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TECHNICAL REPORT 90-02

NUMERICAL MODELING OF THE CREEP BEHAVIOR OF CLAYS WITH EMPHASIS ON TUNNELS AND UNDERGROUND OPENINGS

A Critical Review of the State-of-the-Art

FEBRUARY 1990

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MATHEMATICAL MODELS DEPARTMENT
BERGAMO, ITALY

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SUMMARY

This report presents an interpretive overview and critical assessment of the state-of-the-art for numerical modeling of the creep behavior of clays. The overview and assessment is focused upon application to underground openings. Field and laboratory observations of time-dependent behavior, constitutive modeling of creep behavior, and numerical implementation of constitutive equations are addressed. A critical assessment of the ability of existing models to predict aspects of creep behavior relevant to waste repository design and suggestions for improved analyses that can be developed with existing technology are provided.

Observations of creep behavior in the laboratory and in the field are reviewed to provide a background upon which to assess the adequacy of existing constitutive equations and numerical models for reproducing important facets of creep behavior in clays. The lack of data on creep under long term drained conditions is noted. Attention is also called to the lack of significant data on the creep of stiff clays and clayey rocks, on temperature effects, and on creep under drained conditions for other than one-dimensional stress-states.

Both heuristic and mathematical constitutive models are reviewed. Heuristic models provide a basis for evaluation of the required parameters for the continuum mechanics based mathematical models. The continuum mechanics models are required for numerical analysis. It has been demonstrated that, by using iterative and incremental analysis, virtually any viscous or inviscid continuum mechanics material model can be adapted to consider time-dependent behavior.

Available numerical models for numerical analysis of geotechnical problems involving creep deformations are reviewed. Models for thermo-mechanical coupling are also addressed in this review. Cases where creep-inclusive analyses have been applied to analysis of prototype behavior are cited. However, the lack of well documented case histories of time-dependent deformations over significant time spans is identified as a major obstacle to model verification.

Recommendations are made for an alternative design approach capable of guaranteeing the very long term mechanical integrity of the liner.

ZUSAMMENFASSUNG

Dieser Bericht behandelt in Form einer interpretierenden Übersicht und kritischen Beurteilung den Wissensstand über numerische Modelle für das Kriechverhalten von Tonen. Diese Übersicht konzentriert sich auf deren Verwendung für unterirdische Öffnungen. Es werden Feld- und Laborbeobachtungen von zeitabhängigem Verhalten, die konstitutive Modellierung von Kriechverhalten und die numerische Ausführung von konstitutiven Gleichungen behandelt. Es wird eine kritische Beurteilung bezüglich der Möglichkeiten gegeben, auf Grund von existierenden Modellen eine Voraussage über Aspekte des Kriechverhaltens zu geben, welche für die Konstruktion von Endlagern relevant sind, und es werden auch Vorschläge für eine verbesserte Analyse gemacht, die mit vorhandener Technologie entwickelt werden kann.

Beobachtungen von Kriechverhalten im Labor und im Feld werden diskutiert um einen Hintergrund zu vermitteln, auf Grund dessen die Zulänglichkeit existierender konstitutiver Gleichungen und numerischer Modelle zur Reproduktion von wichtigen Aspekten des Kriechverhaltens in Tonen beurteilt werden kann. Es wird auf den Mangel an Daten bezüglich Kriechverhaltens während längerer Entwässerung hingewiesen. Hingewiesen wird ausserdem auf den Mangel an aussagekräftigen Daten bezüglich des Kriechverhaltens von zähen Tonen und tonigen Gesteinen, bezüglich Temperatureinflüssen und bezüglich des Kriechverhaltens unter entwässerten Bedingungen unter Belastungszuständen, welche nicht eindimensional sind.

Es werden sowohl die heuristischen als auch die mathematischen Modelle behandelt. Heuristische Modelle vermitteln eine Basis zur Einschätzung der Parameter, welche für die mathematischen Modelle, die auf Kontinuumsmechanik basieren, benötigt werden. Die Kontinuumsmechanik-Modelle werden zur numerischen Analyse benötigt. Es hat sich gezeigt, dass unter Anwendung von iterativen und inkrementalen Analysen, praktisch jegliches viskoses bzw. unviskoses Kontinuumsmechanik-Modell angepasst werden kann, um zeitabhängiges Verhalten zu betrachten.

Die vorhandenen numerischen Modelle zur Analyse von geotechnischen Problemen, welche Kriechverformungen darstellen, werden diskutiert. Es wird auch auf Modelle der thermomechanischen Kopplung hingewiesen. Es werden Fälle angeführt, in denen Kriechverhalten einschliessende Analysen angewandt wurden, um das Verhalten von Prototypen auszuwerten. Es muss jedoch darauf hingewiesen werden, dass der Mangel an gut dokumentierten Fallstudien über zeitabhängige Deformationen während einer bedeutenden Zeitspanne als wichtiges Hindernis zur Verifikation der Modelle angesehen werden muss.

Es werden Vorschläge für eine alternative Methodik gemacht, welche eine sehr langfristige mechanische Integrität der Auskleidung garantieren kann.

RESUME

Ce rapport présente une vue d'ensemble et une évaluation critique de la situation actuelle de la modélisation numérique du comportement au fluage des argiles. Cet aperçu et cette évaluation se concentrent sur les applications relatives aux excavations souterraines. Sont abordées des observations de terrain et en laboratoire du comportement dans le temps, la modélisation du fluage et l'application numérique des équations représentatives. Une évaluation critique de l'aptitude des modèles actuels à prédire les aspects du comportement au fluage relatifs à la conception de dépôts finals pour déchets est présentée, complétée de suggestions destinées à améliorer les analyses qui pourraient être développées sur la base des technologies existantes.

Des observations relatives au comportement au fluage en laboratoire et dans le terrain sont passées en revue afin de dresser les éléments sur la base desquels on pourra évaluer l'aptitude des équations constitutives et des modèles numériques à reproduire les aspects importants des phénomènes de fluages d'argiles. On relève le manque de données relatives au fluage à long terme sous conditions de drainage. On attire aussi l'attention sur le manque de données significatives sur le fluage d'argiles rigides et de roches argileuses, sur l'effet de la température et sur le fluage sous conditions de drainage pour des situations autres que celles sous contraintes unidimensionnelles.

Des modèles heuristiques et mathématiques sont passés en revue. Les modèles heuristiques fournissent une base pour l'évaluation des paramètres nécessaires pour la mécanique en milieu continu. Des modèles mécaniques en milieu continu sont nécessaires pour une analyse numérique. Il a été démontré que, grâce à l'utilisation d'analyses itératives et par incréments, tout modèle mécanique d'un milieu continu visqueux, ou totalement dépourvu de viscosité, pouvait pratiquement être adapté pour prendre en compte le comportement dans le temps.

Les modèles numériques disponibles pour l'étude de problèmes géotechniques concernant des déformations par fluage sont passés en revue. Des modèles traitant du couplage thermo-mécanique sont également examinés. On cite des cas où une analyse incluant le fluage pour l'étude du comportement de prototypes a été utilisée. Le manque d'exemples bien documentés de déformations dans le cours du temps pour une durée significative représente toutefois un obstacle majeur à la vérification des modèles.

Des recommendations sont présentées pour une approche différente de la conception de revêtements intérieurs permettant de garantir le maintien de leur intégrité mécanique à très long terme.

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1.0 INTRODUCTION

1.1 Scope of the Review

The objective of this report is to present an overview and critical assessment of the state-of-the-art for numerical modeling of the creep behavior of clays. The overview and assessment is focused upon application of numerical analysis to design and construction of underground openings. Constitutive modeling of creep behavior and numerical implementation of constitutive equations are both addressed. Observations of creep behavior in the laboratory and in the field are reviewed to provide a background upon which to assess the adequacy of existing constitutive equations and numerical models for reproducing important facets of creep behavior in clays.

The review of available information on material behavior encompasses soft clays, hard clays, and soft clayey rocks. The review of constitutive models includes phenomenological models based upon observed stress-strain-temperature-time behavior as well as formal mathematical models based upon elastic and plastic continuum mechanics theories.

Evaluation of numerical models is oriented towards modeling of underground openings. Examples of numerical analysis of problems other than underground openings, including foundation bearing capacity and deformation, are noted to illustrate the applicability of existing models. Comparisons of predicted to observed behavior are critically reviewed.

1.2 Methodology

The methodology employed in the development and presentation of this state-of-the-art review is as follows:
First, laboratory and field observations of time-dependent creep phenomenon are presented. These observations are presented as a background for critical evaluation of the various constitutive and numerical models. The lack of information over the time spans of interest in waste repository design is noted. Particular attention is paid to deduced generalized patterns of behavior that provide the bases for phenomenological constitutive and numerical models.

Next, constitutive models for the creep behavior of geologic materials are reviewed. The review

includes micro-mechanical models that attempt to derive constitutive equations starting from the basic inter-particle interaction level, formal mathematical models from continuum mechanics, and phenomenological models based upon empirical observations of soil response. Evaluation of model parameters and the application of these models to soft and stiff clays and to clayey rocks is discussed.

After reviewing available constitutive models, numerical models for implementation of the constitutive equations for prediction of behavior in geotechnical problems are addressed. Models for thermo-mechanical coupling are included in this review. Examples of the application of numerical models to engineering problems and comparisons between predicted and observed behavior are presented where available. Theoretical and practical limitations of the various methods are discussed.

2.0 LABORATORY OBSERVATIONS OF CREEP BEHAVIOR

The great majority of laboratory data on creep of soils is from tests performed at constant temperature on normally consolidated to lightly over-consolidated soft saturated clay soils. For most conventional problems in geotechnical engineering, the only time dependent strains of concern are those associated with hydrodynamic lag (consolidation). Time-dependent deformation under constant effective stress (creep) are typically only considered for soft plastic clays. Deformations due to hydrodynamic lag have been observed for a wide range of soil types and situations and can be characterized analytically with good confidence. These consolidation deformations will not be addressed in this section except for the extent to which they influence observations of deformations at constant effective stress.

Deformations and instabilities which occur in stiff clays after application of load are usually attributed to consolidation induced softening from water absorbtion (shear loads induced dilation and negative excess pore pressure) and/or to discontinuities (fissures) in the soil structure. Because of the long term softening, initial shear stress levels in stiff clays are typically too low to induce creep strain rates of importance in practice.

A limited amount of laboratory test data is available on temperature effects and on the creep behavior of stiff and overconsolidated clays. Very limited data is available on creep deformation of rock.

2.1 Observations on Clayey Soils

2.1.1 Volumetric creep behavior

Time-dependent volumetric deformations in clay soils are attributable to both creep, or viscous behavior of the soil skeleton, and hydrodynamic lag, or the time required for excess pore pressure dissipation. The phenomena of hydrodynamic lag significantly complicates the interpretation of the results of laboratory tests on clay soils.

Secondary compression, volumetric deformation that occurs at constant effective stress after the end of pore pressure dissipation (primary consolidation), was the first and remains the most common aspect of creep behavior studied in the

laboratory. Secondary compression during one-dimensional loading is perhaps the most important aspect of creep behavior in the design of foundations. Secondary compression may be somewhat less important to tunnel engineering than to foundation design as tunnel excavation may reduce confining stress and induce swelling in the surrounding soil. However, laboratory test results do indicate that secondary compression does resume after unloading and swelling, though at a reduced rate compared to virgin compression loading.

Taylor's (149) laboratory investigations of one-dimensional consolidation behavior provided one of the first systematic studies of secondary compression. Taylor's logarithmic creep law (Figure 1) is still the most commonly used function for secondary compression. According to Taylor, the decrease in void ratio after the end of primary consolidation, Δe_s, is approximated by:

$$\Delta e_s = C_\alpha \log (t/t_p) \qquad (1)$$

where C_α is called the coefficient of secondary compression, t is the time since the start of primary consolidation, and t_p is the time required to complete primary consolidation. Taylor postulated that, for one-dimensional compression, $C\alpha$ was constant, independent of applied stress. Taylor's equation has subsequently been inverted to provide the standard definition for the rate of secondary compression:

$$\frac{\partial e_s}{\partial t} = \frac{0.434\ C_\alpha}{t} \qquad (2)$$

Subsequent studies of secondary compression in one-dimensional and triaxial and plane strain compression tests have sought to determine how C_α varies with time, confining pressure, applied shear stress and soil type. Mesri (92) and Mesri and Godlewski (96) compiled information on the value of C_α for a wide variety of soils. The results of their analyses indicate that the ratio C_α/C_c, where C_c is the virgin compression index, the slope of the void ratio-log stress curve in primary compression, falls in a narrow range for natural soils. Based upon the data shown in Table 1, these investigators concluded that, for practical purposes, the ratio of C_α/C_c can be assumed equal to 0.05 for inorganic natural clay soils. The dashed line in Figure 2 represents a suggested relationship between C_α and natural water content. Ladd and Preston (79) looked at the influence of

deviatoric stress level on C_α using one-dimensional and triaxial test data from three different soils. In the triaxial tests, the specimen was consolidated to its final stress state in several increments. The ratio of axial stress and confining pressure was the same for each increment in a given test. After the last increment was applied the soil was left to deform under constant stress conditions for at least 2 log cycles of time after the end of primary consolidation. On the basis of these tests, Ladd and Preston postulated that C_α could reasonably be assumed constant at stress ratios between hydrostatic and one-dimensional conditions. For principal stress ratios greater than that encountered in one-dimensional conditions, they suggested that C_α may increase significantly but presented no conclusive data on this point.

Walker (163) studied secondary compression under triaxial and simple shear conditions for applied shear stresses between 15 and 85 percent of the peak shear stress. While he found little influence of either confining pressure or applied shear stress level on C_α, Walker did find a difference between creep rates under triaxial and plane strain stress conditions. Fuleihan and Ladd (43), however, reported a systematic increase in C_α with shear stress level that could be represented by a bi-linear relationship (Figure 3). No dependence of C_α on confining stress was reported by these investigators. However, Mesri and his co-workers do report a dependence of C_α on confining pressure. The concept of a constant C_α/C_c ratio was developed to relate this dependence to a change in compressibility.

Hence, a decrease in C_α at low void ratios can be related to the corresponding decrease in C_c typically observed at high confining pressures (27). Note that for most engineering problems C_c is assumed constant, providing consistency between Mesri's model and the common assumption of constant C_α.

While the assumption of dependence only upon shear stress level and not upon confining pressure or time may be appropriate for most engineering applications which consider a 10 to 100 year design life and a limited range of confining pressure, C_α clearly cannot remain constant over geologic time scales. Typical laboratory C_α values extrapolated to geologic time scales would result in void ratios much lower than observed in nature. Unfortunately, almost no laboratory data on secondary compression rates after sustained loading of long duration is available. However, the data for intermediate

times, and the geologic reasoning cited above, suggest that C_α may decrease at long times. Dawson et al. (34) have proposed an exponential function for volumetric creep that becomes asymptotic at long times.

By definition, secondary compression occurs under conditions of zero excess pore pressure and constant effective stress. However, arresting secondary compression by preventing drainage will result in generation of excess pore pressure (20,72,98). Figure 4 shows that even in isotropic triaxial compression, arresting drainage will induce excess pore pressure generation. Hence, secondary compression may be considered a condition wherein pore pressure generation occurs at a slow enough rate to allow for complete pore pressure dissipation.

While secondary compression deformations can be considered entirely creep, or time-dependent deformations, considerable controversy exists over whether or not volumetric creep strains can accumulate during primary consolidation (60,72,94). Conflicting laboratory data has been cited on this topic (59). Berre and Iverson (17), Leroueil et. al. (83) and other investigators have reported a dependence of the magnitude of primary compression on length of drainage path. Mesri and Choi (94) report evidence to the contrary. Kavazanjian has suggested that creep during primary compression may be a self-compensating process. As long as the rate of primary compression, governed primarily by the rate of pore pressure dissipation, exceedes the creep rate, creep strains will not accumulate as they are overridden by the primary strains. Only at the end of primary consolidation, when the rate of pore pressure dissipation is small, do creep strains begin to accumulate. This would explain why laboratory test results on small specimens have been satisfactorily used to predict field consolidation behavior for years.

2.1.2 Deviatoric creep behavior

Undrained triaxial creep tests, in which isotropically consolidated specimens are subject to undrained deviatoric loading at constant shear stress, are probably the second most common type of laboratory creep test. Typically, triaxial specimens are isotropically consolidated and then subjected to either constant shear stress or constant shear load under undrained conditions. In constant shear stress tests, the shear load is increased as the sample compresses to compensate the increase in cross-sectional area. During

undrained deformation, measurements are usually made of axial strain and pore pressure at the base of the specimen. Figure 5 from Mitchell (98) shows a wide range of creep behavior reported for clay soils.
Data exist on undrained triaxial creep for a wide variety of soft clays (9, 20, 28, 35, 43, 45, 51, 52, 56, 75, 95, 99, 103, 128, 139, 148, 151, 155, 157). Very little data exists on deviatoric creep of stiff clays and rocks (7, 13, 18, 50, 112, 135, 146, 153). There is some data on the creep behavior of frozen soils (78, 113, 130, 154).

Interpretation of undrained creep tests are complicated by the generation of pore pressure during undrained creep, resulting in varying effective stress and principal stress ratio over the duration of the test. Pore pressures are generated because of the suppression of volume change due to both creep shear strains and the arresting of secondary compression (9,20,72). In general, suppression of volume change in saturated soil results in excess pore pressure generation. As both shear deformations and sustained volumetric strains induced volume change under drained conditions, under undrained conditions both phenomenon may be expected to induce excess pore pressure. The inter-relationship of these two pore pressure generation mechanisms is not well understood. The rate at which pore pressure develops in laboratory tests may also be influenced significantly by test conditions. Leakage around O-ring seals, osmosis through the membrane, and membrane penetration into the specimen have all been cited as external factors which may influence pore pressure development in laboratory tests. Virtually no laboratory data exists which is not subject to this type of critics. One means of accounting for the influence of pore pressure generation on the undrained creep deformation rate is to consider the creep rate a function of applied stress level, defined as the ratio of the applied shear stress to the shear stress at failure (72,109,140). The shear stress at failure can be considered uniquely related to void ratio, and hence constant during undrained deformation. Thus, during undrained creep, if the applied shear stress is constant the stress ratio remains constant. Thus strain development is unaffected by pore pressure generation, at least up to the point of creep rupture (failure), or the transition to tertiary creep (defined below).

Creep studies of engineering materials typically identify three distinct stages of deviatoric creep deformation: primary, secondary, and tertiary

(Figure 6). During primary creep the strain rate steadily decreases. During secondary creep the strain rate remains constant. During tertiary creep the strain rate accelerates to failure, or creep rupture. Virtually all soils show primary creep, while most soils show tertiary creep when loaded to high enough deviatoric stress levels for long enough times. Few soils show secondary creep behavior for sustained periods of time. Either primary creep continues indefinitely, or a relatively rapid transition occurs from primary to tertiary creep.

The existence of an "upper yield limit" (not to be confused with a yield stress), a stress level below which tertiary creep will not occur, has been postulated by some investigators (104). Opponents of this hypothesis argue that the laboratory tests were simply terminated before the onset of creep rupture. The existence of an "upper yield limit" can be important in the formulation of appropriate visco-plastic constitutive equations for a soil. The existence of a "lower yield limit", below which no creep occurs, has also been suggested. However, it may be that creep strains were simply smaller than measurable by test apparatus. This, too, can be important in formulating visco-elastoplastic theories.

Most studies of the primary creep behavior of clays seem to confirm a linear relationship between the log of creep strain rate and the log of time (2, 20, 28, 43, 51, 77, 100, 140, 157). Creep behavior is typically expressed either in terms of shear or shear stress level. Since the magnitude of the shear stress is unaffected by excess pore pressure, and since stress level usually is defined in terms of applied shear stress and initial void ratio, effective stresses are not explicitly considered by most creep investigators. Many of these investigators report the slope of the log strain rate-log time relationship to be constant for constant deviatoric stress level creep, independent of the magnitude of confining pressure or deviatoric stress level (Figure 7). Singh and Mitchell (140) report such data on a variety of soils. These investigators use the resulting mathematical relationship in developing their widely employed general stress-strain-time function, discussed subsequently in this report.

Another generally accepted pattern of undrained creep behavior used by Singh and Mitchell to develop their equation is the linear relationship between log strain rate and deviatoric stress level for constant duration of sustained loading. Once

again, the slope of this relationship is usually considered constant, at least for stress levels between 15 and 85 percent of the stress level at failure (Figure 7). At stress levels below 15%, the slope of the curve decreases markedly, and at stress levels above 85% the slope increases.

