000332
5760	0.000098	0.000076	0.000057	0.000104	0.000127	0.000173
10080	0.000058	0.000045	0.000034	0.000062	0.000075	0.000102

Table-VIII Strain Rates to Plot log Strain Rate Versus Creep Stress Curve for Soil 1.

Creep stress % of q_u	Strain rate, percent per min.				
	Time				
	1	10	100	1000	10000
20	0.0900000	0.0178476	0.0027448	0.0003838	0.0000476
40	0.2649975	0.0616816	0.0083096	0.0009993	0.0001134
60	0.2033325	0.0387163	0.0051972	0.0006509	0.0000771
80	0.2800024	0.0588972	0.0080216	0.0009500	0.0001080
100	0.3550000	0.0764758	0.0109066	0.0012811	0.0001429
120	0.2449975	0.0518440	0.0071745	0.0008496	0.0000956

Table-IX Strain Rates to Plot log Strain Rate Vs. Creep Stress Curve for Soil 2.

Creep stress % of q_u	Strain rate, percent per min. Time				
	1	10	100	1000	10000
30	0.1349951	0.0237825	0.0032247	0.0003951	0.0000752
50	0.1249999	0.0236533	0.0036290	0.0004674	0.0001142
60	0.1216674	0.0249758	0.0037289	0.0004577	0.0000726
80	0.2066674	0.0403503	0.0054268	0.0006425	0.0000531
100	0.2800000	0.0588755	0.0086527	0.0010285	0.0000563
120	0.1383349	0.0319429	0.0049770	0.0006345	0.0000512

Table-X Strain Rates to Plot log Strain Rate Vs. Creep Stress Curve for Soil 3.

Creep stress % of q_u	Strain rate, percent per min. Time				
	1	10	100	1000	10000
30	0.1499996	0.0304917	0.0041286	0.0005002	0.0000591
50	0.0699975	0.0195137	0.0030931	0.0003855	0.0000460
60	0.0583350	0.0124072	0.0019830	0.0002750	0.0000348
80	0.1283276	0.0291902	0.0041346	0.0005183	0.0000629
100	0.1766656	0.0384370	0.0053586	0.0006574	0.0000757
120	0.2183276	0.0469089	0.0071737	0.0008789	0.0001029

