Evaluation of Time-Dependent Behavior of Soils
Anders Augustesen\textsuperscript{1}; Morten Liingaard\textsuperscript{2}; and Poul V. Lade, M.ASCE\textsuperscript{3}

Abstract: The time-dependent behavior of soils has been investigated extensively through one-dimensional and triaxial test conditions. Most of the observations in literature have focused on the determination of the time-dependent behavior of clayey soils, whereas the reported experimental studies of granular materials are few. This paper presents an up-to-date review of the various observed time- and rate-dependent phenomena that are known to exist for both clay and sand. The description is carried out separately for creep, stress relaxation, rate dependency, and structuration in laboratory experiments. All of the above-mentioned phenomena are present in both sand and clay. The time-dependent phenomena are more pronounced in clay than sand. However, sand exhibits relatively large deformations at high confining pressures because of grain crushing. Furthermore, the review revealed an essential characteristic situation for soils. That is whether the time-dependent behavior can be characterized as isochot or nonisochot. It seems that the isochot behavior is adequate for describing the time effects in clays in most situations. But for sand, the isochot description is inadequate. Further, the phenomenon of structuration plays a role in both clay and sand.

DOI: 10.1061/(ASCE)1532-3641(2004)4:3(137)

CE Database subject headings: Clays; Creep; Sand; Time dependence; Soil properties.

Introduction

From real construction projects, it is well known that clay exhibits rheological behavior, see, e.g., Crawford and Morrison (1996), but it is not widely accepted that sand shows considerable amounts of time-dependent behavior as well (Komornik et al. 1972; Hannink 1994; Leung et al. 1996). To model the observed time-dependent behavior of sand and clay, constitutive models must be developed based on laboratory tests.

Most of the past laboratory studies of the rheological behavior of soils have been focused on the characteristics of clayey soils. This is especially true for the studies of confined conditions where the tests have been entirely based on cohesive soils, such as clay and peat. The studies of rheological behavior of soils in connection with triaxial test conditions are usually carried out on clays, but in the last few years increasing attention has been paid to experimental research into the behavior of granular materials. Sand, which is generally considered as nonviscous, exhibits time-dependent behavior. Experimental results show that the creep strains are not negligible and can reach 10% of the monotonic loading strain (usually understood as the elastoplastic strain). In addition, when loading after creep and relaxation periods, the response is much more rigid than if the time-dependent processes had not occurred (Tatsuoka et al. 2000).

The main purpose of this paper is to present the different time-related effects observed in soils and to remove the confusion regarding definitions, which are found in literature, i.e., a concise review is presented, which describes the observed trends within the field of time-dependent behavior of soil. To keep the review within manageable limits, the following assumptions have been made:

• The descriptions are restricted to factors that concern the mechanical properties, such as stress, time, and strain.
• Temperature dependence has not been considered. For further studies, see Leroueil and Marques (1996).
• Only observations obtained from laboratory tests in one-dimensional oedometer tests and triaxial test conditions are considered. Tests under in situ conditions are not taken into account, see, e.g., Leroueil and Marques (1996) for other types of tests.

In this paper, the description of time-dependent observations in one-dimensional tests and triaxial tests are divided into the following subsections: Creep, relaxation, rate dependency, and accumulated effects. The characteristics of the observed behavior of clay and sand are explained separately within the four subsections. Definitions of creep, stress relaxation, and rate dependency are also presented in the section next.

It should be noticed that there are several ways in which the word “time” can be understood. In this paper, time has nothing to do with dynamic effects where inertial forces are involved. Instead, time and time dependency is here assumed to be related to viscous effects in the soil skeleton, such as creep, stress relaxation, and strain-rate effects. Therefore, the process of consolidation is not regarded as a true time effect either.

Basic Descriptions of Time Effects

There are three standard tests used to identify the time-dependent response of soil: creep tests, stress relaxation tests, and constant rate of strain tests (CRS tests). In the following, these tests and the soil response will be discussed because some confusion is found in literature, especially regarding the definition of creep.
Creep

A creep test (strain path \( A \rightarrow B \)) is illustrated in Fig. 1. Consider a soil sheared to the stress–strain state at point \( A \) [Fig. 1(a)]. At this point, a creep process is initiated by letting the stress be constant over time [Fig. 1(b)]. As times advances, the strain state moves toward \( B \). During this process, the strain is gradually increasing, i.e., the soil exhibits creep behavior [Fig. 1(c)]. Therefore, it can be concluded that during a creep test, which is characterized by constant stress, the strain increases.

Definition of Creep Stages

The results of a creep process performed at constant stress in a triaxial apparatus may be plotted in a strain–time diagram with arithmetic axes, as shown in Fig. 2. The process can be divided into three parts: (1) Primary creep or transient creep, (2) secondary creep or stationary creep, and (3) tertiary creep or acceleration creep.

A decreasing, a constant, and an increasing strain rate characterize the primary, secondary, and tertiary phases, respectively. This fact is further demonstrated in Fig. 2(b) where the logarithm of the strain rate is plotted against the logarithm of time. It should be noted that tertiary creep eventually leads to failure of the soil. This kind of failure is denoted as creep failure or creep rupture. The above mentioned is only valid for a creep test performed by means of a triaxial device.

