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STAND-UP TIME OF TUNNELS IN SQUEEZING GROUND

Part II: A General Constitutive Relationship

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16. Abstract

The effect of tunnel size, advance rate, and depth of cover on the stand-up time of tunnels in squeezing ground was investigated through a series of 12 physical model tests. The stand-up time, defined as the time elapsed before instability develops, was found to be characterized by increasing deformations and deformation rates rather than a catastrophic collapse of the tunnel.

Test results showed a 25% increase in stand-up time was realized by halving the size of the opening (from 5.0 m dia. to 2.4 m dia. when scaled to prototype dimensions) or by decreasing the advance rate by a factor of four (from 1.3 m/hr to 0.3 m/hr for the 5.0 m dia. tunnel). Depth of cover was described in terms of the ratio of confining pressure to material strength. Decreasing the depth (or increasing material strength) by 10% also increased stand-up time by 25%.

In order to establish a predictive capability, a constitutive theory describing the time dependent behavior of soft clays has been developed. By generalizing existing empirical rules developed for fixed boundary conditions and then unifying these empirical rules with a tensor framework, a multi-axial constitutive equation describing the stress-strain-time behavior of normally loaded soft clays was formulated.

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PART II

A GENERAL CONSTITUTIVE RELATIONSHIP FOR SOFT SOILS

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J. K. Mitchell

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LIST OF SYMBOLS

A	Hypothetical strain rate at t_1 and $\overline{D} = 0$ in Singh-Mitchell creep equation
a	Subscript meaning "axial"
B	Skempton's pore pressure parameter, equal to $\frac{\Delta u}{\Delta \sigma_c}$ in triaxial compression
CIU	"Consolidated Isotropic Undrained"
C_c	Virgin compression ratio
c_v	Coefficient of compressibility
C_α	Coefficient of secondary compression
D	Subscript meaning "deviatoric"
D	Deviator stress, $(\sigma_1 - \sigma_3)$
$\overline{D}$	Deviator stress level, $(\sigma_1 - \sigma_3)/(\sigma_1 - \sigma_3)_f$
d	Subscript meaning "delayed"
E_i	Initial tangent modulus (Duncan & Chang, 1970)
e	Void ratio
f	Subscript meaning "failure"
G	Shear modulus from elasticity theory
$\hat{G}$	Deviatoric strain operator
i	Subscript meaning "immediate"
J	Creep operator
K	Stress ratio, σ_3/σ_1
K	Bulk modulus from elasticity theory
$\hat{K}$	Volumetric strain operator
ℓ	Intercept for Kondner's hyperbolic model
m	Slope of plot of log $\dot{\varepsilon}$ vs. log t for Singh-Mitchell creep equation

n	Slope for Kondner's hyperbolic model
o	Subscript meaning "in-situ" or "initial"
OCT	Subscript meaning "octahedral"
t	Time
t_p	Time at the end of primary compression
t_{100}	Time for 100% consolidation to occur
u	Pore pressure
V	Subscript meaning volumetric
w	Water content
$\overline{\alpha}$	Slope of linear portion of log $\dot{\varepsilon}$ vs. $\overline{\Delta}$ plot for Singh-Mitchell creep equation
ε	Strain
$\dot{\varepsilon}$	Strain rate
$\underset{\sim}{\varepsilon}$	Strain tensor
σ	Stress (with two subscripts, i.e. σ_{ii}) or normal stress
$\underset{\sim}{\sigma}$	Stress tensor
τ	Shear stress
'	Superscript meaning "effective stress"
$\overline{}$	Superscript meaning "with measurement of pore pressures" (as in $\overline{\text{CIU}}$)

GENERAL CONSTITUTIVE RELATIONSHIP FOR SOFT SOILS

INTRODUCTION

Most existing constitutive models for predicting the strength and deformation behavior of soft (or squeezing) ground consider hydro-dynamic lag as the sole time-dependent effect. No consideration is given to time-dependent deformation characteristics of the soil skeleton. Thus, when investigating a problem such as the stand-up time of tunnels in squeezing ground, which by its very definition involves time dependency, it should come as no surprise that these existing models are inadequate. In studies performed during the second year of research under this contract, a general constitutive theory for predicting the stress-strain-time behavior of soft clays was developed. A laboratory test program pointed towards evaluation of the theory has been initiated. This experimental work and the accompanying analysis should be completed by the end of the third contract year.

CONSTITUTIVE THEORY

Introduction

In the 50 years since Terzaghi inaugurated the modern study of soil mechanics, a great deal of information on the time-dependent behavior of soft clays has accumulated. Most of this data relates to certain special boundary conditions common in soil mechanics: one-dimensional compression, plane strain, triaxial compression. No general constitutive model for predicting the time-dependent strength and deformation of soft ground is currently available. By generalizing the existing information pertaining to fixed boundary conditions, and by unifying this generalized information within the framework of tensor notation, a general theory for the stress-strain-time behavior of soft clays under multi-axial states of stress has been developed.

Tensor Notation

The stress state at a point within a continuum can be described by a symmetric, second order, Cartesian tensor. The principal stress tensor, $\underset{\sim}{\sigma}$, can be separated into one volumetric and three deviatoric components (Fig. 1). The volumetric component; $\underset{\sim}{\sigma}_V$, represents the mean (sometimes called hydrostatic) stress in the soil element. The volumetric stress is also referred to as the octahedral normal stress. The three deviatoric components, representing the stress difference acting across each of the three principal planes, can be combined into a single deviatoric tensor, $\underset{\sim}{\sigma}_D$.

Each stress tensor component has a corresponding strain tensor component, which together form a strain tensor. For a homogeneous, isotropic material, the strain tensor is also a second order symmetric tensor. Strain tensor components are related to stress tensor components by strain operators. Strain increments are computed by operating on stress increment tensor components.

$$\Delta\underset{\sim}{\varepsilon} = \Delta\underset{\sim}{\varepsilon}_V + \Delta\underset{\sim}{\varepsilon}_D = \frac{1}{\hat{K}} \Delta\underset{\sim}{\sigma}_V + \frac{1}{\hat{G}} \Delta\underset{\sim}{\sigma}_D \tag{1a}$$

For a linear elastic material, these strain operators reduce to multiplicative constants, K and G, often referred to as the bulk and shear modulus:

$$\Delta\underset{\sim}{\varepsilon}_E = \frac{1}{K} \Delta\underset{\sim}{\sigma}_V + \frac{1}{G} \Delta\underset{\sim}{\sigma}_D \tag{1b}$$

where

$\Delta\underset{\sim}{\varepsilon}$ = strain increment tensor

$\Delta\underset{\sim}{\varepsilon}_V$ = volumetric component of strain increment tensor

$\Delta\underset{\sim}{\varepsilon}_D$ = deviatoric increment of strain increment tensor

$\hat{K}$ = volumetric strain operator

$\hat{G}$ = deviatoric strain operator

$\Delta\underset{\sim}{\varepsilon}_E$ = a linearly elastic strain increment.

$$\begin{bmatrix} \sigma_x & \tau_{xy} & \tau_{xz} \\ \tau_{yx} & \sigma_y & \tau_{yz} \\ \tau_{zx} & \tau_{zy} & \sigma_z \end{bmatrix} = \begin{bmatrix} \sigma_{11} & \sigma_{12} & \sigma_{13} \\ \sigma_{21} & \sigma_{22} & \sigma_{23} \\ \sigma_{31} & \sigma_{32} & \sigma_{33} \end{bmatrix}$$

STRESS TENSOR

$$\begin{bmatrix} \sigma_1 & 0 & 0 \\ 0 & \sigma_2 & 0 \\ 0 & 0 & \sigma_3 \end{bmatrix} = \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3} \begin{bmatrix} 1 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & 1 \end{bmatrix}$$

PRINCIPAL STRESS TENSOR $\underset{\sim}{\sigma}$

VOLUMETRIC COMPONENT $\underset{\sim}{\sigma}_v$

DEVIATORIC COMPONENTS $\underset{\sim}{\sigma}_D$

$$\left\{ \begin{array}{l} + \dfrac{(\sigma_1 - \sigma_3)}{3} \begin{bmatrix} 1 & 0 & 0 \\ 0 & -1 & 0 \\ 0 & 0 & 0 \end{bmatrix} \\ + \dfrac{(\sigma_2 - \sigma_3)}{3} \begin{bmatrix} 0 & 0 & 0 \\ 0 & 1 & 0 \\ 0 & 0 & -1 \end{bmatrix} \\ + \dfrac{(\sigma_3 - \sigma_1)}{3} \begin{bmatrix} -1 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & 1 \end{bmatrix} \end{array} \right.$$

$$\underset{\sim}{\sigma} = \underset{\sim}{\sigma}_V + \begin{bmatrix} \dfrac{2(\sigma_1 - \sigma_2 - \sigma_3)}{3} & 0 & 0 \\ 0 & \dfrac{(2\sigma_2 - \sigma_3 - \sigma_1)}{3} & 0 \\ 0 & 0 & \dfrac{(2\sigma_3 - \sigma_1 - \sigma_2)}{3} \end{bmatrix}$$

FIG. 1 STRESS STATE AT A POINT

When considering time dependent stress-strain relationships, one must consider visco-elastic material properties. In the theory of visco-elasticity, the strain operators, $\hat{G}$ and $\hat{K}$, are replaced by creep functions, J_D and J_V. A linear volumetric creep function, J_V, is related to the volumetric strain operator, $\hat{K}$, by the equations:

$$\underset{\sim}{\varepsilon}_V(t) = J_V(t)\underset{\sim}{\sigma}_{VO} + \int_o^t J_V\,(t - \tau)d\underset{\sim}{\sigma}_V(\tau) \qquad (2a)$$

$$\underset{\sim}{\varepsilon}_V(t) = [\hat{K}(t)]^{-1}\underset{\sim}{\sigma}_V \qquad (2b)$$

For a constant stress, $\tilde{\sigma}_{VO}$, these equations reduce to:

$$\underset{\sim}{\varepsilon}_V(t) = J_V(t)\underset{\sim}{\sigma}_{VO} = [\hat{K}(t)]^{-1}\underset{\sim}{\sigma}_{VO} \qquad (2c)$$

A similar set of equations describe the relationship between the deviatoric operators J_D and $\hat{G}$.

General Formulation of Constitutive Relationship

Soil is neither linear, elastic, nor isotropic. We can account for non-linearity and inelasticity by appropriate rules for the superposition of load (and unload) increments. The assumption of isotropy is necessary to make the tensor framework mathematically tractable.

The general equations describing the relationship between stress and strain within the tensor framework for a non-linear inelastic system are:

$$\underset{\sim}{\varepsilon} = \underset{\sim}{\varepsilon}_V + \underset{\sim}{\varepsilon}_D \qquad (3a)$$

$$\underset{\sim}{\varepsilon}_V = \hat{K}^{-1}\underset{\sim}{\sigma}_V \qquad (3b)$$

$$\underset{\sim}{\varepsilon}_D = \hat{G}^{-1}\underset{\sim}{\sigma}_D \qquad (3c)$$

where $\hat{K}$ and $\hat{G}$ may be functions of time, stress history, density (void ratio), and stress level.