APPENDIX-V LISTING OF THE COMPUTER PROGRAMME UBJMAINT TO CALCULATE STRAIN RATES.

```
./ LIST

./ END

8M451   SYS004 IS 04D - 354C - BLKSIZE IS 0080 - RECSIZE IS 80

* $$ JOB JNM=CF37SOIL,DISP=D,CLASS=A
* $$ LST DISP=D
************************************************************************

// JOB CF37SOIL CF37
// OPTION LINK,LIST
// EXEC FFORTRAN
C      PROGRAM FOR THE DETERMINATION OF SINGH-MITCHELL PARAMETERS
C
       DIMENSION XX(25),Y(25),X(25),TIME(25),DIFF(25),XY(25),
      ,XSQ(25),RSDL(25),CHART1(25),CHART2(6,25),YY(25),STLVEL(6)
     1 FORMAT(1H1,///56X,'** SPECIMEN NO. ',I1,' **')
     2 FORMAT(3I2)
     3 FORMAT(8F10.5)
     4 FORMAT(10F8.2)
     5 FORMAT(2F8.5)
     6 FORMAT(//34X,'*** AT STRESS LEVEL OF ',F4.2,' OF UNCON'
      ,'FINED COMPRESSIVE STRENGTH ***'//21X,'I',5X,'XX(I)',
      ,6X,'X(I)',5X,'Y(I)',5X,'XY(I)',6X,'XSQ(I)',4X,'I',5X,
      ,'XX(I)',5X,'YY(I)',4X,'RSDL(I)'/)
C
     7 FORMAT(20X,28('*'),/20X,'Y=',F10.5,'+ ',F10.5,' *X(I)'
      ,//20X,'RESIDUAL=',F12.3,//20X,'FOR TIME= ',F6.0,3X,'Y(I
      ,)= ',F10.2//)
     8 FORMAT(//21X,'** THE FOLLOWING ARE THE TABLES TO DETER'
      ,'MINE THE SINGH-MITCHELL PARAMETERS-A, ALPHA AND M **'
      ,/34X,'*** AT STRESS LEVEL OF ',F4.2,' OF UNCONFINED CO'
      ,'MPRESSIVE STRENGTH ***'//45X,'** TABLE FOR THE DETER'
      ,'MINATION OF ALPHA **'//50X,'TIME',6X,'STRAIN',5X,'STR'
      ,'AIN-RATE'/50X,'(MIN)',4X,'(PERCENT)',2X,'(PERCENT/MIN)'
      ,/(49X,F6.0,F10.2,F16.7))
C
     9 FORMAT(20X,I2,5F10.2,I6,3F10.2)
    12 FORMAT(//47X,'** TABLE FOR THE DETERMINATION OF M **'/
      ,50X,'TIME',6X,'STRAIN',5X,'STRAIN-RATE',/50X,'(MIN)',
      ,4X,'(PERCENT)',2X,'(PERCENT/MIN)'/(49X,F6.0,F10.2,F16.7))
C
    14 FORMAT(//45X,'** TABLE FOR THE DETERMINATION OF ALPHA **'
      ,//15X,'STRESS-LEVEL',6X,F6.0,4(10X,F6.0)/)
    16 FORMAT(19X,F4.2,2X,5F16.7)
C
C      TIME=TIME AT WHICH READING IS REQUIRED
       READ(1,4) (TIME(J),J=1,19)
C
C      KATANL=SOIL SPECIMEN NO.
C      N= NO. OF READINGS
C      M= TOTAL NO. OF SPECIMENS
C      NS= NO. OF STRESS LEVELS
C
       READ(1,2) N,M,NS
       DO 87 KATANL=1,M
       WRITE(3,1) KATANL
C      STLVEL= STRESS LEVEL
C      DD= CORRECTED ZERO READINGS
C
       DO 86 JK=1,NS
       READ(1,5) STLVEL(JK),DD
       WRITE(3,6) STLVEL(JK)
C      XX= TIME OF READINGS IN MINS.
C      Y= COMPRESSION READING
C
       READ(1,3) (XX(I),Y(I),I=1,N)
       DO 29 I=1,N
    29 X(I)=ALOG10(XX(I))
       IN=0
       DO 100 II=1,19
       SUMX=0
       SUMY=0
       SUMXY=0
       SUMXSQ=0
       DO 1? I=1,N
       DIFF(I)=XX(I)-TIME(II)
```

```
      IF(DIFF(I)) 10,21,33
   10 CONTINUE
   33 IF(ABS(DIFF(I)) .LT. ABS(DIFF(I-1))) GO TO 21
      NN I-1
      GO TO 22
   21 NN=I
   22 DO 20 I=1,NN
      XY(I)=X(I)*Y(I)
      XSQ(I)=X(I)*X(I)
      SUMX=SUMX+X(I)
      SUMY=SUMY+Y(I)
      SUMXY=SUMXY+XY(I)
   20 SUMXSQ=SUMXSQ+XSQ(I)
      CA L EQUATN(NN,SUMX,SUMXSQ,SUMY,SUMXY,RATIO0,RATIO1)
      SMRSDL=0
      DO 30 I=1,NN
      YY(I)= RATIO0+RATIO1*X(I)
      RSDL(I)=(YY(I)-Y(I))*(YY(I)-Y(I))
      WRITE(3,9)I,XX(I),Y(I),XY(I),XSQ(I),I,XX(I),YY(I),RSDL(I)
   30 SMRSDL=SMRSDL+RSDL(I)
C
C     RESULT= READING AT PARTICULAR TIME
      RESULT= RATIO0+RATIO1*ALOG10(TIME(II))
      WRITE(3,7) RATIO0,RATIO1,SMRSDL,TIME(II),RESULT
C
      CHART=(RESULT-DC)*100.*.0001
      IM= IM+1
      CHART1(IM)=CHART
      CHART2(JK,IM)=CHART1(IM)/TIME(IM)
  100 CONTINUE
      WRITE(3,8) STLVEL(JK),(TIME(I),CHART1(I),CHART2(JK,I),
     *I=1,5)
      WRITE(3,12)(TIME(I),CHART1(I),CHART2(JK,I),I=6,IM)
   86 CONTINUE
      WRITE(3,14) (TIME(I),I=1,5)
      DO 91 J1=1,NS
   91 WRITE(3,16) STLVEL(J1),(CHART2(J1,I),I=1,5)
   87 CONTINUE
      STOP
      END
      SUBROUTINE EQUATN(N,B,C,D,E,RATIO0,RATIO1)
      A=FLOAT(N)
      FACTOR=B/A
      A=A *FACTOR
      B=B *FACTOR
      D=D *FACTOR
      RATIO1=(E-D)/(C-B)
      RATIO0=(D-B*RATIO1)/A
      RETURN
      END
/*
// EXEC LNKEDT
// EXEC
/*
/&
* $$ /*
* $$ EOJ
BM801   TOTAL STMTS IN JOB -        125
BM811   UDJMAINT STATISTICS   SYS004=000127, SYS005=000000, SYSLST=001..3
```

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Some Abbreviations:

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ICSMFE: International Conference on Soil Mechanics and Foundation Engineering.

JGD : Journal of the Geotechnical Division.

JSMFD : Journal of the Soil Mechanics and Foundation Division.

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