Other mathematical forms of logarithmic and power laws have been fit to the results of undrained creep tests (151, 161, 162). Often times, lumped parameter non-linear rheological models are composed as physical analogs for the problem (25, 49, 73, 104). However, these equations have been developed almost exclusively to model primary creep.

A variety of empirical rules have been proposed to predict the onset of creep rupture (28, 103, 128, 139, 141). Relationships have been proposed that relate the time to creep rupture, or failure, to strain, strain rate and effective stress. Due to the inter-relationship among these parameters, these different creep rupture relationships are not necessarily inconsistent. Campanella and Vaid (28) present evidence to show that the effective stress at which creep rupture occurs falls along a straight line that passes through the origin of the p-q stress plot, equivalent to the Mohr-Coulomb failure envelope (Figure 8). This relationship was found valid for both undisturbed and remolded soils. However, there was observed an influence of consolidation stress on the slope of the failure envelope which Campanella and Vaid suggest is attributable to the influence of consolidation stress on shear strength. These investigators (28, 157) also demonstrated that the ratio of the time to creep rupture, t_f, to the minimum creep strain rate, $\dot{\epsilon}_{min}$, could be considered a dimensionless constant, C:

$$\dot{\epsilon}_{min} = \frac{C}{t_f} \qquad (3)$$

where the minimum creep strain rate is defined as the strain rate immediately prior to the onset of tertiary creep. The transient minimum strain rate is the mobilization limit for inception of the rupture phase. Following Ter-Stepanian (151) the creep process consists of a series of jump-like reorganizations of soil fabric. During mobilization, re-orientation occurs in order to develop the maximum possible frictional resistance. If this happens to be insufficient to oppose the applied shear stress, the mobilization limit is attained. Thus, as the minimum strain rate during

primary creep decreases, the expected life of the specimen (the time to failure) increases (Figure 9).

Finn and Shead (85) found that minimum creep rate - rupture life relationship is valid also for overconsolidated clays, irrespectively of strain history and stress level.

Other investigators have postulated unique relationships among stress, strain, and time (20,72,96) and stress, strain, and strain rate (83,153,158) and between strain and excess pore pressure (20,89) from laboratory test data. Assuming both types of relationships are valid, mutually consistent relationships between creep rupture and pore pressure, strain, and strain rate can be established. Furthermore, the rupture line on the p-q diagram can be deduced to also represent the locus of the states of minimum mean effective stress for undrained shear of normally consolidated clay at constant strain rate, as shown for the "typical" effective stress path in Figure 11a. Note that if a unique relationship among stress, strain, and strain-rate or time is assumed, tests performed at different strain rates can be used to deduce creep properties and creep behavior (155). Furthermore, this relationship would imply that testing at very low strain rate could eliminate the need for tests of very long duration if adequate resolution in stress and strain measurement can be achieved. Finally, such relationships may mitigate the importance of membrane leakage and other testing factors on pore pressures measured in undrained tests.

Relatively little information exists on drained creep for other than one-dimensional loading conditions. This may be attributable to the difficulties involved in performing and interpreting test results due to the influence of hydrodynamic lag. The low permeability of clay soils require that either the load be applied exceedingly slowly to allow for complete pore pressure dissipation or that the creep load be applied at the end of a consolidation interval. To apply the load slow enough to prevent generation of excess pore pressure may take several days, and creep strain may accumulate during the loading interval. Applying the load at the end of a consolidation interval means that additional consolidation must occur prior to reaching the drained state. Hence, drained creep test data is rare and subject to interpretational problems. However Singh and Mitchell (140) showed that drained creep behavior appeared to follow the same

broad patterns of behavior shown in Figures 6 and 7 for undrained behavior.

Only one case of creep rupture in drained creep tests has been reported in the literature. Bishop and Lovenbury (18) reported on creep rupture in a long term drained laboratory test on a stiff fissured clay. However, the fissured, discontinuous nature of the material and the absence of any similar data indicates that the observed rupture was probably due to strain localization along a pre-existing discontinuity, rather than to creep rupture of a homogeneous material.

There is a significant amount of data on creep rupture in frozen soils. Laboratory test data indicate frozen soils follow patterns of creep rupture similar to undrained clay, particularly with respect to the relationship between creep strain rate and the time to failure (78, 113, 130, 154). Since the solid nature of the frozen pore phase prevents compression and concurrent stiffening of the soil skeleton, it is not surprising that frozen soils behave more like undrained soils than drained soils in this respect.

Limited studies on the influence of the stress system, or intermediate principal stress, on creep have been conducted. Most of the available data compares triaxial compression creep to plane strain creep (20,23,28,158). Analysis indicate that by expressing shear stress and shear stress level as a function of the invariant octahedral shear stress, the results of these two types of tests can be shown to follow the same empirical relationships (20, 23, 70).

Lacerda (77) studied stress relaxation in triaxial compression. This type of data can be useful in verifying the applicability of creep laws derived from triaxial or plane strain stress-controlled tests to three dimensional stress states under deformation controlled loading.

Studies of temperature effects on the deviatoric creep of clays have been performed using triaxial compression tests (46, 56, 99). Though these tests are typically performed undrained, they are not performed at constant volume due to thermal expansion. Differential thermal expansion of the soil skeleton and pore water create positive excess pore pressures during undrained thermal expansion.

Furthermore, thermal loading causes the deviatoric creep rate to increase. Analysis of data on temperature affects indicate there is an initial

thermal volumetric strain due to expansion of the soil water and minerals and an increase in the viscous strain rate under constant stress loading.

The initial thermal strain can be predicted from knowledge of the proportions and thermal expansion coefficients of the constituent materials in the soil mass. Most laboratory studies of thermally accelerated creep rates have been oriented towards modeling creep as a rate-activated process, as discussed in the subsequent section on rate process theory.

In general, all laboratory testing of creep behavior is limited to some extent by laboratory equipment. Measurements of volume change are typically limited to measurement of water expelled or imbibed through the base of the specimen. Penetration of the membrane into the specimen, compression of gas in soil voids, diffusion of water through the membrane, and leakage around the O-ring seals between membrane and the top cap and cell base may all influence sample volume change. In practice, neither mechanical nor thermal loads can be applied instantaneously, and compliance in test apparatus may significantly influence the measurement of strains immediately after loading. Furthermore, analysis of test results generally assumes material homogeneity and uniformity of deformation. These details of testing and analysis may significantly influence interpretation of test results, particularly considering the low strain rates and small deformations typically encountered in creep tests.

2.1.3 Combined creep loading

Information on soil response to combined loadings is required to extrapolate theories on creep behavior derived from simple laboratory tests to generalized loading situations. The term combined loading is used to refer to the imposition of additional loads after periods of sustained creep. The most common type of combined loading is loading after a sustained period of secondary compression. Leonards and Ramiah (83) showed that soils which undergo one-dimensional secondary compression demonstrate an apparent stiffening when subjected to additional load. This is now an accepted facet of one-dimensional compression behavior termed quasi-preconsolidation.

There is also ample evidence from triaxial compression tests that soils which undergo secondary compression also demonstrate an apparent stiffening when subjected to shear loading (11, 20,

72). Soils subjected to sustained deviatoric creep show a similar initial stiffening when subjected to additional shear stress (52). For both sustained volumetric and deviatoric creep, the creep rate decreases markedly when unloading occurs. A short period of no creep strains after unloading, followed by resumption of creep at a slower rate than before is typically observed in one dimensional and undrained triaxial compression laboratory studies of creep after unloading (13, 77).

Stress relaxation may be considered a form of combined loading, as the deviatoric stress and stress level may change during stress relaxation. Stress relaxation data can be used to validate creep models and to provide insight into facets of creep behavior. One stress relaxation phenomenon of fundamental importance to the understanding of creep behavior and also of importance to the long term behavior of underground openings is the change (or lack thereof) of the lateral stress during one-dimensional secondary compression.

Conflicting laboratory data has been presented on the change in lateral stress during one-dimensional compression (53, 55, 59, 77, 93). Conflicting theories have been proposed to show that the lateral stress should be increasing, decreasing, and constant under one- dimensional conditions. Arguments about which pattern of behavior to expect reflect fundamental differences in hypothesis on creep behavior. While there is some laboratory data to suggest the lateral stress in soft clays may decrease or remain constant, most test data seem to support at least a limited increase in lateral stress after the end of primary consolidation. It is difficult to hypothesize a mechanism other than thixotropic hardening caused by physico-chemical change that could account for a decrease in lateral stress during secondary compression.

2.1.4 Representative Test Results

Development of realistic constitutive equations for soil require data sets from which constitutive parameters can be derived and to which predictions made using the constitutive equations may be compared. Ideally, the tests from which the constitutive parameters are derived should be basic, standard laboratory tests to facilitate use of the model and the tests, to which predictions are compared, should follow different stress paths than the tests used to determine model parameters. The wider the variety of laboratory stress paths for which model validity can be established, the

greater the confidence with which the constitutive model may be used to predict field behavior. Hence, representitive data sets for model development and verification should include the widest possible range of test conditions.

At a minimum, representitive data sets for model development and verification should include information on primary consolidation behavior, secondary compression, deviatoric stiffness during immediate undrained loading and deviatoric creep behavior. Data on the influence of confining pressure (or void ratio) and stress history on deviatoric behavior is also required. Standard laboratory tests from which this information can be deduced include one dimensional and isotropic triaxial consolidation tests, drained and undrained stress or strain controlled triaxial compression tests, and undrained triaxial creep tests. Non-standard tests from which additional information on model parameters and model validation can be obtained include plane strain and true triaxial test data, undrained consolidated isotropic compression tests, drained creep tests, and stress relaxation tests.

The most restrictive requirement in selecting representitive data sets for model validation is the requirement for data on deviatoric creep behavior. While information on consolidation, secondary compression, and immediate deviatoric shear behavior, exists for a large number of natural soils, the number of those soils for which deviatoric creep behavior is available is significantly less. In fact, only two natural soils can be identified for which the requirement of comprehensive data is readily available from the technical literature: San Francisco Bay Mud and Atchafalaya Basin Clay. Several other soils can be identified for which nearly complete data sets are available, including Osaka Clay, Haney Clay, and Boston Blue Clay. Comprehensive data sets are available for several remoulded or reconstituted clays, but for undisturbed natural clays only the five soils cited above have sufficient data available.

Atchafalaya Levee Clay is a very soft highly plastic alluvial clay with natural water content typically between 60 and 80 percent. Fuleihan and Ladd (43) provide a summary of four separate sampling and testing programs including one performed for their study. Test data from the four programs is unusually complete, including shear data from triaxial compression, plane strain compression, and direct simple shear tests,

secondary compression data at different stress levels, and creep data for a variety of stress states.

Figure 10 presents some typical creep test data on Atchafalaya Clay illustrating, the influence of stress system on creep behavior. Table 2 summarizes the material properties on Atchafalaya Clay.

The most comprehensive data set for any natural soil is probably the data set for San Francisco Bay Mud. San Francisco Bay Mud is a soft gray marine silty clay found along the margins of San Francisco Bay throughout the San Francisco Bay area. The clay has a low to intermediate sensitivity and a natural water content typically around 90 percent. While the clay does vary locally around the bay, test data on deposits that are widely separated geographically but have similar index properties show similar engineering properties. Probably the most extensively tested deposit is at the Hamilton Air Force Base site where the University of California performed field and laboratory tests for over 20 years.

Bonaparte and Mitchell (21) collected test data on Hamilton Air Force Base Bay Mud for a wide variety of field and laboratory tests, including isotropically consolidated triaxial compression tests, plane strain tests, undrained creep tests, and stress relaxation tests. Table 3 provides a summary of the Hamilton Air Force Base soil properties compiled by these investigators. Additional data on Bay Mud deposits with similar index and engineering properties is available from Sitar and Clough (1977), Finno and Clough (42), Holzer et. al. (54), and Arulanandan et. al. (9), including test data on pore pressure development due to the arresting of secondary compression (Figure 4).

Boston Blue Clay is another extensively investigated soil. Lambe et. al. (81) and Ladd and his co-workers (80) provide extensive data on the properties of Boston Blue Clay, including consolidation, triaxial, and plane strain laboratory test data. Unfortunately, little data exist on the undrained creep behavior of Boston Blue Clay. The data reported in the literature include the results of only two undrained creep tests. Hence, creep parameters determined from these tests must be considered suspect. Table 4 summarizes the properties of Boston Blue Clay at the M.I.T. test section.

Osaka alluvial clay is a normally consolidated clay of low sensitivity and a natural water content of approximately 45 percent. Murayama et. al. (reported by Oka, (111)) and Adachi et. al. (3) independently report the results of consolidation, strain controlled triaxial compression, and undrained triaxial creep tests. Figure 11 a, b, c, present summary tables of material properties at two different confining pressures and typical test results for both constant strain rate and constant stress triaxial tests. No information on secondary compression is available for this.

Haney Clay is a marine illitic clay of intermediate sensitivity with a natural water content of approximately 40 percent. Campanella and Vaid (28, 156) present shear test data along with comprehensive information on undrained creep behavior is available. However no consolidation data are available for this soil.

2.2 Observations on Rocks

There is relatively little laboratory data on the creep behavior of rocks. What little data is available (7, 13, 50, 135) is oriented towards deviatoric creep behavior, with little attention paid to creep related volume change. Salt represents the rock type for which the greatest amount of creep data is available (13, 50, 138) due to its popularity as a host rock for waste repositories. Additional limited information is available on clay marl, altered rock, and rock interbedded with clay seams (135,146,153). Laboratory creep test data on rocks typically shows only primary creep. Usually, either exponential or logarithmic equations are fitted to the primary creep curve.

Creep deformation in rocks and stiff clays is usually considered to occur with zero volume change, although it is likely that this is simply due to test duration too short for measurable volume change to occur. However, it may also be that the tendency for shear stress induced dilation, or expansion, in stiff clays and rocks is counter-balanced by volumetric stress induced secondary compression, resulting in zero net volume change for practical purposes.

Myer et. al. (105) have presented a suite of laboratory creep test results on an artificial sandstone-like material composed of a sand-wax mixture, mixed and compacted warm to enable the wax to function as a cementing agent. These investigators also present the results of physical

model tests in which different diameter tunnels were driven into a pressurized tank filled with compacted sand and wax. Internal sand-wax deformations measured along radial and axial lines by inductance gauges provided laboratory data obtained in a controlled environment that can be useful for validating numerical models.

2.3 Observations on stiff clays

Rousset (126) reports on undrained creep triaxial tests (Fig. 19) on Boom clay, which is a plastic overconsolidated (OCR=4) , natural clay , homogeneous, with 20% average water content. This clay comes from 230-240m depth and has been tested under a confining pressure of 5 MPa, which is higher than that likely to act in situ (2 MPa) (14) to avoid experimental dispersion between test results. Tests were run in deviatoric steps of 500 hours.

This clay shows primary, but also secondary creep (Table 5). In Table 6 secondary creep rates values are summarized. Above 75% of maximum deviatoric stress, tertiary creep leading to failure develops. A lower deviatoric limit for creep inception is reported to exist (1.5 MPa).

In his study also conclusions on creep tests performed on other 5 clays taken at great depth are cited but no data were provided. The clays came from 300 m to 1000 m depth and are essentially marls characterized by different carbonate contents (Calcium contents ranging from 7% to 38%) and very low water contents (3% - 10%).

The marls were reported not showing any creep effect in triaxial tests, but to start showing it when tested saturated. Details on test procedure for this case are not provided however.

Nguyen Minh (106) reports on laboratory geomechanical characterization of hard marls encountered in driving a deep tunnel. The initial state of stress has been estimated as 10 MPa isotropic. Tests have been run at a slow loading rate, which is however not specified.

Below axial stress of 8-9 MPa strains are shown to stabilize. Above this stress value, the three creep stages can be observed.
No indications on the evolution of creep phenomena is given with confining stress (tri-axial tests).

3.0 FIELD OBSERVATIONS OF CREEP BEHAVIOR

3.1 Observations in Underground Openings

Observations of creep and other time-related phenomena in underground openings typically are concerned with the convergence of the opening (6,30,31). Studies of the deformed shape of tunnel linings have been used to deduce information on the distribution of loads on the lining for long term design. Rheological models for material behavior are often calibrated with field measurements of convergence for use in estimating long term loads on the support system (31) and for use in establishing predictions for comparison to measured deformations from monitoring programs during and after construction.

Peck (114) makes the observation that the load acting on a tunnel liner on the horizontal diameter of the liner are generally observed to increase with time. Kavazanjian and Mitchell (69) attribute the load increase to stress relaxation and speculate that stresses around the tunnel should approach a hydrostatic state with time. Leonards (82) challenged this logic, suggesting that the lengthening of the horizontal diameter is incompatible with a move towards the hydrostatic stress state.

Long term data on liner loads and diameter change is available for tunnels in London (Ward and Thomas, (164) and Terzaghi (152)). Data from both cases, presented in Figure 12 a, b, show the liner load and horizontal diameter continuing to increase with time without any strong indication of asymptotic behavior over the period of the measurements.

Nguyen Minh [106] reports on the time dependent behaviour of a pilot gallery driven in hard marls. The total overburden is about 400 m associated to an initial state of stress of 10 MPa, isotropic.

Evolution of convergences (Fig. 20) of the gallery has been observed, due to the time dependent behaviour of the ground. After 300 days, the deformation rate is reported as low but not vanishing and evaluated about 10^{-6} per days in one section of the gallery.

Pressure on the lining varies between 25% in the case of a rigid concrete ring and 6% with a light support. These values refer to a 26 months observation period.

Rousset (126) illustrates the results of convergence measures taken in a pilot gallery at -250 m level in stiff saturated Boom clay. The liner is made of precast concrete rings.

Measures have been taken for 3÷4 years long. As may be seen in Fig. 21, after the end of the excavation phase (16 days), time dependent effects of the rock take place.
They decrease, but even if small, they do not stabilize in the time period of observation. After four years the pressure against the liner is about 30% of the lithostatic value. The gallery ovalized in the horizontal direction.
The same trend has been observed in the evolution of lining convergences (Fig. 22).

Einstein (39) reports on significant time-dependent effects in tunnels driven in marls in Switzerland. Invert heave rates of 0.5÷1.0 cm/year in 75-100 years old tunnels are reported as not uncommon. This effect is ascribed to electro-chemical mechanically enhanced swelling which is very strong in presence of active clay minerals, as smectite.

In the Ricken tunnel, driven in marls in the 1908, an average invert heave 0.9÷1.9 cm/year has been measured. The first value (0.9) refers to 510 days measure time during 1966-1967, the second (1.9) to 14 months measure time during 1916-1917.

The stand-up time, or time-dependent stability, of underground openings is probably the most important creep phenomenon with respect to construction of underground openings. Delayed failures of unsupported openings have in the past been related to creep rupture phenomena. However, in a comprehensive survey of the stand-up time of tunnels, Myer et. al. (105) report that, with the exception of delayed failures in stiff clays due to negative excess pore pressure dissipation and concurrent swelling and softening, all reported cases of delayed failure in tunnels could be attributable to geologic discontinuities, and not to creep rupture. In accompanying laboratory scale model studies in a pressurized tank filled with a compacted sand-wax mixture these investigators reported that "failure" in this homogeneous mass was characterized by excessive deformations rather than by catastrophic collapse. The conclusion about the role of discontinuities in the stability of underground opening is particulary significant with respect to numerical modeling and is discussed subsequently.

Driven by recent interest in the design of nuclear waste repositories, in-situ experiments on thermo-mechanical coupling of deformations have been conducted or are planned in a number of different places around the world (58, 143, 160). While the variable thermally induced stresses during these tests invalidate them as creep tests per se, analysis of the data from these tests may provide insights into thermally-activated deformation, and test results provide case histories for use in validation of numerical models.

Pyrah et. al. (121) report on pressuremeter creep tests, in which the radial expansion pressure is held constant. Test results for relatively short durations were compared to numerical analyses which assumed undrained conditions for the test duration.

3.2 Observations on Surface Loadings

Field observations of creep behavior of surface structures are available for building and embankment foundations, for braced excavations and cut slopes, and for in-situ tests. Observations of creep-related phenomenon for embankment foundations are the best documented of these field observations (43, 122). Creep effects beneath embankments are of significance due to evidence of delayed failure and retarded primary consolidation due to creep. Settlement of embankments and structures due to secondary consolidation can also be of engineering significance.