In the following sections, models for both the volumetric and deviatoric strain operators, $\hat{K}$ and $\hat{G}$, are developed for normally loaded saturated clays of low sensitivity. The concept of immediate and delayed components of deformation, described by Bjerrum (1967), is applied in the development of both models. Bjerrum assumed that one dimensional deformation of the soil skeleton consisted of a time independent contribution which takes place immediately upon application of an effective stress, followed by a time dependent, or delayed contribution. The general equation describing this concept can be written:

$$\underset{\sim}{\varepsilon}_V = \underset{\sim}{\varepsilon}_{V_i} + \underset{\sim}{\varepsilon}_{V_d} \tag{4a}$$

$$\underset{\sim}{\varepsilon}_D = \underset{\sim}{\varepsilon}_{D_i} + \underset{\sim}{\varepsilon}_{D_d} \tag{4b}$$

where the subscript i designates the immediate component of the deformation and the subscript d denotes the delayed component.

Empirical models have been developed for each of the four components of equations (4a) and (4b). In each case, an existing model of soil behavior was adopted based on criteria of compatibility, both with the other model components and the tensor framework, and a good fit with real soil behavior.

Volumetric Model

The volume change behavior of cohesive soils under one-dimensional conditions is probably the most extensively investigated problems in geotechnical engineering. Terzaghi's work on this subject inaugurated the modern era in soil mechanics more than fifty years ago. The Seventh Rankine Lecture, delivered by Bjerrum in 1967 is a definitive work on the subject. Information on the volumetric deformations under stress states other than one-dimensional is more limited. There is, however, enough information extant from which to formulate a general model for the volumetric tensor component. Thus, by generalizing Bjerrum's one-dimensional model to encompass all stress states, a unique relationship between void ratio, effective stress, and time can be hypothesized.

Bjerrum (1967) suggested that the deformation of soil is composed of immediate and delayed components. The immediate (or instant) component is that deformation which takes place immediately upon increase in the effective stress. If there were no hydro-dynamic lag volumetric compression would occur instantaneously upon load application, as shown by the dashed line in Fig. 2a. This is contrary to the standard assumption that the magnitude of deformation occurring during primary compression, defined as compression occurring prior to complete dissipation of excess pore pressures in a consolidation test, is a material constant. In Bjerrum's model, primary compression can contain both immediate and delayed contributions.

Due to the non-uniformity of excess pore pressures throughout a laboratory test specimen, portions of the sample near a drainage boundary may be well into the delayed deformation mode at the end of primary compression, while the center of the specimen is just completing its immediate compression phase. This implies that the magnitude of primary compression depends on the drainage boundary conditions; i.e., the time required for complete pore pressure dissipation. That it does has been illustrated by consolidation tests on samples of different heights (Berre and Iverson, 1972).

The elimination of hydro-dynamic lag in a fine grained soil is not possible without altering the properties of the soil. Therefore, immediate compression cannot be measured directly. Secondary compression, however, consists solely of delayed deformations. If laws governing the delayed volumetric compression of soils can be deduced from observations of secondary compression, a subject on which there exists a plethora of information, the immediate

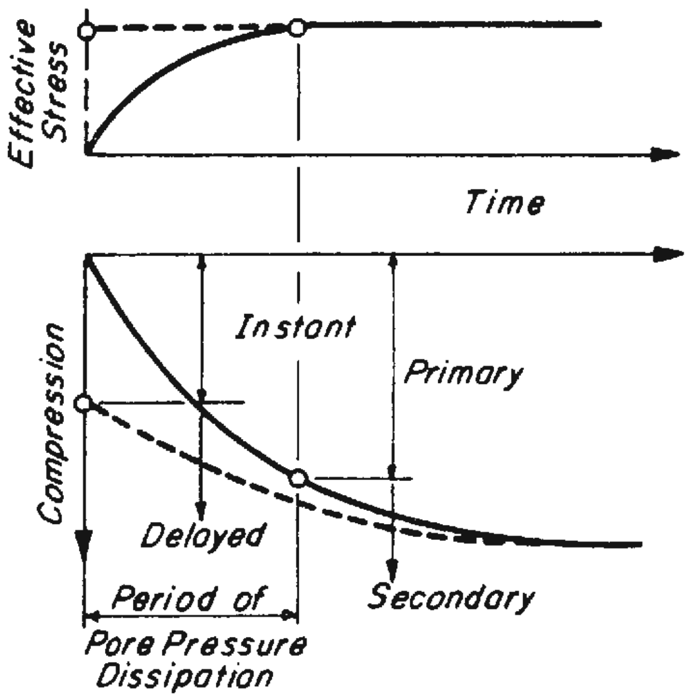

FIG. 2a INSTANT AND DELAYED COMPRESSION
(Bjerrum, 1967)

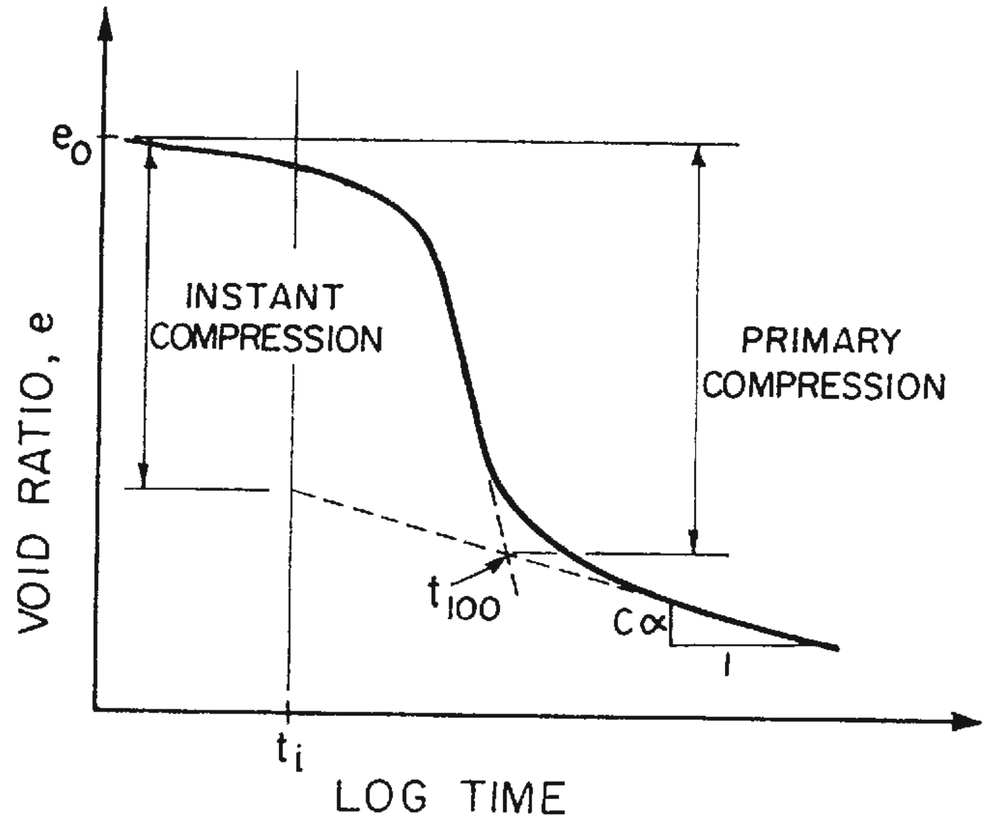

FIG. 2b INSTANT COMPRESSION BACKFIGURED FROM CONSOLIDATION TEST

deformation component can then be determined by subtracting the delayed contribution from the primary deformations.

The standard assumption made in engineering analysis concerning secondary compression is that the deformation is linear with the log of time (Buisman, 1936; Taylor, 1942), and actual behavior agrees well with this assumption. This is commonly expressed as:

$$\Delta e_S = C_\alpha \log(t/tp) \tag{5}$$

where

Δe_S = the change in void ratio due to secondary compression

C_α = the coefficient of secondary compression

tp = the time at which primary compression concluded

t = the time since the start of the load increment, $t > tp$

Detailed investigations by Ladd and Preston (1965) into the one-dimensional condolidation behavior of soft clays showed that for a clay with a constant virgin compression index (C_c), the usual case for a normally loaded clay of low sensitivity, C_α can be considered as constant regardless of the value of the vertical effective stress. In view of the problems associated with non-uniform pore pressure dissipation, these authors recommended evaluating C_α at a point one to two log cycles of time beyond tp. In studies on the secondary compression of silts and clays, Mesri and Godlewski (1977) concluded that the ratio of C_α/C_c was a material property, independent of the stress at which it is evaluated. This reduces to the assumption of C_α constant for a soil with a constant C_c. The implication of such an assumption is that C_α, the coefficient of secondary compression, is independent of bulk stress level (for normally loaded clays of low sensitivity).

Ladd and Preston also showed that for principal stress ratios between the hydrostatic case (isotropic triaxial consolidation) and the one-dimensional case, it is reasonable to assume a constant value for C_α. For principal stress ratios greater than those encountered in one-dimensional compression, they suggest C_α may increase significantly, though they present no data on this effect.

Walker (1969) studied volumetric creep (secondary compression) under triaxial and simple shear conditions for stress levels of less than 15% to greater than 85% of the peak shear stress. He found the volumetric creep rate (C_α) to be essentially independent of stress level. Walker did, however, find a difference between values of C_α evaluated from triaxial compression tests and those values evaluated from plane strain tests. In his studies of the creep settlement of foundations, de Ambrosis (1974) concluded that the effect of deviator stress level on volumetric creep was insignificant.

Based on these arguments, a model for the delayed volumetric compression of soft soils, consisting of a linear relationship between deformation and the log of time, which is independent of both bulk and deviator stress level would appear to be valid over a wide range of conditions encountered within normal engineering practice. Thus, equation (5) becomes:

$$\varepsilon_{Vd} = \varepsilon_{1d} + \varepsilon_{2d} + \varepsilon_{3d} = \frac{C_\alpha}{1 + e_o} \log(t/t_i) \qquad (6)$$

$$(t > t_i)$$

where

ε_{Vd} = the delayed volumetric strain

e_o = the initial void ratio ($\varepsilon_V = \Delta e/(1 + e_o)$)

ε_{id} = the delayed strain along principal strain axis i, i = 1, 3.

t_i = a reference time by which all "immediate" compression is expected to have occurred.

Once a relationship describing delayed compression has been developed immediate volumetric deformations can be back calculated from primary deformations (Fig. 2b).

For normally loaded clays of low sensitivity, it is generally assumed that primary deformations are described by a straight line on a plot of void ratio (or volumetric strain) versus the log of the vertical effective stress. If we assume that K_o, the coefficient of lateral earth pressure at rest, is independent of stress level, then the stress axis can be transformed from vertical effective stress to octahedral (or volumetric) effective stress without changing the slope of the primary deformation line.