Data is available on secondary settlements of structures (26, 75) for relatively long time spans (on the order of 70 years) compared to laboratory data, but still short compared to the design life of a waste repository.

The available field data on the creep of clay indicates that field observations of creep follows much the same patterns of behavior as laboratory observations, including the potential for creep rupture in undrained shear, the lack of observed rupture phenomena for drained cases, and an approximately linear relationship between the amount of secondary compression and the logarithm of time.

There are also studies of the time-dependent behavior of slopes in stiff clays (142). Most of these failures are attributed to either progressive failure induced by environmental changes or failure along a pre-existing discontinuity or plane of weakness. No cases of delayed failure after the end of primary consolidation have been attributed to

creep rupture in homogeneous materials.

Reports of anomalous pore pressure behavior beneath embankments during primary and secondary compression (101) indicate the potential for creep-induced instabilities in sensitive materials. Anomalous behavior includes a halt in pore pressure dissipation and even pore pressure generation during primary consolidation under constant load, and episodes after the end of the initial phase of primary consolidation wherein excess pore pressure is generated under constant load conditions. These field anomalies are similar to the instabilities observed during laboratory consolidation of sensitive clays. Tse (101) reports on a variety of field and laboratory studies that show pore pressure continuing to increase after the end of loading, an increase in pore pressure towards the end of consolidation, after considerable dissipation has occurred and pore pressures being generated after all initial excess pore pressures had dissipated and the soil had gone into secondary deformation. He attributes these phenomenon to structural breakdown, or creep, of the soil skeleton.

4.0 CONSTITUTIVE MODELS FOR TIME-DEPENDENT BEHAVIOR

4.1 Introduction

Constitutive equations are mathematical relationships characterizing the behavior of a material with respect to its reaction to applied loads. Constitutive relationships for the deformation behavior of solids typically relate stress to strain as a function of environmental quantities such as time and temperature and parameters considered as material properties. A general constitutive relationship can be written symbolically as:

$$\underset{\sim}{\sigma}(X,t) = F\,[\underset{\sim}{\epsilon}(X,t),\ T(X,t),\ e(X,t),\ w((X,t),\ldots,X,t)] \quad (4)$$

where F is the general non-linear constitutive material response function, $\underline{\sigma}$ is the second order stress tensor, $\underline{\epsilon}$ is the second order strain tensor, X is the Lagrangian position vector, defined with respect to a point in the initial, undeformed, body, t is time, T is temperature, e is void ratio, w is water content, and the unspecified terms represent any other state parameters that need to be accounted for to describe the constitutive stress-strain relationship.

There are two general approaches to evaluating the material response function, F. A phenomenological, or heuristic, approach can be used, wherein invariant stress and strain quantities are related by general non-linear environmental functions, e.g.

$$\sigma_{oct} = f_1 (\epsilon_{oct}) + f_2 (\gamma_{oct}) \tag{5A}$$

$$\tau_{oct} = f_3 (\epsilon_{oct}) + f_4 (\gamma_{oct})$$

where τ is shear stress, γ is shear strain, the subscript "oct" refers to the invariant value on the octahedral plane, and f_1 through f_4 are the material response operators dictated by experiment and observation.

Alternatively, a specific mathematical form dictated by continuum mechanics can be hypothesized for the material response function and functional parameters can be determined by calibration with observed material behavior. For instance, if the influence of all state variables except ϵ are ignored, the constitutive equation is reduced to:

$$\underset{\sim}{r} = F(\underset{\sim}{e}(X,t)) \tag{6}$$

If material behavior is assumed linear, or the constitutive equation is linearized for use in incremental analyses, equation 6 becomes:

$$\underset{\sim}{\sigma} = \underset{\sim}{C}_{ijkl}\ \underset{\sim}{e}(X,t) \tag{7A}$$

or

$$\Delta\underset{\sim}{\sigma} = \underset{\sim}{C}'_{ijkl}\ \Delta\underset{\sim}{\epsilon}(X,t) \tag{7B}$$

where C and C′ are fourth order stiffness tensors consisting of 81 elements. Equations 7 are the general constitutive equations for linear and incrementally linear rate-dependent materials. By ignoring the influence of t, rate-independent equations result and the equations of linear elasticity can be derived.

Continuum mechanics based constitutive equations are required for numerical analysis of problems involving more than a single idealized soil element. By discretizing the problem into sufficiently small load steps, the linearized incremental approach characterized by equation 7B can be used to create a model which conforms to virtually any internally consistent phenomenological model. Internal consistency

requires that combinations of load, such as deviatoric shear after sustained secondary compression, not lead to inadmissible modes of behavior (e.g. negative shear moduli). While this approach may not be theoretically "elegant" or computationally efficient, it has been used effectively on many problems.

The general constitutive function in Equation 4 can also be expressed in rate form, relating stress rate to strain rate. Equation 4 can be further generalized by decomposition of the strain (or strain rate) tensor. Decompositions of the strain tensor into various combinations of elastic (recoverable), plastic (irrecoverable), immediate (inviscid) and delayed (viscous) components have been employed by various investigators to suit their particular constitutive requirements.

4.2 Phenomenological Models

4.2.1 Particulate mechanics

One approach to developing phenomenological constitutive models is to derive the material response function from a micro-scale model of the interactions between individual soil particles. The most popular micro-mechanical approach to modeling clay soil behavior is consideration of soil deformation as a "rate activated process" (29, 98, 99, 104, 118). In so-called rate-process theory, deformation is assumed to occur when interparticle atomic bonds break, and bonds break when atoms vibrating around points of metastable equilibrium excede a local energy maximum, the so-called activation energy, and move to the next metastable state of equilibrium. Applied shear stresses distort interparticle energy barriers such that the barrier in the direction of the shear stress is lowered, increasing the deformation rate and creating a preferred direction of deformation (Figure 13). The global deformation rate is evaluated by consideration of the rate at which bonds break, the bond density, and the deformation which occurs when a bond breaks. The variability of these quantities throughout a soil element makes rate process theory an ideal application for stochastic modeling and statistical mechanics.

A wide variety of constitutive theories based upon rate process theory have been developed for soils. Depending on the parameters considered in the development of the constitutive equations, rate process theory can be used to describe the influence of such environmental parameters as time, temperature, particle density, and chemical

potential on soil deformation. While to some extent the predicted influence of these environmental factors depends upon assumptions made in the physical model, virtually all rate process theories are consistent with respect to the influence of temperature on deformation behavior. Rate process theory predicts strain rate is related to temperature by the Arrhenius equation:

$$\dot{\epsilon} = A \exp (E / RT) \tag{8}$$

where A is a reference strain rate t, E is the activation energy, and R is the universal gas constant. The reference strain rate, A, is related by Mitchell (98) to temperature, Boltzmann's constant, k, Planck's constant, h, and the proportion of successful barrier crossings, X as:

$$A = X (kT/h) \tag{9}$$

Mitchell suggests the A may be independent of temperature during undrained creep.

Equations 8 and 9 represent a logical and simple form for accounting for the influence of temperature on the creep behavior of soils.

Other particulate mechanics approaches to constitutive modeling include micro-plane slip theory and consideration of soil as a dissipative material (15,16). In all micro-mechanical models, the modeler has significant latitude with respect to governing assumptions and equations similar to those from rate process theory can be derived. Where possible, micro-mechanical models should be validated and calibrated with observed material behavior to be useful for engineering purposes. More typically, the functional forms chosen for empirical phenomenological models, described in the next section, are based upon equations derived from rate process theory. Perhaps the most useful function of rate process models of soil deformations is that they provide a basis for extrapolating macro-scale constitutive models to time scale and environmental conditions for which little observational data exists. This could be particularly useful for problems involving geologic time scales such as the design of high level radioactive waste repositories, wherein observational data for periods approaching the life span of the facility are not available.

4.2.2 Empirical models

Empirical phenomenological models are constitutive models in which functional forms have been fitted to systematic patterns of stress- strain behavior deduced through observation. Hyperbolic-shaped stress-strain curves, linear void ratio-logarithm stress relationships, and Taylor's secondary compression theory are all examples of empirical constitutive stress-strain models for soils. The majority of empirical models developed for clay soils are for one-dimensional compression behavior. However, models are also available for certain triaxial stress conditions and several of the one-dimensional models have been generalized to two and three dimensional stress states.

Bjerrum (19) hypothesized a unique relationship between void ratio, effective stress, and time of sustained loading (or geologic age) for one-dimensional compression. By assuming constant values of C_c and C_α/C_c, Bjerrum's model can be depicted by a series of parallel isochrones, or lines of constant time, on a void ratio-log effective stress diagram (Figure 14). Bjerrum's model is notable for its decomposition of the strain tensor into an immediate, time-independent component, which occurs immediately upon imposition of effective stress, and a delayed, time-dependent component, which also starts to accumulate immediately upon load application.

Mesri and his co-workers developed a theory for one-dimensional stress-strain-time behavior based upon the concept of a constant C_α/C_c ratio (93, 94, 96). These investigators decomposed the strain rate tensor into a time-independent component due solely to changes in effective stress and an interrelated time-dependent component that occurs only after the completion of pore pressure dissipation. The noteworthy aspect of this model is the decomposition of the strain tensor such that no volumetric creep deformations can occur during primary compression. This decomposition, while convenient, has been attacked on theoretical and physical grounds as non-objective and contradictory.

Leroueil et. al. (83) hypothesize a unique stress-strain-strain rate relationship for natural clays. This model, developed for one- dimensional compression, was extended to two-dimensional stress states using the limit state concept in the YLIGHT constitutive model. In YLIGHT, the limit state, or yield criterion, is considered a function of strain rate, and provisions are made for evolution of the

shape and size of the limit state due to "destructure", or remolding of the soil skeleton.

After one-dimensional compression, the most thoroughly investigated aspect of the stress-strain-time behavior of soils is triaxial creep, typically under undrained conditions. While a large variety of power laws, logarithmic and exponential relationships have been proposed for undrained creep, the Singh-Mitchell creep function (140) is by far the most commonly used creep function for soils. Based upon the patterns of behavior portrayed in Figures 7 and 9 Singh and Mitchell developed the following equation for the strain rate during undrained creep:

$$\dot{\epsilon} = A \exp(\alpha \bar{D}) (t/t_i)^m \qquad (10)$$

where D is the deviatoric stress level, α and M are the material constants defined in Figure 7, t_i is a reference time, usually set equal to 1.0, and A is a material parameter which provides a reference strain rate. Singh and Mitchell showed that an equation of similar form to this could be derived from rate process theory if the appropriate assumptions were made.

Dawson (34) has proposed a volumetric creep rate equation similar in form to the Singh-Mitchell rate process based function (equation 10). Dawson related volumetric creep rate to an internal variable he termed the consolidation stress state. The innovative facet of Dawson's work is that all volumetric deformation of the soil skeleton is considered viscous, or time-dependent. The deformation of saturated soil is treated as a mixture of two viscous fluids.

After soft clays, the material most frequently evaluated for creep potential is salt. A great deal of attention has been paid to the creep behavior of salt as part of the U.S. waste repository design program. A variety of logarithmic and exponential creep functions have been fit to creep data on salt (50,138).

Kavazanjian and his co-workers have created a unified phenomenological model for the three-dimensional stress-strain-time behavior of soft clays (70, 72). Based upon Bjerrum's decomposition of the strain tensor into immediate and delayed components, this model unifies a variety of commonly used models for clay soil behavior under restricted boundary conditions within a general

three-dimensional framework. The component models for restricted boundary conditions employed to develop the unified model include Bjerrum's one dimensional compression model, Ladd and Foott's normalized soil properties concept, Taylor's secondary compression law, and the Singh-Mitchell creep equation. Consistent rules for combined loading and super- position of load increments were established based upon laboratory test data.

Kavazanjian, Mitchell, and Bonaparte (/70/) developed a heuristic extension of the initial model for triaxial stress states to three dimensional stress states. Model parameters are conventional parameters familiar to most practicing geotechnical engineers and can be derived from standard laboratory tests, from correlation with index properties, or on the basis of past experience and engineering judgment.

4.3 Continuum Models

A second class of constitutive models are the theoretical models developed from continuum mechanics by making assumptions on the mathematical form of the material response function. The applicability of these models depends somewhat upon the accuracy with which the assumed mathematical relationships conform to real soil behavior However, incremental and iterative approaches can be used to significantly weaken this dependence. If analyses using either closed form solutions or finite or boundary element type methods are to be used to make numerical predictions, material behavior must be fitted to one of the various constitutive models from continuum mechanics.

4.3.1 Inviscid and viscous elastic and plastic

Although conventional elastic and elasto-plastic constitutive models do not explicitly consider time, iterative and incremental approaches can be used to model time-dependent behavior. The simplest approach to modeling time-dependent behavior is to define time-dependent elastic moduli. Time-dependent plastic moduli can also be defined for iterative analyses. Alternatively, in the numerical solution of the problem, creep strain can be "moved to the other side of the equation," treated as an increment of stress relaxation, and accommodated by applying fictitious internal forces. More detail on these approaches are provided in the next section on numerical implementation.

Time dependent deformations can also be accommodated by inclusion of explicit viscous, rate

dependent terms in the constitutive equations. Constitutive equations with explicit viscous terms can be classified as viscous-elastic, viscous-plastic, and elastic and/or plastic plastic-viscous. Lumped parameter mechanical analogs are used in Figure 12 to illustrate the various possibilities. Figure 12a shows a viscous elastic model, in which the load on the soil skeleton is shared by the viscous and elastic elements. In this type of model, the recoverable visco-elastic strains slowly stabilize as the elastic deformation gradually assumes the entire load. In viscoplastic models, as shown in Figure 12B, irrecoverable viscous strains develop whenever the yield stress is exceeded and continue as long as the applied stress exceeds the yield stress at a rate governed by the difference between the applied stress and yield stress. In an elastic or plastic plastic-viscous model (Figure 12) irrecoverable rate sensitive strains start to accumulate immediately upon load application and continue as long as a load is applied to the soil skeleton.

4.3.2 Time-dependent elasticity and plasticity theory

Booker and Small (22) developed a time-dependent model for soil behavior using the concept of time dependent elastic moduli, or strain operators. Volumetric and deviatoric strain operators are formulated to provide time-dependent secant values of the bulk and shear moduli for use in the elastic equations of continuum mechanics. The form of the strain operators is based upon the phenomenological type of models described in the previous section. After each discrete time or load interval, the value of each strain operator is redefined and stress and strain distribution within the soil mass recalculated. If small enough time steps are taken and strain operators are defined based upon valid and consistent phenomenological relationships, time-dependent elasticity theory is a simple means by which phenomenological patterns of soil behavior can be incorporated into numerical analysis.

Borja and Kavazanjian (23,24) have demonstrated how conventional inviscid plasticity theory can be used to model time-dependent behavior. The strain tensor is decomposed into elastic, plastic, and time-dependent components:

$$\dot{\epsilon} = \dot{\epsilon}_e + \dot{\epsilon}_p + \dot{\epsilon}_t \qquad (11)$$

The elastic and plastic strain rates, $\dot{\epsilon}_e$ and $\dot{\epsilon}_p$, are independent of time. The time-dependent (creep) component, ϵ_t, is a time-dependent plastic deformation component. This time-dependent model is

of the elasto-plastic-viscous type. Defining an elasto- plastic response function C_{ep}, Borja and Kavazanjian wrote the governing constitutive equation in form:

$$\dot{\sigma} = C_{ep} (\dot{\epsilon} - \dot{\epsilon}_t) \qquad (12A)$$

or

$$\dot{\sigma} = C_{ep} \dot{\epsilon} - \dot{\sigma}_t \qquad (12B)$$

where $\dot{\sigma}_t$, equal to $C_{ep}\ \dot{\epsilon}_t$, is termed the stress relaxation rate.

Borja and Kavazanjian developed a constitutive model using equation 12B based upon modified Cam-Clay plasticity. During sustained loading, creep strain accumulate in their model according to either the rate of secondary compression or the Singh-Mitchell creep equation. The elliptical Cam-clay associative yield surface expands with time to account for the quasi-preconsolidation effect, or time-dependent stiffening observed in clay soils during secondary compression. Figure 16 portrays this yield surface expansion and the accompanying stiffening.

In the Borja and Kavazanjian model, volumetric and deviatoric phenomenological creep relationships cannot be satisfied simultaneously. If creep strains are scaled according to the volumetric relationship (using C_α), deviatoric creep rates are underpredicted at very low stress levels and are seriously over predicted at high deviatoric stress levels.
Furthermore, the deviatoric strain rate at failure is undefined. If deviatoric scaling is used, the volumetric creep rate is zero, during isotropic compression and at failure, and is seriously underpredicted at high and low stress levels.

Hsieh and Kavazanjian (55) have applied this time-dependent plasticity approach to a "double-yield surface" Cam-Clay type model. The second yield surface is a horizontal, perfectly plastic, Von-Mises type yield criterion embedded within the yield ellipse (Figure 19). In this non-associative model is that independent phenomenolgoical models for both deviatoric and volumetric deformations can be satified simultaneously, e.g. secondary compression and Singh-Mitchell creep equations can both be satisfied. In associative plasticity models, the ratio of volumetric to deviatoric strains is fixed by the shape of the yield

criterion, hence independent relationships for shear and volume strains cannot be satisfied simultaneously. Kavazanjian and Hsieh (67) have demonstrated that by appropriate choice of hardening rules based upon phenomenological relationships, the double yield surface model can be made to conform almost exactly to the Kavazanjian and Mitchell phenomenological model (72).

The time-dependent plasticity models of Kavazanjian and his co-workers have only been applied to primary creep problems. Theoretically, tertiary creep could be modeled by setting the parameter M in the Singh-Mitchell creep function equal to a value greater than 1.0. However this approach would result in tertiary creep for all stress levels and might result in numerical instability.
If numerical stability could be achieved, a hybrid approach using the Singh-Mitchell creep equation and Campanella and Vaid's creep rupture relationships could be developed within the time-dependent plasticity framework.

Time-dependent elasticity theory and time-dependent plasticity theory provide simple means by which time-dependent behavior can be modeled. The advantage of these types of models is that selected phenomenological patterns of behavior can be accurately reproduced. The disadvantage of these models is that they are often computationally cumbersome due to the small time increments and iteration required for the solution to converge. Care must be taken that the phenomenological patterns on which the numerical parameters are calibrated are consistent and objective to insure stability and convergence.

4.3.3 Viscous elasticity and plasticity theory

While visco-elastic models have been developed for geotechnical analyses, (25,49,76,122) this type of model should probably be restricted to analysis of problems in soil dynamics, where the assumption of linear elastic material response makes possible simplified frequency-domain analyses of problems. For pseudo-static loadings, where the rate of load application is slow enough to ignore dynamic affects, visco-elastic models cannot provide patterns of material response that conform to real soil behavior unless a path-dependent modulus is defined. Otherwise, visco-elastic models require that all strains are recoverable and that only primary creep can occur.

A wide variety of visco-plastic constitutive models have been proposed (1-5, 8, 12, 32, 33, 41, 47, 64, 66, 87, 107, 127, 159, 162). Visco-plasticity has been applied to isotropic work hardening plasticity, Kinematic hardening plasticity, and bounding surface plasticity. Endochronic theory represents a unique class of visco-plastic constitutive theory.

Most visco-plastic constitutive theories are "overstress" models based upon Perzyna's theory (115), in which viscous plastic strains occur only when the stress exceeds some limiting value. The magnitude (or rate) of the viscous plastic deformation is related to the difference between the current stress and the limiting stress, termed the over- stress. Adachi, Oka, and others (1-5, 66, 107, 110) developed overstress models based upon isotropic work hardening Cam-Clay type plasticity theory, while Dafailias (33), Kaliakin (64), and Liang et al. (87) have developed visco-plastic bounding surface plasticity overstress models. The primary shortcoming of the early overstress models was that they would only predict primary creep under constant stress loading. Aubry (12) introduced an internal damage parameter, D, to model creep rupture. Aubry proposed an equation of the form:

$$\dot{\epsilon}_{vp} = \dot{\epsilon}_{vp\phi} (1 - D/D_f) \qquad (13)$$

where $\dot{\epsilon}_{vp}$ is the viscoplastic strain rate, the subscript "ϕ" refers to the viscoplastic strain in the undamaged state, and D_f is the value of the internal damage variable at the failure, or at the time of creep rupture.