If we assume constant values for both C_α (as shown in the previous section) and t_p (assumes, as did Terzaghi, that c_v, the coefficient of compressibility, is constant) then immediate deformations will fall along a line parallel to the primary line; i.e., a line with slope C_c. Furthermore, because C_α is assumed constant, the void ratios corresponding to a specified time of delayed compression would fall along a line (isochrone) parallel to the immediate (and primary) line. Using a log time law for C_α, isochrones representing equal intervals of log time would be equally spaced. This concept, first proposed by Ladd and Preston with respect to primary and secondary compression (Fig. 3a), was employed by Bjerrum in his general model for the one-dimensional consolidation behavior of clays (Fig. 3b). In this model, Bjerrum established rules for the superposition of load increments based on the concept of a quasi-preconsolidation pressure described by Leonards and Ramiah (1960). Garlanger (1972) successfully adopted Bjerrum's model to predict the results of the aforementioned oedometer tests of Berre and Iverson.

The one-dimensional case represents soil behavior at a particular deviator stress level $((\sigma_1 - \sigma_3)/(\sigma_1 - \sigma_3)_f) = \overline{D}$ or stress ratio $(\sigma_3'/\sigma_1' = K_o')$. To

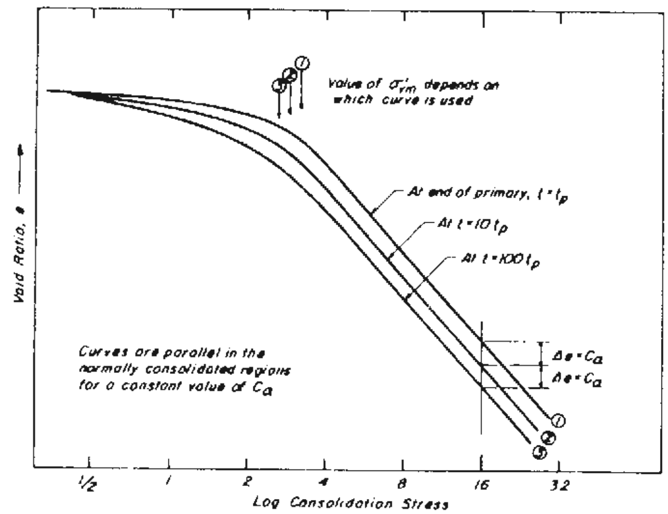

FIG. 3a EFFECT OF SECONDARY COMPRESSION ON THE LOCATION OF THE COMPRESSION CURVE

(Ladd and Preston, 1965)

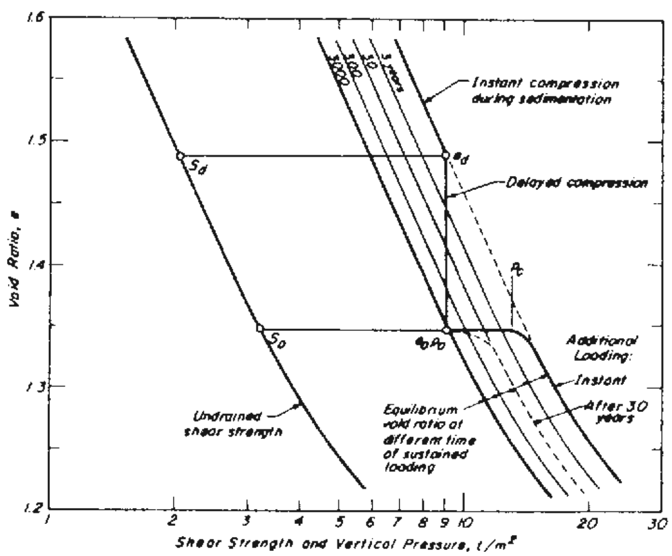

FIG. 3b BJERRUM'S (1967) MODEL FOR THE STRESS-STRAIN-TIME BEHAVIOR OF SOIL IN ONE-DIMENSIONAL COMPRESSION

establish a general model based on the one-dimensional model, we must establish rules for the effect of deviator stress level on soil behavior.

Rendulic proposed that if lines representing constant water contents (or void ratios) were plotted in a three dimensional stress space for the case of triaxial compression, they would create a series of concentric arcs (Fig. 4). Ladd (1971), restating this theory in terms of the stress path concept (Lambe, 1967), showed similar effective stress paths for saturated cohesive soils in triaxial compression (Fig. 5). By definition of similarity, the locus of points at a constant stress ratio (or deviator stress level) on a void ratio-log stress plot forms a line parallel to the locus of points for any other stress ratio. Taylor (1948) must have been thinking along similar lines when he proposed his void ratio-log stress plot with parallel contours for vertical effective stress, isotropic consolidation, and deviator stress at failure, the slope of these contours being C_c (Fig. 6).

Based on the above arguments, we can construct a plot of void ratio versus log octahedral stress with contours of constant deviator stress level (or stress ratio), each contour having the same slope, C_c (Fig. 7). This plot, along with our earlier conclusion that C_α was independent of deviator stress level, enables us to generalize Bjerrum's one-dimensional compression model to general, three-dimensional stress states. If we were to try to represent this model graphically in three dimensional void ratio-log stress-log time space, we would have a family of parallel planes, each plane representing a constant deviator stress level (Fig. 8).

Changes in volume of a soil element during consolidation can be determined on the basis of net inflow or outflow of water computed using Darcy's law, boundary pore water pressures, and soil permeabilities. Volume changes can be related to changes in octahedral effective stress on the basis of Fig. 8. Changes in pore water pressure are equal to changes in the octahedral effective stress.