The internal damage variable may be related to shear strain, pore pressure, duration of loading, or any other intrinsic parameter. Adachi and Oka have shown that pore pressure accumulation and deviatoric stress level can serve as the damage parameter for soft clays, and have extended their overstress theories to consideration of creep rupture effects. Figure 18 shows the patterns of creep behavior obtained by the Adachi and Oka model. Note the close conformance of the idealized patterns of deviatoric creep behavior shown in Figure 7.

A second approach to modeling creep rupture using viscoplasticity is through the use of a non-stationary flow surface (91, 97, 134). If the flow surface evolves during creep such that the "overstress" increases, creep strain rates can

increase correspondingly. A non- stationary flow surface model in which the flow surface degrades is similar in concept to an overstress model with a damage function. Evolutionary rules for the non-stationary flow surface can be chosen to reproduce either primary or tertiary creep phenomenon.

Runesson (127) presents an interesting approach to elasto-viscoplasticity for soft clays. Using a strain decomposition similar to Borja and Kavazanjian, a plastic-viscous model was formulated such that plastic strains developed in all stress ranges. Consolidation behavior was considered using Biot's theory, and was uncoupled from the soil deformation.

Visco-plastic models are in an early phase of development and only a few were verified both on a sample level and/or a tunnel problem level.

The above models can probably predict as they are, or be adjusted to predict, the so called " undrained creep failure" in normally consolidated materials. It is however to see how could they predict creep failure in overconsolidated clays, which seem more of interest here. However, according to Finn and Shead (85) and mostly to Richardson and Whitman (124) strain rate effects are stronger in overconsolidated materials than in normally consolidated materials. Majority of the continuum models based on Perzyna " overstress" concept would fail to predict overconsolidated creep, because this would require a visco-elastic creep. Summing up, visco-plasticity models are a good potential tool. Additional, conceptual and theoretical work, supported experimentally is needed to make them an operative and reliable source for prediction.

Endochronic theory presents an innovative approach to visco-plasticity in which no yield surface is used. Plastic strain is expressed as a function of a scalar damage parameter called intrinsic time (8, 159). Intrinsic time, and hence plastic strain, is defined as a non-decreasing function of load and time. While endochronic theory has proven successful for some applications in metal plasticity, where perfectly plastic flow may be assumed, it has not been widely accepted in geotechnical practice. Volume change during soil deformation greatly complicates the formulation of endochronic constitutive equations for soils. A complete endochronic formulation for soils requires determination of a large number of material parameters through (8). These parameters must be back calculated from suites of detailed laboratory

test results. The interactions between material parameters and the sensitivity of model behavior to parameter values are not well understood. For these reasons, endochronic plasticity theory has not been widely employed for geotechnical applications.

5.0 NUMERICAL MODELING OF TIME-DEPENDENT BEHAVIOR

A variety of numerical models of time-dependent creep behavior of soils have been developed using viscous and inviscid elastic and elasto-plastic models. Numerical implementation of some of these models include hydrodynamic (consolidation) and thermo-mechanical coupling effects. Model parameters are typically assessed by fitting numerical predictions to the results of laboratory tests. With the notable exception of endochronic theory, this is usually a straightforward process if a numerical model is available. In several cases, comparisons between numerical predictions and observations of field behavior are available. However, most of these comparisons are for surface embankment loadings, as this is the conventional problem for which creep deformations are usually of greatest concern. Only limited field data on creep in underground openings is available. However, some case history analyses have been made.

5.1 Time-Dependent Elasticity

Time-dependent linear elasticity provides a computationally simple but inefficient approach to numerical modeling of time-dependent soil behavior. Through the use of either a tangent modulus incremental form of the constitutive equation or a secant modulus global formulation, time-independent linear elasticity theory can be used to model path dependent non-linear time-dependent material behavior. However, small time increments and an iterative approach are required to achieve stability and convergence.

The application of effective stress based incremental elasticity theory to the analysis of time-dependent settlement of foundations has been demonstrated by Booker and Small (22). Using time-dependent moduli, these investigators achieved good agreement between observed and predicted vertical settlement. Kuppasamy and Buslor (76) demonstrated the use of a time-dependent total stress secant modulus approach to analysis of the creep behavior of laterally loaded piles.

The introduction of pore pressure and consolidation into a time-dependent elasticity model greatly complicates definition of the time-dependent moduli. Volume change during and after consolidation must be considered in defining the time-dependent moduli values. However, the problem is not intractable, and time-dependent elasticity analyses of creep deformation of embankment

foundations can be performed. Time-dependent moduli can be defined using a phenomenological model like the one developed by Kavazanjian, Mitchell, and Bonaparte and input to incremental elastic analyses.

5.2 Time-Dependent Plasticity

Kavazanjian and his co-workers (23,24,55,67) have installed their time-dependent plasticity models in a finite element program that inclues consideration of hydro-dynamic lag, finite deformation theory, and a non-linear void ratio-anisotropic permeability relationship. Both the associative, single yield surface Cam-clay constitutive model and the non-associative double yield surface model have been so installed. Both models have been used by the developers to analyze a well-documented case history of embankment foundation deformations.

The associative model of Borja and Kavazanjian (23, 24) was used independently to evaluate the results of centrifuge model tests of embankment foundation deformation (86). A comparison of observed and predicted deformations show the model to predict time dependent deformations well, but revealed a significant discrepancy between observed and predicted initial, undrained shear deformations. This discrepancy for initial strains is characteristic of associative clay models, which tend to underestimate shear distortion in soft clays.
The double yield surface model was developed by Kavazanjian and Hsieh (67) to remedy this deficiency by allowing additional shear strain within the yield ellipse. The additional shear strain potential field is mapped according to the results of laboratory undrained triaxial tests.

Due to uncertainty about creep during combined loading, several different creep options are provided for these two time-dependent plasticity models. The single yield surface model provides the option for phenomenological scaling of creep strains to conform to either the secondary compression law or the Singh-Mitchell creep function. The user of the double yield surface model can choose to either satisfy secondary compression and Singh-Mitchell simultaneously or use a hybrid creep rule that combines these two phenomenological relationships. Numerical analyses of the embankment foundation deformations were performed using all of these options. Results of the analyses show no clear superiority of any one of these phenomenological scaling laws over the others when numerical predictions are compared to

observed behavior for complex boundary value problems.

5.3 Visco-Elasticity

Asymptotic visco-elastic models have been applied to the analysis of both embankment and tunnel problems (8,30,122,144). Redman and Poulos (122) applied a visco-elastic model analysis of two case histories of undrained deformation of soft clay foundation soils beneath embankments. Akagi (8), Cividini et. al. (30), and Swoboda et. al. (144) used visco-elastic models to analyze tunnel behavior. These investigations calibrated the rheological parameters of their models against convergence measurements of tunnel linings.

Visco-elastic models offer advantages of efficiency and objectivity over time-dependent elastic models because the viscous resistance is explicit in the formulation of the problem. However, visco-elastic models are constrained to producing only primary creep. This could be remedied by allowing the stiffness of the elastic element to degrade with time, producing a time-dependent visco-elastic model that would be little different from a time-dependent inviscid elastic model. Koon and Lytton (74) have demonstrated that, mathematically, Laplace transforms can be used to reduce the equations of visco-elasticity to time-dependent equations of elasticity, hence there is little practical difference between time-dependent elasticity and visco-elasticity.

5.4 Visco-Plasticity

Several visco-plastic models have been implemented within finite element computer codes (2,33,64,97,127,143). Computationally, the visco-plastic models have the advantage of explicit consideration of time-dependent effects in the governing mathematical relationships. Hence, iterative and approximate approaches to calculating time-dependent material parameters and stress relaxation terms are not required. The disadvantage inherent in this approach is that the mathematical formalism imposes certain restrictions on the patterns of deformation behavior the model can reproduce. Associative relationships pre-define the relationship between volumetric and deviatoric strains, while non-associative relationships create computational difficulties. Furthermore, most visco-plastic relationships do not account for time-dependent behavior at low stress levels (below the yield stress) or after unloading. However, as with elastic analyses, a great deal of flexibility

can be achieved by adjusting material properties after each increment to account for non-linearities in material response.

In most cases, numerical modeling has been restricted to analyses of the behavior of laboratory specimens. However, in several cases the models have been applied to field problems. Examples of application of visco-plastic models to tunnels (143), embankments (110), and strip foundations (97) can be found in the literature. Comparisons with observed field and laboratory behavior typically show excellent agreement, as model parametres are often calibrated with the cases for which the predictions are shown. Because the models so calibrated are not then tested against additional data, there is little conclusive evidence on the reliability of these models for predicting soil response for conditions other than those on which they were calibrated.

5.5 Thermal Coupling

While analyses of hydrodynamic coupling in the deformation of saturated clays are now commonplace due to the importance of the problem, relatively few analyses of thermo-mechanical coupling have been developed due to the limited situations in which thermal effects are important. A non-stationary thermal field has several impacts on soil behavior, including thermo-mechanical coupling, thermo-hydraulic coupling, and thermally activated creep strains. Numerical models have been developed to account for some of these affects in the design of nuclear waste repositories (10, 57, 120, 143). However, no single method is currently available to account for all of these affects simultaneously.

Thermo-mechanical coupling refers to the changes in stress to due differential thermal expansion and contraction. Thermo-mechanical coupling is generally considered the most important thermal influence on waste repository design.

Arulmoli and St. John (10, 143) describe a decoupled thermo-mechanical analysis. Temperature time-histories calculated using a two-dimensional finite element program for transient heat flow were input as the thermal loads to a visco-plastic finite element program for deformation analysis. No hydrodynamic (hydraulic) coupling ws considered in these analyses as the host rock (salt) was dry.

Pusch and Adey (120) present an analysis of thermally activated creep settlement of radioactive

waste canisters based upon rate process theory. Creep rates, hence settlement rates, are adjusted according to predicted thermal fields according to relationships based upon rate process theory.

Hueckel et. al. (56,57) have developed a fully coupled elasto-plastic finite element program for the thermal-mechanical-hydraulic response of clay soils for waste repository design in Europe. This is the only fully coupled analysis reported in the literature. However, creep effects on soil deformation were not considered. Despite there various analyses of different aspects of thermal coupling, no comprehensive analysis of thermo-mechanical-hydraulic creep-inclusive deformations has yet to be reported in the literature.

5.6 Numerical Analyses of Underground Openings

Drained and undrained time-indipendent analyses of underground openings are fairly common. Such analyses are typically used to estimate loads on liners and ground surface deformations associated with tunneling. However, analyses of time dependent effects, including consolidation effects but exclusive of time effects related to the advance of the tunnel heading are quite rare.

Johnston and Clough (62) define four major causes of time-dependent behavior during tunneling:

* Effects due to the advance of the tunnel heading.
* Effects due to soil creep and/or secondary compression.
* Effects due to consolidation from drainage into the tunnel.
* Effects due to consolidation of the soil around the tunnel due to tunnel construction.

Analyses of effects due to advance of the tunnel heading require either a true 3 dimensional analysis of the advancing tunnel heading or an axi-symmetric two-dimensional approximation. While results of linear elastic three dimensional analyses have shown that the axi-symmetric approximation is not appropriate for shallow tunnels (Kasali and Clough, (65)), this two-dimensional approach does not seem unreasonable for relatively deep tunnels. Although no comparative analyses are available, it is not unreasonable to postulate that the axi-symmetric approximation will yield reasonable results when the stiffness of the ground is relatively constant within the zone of influence of the tunnel heading and when the free surface does not lie within the influence zone,

typically 5 to 7 tunnel diameters radially from the tunnel center line.

An alternative to the axi-symmetric approximation of an advancing tunnel heading is a plane strain longitudinal section in which the tunnel is treated as a slot of infinite width. Finno and Clough (42) have shown that this approach can yield reasonable results for points directly above the crown of the tunnel and for points in front of the heading on the tunnel center line. Results from such an analysis are only applicable to points on the vertical plane that includes the tunnel axis. At any other point the plane strain assumption will introduce significant discrepancies.

Several investigators have modeled the influence of-deviatoric creep on liner stresses and deformations. Ghaboussi and Gioda (44), Sakurai (129), and Cristescu et. al (31), used visco-elastic models in two dimensinal analyses at stresses around tunnel openings. Cividini et. al. (30) and Akagi et. al. (6) use lumped-parameter visco-plastic rheological model to simulate yield and convergence of tunnel openings Swoboda et. al. (144) used a coupled finite element-boundary element analysis.

Finite elements were used to represent the viscoplastic properties of the rock mass surrounding the tunnel, while the tunnel liner was treated as a visco-elastic material.
Beyond the zone of influence of the tunnel, elastic boundary elements are used to save computing time. Tan and Clough (145) developed a finite element model for undrained creep using the Singh-Mitchell creep equation and applied it to the analysis at tunnel openings. While all of these analyses demonstrate the influence of creep in increasing liner loads and redistributing and relaxing shear stress in the ground around the tunnel, only Cividini et. al. (30) provide a comparison between observed and predicted behavior, and in this case rheological parameters were back calculated from short term observations for use in predictions of long term behavior. No guidance is given for evaluating rheological parameters prior to the start of construction.

Modeling of consolidation deformations due to drainage into the tunnel and due to tunnel construction are somewhat interrelated. Both effects require a coupled deformation pore pressure analysis.
However modelling of drainage into the tunnel is somewhat simplistic as tunnel construction need not

necessarily be modeled. Johnston and Clough (62) Seneviratne and Gunn (136), and Finno and Clough (42) all report on two-dimensional finite-element analyses of consolidation behavior around tunnels in soft ground. All three investigators used the modified Cam-Clay model of soil behavior in their analyses. While the Johnston and Clough study was purely hypothetical, both Seneviratne and Gunn and Finno and Clough compare predictions made using paramenters calculated from laboratory test data to observed behavior. Seneviratne and Gunn compare their predictions of deformation to results from physical model tests, while Finno and Clough compared their predictions to field measurements on an actual tunnel project in San Francisco.

Finno and Clough compare predicted deformations from both longitudinal and transverse sections to observations made during construction of a 3.05 m diameter 9.75 m deep sewer tunnel in a deposit of recent San Francisco Bay Mud properties where determined from laboratory test results and past experience with Bay Mud. Results of analyses showed that when the heave induced when the shield was showed forward and the tail void closure behind the shield were modeled, the longitudinal analysis yield good results when compared to deformations and ground water conditions observed during advance of the shield and the transverse analyses compared well with observations of ground surface settlement and lateral displacements in towards the tunnel. Unfortunately, field monitoring was discontinued relatively soon after the end of construction, hence no comparisons between observed and predicted long term behavior are available.

6.0 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

This report deals with the assessment of the influence of creep strain on the long - term stress-strain conditions occurring around a deep excavated tunnel in the time range of interest for waste disposal. Namely, the time range usually considered is of the order of magnitude of 10^4 years.

The tunnel has to be excavated in a clay medium . Clayey materials which have been considered in this study encompass mud rocks (water content 5%-10%, very low plasticity), stiff clays (water content 10%-30%), soft clays (very high water content, 50%-90%, high plasticity).

Mechanically, different patterns of response to the applied loads depend in these materials also on the initial state of stress compared to the maximum past experienced stress. Clay rocks behave as strongly overconsolidated materials in the range of stresses of interest.
The mineralogical characteristics of the clayey fraction is responsible for very different response patterns to applied loads.

Experimental evidences available to support mathematical modeling have been reviewed here as well as possible approaches capable to predict long-term stress conditions acting on the tunnel liner for clays with different characteristics.

Data from laboratory tests as well as from in situ measurements will be discussed in the next two sections. Available mathematical models will be reviewed in the third section.

6.1 Data from laboratory experiments

Laboratory tests available in the technical literature are performed following specific load paths and boundary conditions.

In clay soils, volumetric creep strains are conventionally studied by means of oedometric tests in which the so-called secondary compression is observed. This phase is observed also during consolidation in triaxial devices where volumetric strains are typically obtained by measuring the amount of water expelled by the specimen. Alternatively, when drainage is not allowed during the secondary compression phase, pore pressure increase is observed . Secondary compression does

not find explanation in terms of pore over-pressure dissipation following the theoretical framework of the consolidation theory. The low pore pressure gradients may alter the conditions under which this flux occurs, however, with respect to those assumed in consolidation theory.
For such tests, moreover, interpretation in terms of creep is far from straightforward.

Usually, in soft saturated clays, the secondary compression effect is negligible with respect to the primary consolidation one. The coefficient of secondary consolidation, $C\alpha$, is constant and around 5% of the virgin compression index, which accounts for the elasto-plastic consolidation strains. However, its influence over large time spans could become significant. In principle, it should decrease for very long times, as discussed in Sect. 2.1.1 .

Creep in clay soils under deviatoric stress conditions is typically based on shear and triaxial tests under undrained conditions. Clays are usually tested saturated.
In clay rocks, uniaxial tests are also used; they are typically performed at the natural water content.

It may be first observed that in saturated clays, due to the undrained conditions, it is difficult to interpret results following a rationale which enables the identification of the role of the liquid phase in the medium.

Further, the following considerations apply.

The large majority of tests are run at low stresses (tens to hundreds kPa) and very short time (typically one week for a loading step) compared to that of interest, and too short to obtain a comprehensive picture of the responses for every deviatoric stress level.
Systematic observations on changes in the observed deviatoric response patterns encompassing from low to high confining stress states could not be found in the technical literature.

Characteristics of available test equipment make it difficult to get representative test results for long times and at very low strain rates. In particular, this aspect becomes critical when testing saturated plastic clays as discussed in Sect. 2.1.2..

The phenomenologic behavior observed in test results is the following.

In normally consolidated soft clays with high water content, creep strains develop at a decreasing rate at constant total stress, followed, possibly, by an increasing rate phase leading to failure. The existence of a stationary phase is considered not present in these materials by most authors. Further, there is no experimental evidence that creep rupture occurs for every stress level in the time span investigated.

Data for stiff clays are scarce.
In the stiff (limited plasticity, high stress values) lightly overconsolidated natural Boom clay taken at very high depth (220-240 m), the development of a stationary phase for deviatoric creep is observed at intermediate deviatoric stress levels (57-71 %). The extent of the stationary phase is not known for the time spans investigated for the cases where failure has not been observed. A bound below which only primary creep occurs has been found at deviatoric stress levels of 50 % Sect.2.3). The existence of a lower bound for creep strains occurence is reported at a deviatoric stress of 1.5 MPa.

Considering marls (Sect. 2.2), creep test results useful for modelling purpose are scarce. A lower limit for creep occurrence, as in rocks, may be present, but is variable among these materials due to the influence of the nature and actual conditions of the clay fraction. The presence of water seems to affect it significantly. Water is expected to weaken the bonds contributing to the cohesion of the material.

Marls taken from a very high depth (300 to 1150 m) have been reported not to show practically any creep strains when tested in laboratory. They did, however, when tested saturated.
The behavior observed may be dependent on the stress conditions at which they were tested, which were not specified. The difference in stress conditions (e.g. confining stresses) compared to those occuring in situ could result in significant differences for the envisaged application.

6.2 Data from in situ measurements

In situ measurements regard mainly the convergence of tunnel liners. Here the overall response of the tunnel is measured and the ingredients of the time dependent response are extremely difficult to identify.

In any case, this type of measurements are to be referred to a particular liner-soil system rather than to the soil itself. For this reason, these measurements cannot be easily used to fit a model to be used for a sensitivity analysis.

Observations are limited in time and related material models are often referred to the gallery construction phase.

In saturated clay soils, time-dependent results obtained during the first decades are mainly due to the time-dependent effect of dissipation of the excess pore water pressures due to the excavation. These effects are fairly well understood and modeled in the coupled approach of the theory of consolidation. Other relevant time-dependent effects in some clays having active minerals are attributed to electro-chemical swelling enhanced by the mechanical swelling due to the excavation (39). Some time-dependency of this effect can be attributed to availability of water allowing swelling to occur.

Considering now soft rocks, time response due to water flux is negligible,if at all present, with respect to the immediate elasto-plastic and to creep response. Deviatoric stresses induced by excavation are accomodated mainly by these last types of deformation until a stress level is attained at which no creep develops.

In mud rocks, particularly at high stresses, this limit was reported to exist. No data on the value of this level were found in the literature search. However, it should be considered whether its value remains constant in mud rocks over a very long time span, due to potential environmental variations leading to physico-chemical changes of the clay fraction.

6.3 Considerations on the state of the art of numerical modeling of creep effects

The available conceptual models for creep response which were found for clay materials follow two distinct approaches. The effective stress approach is used to explain saturated clay soils behavior; the total stress approach is applied mainly to rocks. The difference between them consists in the consideration of the role of water in the materials response to loading. The effective stress approach considers saturated porous clays as two-phase materials, the total stress approach as a monophasic material.