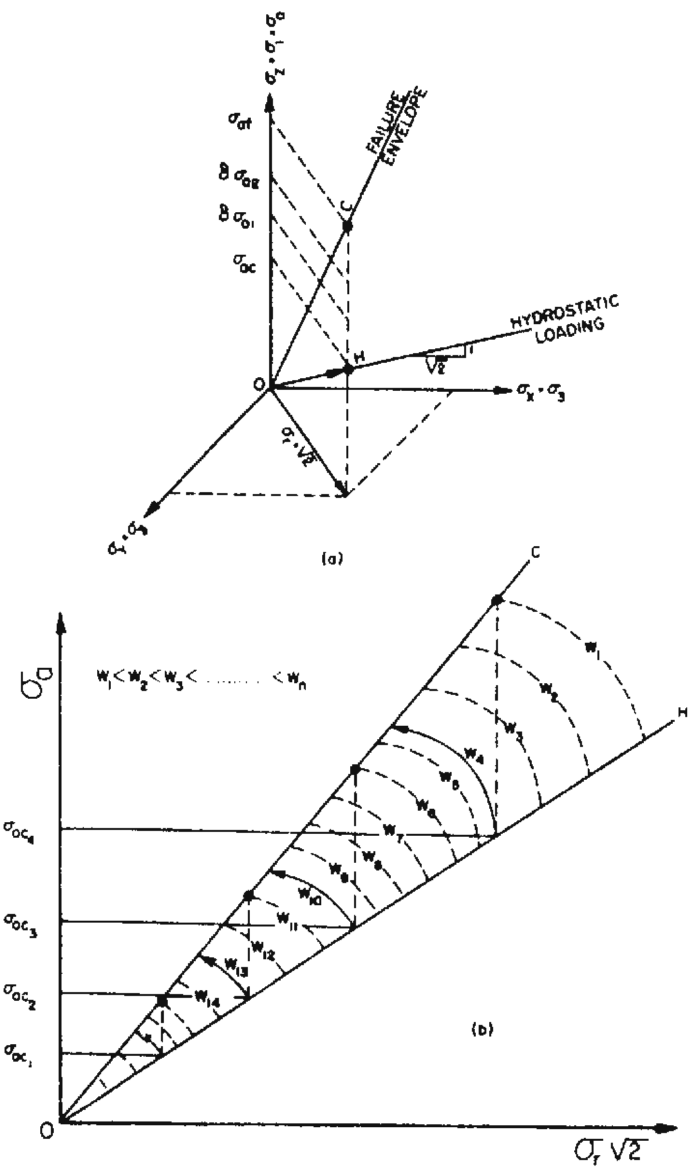

FIG. 4 RENDULIC DIAGRAM

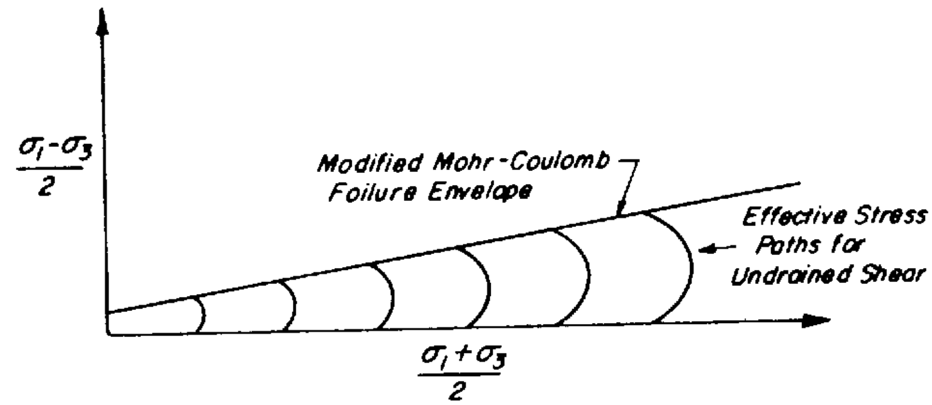

FIG. 5 LADD, 1971

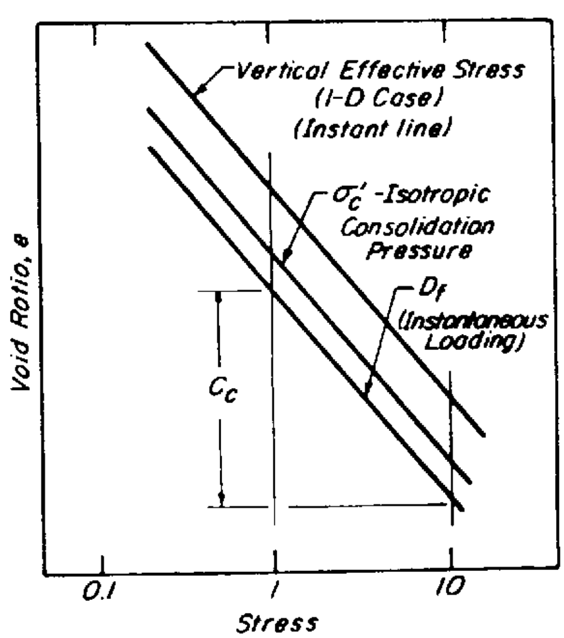

FIG. 6 TAYLOR, 1948

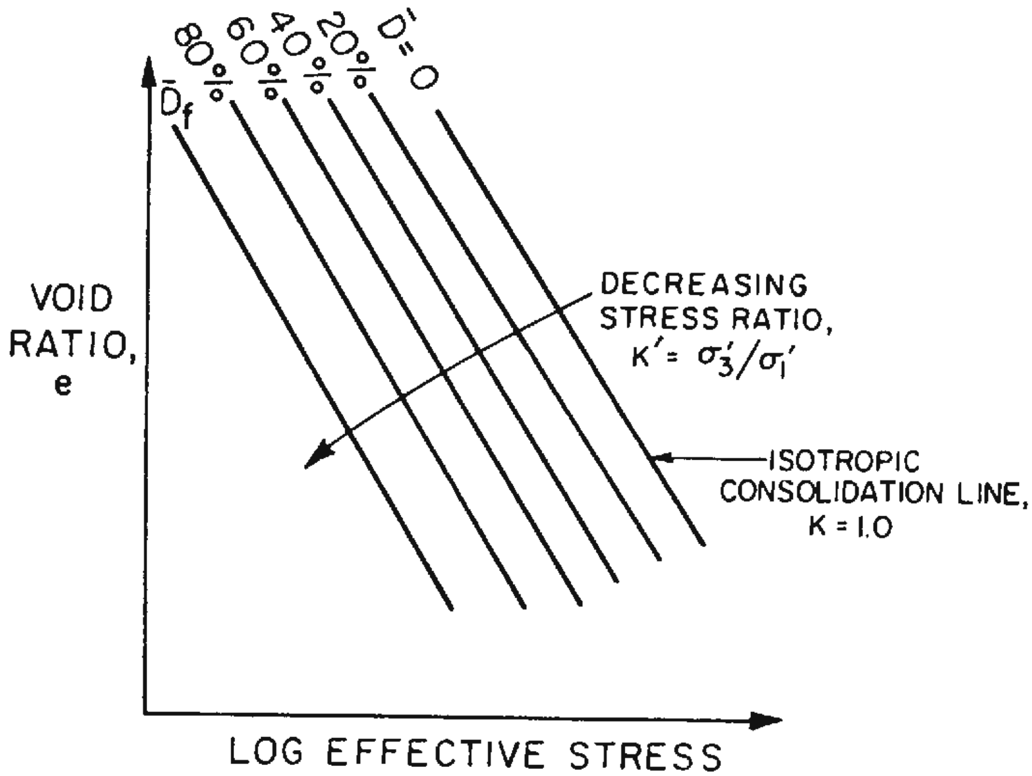

FIG. 7 VOID RATIO - EFFECTIVE STRESS RELATIONSHIPS FOR DIFFERENT PRINCIPAL STRESS RATIOS, ALL AT THE SAME TIME AFTER STRESS APPLICATION

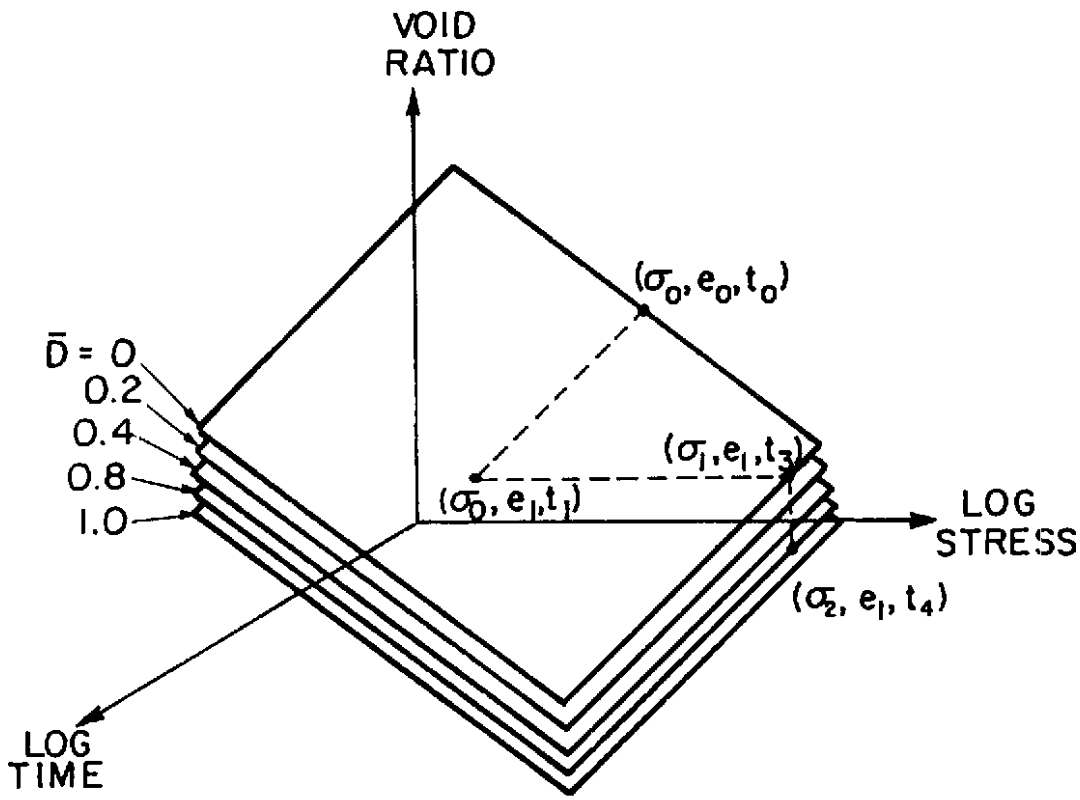

FIG. 8 GENERAL VOLUMETRIC MODEL

Deviatoric Model

The problem of hydro-dynamic lag doesn't enter into the determination of immediate and delayed components of the deviatoric deformations. Since pore water pressure is isotropic, it does not affect the value of the deviator stress. Thus, the deviator stress responds instantaneously to a change in load, and immediate deviatoric deformations can be determined by direct measurement.

The shearing resistance of a soil is logically a function of the density (void ratio), or state of the soil. Since the volumetric model expresses density as a function of bulk effective stress, the deviatoric strains must also be related to bulk effective stress and time.

The dependence of the immediate deviatoric deformation on the state of the material is often expressed as a stress-dependent modulus for undrained shear. Undrained shear tests on saturated soils involve zero volume change, implying pure deviatoric behavior. Duncan and Chang (1971) expressed the initial tangent modulus, E_i, as a function of the minor principal effective stress, σ_3', in their hyperbolic stress-strain formulation. Ladd and Foott (1974) suggest that the shear modulus of normally loaded clays, determined on the basis of undrained triaxial compression tests, can be normalized with respect to σ_c', the isotropic consolidation pressure (Fig. 9).

The isotropic consolidation pressure can be uniquely related to void ratio in the manner described by the volumetric model. For any given void ratio, an equivalent isotropic consolidation pressure can be determined from Fig. 7. Then, Ladd and Foott's normalization technique can be used in conjunction with the concept of a hyperbolic shape for the deviatoric stress-strain curve first proposed by Kondner (1963) to determine the immediate deviatoric deformations. Alternatively, any of the many existing deviatoric

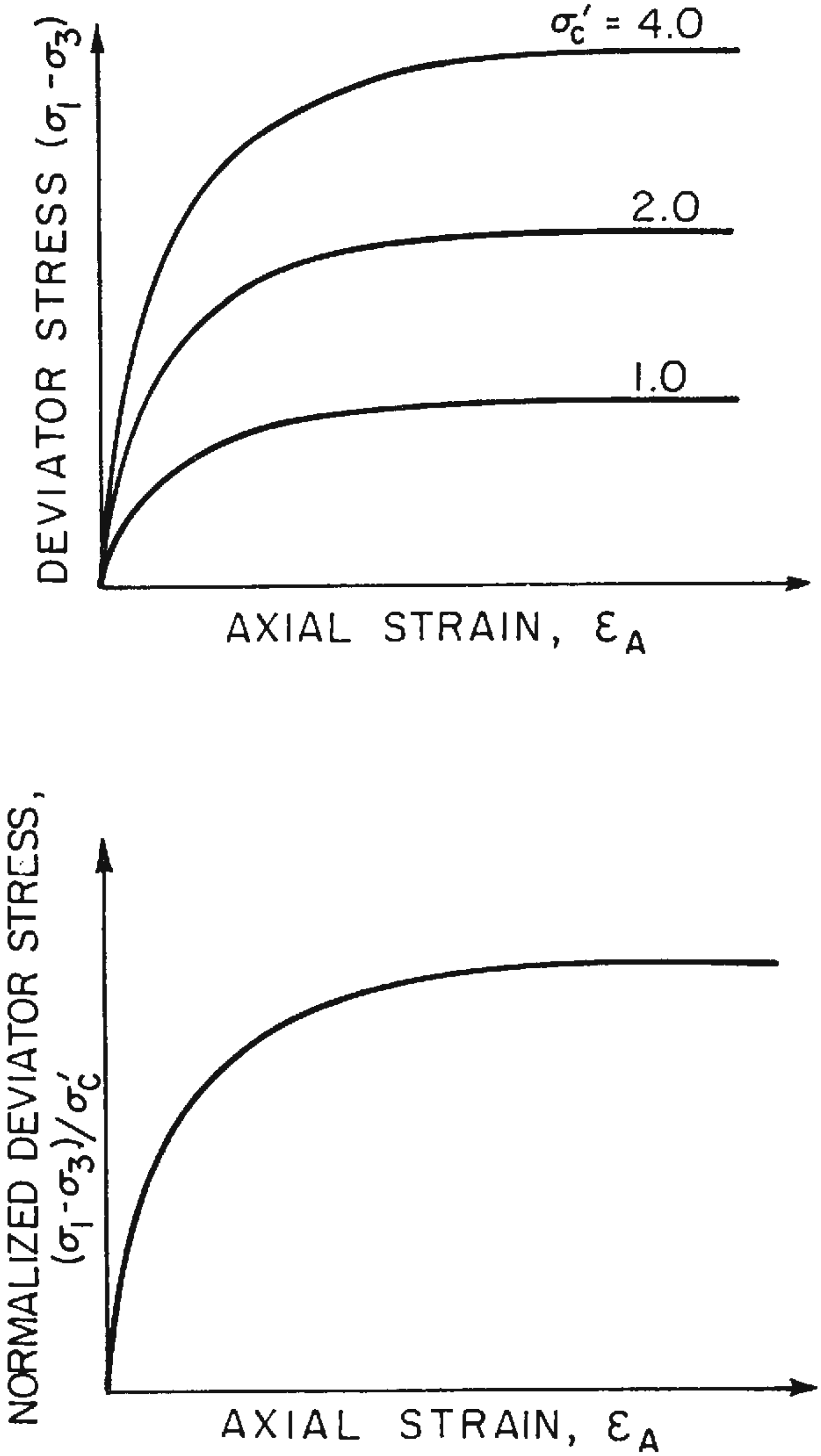

FIG. 9 NORMALIZED SOIL PROPERTIES

(Ladd and Foott, 1974)

stress-strain models which consider deformations to be time independent could be employed.

The normalized properties model is desirable because of compatibility with the volumetric model and simplicity. The existence of unique normalized curves for deviator stress and excess pore pressure versus axial strain is a sufficient condition for the existence of similar stress paths and parallel contours of constant deviator stress level on a void ratio-log stress plot. Furthermore, only two parameters, a slope and an intercept, are required to describe the hyperbolic stress-strain curve, as shown in Fig. 10.

Numerous models have been proposed for the deviatoric creep behavior of clay soils (Singh and Mitchell, 1968; Murayama and Shibata, 1966; Gibson and Lo, 1961; de Ambrosis, 1974). Using the previously established criteria of simplicity and compatibility, the Singh-Mitchell model was chosen as the delayed component of the deviatoric strain operator. Singh and Mitchell proposed a general model which describes the axial deformation of samples in triaxial compression subjected to a constant deviator stress. For undrained samples, the rate of 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 of an isotropically consolidated sample at the same void ratio (D_f).

$$\dot{\varepsilon}_A = A \exp[\bar{\alpha}D/D_f](t_1/t)^m \tag{7}$$

where

t = time under deviator stress D

t_1 = time at which A is defined (usually 1 minute)

$\dot{\varepsilon}_A$ = the axial strain rate at time t.

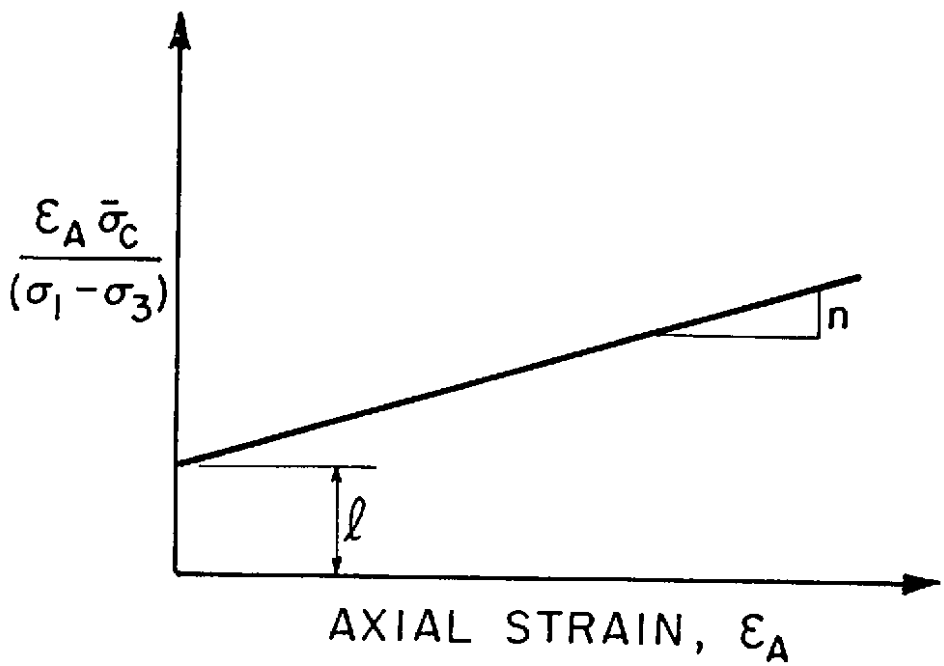

FIG. 10 KONDNER'S HYPERBOLIC MODEL

The stress parameters in equation (7)(D and D_f) are inherent to the other model components (the volumetric component and the immediate deviatoric component). Thus, the Singh-Mitchell equation is ideal for use in describing the delayed deviatoric deformation under a constant stress. Rules for superposition of load increments are necessary to use this relationship in the general model.

Though limited information is available on the effect of deviatoric stress superposition on deformation behavior (Paduana, 1966; Hirst, 1968; Singh, 1968), the indications are that cohesive soils behave much the same under changes in deviatoric stress as they do under changes in volumetric stress. Experiments by Hirst showed that after being subjected to a sustained deviatoric load, a soil specimen will exhibit a "quasi-resistance" to immediate shear deformation under an increased shearing stress (Fig. 11). Hirst described this behavior as a stiffening of the soil skeleton, or work hardening. Thus, the deviatoric equivalent of a quasi-preconsolidation pressure can be established to describe deformation behavior under superimposed deviator stresses. Referring to Fig. 11, if a soil element instantaneously loaded to stress σ_1 and left to creep until time t_1 corresponding to strain ε_1 were instantaneously loaded to stress σ_2, it would then creep at the same rate as a soil element instantaneously loaded to stress σ_2 and allowed to creep to strain ε_1 corresponding to time t_2.

DETERMINATION OF MODEL PARAMETERS FROM LABORATORY TESTS

Seven parameters and one graphical relationship are required to define the general stress-strain-time behavior of a soft clay as described by this model. The required parameters are: C_c, the virgin compression index; C_α,

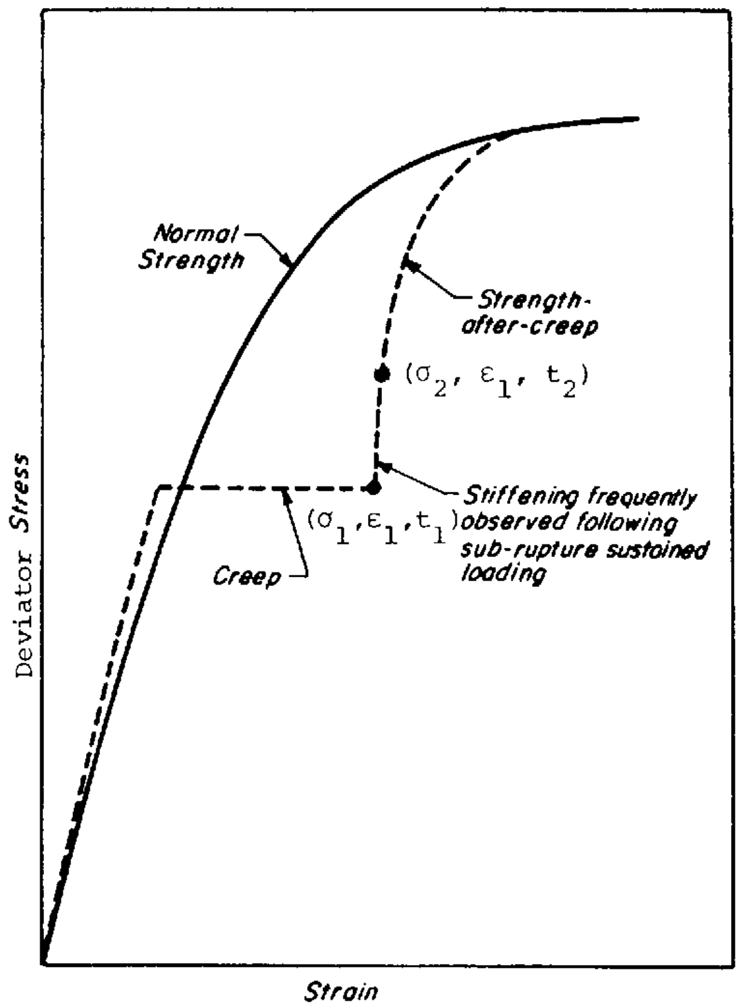

FIG. 11 EFFECT OF CREEP UNDER SUSTAINED DEVIATOR STRESS ON STRESS-STRAIN BEHAVIOR (AFTER HIRST, 1968)

the coefficient of secondary compression; ℓ and n, the two parameters describing the normalized hyperbolic stress-strain curve; and A, $\bar{\alpha}$, and m, the parameters for the Singh-Mitchell equation. There is no closed form manner with which to describe the void ratio-log stress relationship of Fig. 7 at the present time. Therefore, until such a closed form description is developed, this portion of the model must be described graphically.

At a minimum the parameters necessary to completely define the general model can be determined from three laboratory tests; one isotropically consolidated undrained triaxial compression test with measurement of pore pressures ($\overline{\text{CIU}}$), one anisotropically consolidated triaxial creep test, and one consolidation test (either one dimensional, isotropic, or anisotropic).

From the $\overline{\text{CIU}}$ test, the normalized immediate deviatoric stress strain curve (Fig. 9) and the location of the constant stress level contours on the void ratio-log stress plot (Fig. 7) can be obtained. The normalized curve can be described with two parameters (Fig. 10). From the consolidation test, the slope of the void ratio-log stress contours (C_c) Fig. 7, and the rate of delayed compression (C_α) can be determined.

Singh and Mitchell (1968) have shown how the three parameters required to describe the deviatoric creep behavior of soft clays can be determined from a single undrained creep test, although the use of two tests would be preferable.

Unfortunately, due to the natural variability of soil deposits, it is unreasonable to expect that the seven parameters and the relationship in Fig. 7 required to describe soil behavior according to the model can be determined from just three tests. The engineer must use his own discretion in determining the number of tests necessary to satisfactorily define average values for the required parameters.

LABORATORY TEST PROGRAM

Introduction

Experimental studies on remolded San Francisco Bay Mud have been initiated in order to ascertain the validity of the general theory. Remolded soil was chosen for the testing program because it conformed closely to the theoretical assumptions of isotropy, saturation, and low sensitivity. The initial series of tests will determine the parameters necessary to describe the four terms in equation (4) as defined by the general model. These parameters will then be used to predict the axial deformation, pore pressure, and volume change behavior of triaxial samples subjected to complex series of loads and drainage conditions. A second series of laboratory tests will then be performed to evaluate these predictions. At the present time, the tests describing three of the four terms in equation (4) are complete. Only the parameters for the Singh-Mitchell equation have yet to be evaluated. The following sections present the laboratory procedure and the results of tests defining the three above-mentioned model components.

Laboratory Procedure

Laboratory tests were performed on specimens of remolded San Francisco Bay mud. Bay mud was chosen because it was believed to be typical of many soft clays in terms of both composition and behavior. The soil was remolded to conform as closely as possible to the theoretical assumptions of isotropy.

The Bay Mud was obtained from five-inch diameter, thin walled, fixed piston samples taken from depths of three to five meters at Hamilton Air

Force Base near San Rafael, California. Atterberg limits of Hamilton Air Force Base Mud are 88% (± 1%) for the Liquid Limit and between 35% and 44% for the Plastic Limit. These values yield a plasticity index range of 43% to 54%. Air pycnometer determinations of specific gravity resulted in a mean value of 2.71 for the dry soil. The natural water content of the Bay mud was slightly above the Liquid Limit, varying from 90 to 97% for most samples.

The soil was remolded by placing undisturbed chunks of soil weighing approximately 1000 gm. into large, heavy duty, plastic bags. Carbon dioxide was blown into the bag to displace the air and thus facilitate the later saturation of the remolded soil under an applied back pressure. Enough de-aired water was added to raise the average water content of the remolded soil to 100%. The bag was sealed across the top with a sealing iron, taking care to leave a small opening in the corner through which a vacuum could be applied. The bag was evacuated and the soil was thoroughly remolded while maintaining the vacuum. Twenty-four hours later, just prior to re-constituting the test specimens, the vacuum was reapplied and the material was once again reworked.