In saturated clay soils, the effective stress approach accounts for the short term (decades) time dependency of the response to the applied loads at a continuum level, (consolidation). To study creep within this approach the experimental evidence of time dependent behavior in laboratory tests should be gained in drained tests in order to detect the time-dependent effects on the clay skeleton. Undrained tests should be used then to complement the information for the water-solid system. Undrained tests are performed at a constant load and not at constant effective stress: the time dependent effect should not be considered creep in a strict sense.

Data that could be used within the framework of this approach are not available at the present time as explained in Section 2.1.2. Constitutive models have been developed to describe the available time-dependent phenomenological results (mainly undrained deviatoric tests) and in some cases included in effective stress oriented mathematical models. The experimental background information however is not complete.

The use of these models has been restricted to the study of laboratory test results and of specific boundary value problems, in which the hydraulic and stress boundary conditions of the routine laboratory tests (secondary compression in drained conditions) are applicable.

These models describe typically the primary creep effect which is dominant for softer clays with high water content in the time range of interest for conventional geotechnical applications. When included in a visco-plastic mathematical model capable of predicting failure, also the undrained creep failure of normally consolidated clays can be modeled. Undrained creep failure is not a relevant phenomenon for this application; in fact the time periods considered in this study make the material response develop in drained conditions.
The response of these models for overconsolidated clays is moreover doubtful, because they should imply the presence of viscous effects in the elastic stress region.

A considerable amount of experimental results on normally consolidated, soft saturated clays is available : some of them have been selected in Section 2.2. They pertain to some natural clays of different plasticity. The data provided could enable sensitivity analyses with the use of different types of effective stress based models.

The data, however, are related to a phenomenology occurring at low stress levels.

The available results for rocks point to the adequacy of a monophasic material approach. Creep response of the medium is due mainly to that of the crystalline structure and to material dishomogeneities.

A few rheological models discussed in the technical literature follow the monophasic approach also for saturated clays. They, in some sense, could be considered consistent with the boundary conditions adopted for the majority of available deviatoric laboratory results.

A threshold for viscous effects is present, and, in rocks, is assumed to coincide with that of peak resistance. The existence of a lower bound for creep strain occurence different from the peak resistance has been modeled in the case of Boom clay.

It is doubtful how far these models can be adapted to stiff saturated clays. In general, when applied to saturated clays, the extrapolation of data interpreted in terms of rheologic monophasic models to long times is complicated by the implicit ignorance of the role of the liquid phase on the stress evolution.

6.4 Conclusions and design oriented recommendations

From the previous discussion it follows that the quality of creep data available appears to be scarce and probably inadequate to support refined numerical models to run a sensitivity analysis of the problem to be investigated.

This inadequacy can be considered intrinsic in the capabilities of creep measuring techniques equipment.

Laws introducing time directly as a variable are difficult to validate experimentally. The approach aiming at establishing a unique relationship between stress-strain-strain rate should overcome this difficulty. For the problem to be investigated here strain rates of interest could be very low and again significance of experimental results can be questionable considering available testing apparatuses which should work with a very fine resolution. Actually, conventional laboratory tests do not provide resolution at the very low strain rates of interest.

Strain rates of interest for design life greater than 50 years may be of the order of 0.1 percent per year, or less: in the case of Boom clay, they were estimated to be not greater than $10^{-6} \div 10^{-5}$ /year. For specimens of length 7.5 cm, conventional for triaxial soil clay samples, 0.1 of strain rate percent would require resolution at strain rates of less than 2×10^{-5} cm per day, or 10^{-6} cm per hour in laboratory testing. Such slow strain rates are not feasible with conventional equipment.

In order to predict the influence of creep induced strains on the response of the material surrounding a tunnel for time spans ranging from 10^3 to 10^4 years, the way to proceed is via the understanding of the fundamental mechanisms of the time dependent response of the material.

In fact, no data fitting can be done here by means of extrapolation of short term behavior.
This is done for the time spans of interest of conventional engineering projects (tens of years) with the help of in situ measurements.
Predictions of laboratory calibrated constitutive models are compared with in situ measurements. Indeed, the observed trends of the in situ measurements can give confidence in the extrapolation to the time spans of engineering interest.

The research should be thus first oriented at understanding microstructural mechanisms responsible for time-dependent response in clays. In clay rocks emphasis should be given on the influence of the clay fraction. Their understanding should help in designing tests representative for very long time spans.

This fundamental research should also try to establish the mutual role of creep and plasticity, the role of pore water and of clay mineralogy.

It should be also remembered that clays are materials which can modify their structure and, consequently, their response to the applied loads due to physico-chemical changes occurring in the soil mass. Particle contacts may become cemented, or welded, double layer thickness may change and other chemical processes may take place. Most of these changes tend to harden, or stiffen the sample, reducing strain rates and deformations. Scenarios involving factors capable to cause modifications should be identified with reference to site conditions and characteristics of the engineering project.

Considering the clay as not affected by chemical changes, a possible way to proceed to obtain a shortened time-scale could be devised in which analogy tests are run, where well defined creep microstructural phenomena are accelerated due to induced calibrated chemical processes.

From an engineering point of view, it is important to establish at least if a limit for creep occurence exists and how much it is depending on stress, temperature and other environmental conditions (chemical agents).

The existence of a stress limit in clay soils below which no creep strains occur seems to be dependent to a high degree on the presence of water.
If a limit is observed, it will probably be too low not only in saturated normally consolidated clay soils, but also in saturated stiff and lightly overconsolidated clays, to allow the designer to rely.on a load transfer which significantly reduces the values of stress on the liner in the very long term with respect to the in situ state of stress.

It seems worthwhile to undertake research in the form of a test campaign for the case of clay rocks where a reduction on the long term pressure against the tunnel lining with respect to the lithostatic state of stress could be realistic.

The creep limit is often considered as coincident with the rupture limit in rock excavated tunnel practice. Following this approach creep is attributed to changes in the microstructure of the material which has experienced the peak strength conditions. This approach has been adopted to estimate the ultimate response of rocks and also of clay rocks (61).
Following this approach ultimate loads have been predicted to exceed 40% of the in situ original pressure in soft clay rocks even under the hypothesis that the tunnel is excavated above the water table.
It should be considered, however, that the former hypothesis is highly unlikely.

The presence of the clay fraction could contribute to lower the creep limit. The sensitivity of the clay fraction to environmental conditions should be evaluated in the laboratory tests aimed at assessing the creep limit for analyses of rock pressure at very long time.

The investigation on the creep limit dependence should be performed within a range of confining

pressures encompassing the in situ values. The different confinement occurring in situ with respect to that adopted in the laboratory is judged to be one of the causes for the differences in the measurements of time-dependent response.

In parallel, a promising way to evaluate the long-term shear stress resistance of materials is looking at the shear stresses existing in situ. The lesser the deviation from the isotropic state of stress the greater the creep sensitivity of the material. This way to look at the problem should be supported by a study on the conditions of clay deposit formation and of the effects of previous and progressing geologic phenomena.

A measure of the capability to sustain a long term shear stress can be provided then by investigations of the in situ state of stress.

Considering the behavior of the lining, physical characteristics degradation leading to alteration of mechanical properties with time need to be evaluated if integrity over 10^4 years has to be guaranteed. It is then proposed to make assessments on lining performance on the basis of appropriate liner stiffness characteristics obtained with appropriate laboratory tests on artificially aged liner material.

FIGURES

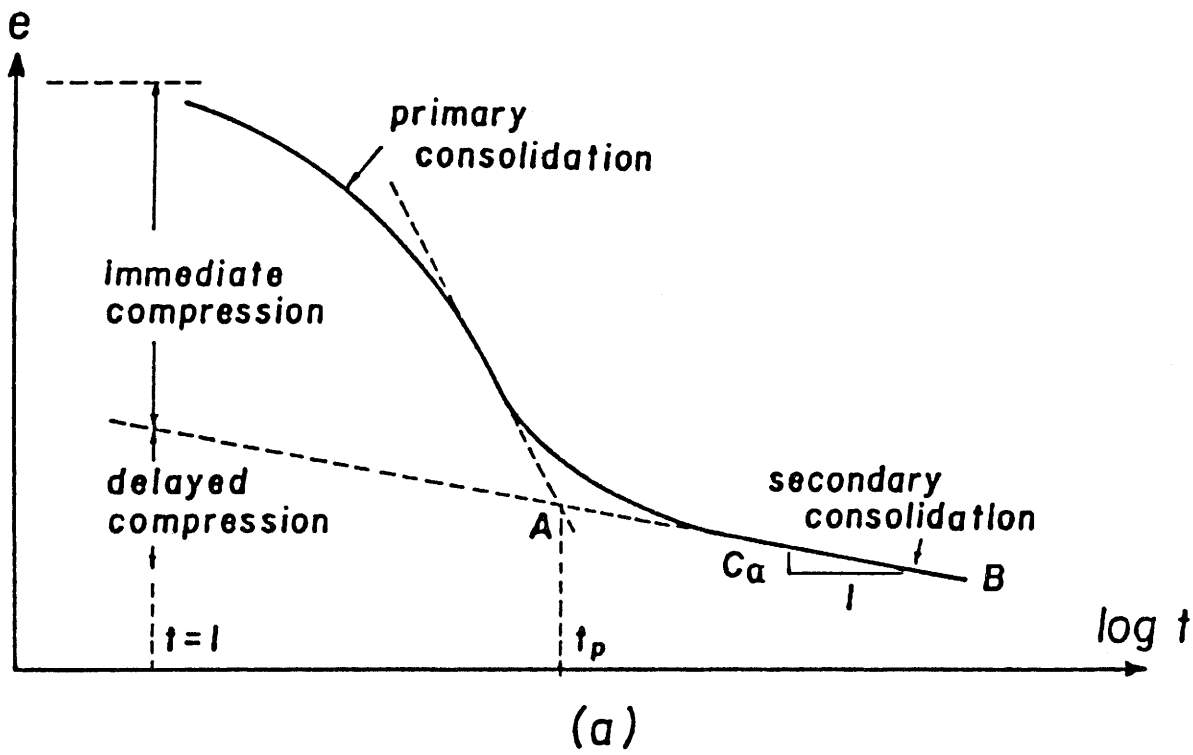


FIGURE 1 Taylor's Secondary Compression Law

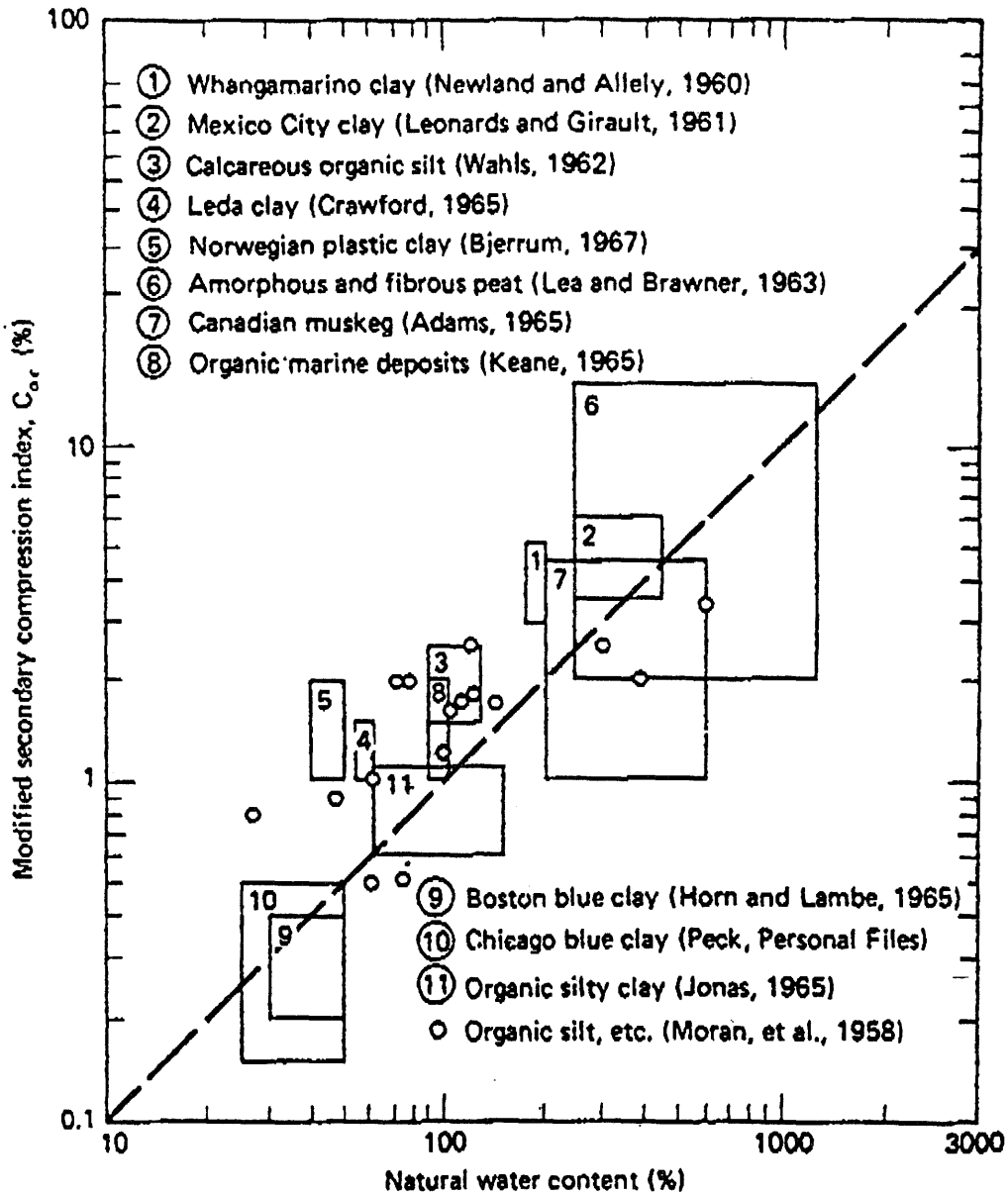
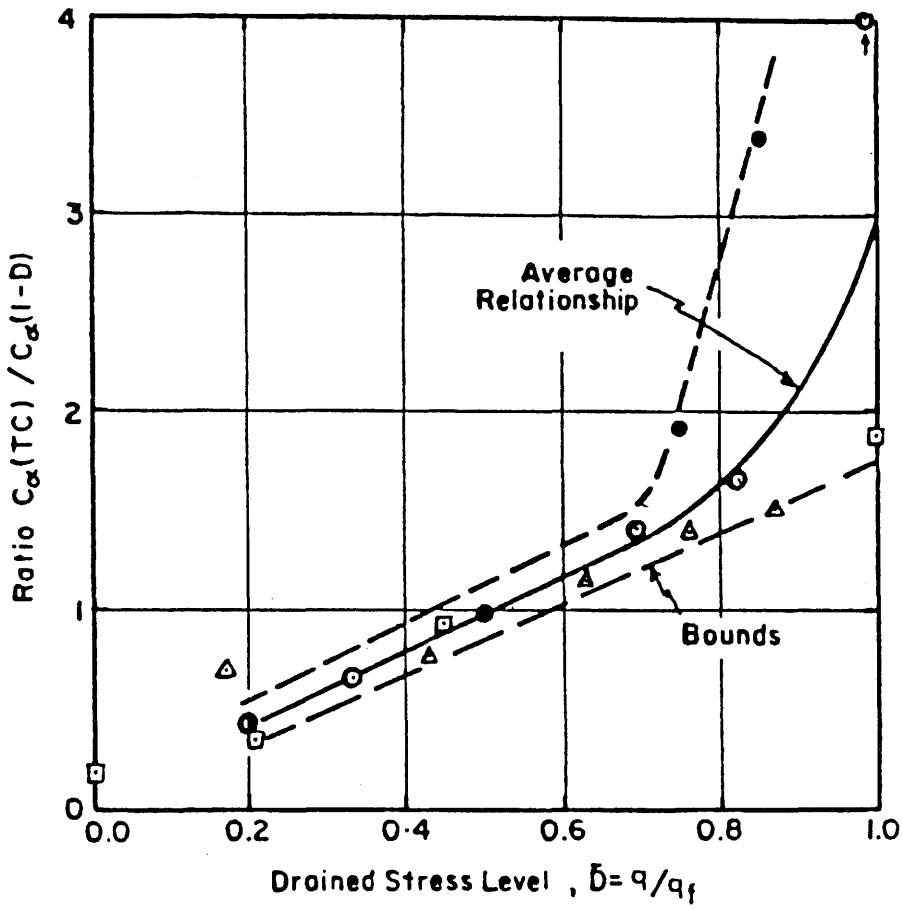


FIGURE 2 Natural Water Content-Secondary Compression Relationship (92)



SYMBOL	RESEARCHERS	CLAY	PI,%
⊡	Barden (1969)	Remoulded Frodsham Clay *	—
●	Bishop and Lovenbury (1969)	Normally Consolidated Pancone Clay (Pisa)	47
△	Bishop and Lovenbury (1969)	Overconsolidated London Clay (Hendon)	47
⊙	Murayama and Shibata (1961)	Normally Consolidated Osaka Clay *	43

* refers to an assumed drained creep strength, q_f

Note: Data from drained triaxial compression tests.
Figure based on assumption that C_α(1-D Compression)=C_α (TC) at drained stress level $\bar{D}$=0.5.