Subsequent to the second remolding of the material, the Bay mud in the plastic bag was sampled with a thin walled double acting air piston, modified to act as a fixed piston sampler, and injected into a split mold set up on the pedestal of the triaxial cell. By cutting the head off, the double acting air piston could be used as a fixed piston sampler (Fig. 12). The outer diameter of the piston (1.375 m) was only slightly less than the inner diameter of the split mold (1.40 m). The mold was lined with a thin rubber membrane. Eight 1/4 in. strips of Whatman's No. 54 high wet strength filter paper were aligned symmetrically within the mold. The strips were tucked beneath the

FIG. 12 MODIFIED DOUBLE ACTING AIR PISTON, SPLIT MOLD, AND BAG OF REMOLDED CLAY

bottom stone (Fig. 13a), run up the side of the mold, folded down the outside of the mold, and held in place with a rubber band (Fig. 13b). A filter disk was placed atop the bottom porous stone. The mold was filled with de-aired water prior to injection of the remolded soil (Fig. 14) to ensure that as little air as possible was entrapped within the specimen prior to consolidation. The remolded soil was then injected into the mold.

After injection of the Bay Mud, the top of the specimen was levelled with a spatula, filter disk, porous stone, and top cap were placed atop the sample, filter strips were released from beneath the rubber band and trimmed to size, and the membrane was released from around the top of the mold and secured about the top cap with an O-ring. Once again, the space between the membrane and the stone and top cap was filled with de-aired water to displace as much air as possible from within the membrane.

After re-constitution, the specimen was left untouched for one day. At the end of the twenty-four hour period, a vacuum back pressure of 0.05 kg/m^2 was applied to the specimen through the base. After another twenty-four hours, the split mold was removed, the specimen was measured with a paper tape to determine initial volume, and the chamber was placed on the cell and filled with water. At this point, the sample preparation process was complete.

Subsequent to filling the cell chamber, each sample was subjected to a back pressure of 1.0 KSC. The cell pressure was increased to whatever pressure was necessary to establish the desired initial effective consolidation pressure. Measured values of the pore pressure parameter B (equal to $\Delta u/\Delta\sigma'_c$) generally fell between 0.98 and 1.0 for the first increment. If they did not, the back pressure was increased until a satisfactory B value (greater than 0.98) was achieved.

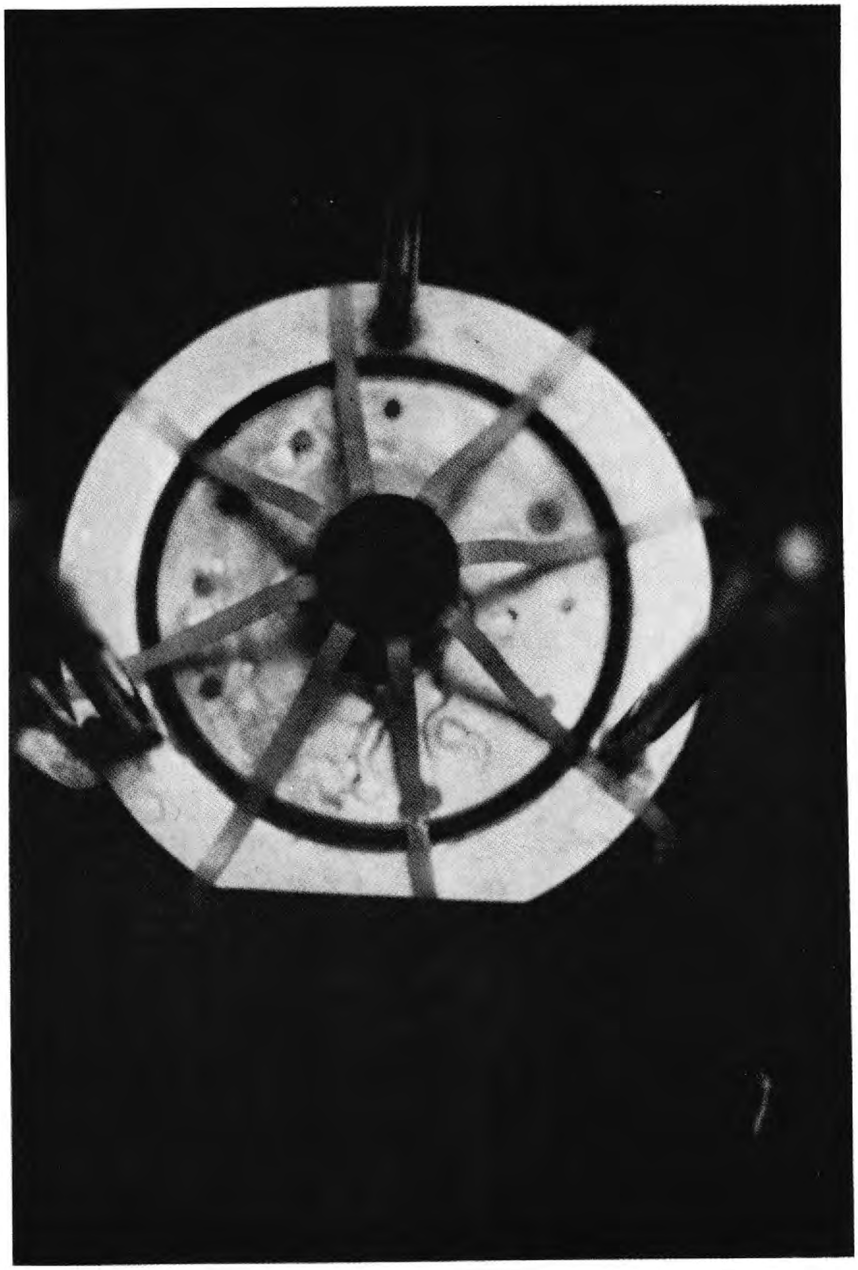

FIG. 13a STRIPS TUCKED BENEATH POROUS STONE

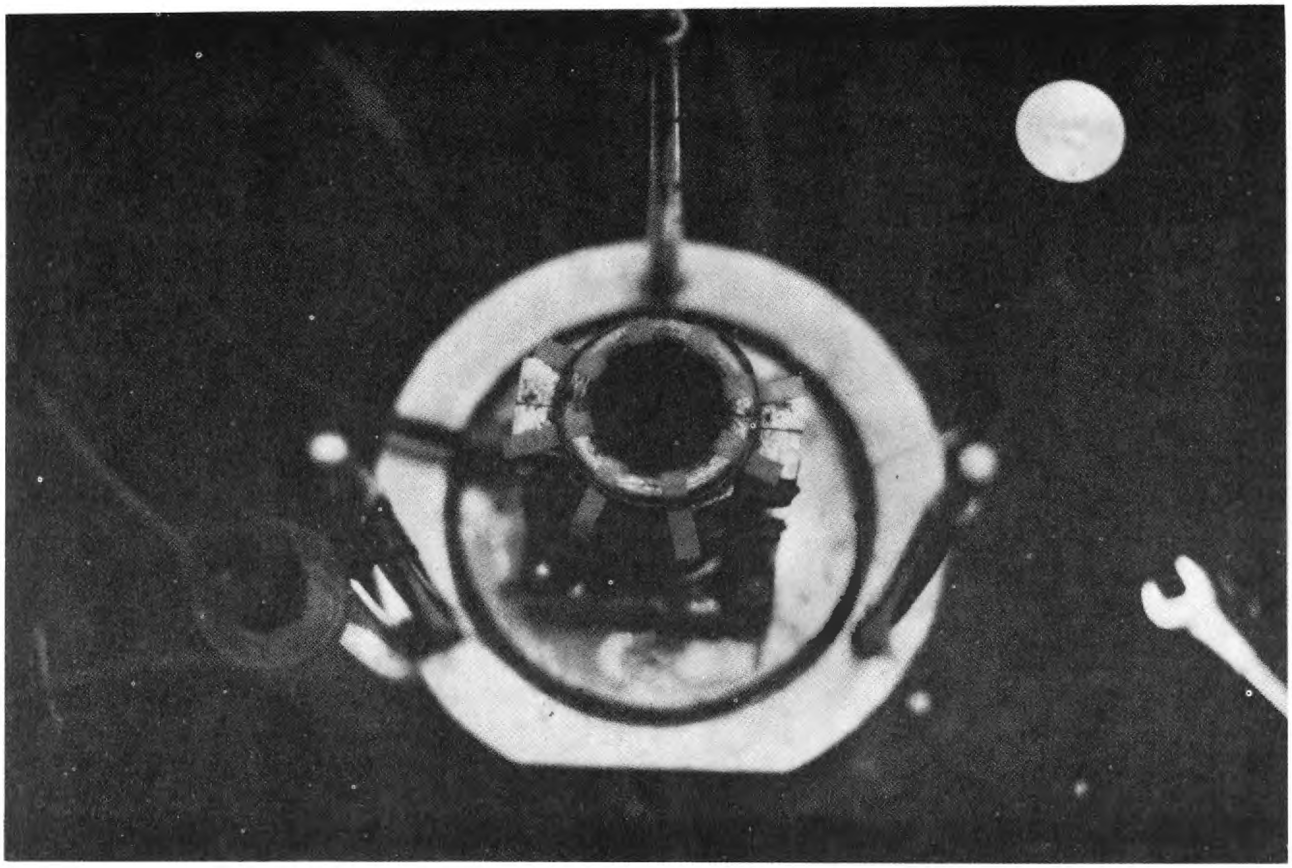

FIG. 13b STRIPS FOLDED DOWN OVER TOP OF SPLIT MOLD

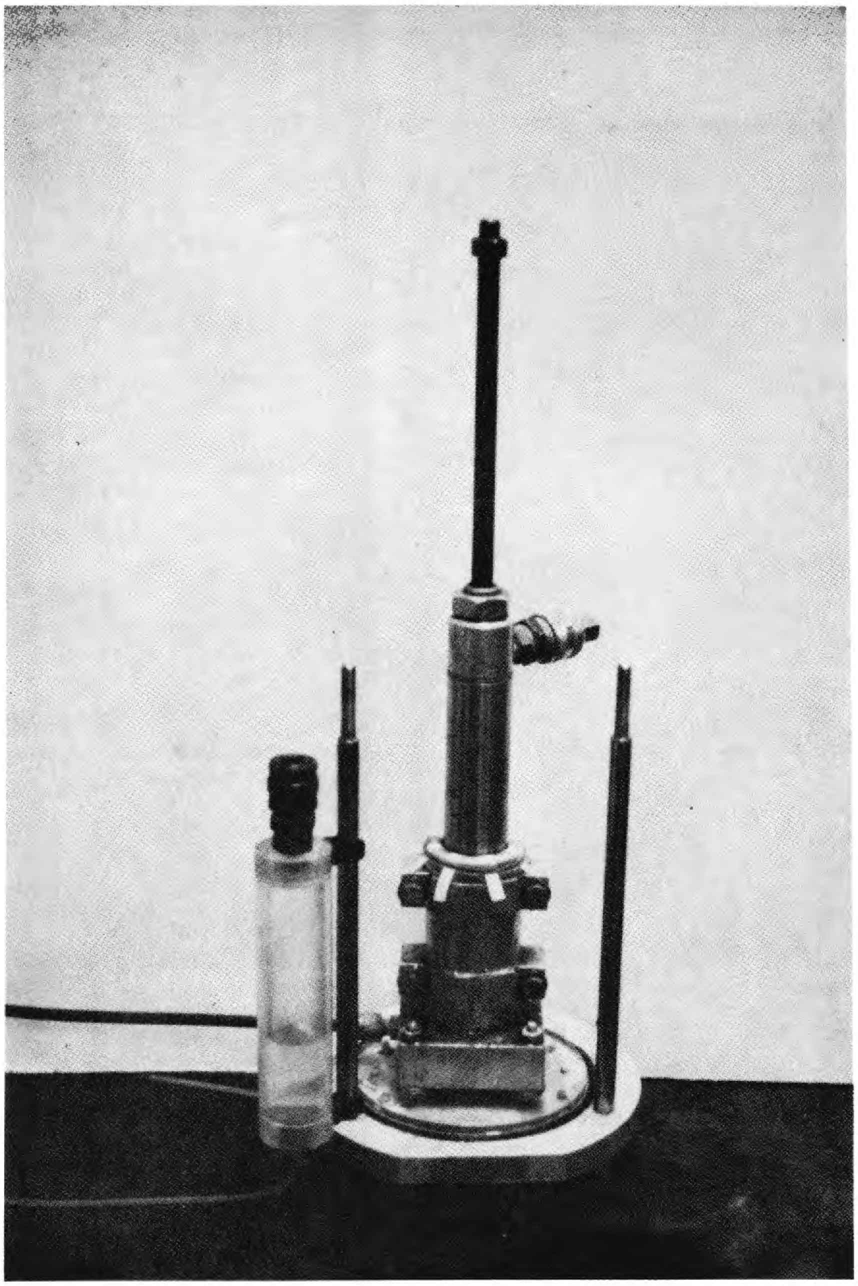

FIG. 14 PISTON INSERTED INTO MOLD FOR INJECTION OF REMOLDED CLAY

At the end of a test, each specimen was broken into three segments. Water contents were taken of each segment to determine total volume of solids in the specimen and specimen uniformity. Results of almost twenty tests have shown consistent uniformity between water contents for top, middle, and bottom sections of each specimen. This implies the sample preparation technique is producing uniform samples and that testing procedures are not creating zones of segregation of pore water.

Loading frame, triaxial cells, volume-change measuring devices, and pore pressure apparatus are all standard University of California equipment. Details of the apparatus can be found elsewhere (Chan and Duncan, 1966). Pore pressures and volume change were measured through the single drainage line from the pedestal on which the specimen stood. Back pressure was applied through the same line (Fig. 15). Cell pressure was applied with compressed air. The air water interface was located in a small reservoir attached to the bottom of the triaxial cell with a 1/8 in. O.D. pressure line.

<u>Test Results</u>

To date, test results include isotropic consolidation (IC) tests and isotropically consolidated, undrained triaxial compression loading tests with measurement of pore pressures ($\overline{\text{ICUL}}$). From the IC tests, the delayed volumetric parameter C_α and the slope of the immediate volumetric contours of constant deviatoric stress level, C_c, were determined. The $\overline{\text{ICUL}}$ tests determine the spacing of the immediate volumetric stress level contours (that is to say, the shape of the similar undrained effective stress paths) and the normalized soil properties.

Void ratios from the $\overline{\text{ICUL}}$ tests were plotted along with the results of the IC tests on a void ratio-log stress plot. A best fit straight line was

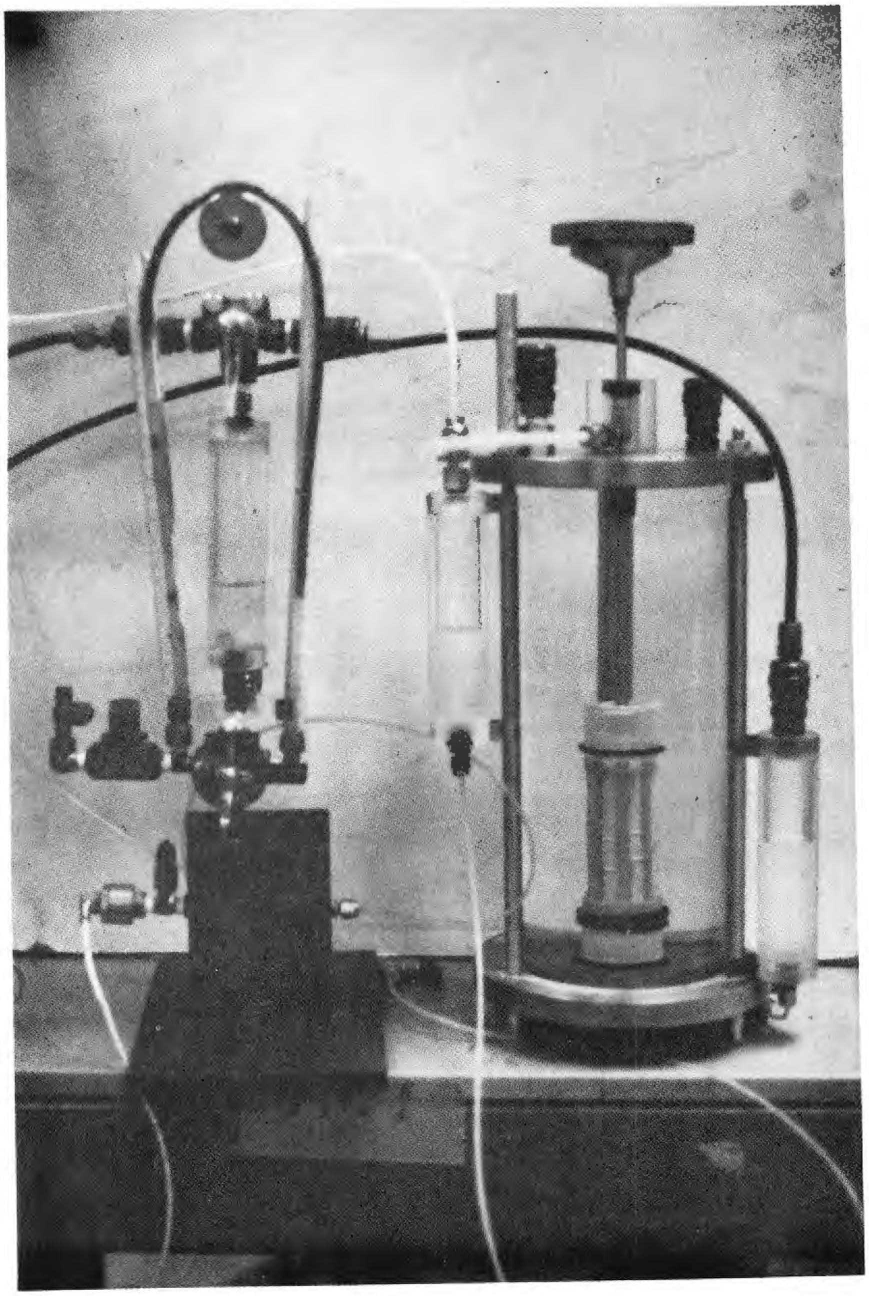

FIG. 15 PICTURE OF VOLUME CHANGE, BACK PRESSURE, CELL SET-UP

drawn among these points, the slope of the line being taken as the value of C_c, the virgin compression index. Results from the IC tests were used to determine the spacing of the delayed compression isochrones. Fig. 16 shows the resulting void ratio-log stress plot with delayed compression isochrones for the case of isotropic compression. The one minute and one year lines shown in this figure are extrapolated lines based on the measured value of C_α.

Fig. 17 presents the normalized plots of deviator stress and excess pore pressure versus axial strain developed from the $\overline{\text{ICUL}}$ tests. Note that in RBM-12, significant excess pore pressures were developed prior to the start of the shear test as a result of mishandling while placing the cell within the loading frame. However, test results still converged on the normalized curve by the end of the test.

The void ratio-log stress plot with contours of constant deviatoric stress level, describing the interrelationship between volumetric deformations and deviatoric stress level, is shown in Fig. 18. Note that the abcissa of this plot is $(\sigma_c' - \sigma_d)$ where σ_c' is the equivalent immediate isotropic consolidation pressure corresponding to the void ratio of the soil. From an analytical point of view, it would be simpler if the abcissa of this plot was octahedral effective stress. Unfortunately, the octahedral effective stress is not uniquely related to deviator stress for a constant void ratio, i.e. in an undrained triaxial compression test, σ'_{OCT} is not always a monotonic quantity--in some cases it may increase first and then decrease with increasing strain. Therefore, σ'_{OCT} cannot be used to describe a unique relationship.

The use of $(\sigma_c' - \sigma_d)$ as the abcissa in Fig. 18 is not the only way by which void ratio and stress can be uniquely related. Other possible abcissa's would be K x σ'_{OCT} and $\overline{D}$ x σ'_{OCT}. The final form in which this void ratio-log stress relationship will be presented has yet to be determined.