VARIATION OF "C_α" WITH DRAINED STRESS LEVEL

FIGURE 3 Influence of Stress Level on Secondary Compression (43)

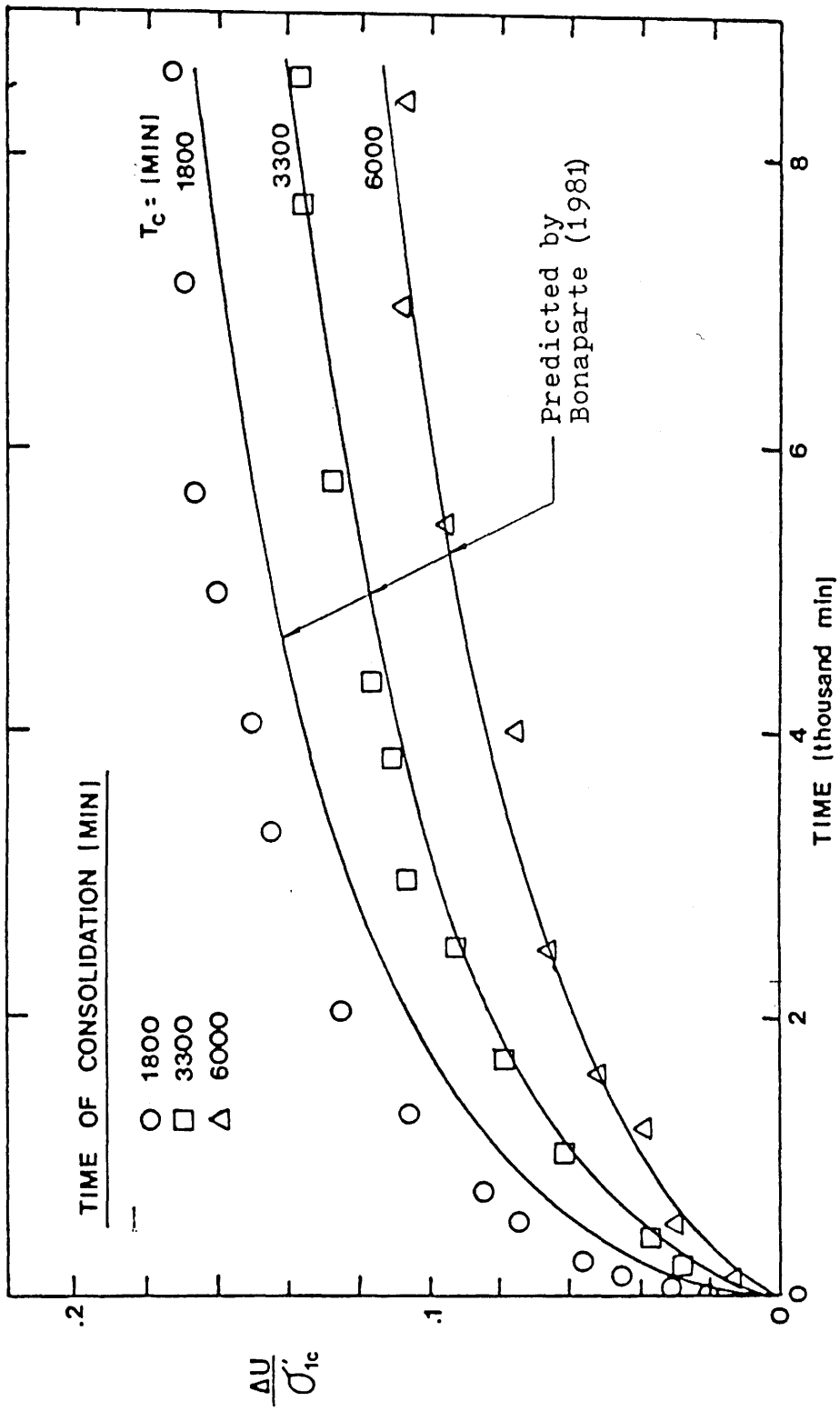


FIGURE 4 Pore Pressure Due to Arresting of Isotropic Secondary Compression (20)

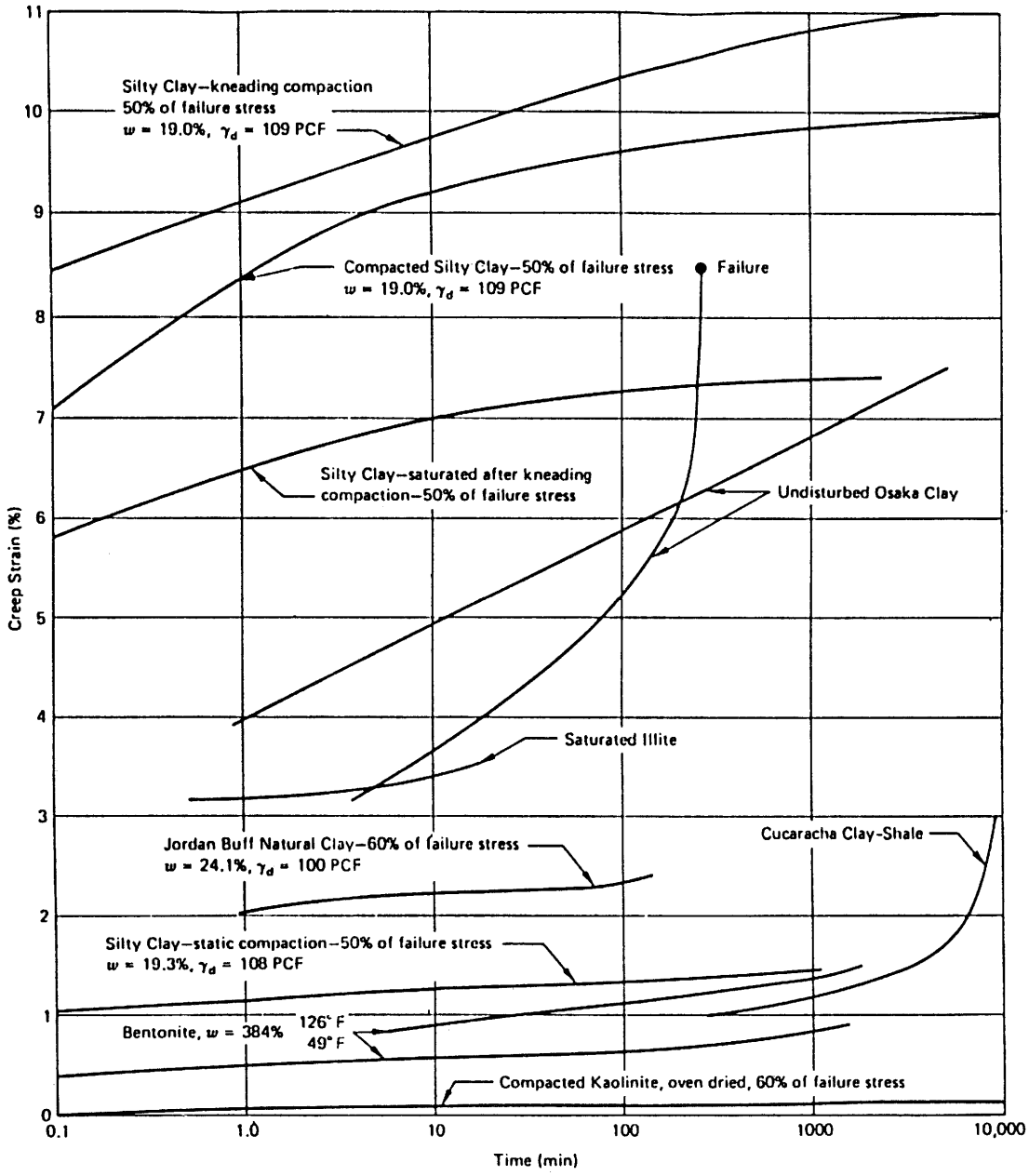


FIGURE 5 Creep Behavior of Natural Soils (98)

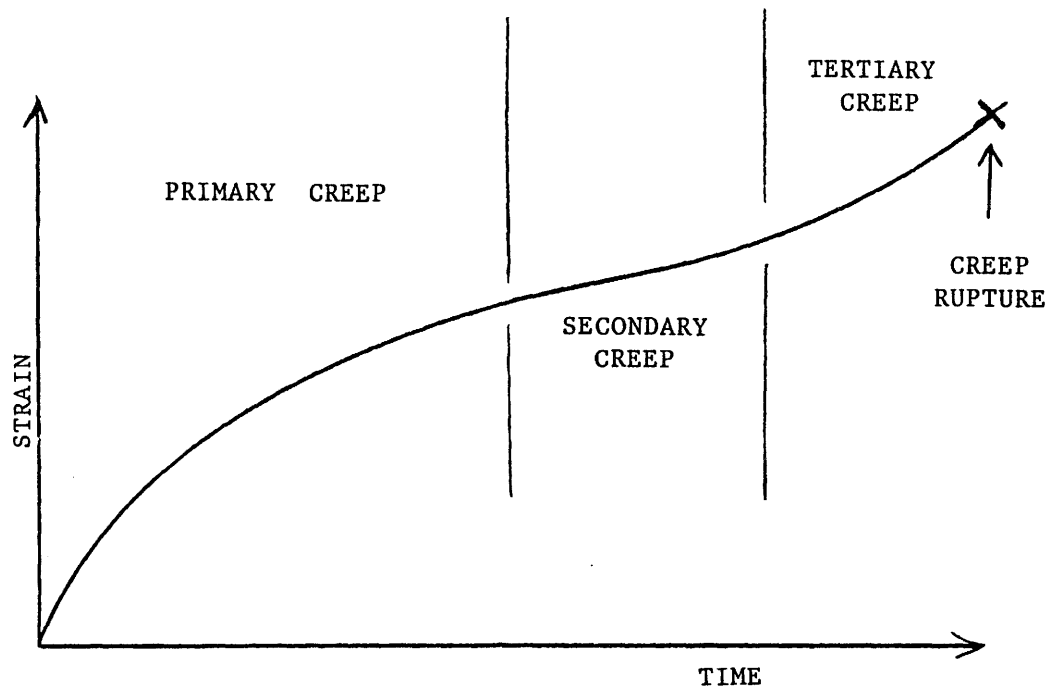


FIGURE 6 The Three Stages of Deviatoric Creep

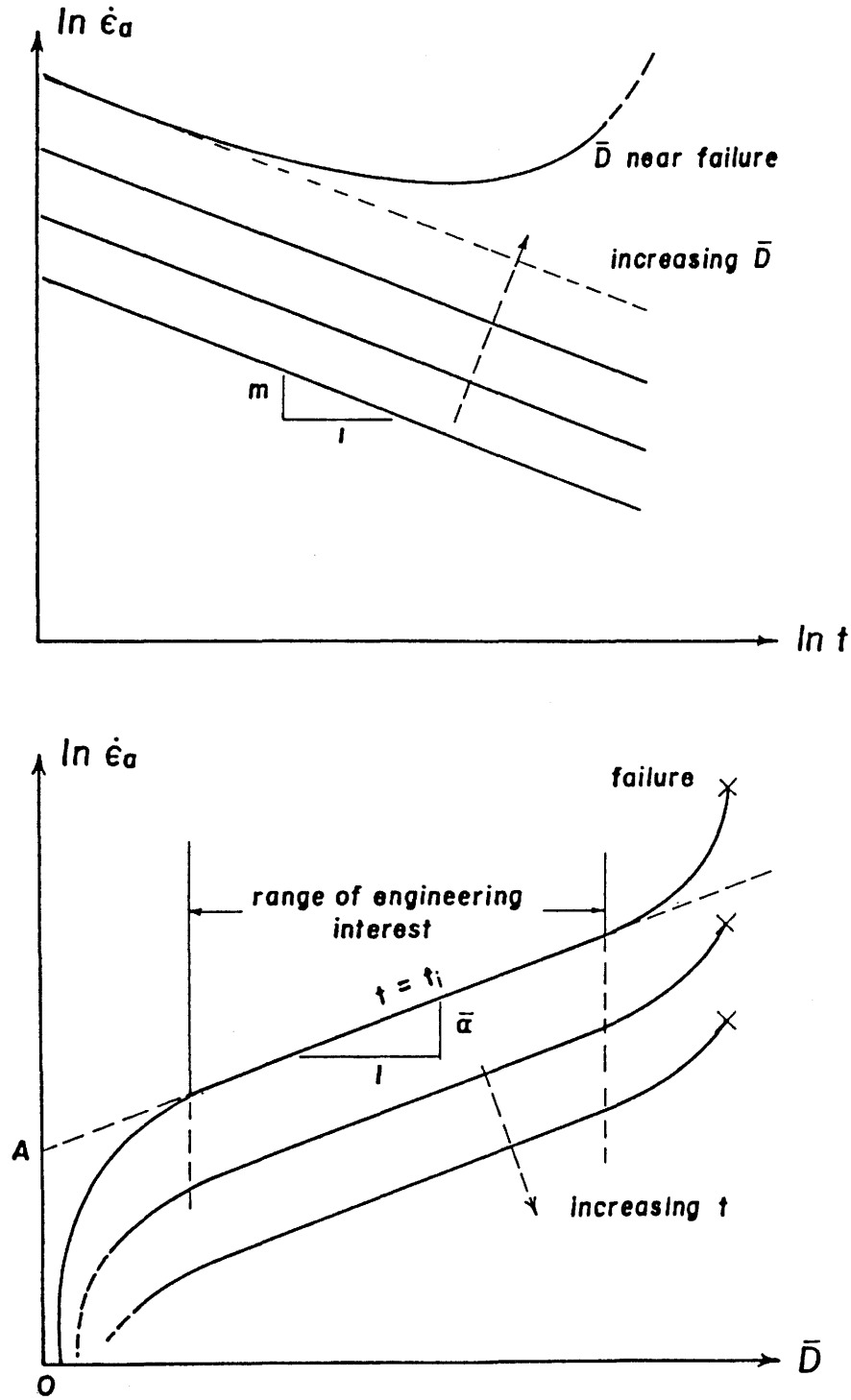
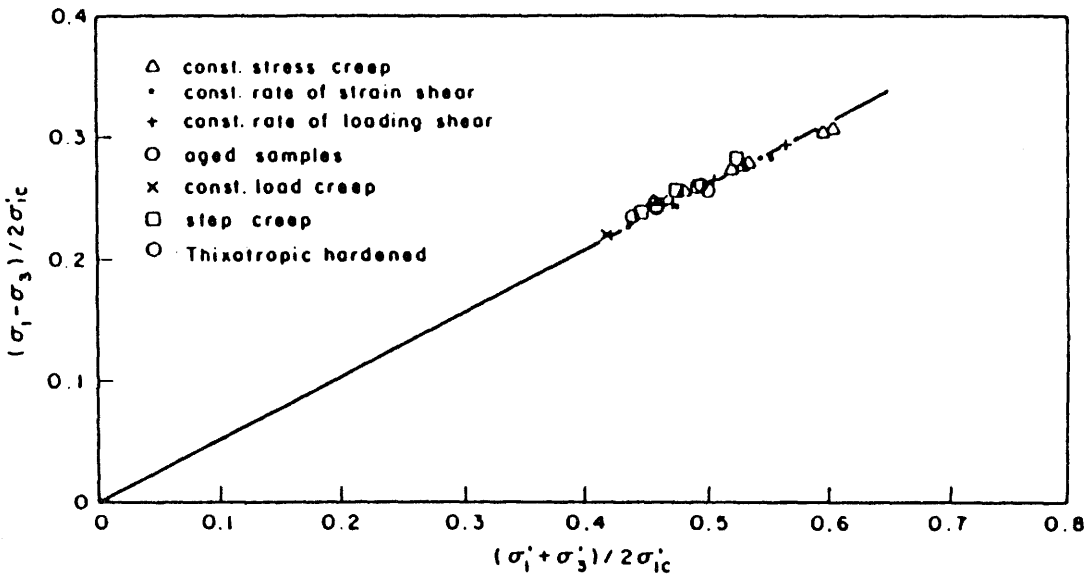


FIGURE 7 Influence of Stress Level on Deviatoric Creep



Stress Conditions at $(\sigma'_1/\sigma'_3)_{max}$

FIGURE 8 Mohr-Coulomb Criteria for Creep Rupture (28)

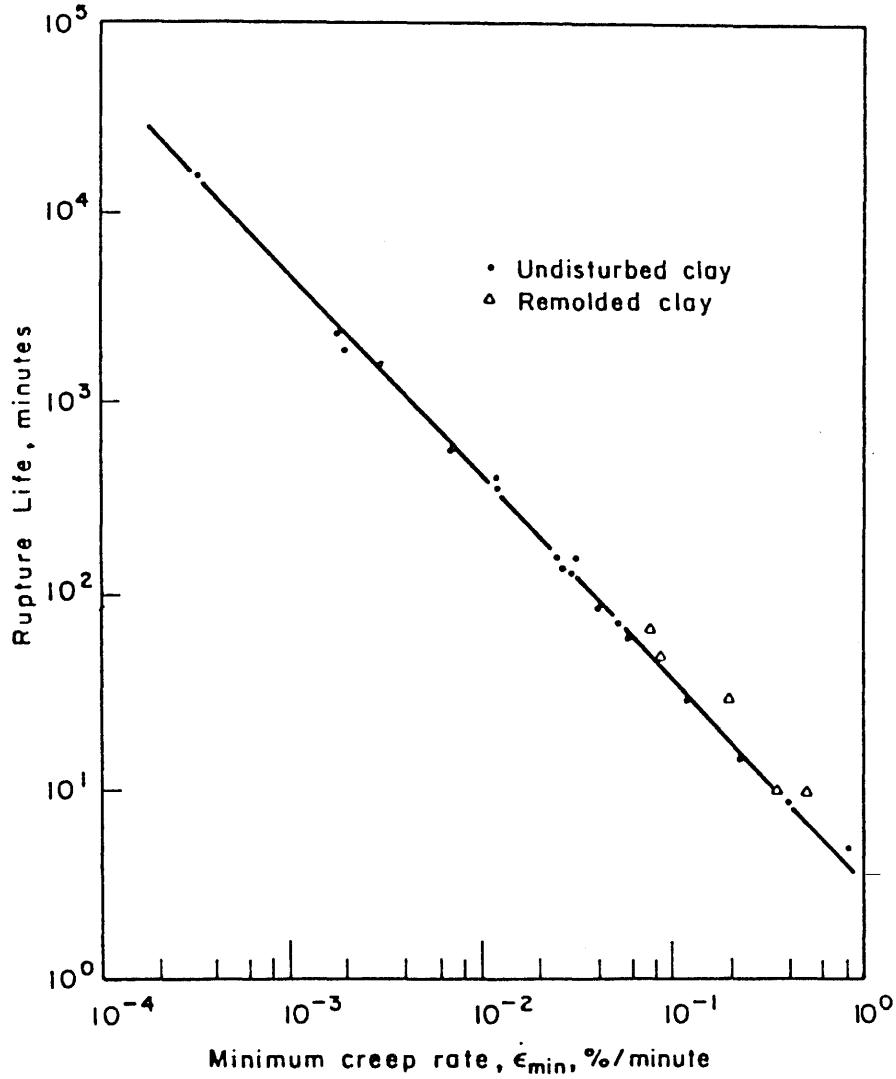
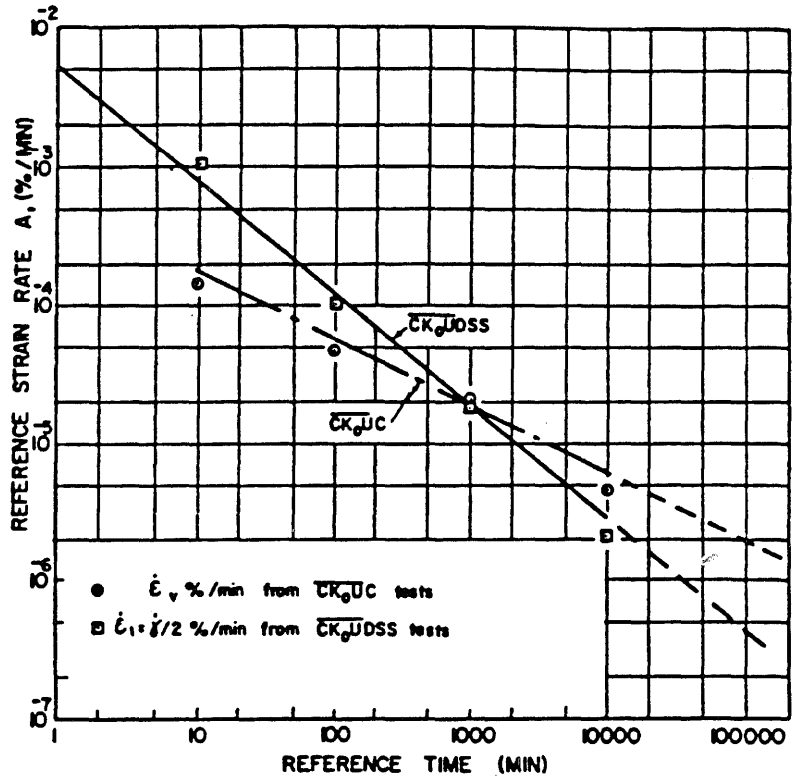
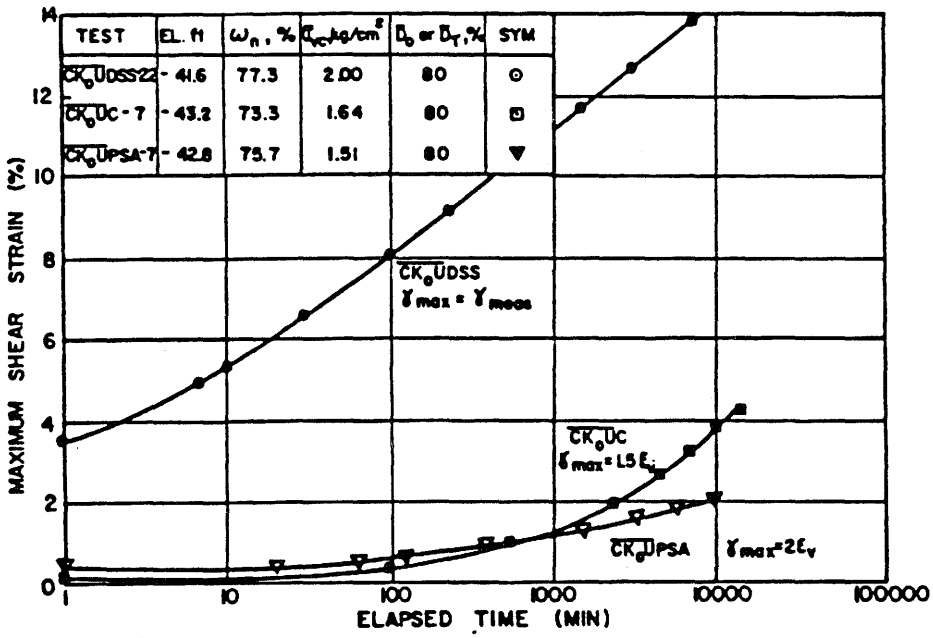


FIGURE 9 Minimum Creep Rate-Rupture Life Relationship (155)



a.



b.