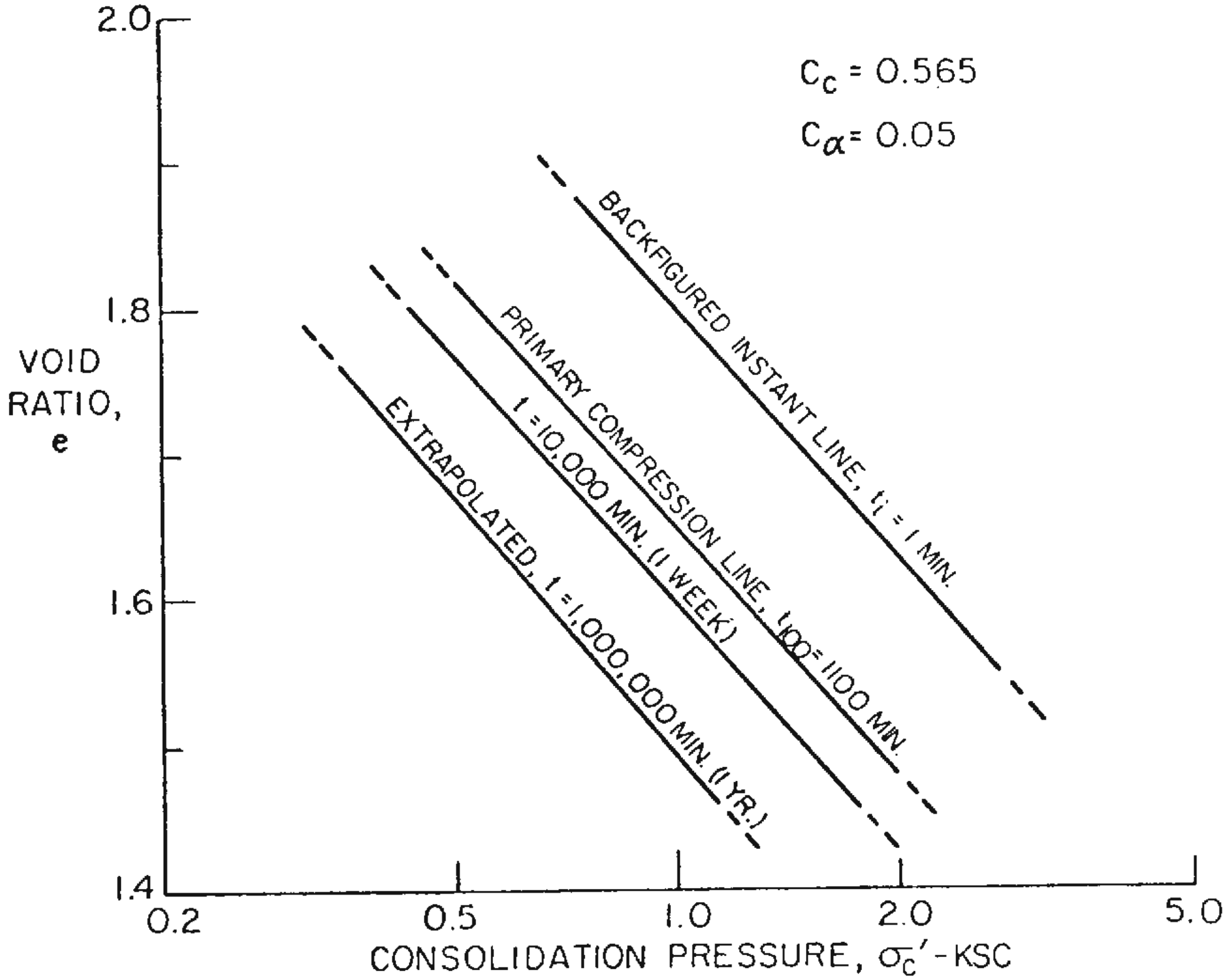

FIG. 16 COMPRESSION ISOCHRONES FOR REMOLDED SAN FRANCISCO BAY MUD - ISOTROPIC COMPRESSION TESTS

(a)

(b)

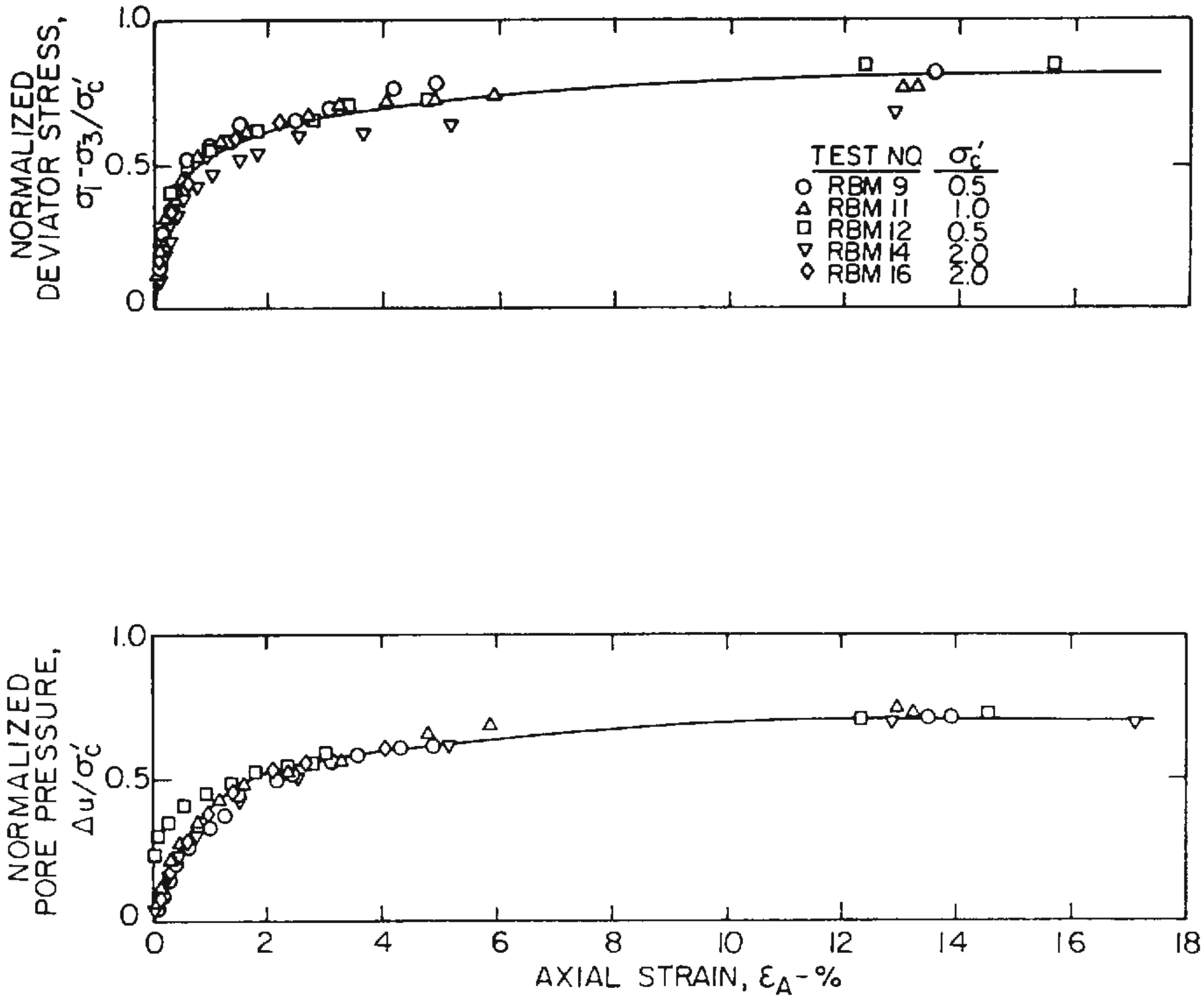

FIG. 17 NORMALIZED STRESS-STRAIN BEHAVIOR OF REMOLDED SAN FRANCISCO BAY MUD

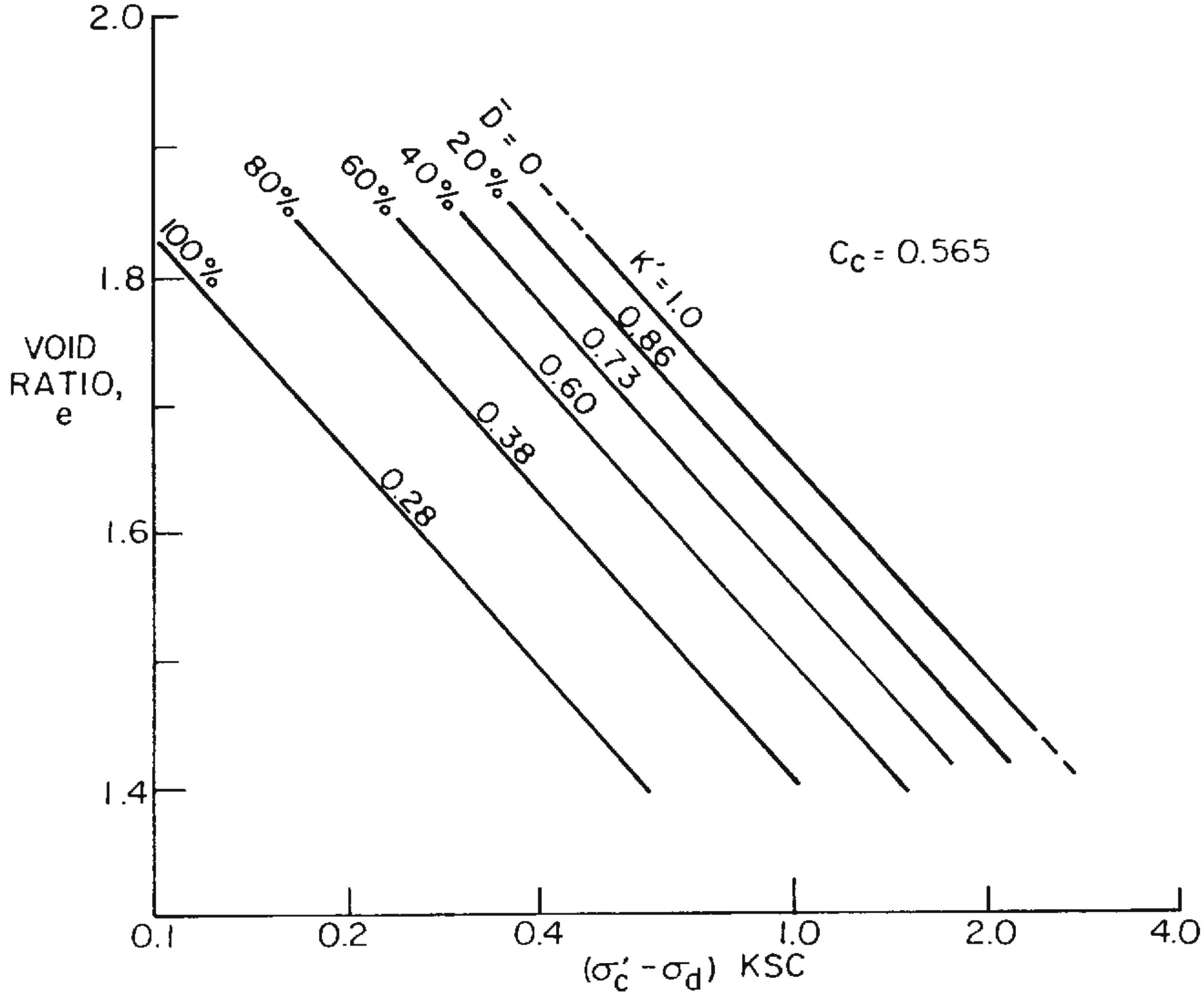

FIG. 18 EFFECT OF DEVIATOR STRESS ON IMMEDIATE VOLUMETRIC COMPRESSION OF REMOLDED SAN FRANCISCO BAY MUD

Future Tests

A series of undrained anisotropic creep tests must be performed to determine the parameters for the Singh-Mitchell equation. Once these values are established, all the parameters required to define the four components of the general model described by equation (4) will be known. These parameters will be input into a numerical model to predict the results of triaxial tests involving a complex series of loads and drainage conditions. Laboratory tests will be performed to evaluate these predictions and thus to ascertain the validity of the general model.

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