FIGURE 10 Undrained Creep Behavior of Atchafalaya Clay (43)

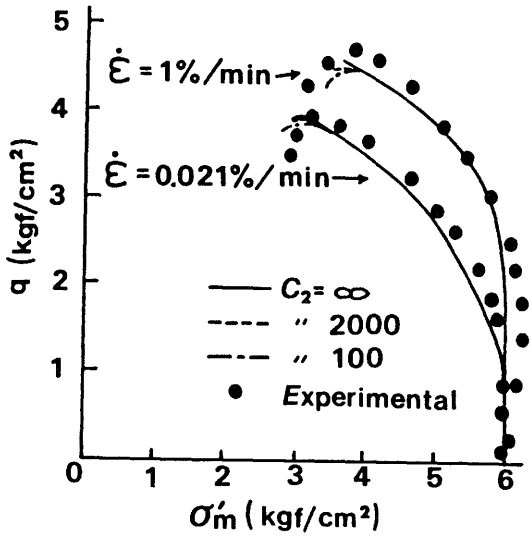


FIGURE 11a Stress paths for clay (3)

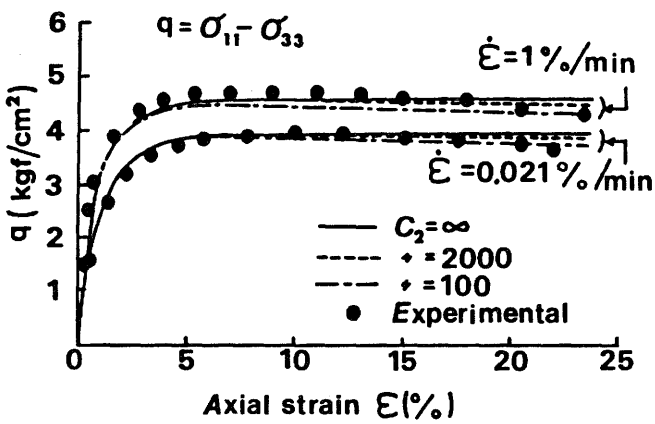


FIGURE 11b Stress-strain curves for clay (3)

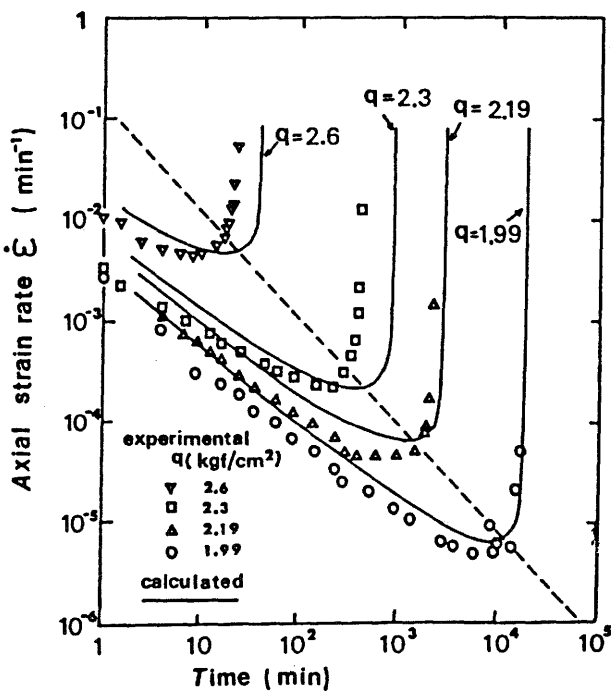


FIGURE 11c Variation in the axial strain rate with time (3)

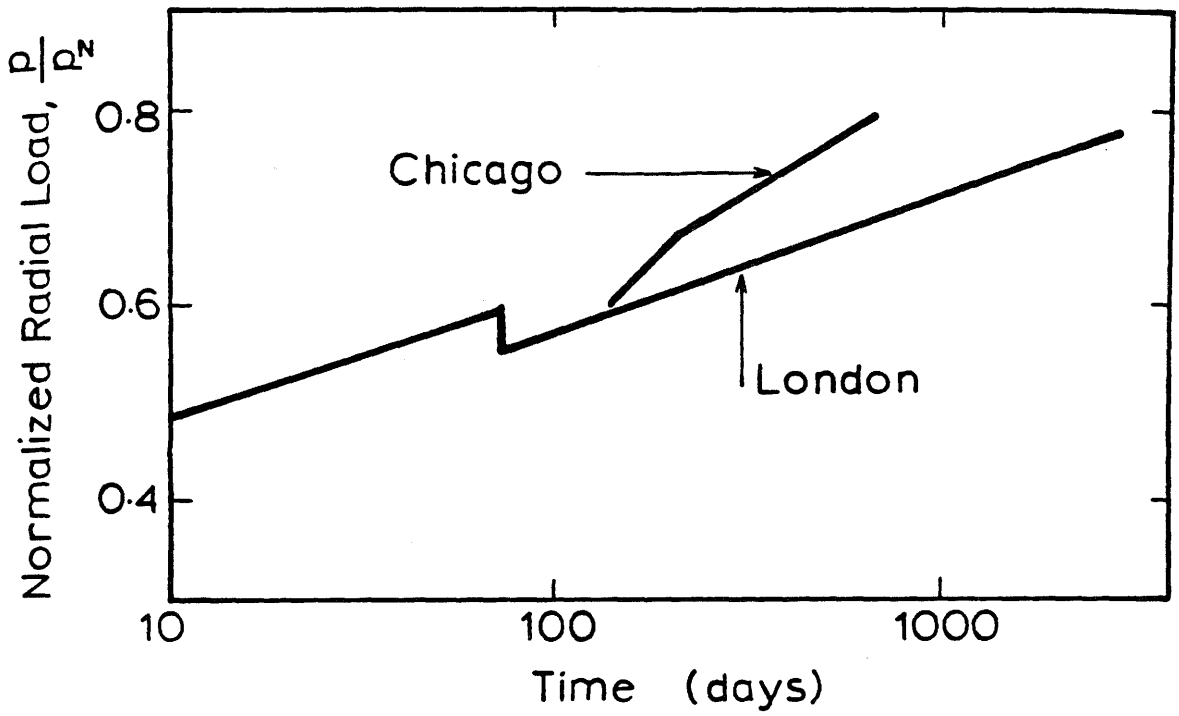


FIGURE 12a Variation of Radial Load on the Liner with Time (62)

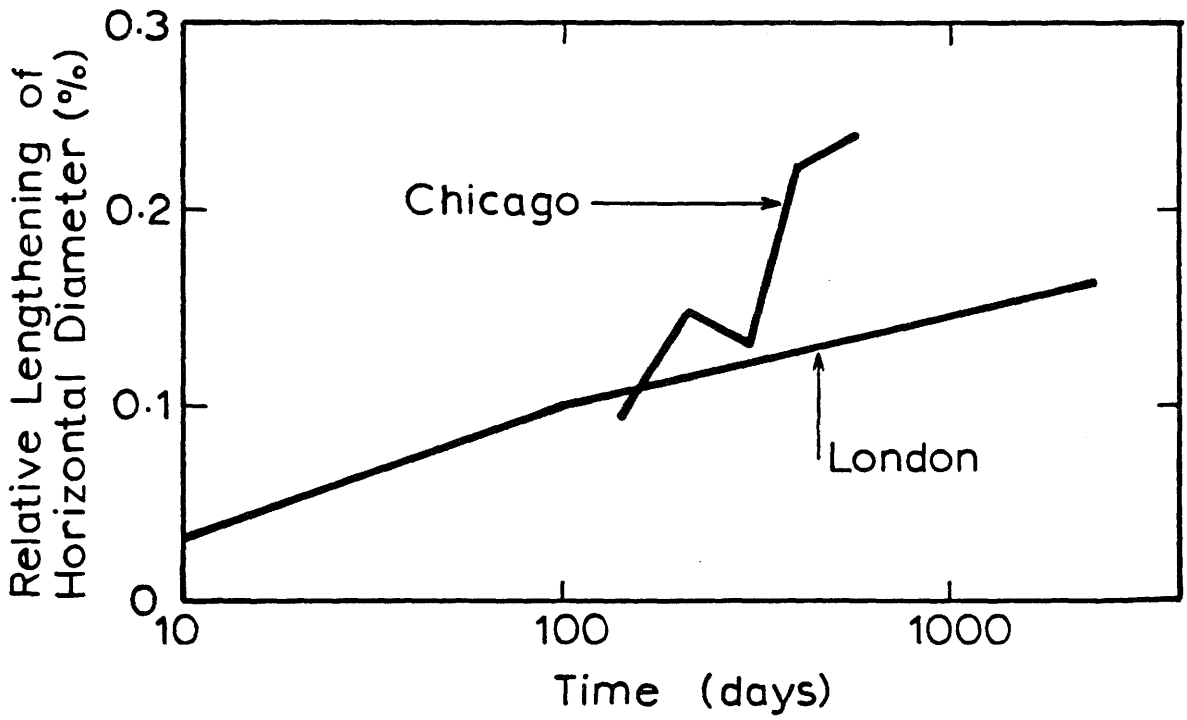


FIGURE 12b Variation of Horizontal Diameter with Time (62)

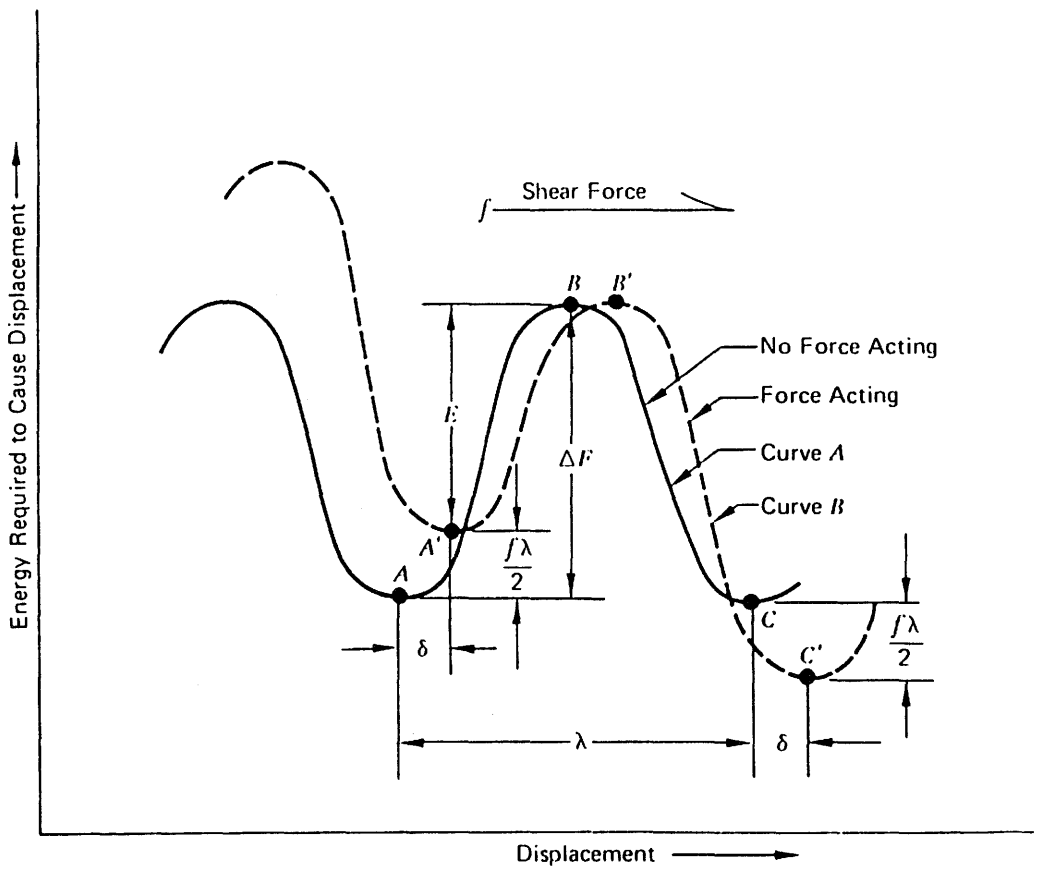


FIGURE 13 Rate Process Deformation Theory (98)

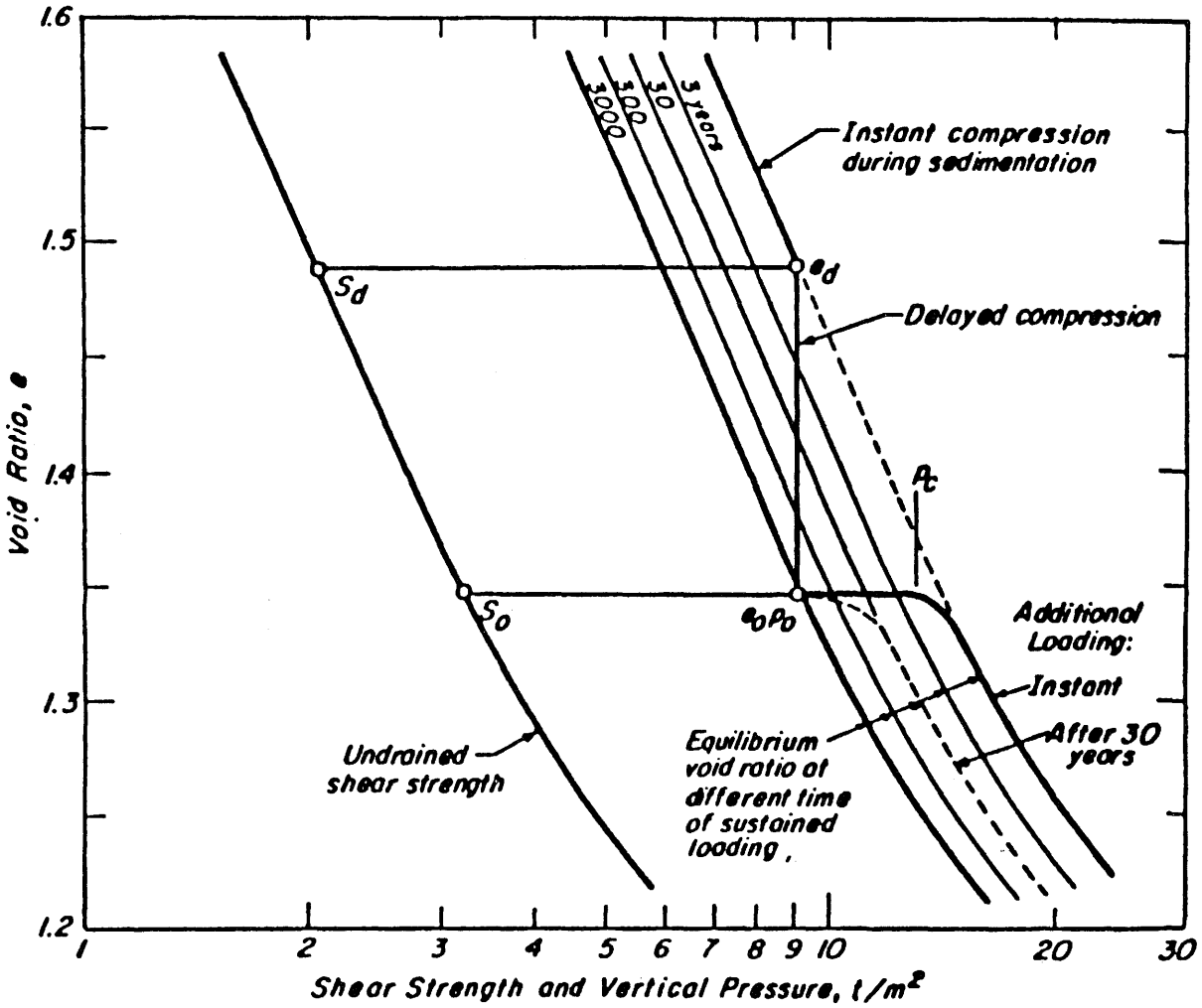
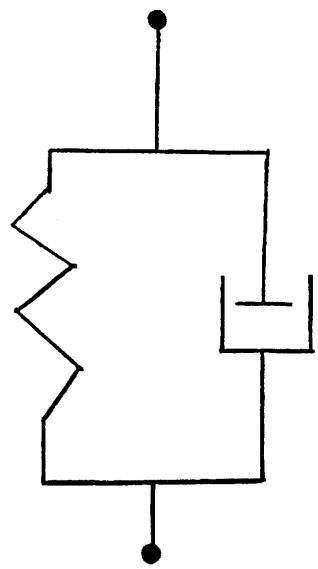
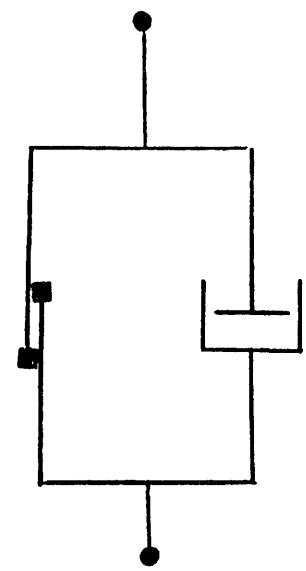


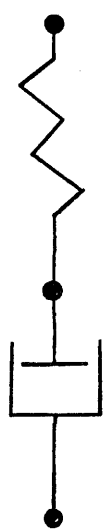
FIGURE 14 Bjerrum's One-Dimensional Compression Model (19)



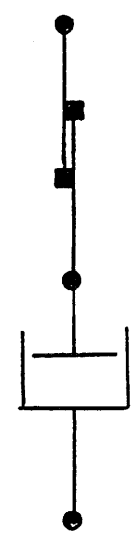
VISCO - ELASTIC



VISCO - PLASTIC



ELASTIC - VISCOUS



PLASTIC - VISCOUS

FIGURE 15 Rheological Models for Viscous Behavior

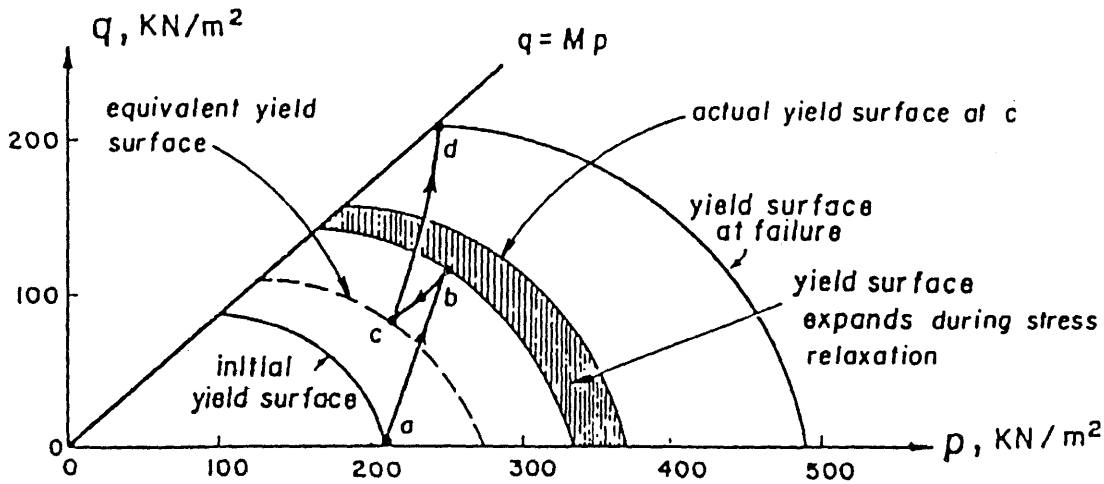


FIGURE 16 Yield Surface Expansion Due to Creep (24)

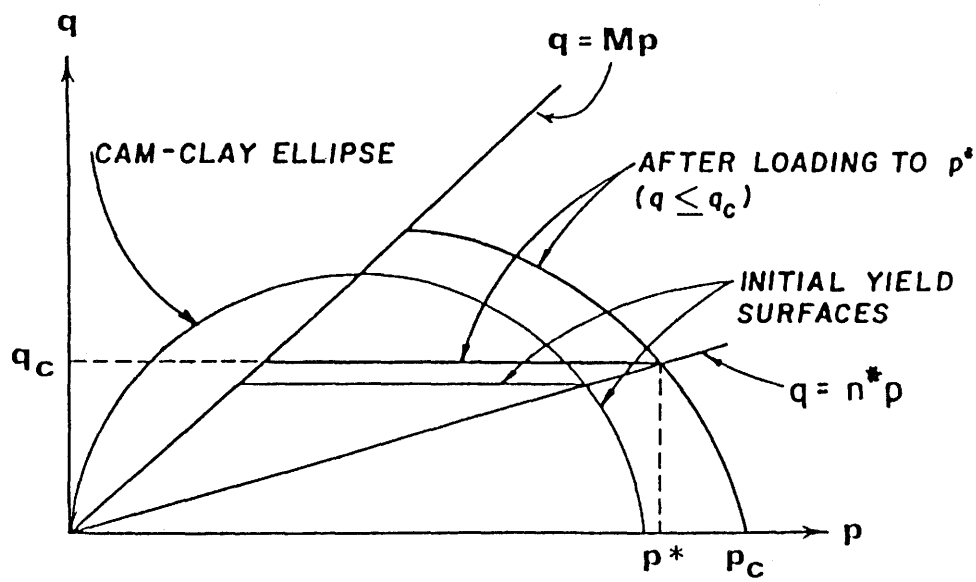


FIGURE 17 Double Yield Surface Plasticity Model (67)

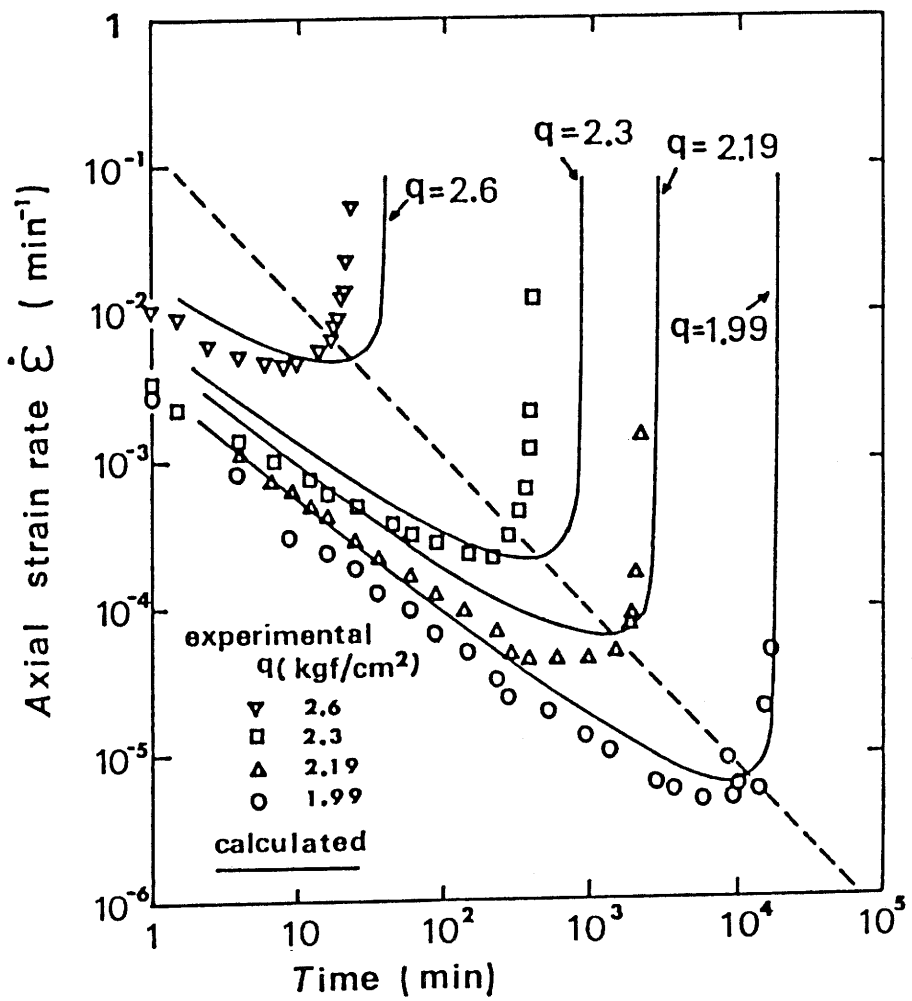


FIGURE 18 Adachi and Oka's Visco-Plastic Creep Model (111)

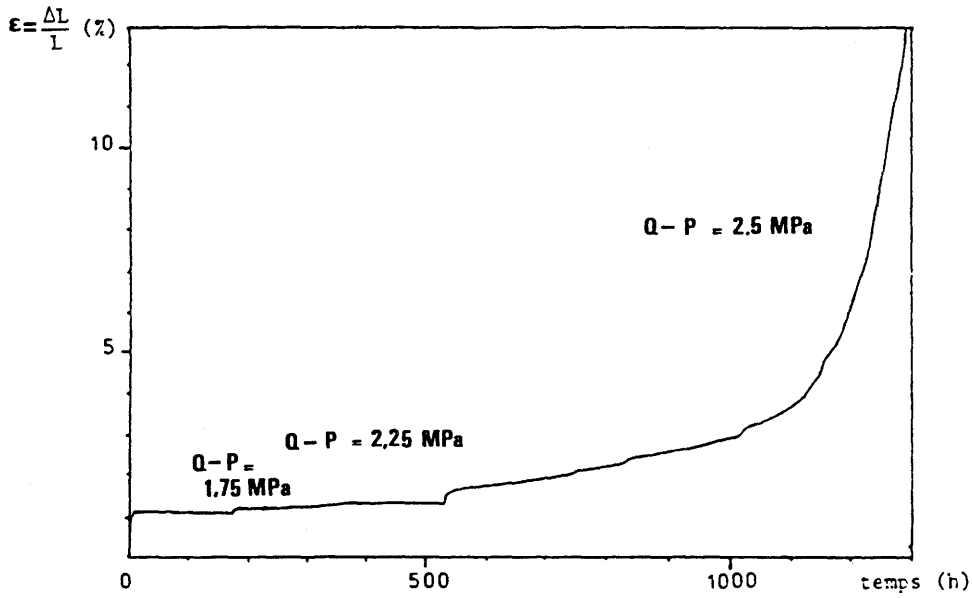


FIGURE 19 Axial strains in a triaxial creep test different deviator stress levels on Boom clay (126); p= 5 MPa.

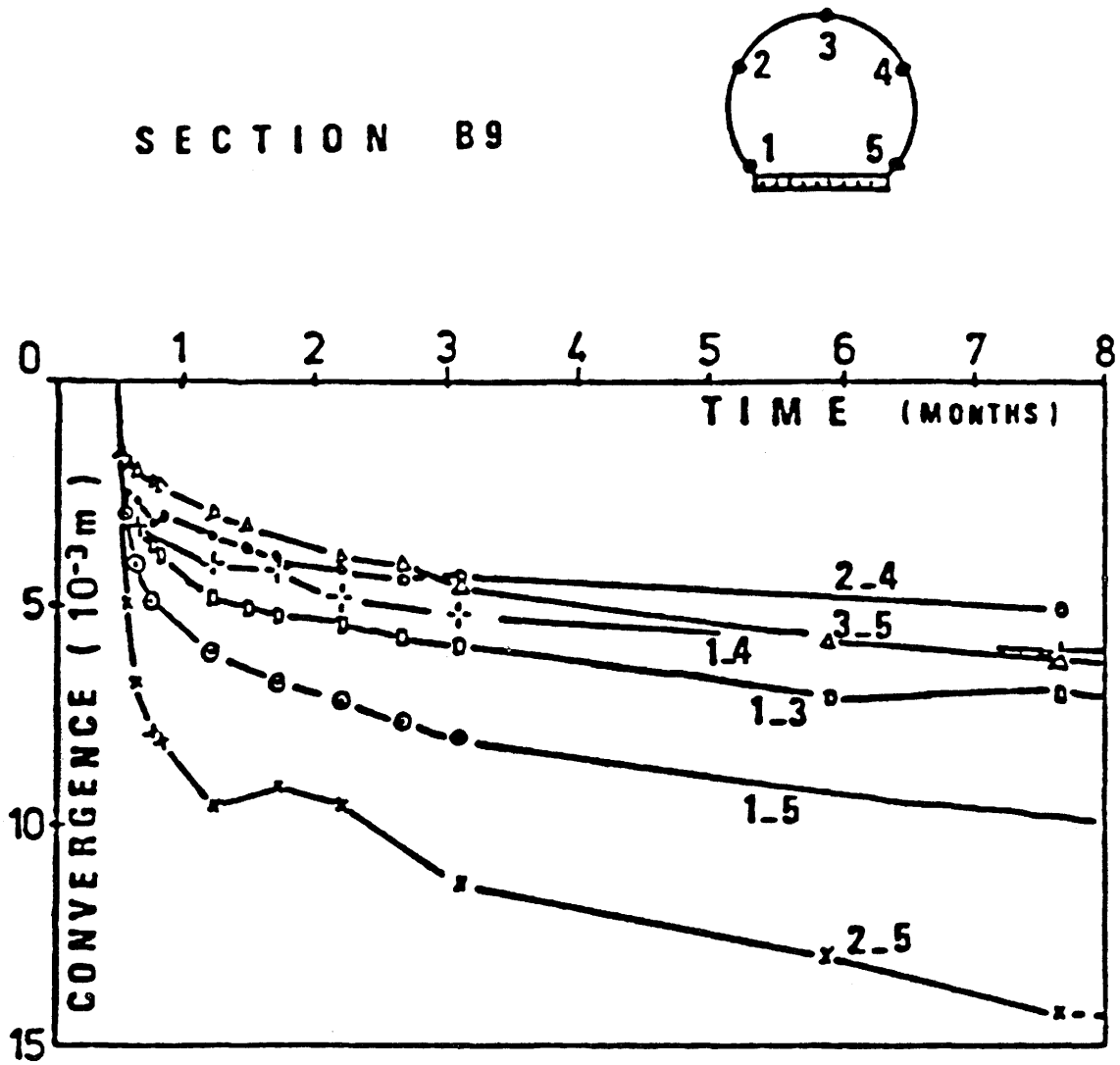


FIGURE 20 Convergence versus time in the Chamoise tunnel (106)

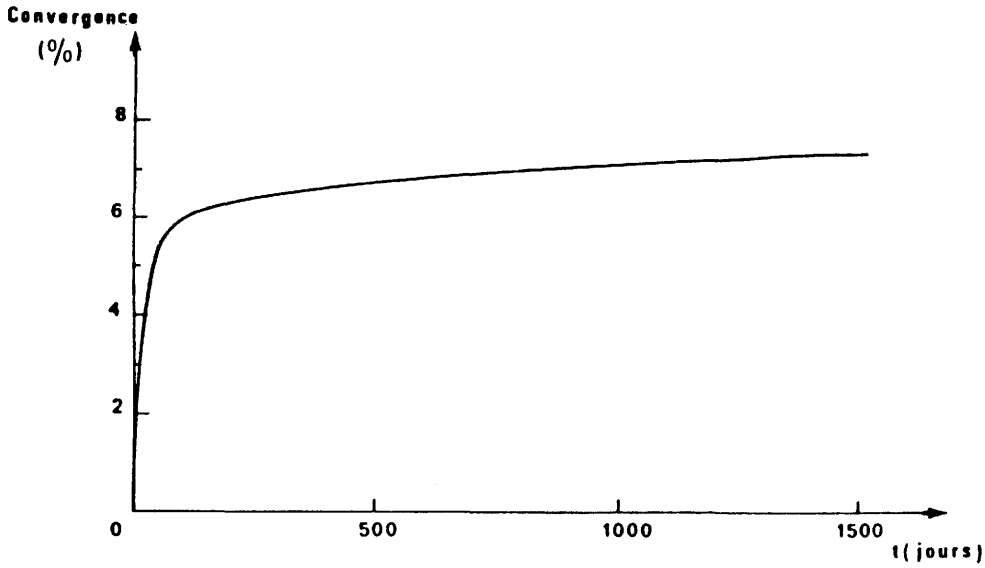


FIGURE 21 Convergence of the gallery in Boom clay; extrapolation (126)

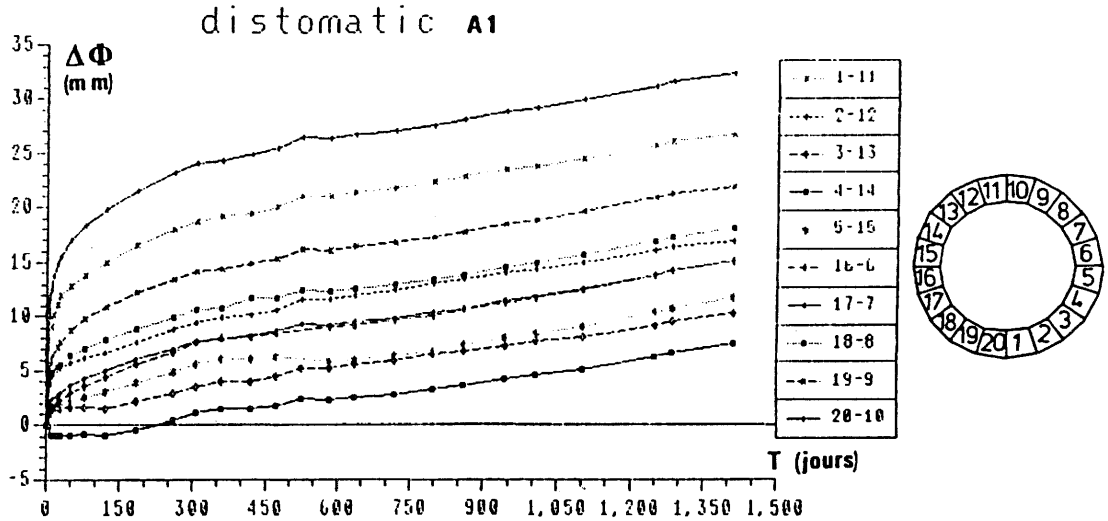


FIGURE 22 Convergence of the lining of the gallery in Boom clay (126)

TABLES

Soil	C_α/C_c	Reference
Whangamarino clay	0.03÷0.04	Newland and Allely (22)
Norfolk organic silt	0.05	Barber (2)
Calcareous organic silt	0.035÷0.06	Wahls (28)
Amorphous and fibrous peat	0.035÷0.083	Lea and Brawner (13)
Canadian muskeg	0.09÷0.10	Adams (1)
Leda clay	0.03÷0.055	Walker and Raymond (31)
Leda clay	0.04÷0.06	Walker and Raymond (32)
Peat	0.075÷0.085	Weber (33)
Post-glacial organic clay	0.05÷0.07	Chang (9)
Soft blue clay	0.026	Crawford and Sutherland (11)
Organic clays and silts	0.04÷0.06	Ladd (12)
Sensitive clay. Portland	0.025÷0.055	Ladd (12)
Peat	0.05÷0.08	Samson and La Rochelle (25)
San Francisco Bay mud	0.04÷0.06	Su and Prysock (26)
New Liskeard varved clay	0.03÷0.06	Quigley and Ogunbadejo (23)
Silty clay C	0.032	Samson and Garneau (24)
Nearshore clays and silts	0.055÷0.075	Brown and Rashid (6)
Fibrous peat	0.06÷0.085	Berry and Vickers (3)
Mexico City clay	0.03÷0.035	Mesri, et al. (19)
Hudson River silt	0.03÷0.06	Mesri, Personal files
Leda clay	0.025÷0.04	Mesri, Godlewski
New Haven organic clay silt	0.04÷0.075	Mesri, Godlewski

Table 1. VALUES OF C_α/C_c FOR NATURAL SOIL DEPOSITS (96)

Virgin Compression Index	C_c =	0.45÷0.58
Recompression Index,	C_R =	0.08
Coefficient of secondary Compression,	C_α =	0.003÷0.01
Hyperbolic Stress-Strain Parameter,	a =	0.02÷0.04
Hyperbolic Stress-Strain Parameter,	b =	2.04
Hyperbolic Failure Ratio,	Rc =	1.0
Singh-Mitchell Creep Parameters	A =	0.00043
	$\bar{\alpha}$ =	2.0
	m =	0.8÷0.9
Slope of the Critical State Line,	M =	0.88

where

$$a = \frac{\sigma'_c}{E_i} \; ; \quad b = \frac{R_f \sigma'_c}{(\sigma_1 - \sigma_3)_f} \; ; \quad R_c = \frac{(\sigma_1 - \sigma_3)_{ULT}}{(\sigma_1 - \sigma_3)_f} \; ; \quad M = \frac{6 \sin \varphi}{3 - \sin \varphi}$$

Table 2 MATERIAL PROPERTIES FOR ATCHAFALAYA CLAYS (63)

Property	Value
Thickness of Deposit	16.764÷17.678 m
Depth of Water Table	0.000÷3.658 m
Winter Average	1.219÷1.829 m
Summer Average	2.438 m
Saturated Unit Weight	1505.692 kg/m^3
Natural Water Content (%)	90
Liquid Limit (%)	88
Plasticity Index (%)	40
Liquidity Index (%)	1
Organic Content (Total carbon - %)	1.5
Compression Index C_c	1.2÷1.8
Just past p'_p	(1.5~average)
For $p' > 2\text{-}3$ kg/cm^2	0.8÷0.9
Recompression Index C_r	0.10÷0.15
Coefficient of Secondary Compression C_α	0.01÷0.02
Coefficient of Consolidation, C_v	0.743÷0.929 m^2/year
Effective Stress Friction Angle, Φ'	
Consolidated undrained triaxial	32.5°÷35° (34° average)
Vertical Plane Strain C-U	38°
Horizontal Plane Strain C-U	35°
Consolidated Drained	31°
Undrained Strength Ratio $S_u/\bar{p}$	
IC-U	0.34
AC-U (K_o)	0.35
UU	0.32
Field Vane	0.31÷0.32
Cone Penetrometer	0.31÷0.32
Self Boring pressuremeter	0.40
IC-U Data Extrapolated (to an OCR of 1.1-1.5)	0.36÷0.42

Table 3 SUMMARY OF ENGINEERING PROPERTIES OF S. F. BAY MUD AT THE UNIVERSITY OF CALIFORNIA HAMILTON AIR FORCE BASE TEST SITE (21). Unit conversion made at ISMES.

Property	Value
Undrained Strength Ratio-Continued	
Vertical Plane Strain	0.29
Horizontal Plane Strain	0.37
Creep Parameters (Undrained Creep on CU Samples)	
$\bar{\alpha}$	4.45
A	0.0035%/min
m	0.7÷0.8
Overconsolidation Ratio	
Crust	3
Normally Consolidated Zone	1.0÷1.5
In-Situ Lateral Stresses	See Text
Modulus Multiplier M ($E = MS_u$)	See Text

Table 3 (Continued)
SUMMARY OF ENGINEERING PROPERTIES OF S.F. BAY MUD AT THE UNIVERSITY OF CALIFORNIA HAMILTON AIR FORCE BASE TEST SITE (21). Unit conversion made at ISMES.

Parameter	Symbol	Value
Virgin compression index	λ C_c	0.147 0.338
Recompression index	κ C_r	0.060 0.138
Secondary compression coefficient	ψ C_α	0.00119 0.00274
Hyperbolic stress-strain parameters	a b R_f	0.0062 2.728 0.90
Singh-Mitchell creep parameters	A α m	1.44×10^{-4} /day 2.475 0.642
Permeability components	kh kv	0.00113 m/day 0.00054 m/day
Slope of critical state line	M	1.05
Void ratio at p_c = 1 KPa	e_a	3.56
Instant volumetric time	$(t_v)_i$	1.00 day
Instant deviatoric time	$(t_d)_i$	1.00 day

Table 4 MODEL PARAMETERS FOR BOSTON BLUE CLAY (23)

Deviatoric stress Q (MPa)	Ratio $(Q-P)/(Q-P)_{MAX}$	
* < 1.5		No creep
* < 1.5 or 1.75	43% or 50%	Primary creep only
* 2 or 2.25	57% or 64%	Secondary creep in most cases moderate strains
* 2.5	71%	Secondary creep with high strain rate. Seldom tertiary creep or failure
* 3	86%	Secondary creep with very high strain rate or failure
* > 3		Immediate failure

P - Confining stress

Table 5 FEATURES OF TRIAXIAL CREEP TESTS ON BOOM CLAY (126)

Deviator (MPa) Test N.	1.5	1.75	2.0	2.25	2.5	2.75	3.0
1	0		2.2		21.7		566
2	0		0	*	58		
3			0	0	35		
			*				
4		0		14.7			
		*					
5		0		7.4		56	
6		*		*		*	*
		*		*		*	*
7			4.2				
8			0	5.8			
9				5.6			
10			4.9				
11			7.4				
12			0				
13					43		
14					16		
15					13		

Table 6 AVERAGE STATIONARY CREEP RATES ON BOOM CLAY (10^{-6} $hour^{-1}$) (126)

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