Some confusion exists regarding primary, secondary, and tertiary creep defined in connection with creep tests performed in a triaxial apparatus on the one hand, and primary, secondary, and tertiary compression defined in connection with step load tests performed in an oedometer apparatus on the other hand. For oedometer tests, primary, secondary, and tertiary compression can be defined by plotting strains versus the logarithm of time, see Fig. 3(a). The primary phase is identical to the primary consolidation, i.e., the primary phase is the phase where excess pore pressure dissipates. The secondary phase is also denoted as secondary consolidation and, in oedometer tests, this phase corresponds to pure creep, i.e., deformations occur due to deformations in the soil skeleton (in this paper, there is no distinction between “secondary consolidation” and “secondary compression”). Tertiary compression corresponds to pure creep, too. The tertiary compression phase is subsequent to the secondary compression phase and it is characterized by a nonlinear relationship between log (time) and strain [see Fig. 3(a)].

By comparing Figs. 2(a) and 3(a), it can be concluded that there are clear differences between primary, secondary, and tertiary creep and compression. By inspecting Fig. 3, nothing can be stated about the changes in strain rate with time \((d^2 e/dt^2 \text{ or } e)\), because strain or strain rate is plotted against logarithm of time. It can be shown from elementary definitions of \(\ln(t)\) and differentiation that (den Haan 1994):

\[
\frac{d^2 e}{dt^2} = \frac{1}{t^2} \left( \frac{d^2 e}{d(\ln t)^2} - \frac{de}{d(ln t)} \right)
\]

where \( t = \text{time} \) and \( e = \text{strain} \). From Eq. (1), it can be concluded that the strain rate is increases if the second derivative of strain with regard to the logarithm of time is larger than the first derivative. The rate remains constant when both derivatives with regard to the logarithm of time are equal. The normal case will be that the rate continues to decrease, for which the second derivative with regard to the logarithm of time is smaller than the first derivative (den Haan 1994). Thus, a steepening strain versus logarithmic of time curve (tertiary compression) corresponds to a decreasing strain rate. In Fig. 3(b), the logarithm of strain rate versus logarithm of time is depicted for a single load increment in an oedometer test. It can be noted that the strain rate decreases with time. Therefore, it can be concluded that in oedometer tests,

Fig. 1. Creep test performed at a low stress level: (a) Stress–strain relationship; (b) stress history; and (c) strain history

Fig. 2. Definition of creep stages when considering a creep test at constant stress performed in a triaxial apparatus: (a) Strain versus time and (b) log(strain rate) versus log(time)
only primary creep (decreasing strain rate) can be observed whereas secondary (constant strain rate) and tertiary creep (increasing strain rate) cannot be observed.

From the above discussion, it can be concluded that primary, secondary, and tertiary creep are associated with a decreasing, constant, and increasing strain rate over time, whereas primary, secondary, and tertiary compression are in all situations associated with a decreasing strain rate over time.

Problem of Reference Time

It should be noted that the problem of defining the end of primary consolidation (EOP) is a contentious issue. It is closely related to the problem of similitude—meaning similarity or equivalence—(problem of reference time) that plays an important role with respect to determining the magnitude of creep in clayey soils. The problem of similitude does not change the fact that secondary compression exists. In other words the problem of similitude concerns the magnitude of the creep strains but not the rate or evolution of the creep strains. The concern is deciding when creep deformation starts, i.e., determining the reference time \( t_i \). There are two aspects when evaluating the reference time:

- The reference time is taken as the time at the EOP. This implies that the value of \( t_i \) should vary with the drainage length or the thickness of the soil.
- The reference time is taken as an intrinsic parameter for a given soil. It means that \( t_i \) is independent of drainage conditions and soil thickness.

Much attention has been paid to the evaluation of reference time in literature, because it is crucial for estimating creep settlements in low permeability soils such as clay. The two aforementioned aspects express the two well-known approaches that have been adopted in the estimation of secondary compression, i.e., hypotheses A and B (Ladd et al. 1977). These approaches are as follows.

- Hypothesis A assumes that sample thickness has no effect on the location of the EOP curve and hence on the value of pre-consolidation pressure. This hypothesis gives unique values of strain at the EOP, which corresponds to the fact that the soil does not show any time-dependent creep behavior during pore pressure dissipation in a manner that affects the strains at EOP. The main assumption of this hypothesis is that the secondary compression (creep) occurs only after primary consolidation. This method has been suggested by Mesri and Choi (Ladd et al. 1977; Leonards 1977; Mesri and Choi 1985a,b).

- Hypothesis B assumes that creep occurs during the whole consolidation process, which means that the strain at the EOP is not unique. This hypothesis, in which the time-dependent strains take place during the primary consolidation, is suggested by Suklje (1957), Wahls (1962), Barden (1969), Bjer-

\[ \log \tau \]

\[ \log \epsilon \]

\[ \log \text{time} \]

Fig. 3. Definition of primary, secondary, and tertiary compression: (a) Strain versus log(time) and (b) log(strain rate) versus log(time)

rum (1967), Leroueil et al. (1985), Crawford (1986), Kabbaniet al. (1986), and Yin (1999) among others.

The predicted strain–time curves in connection with hypotheses A and B are illustrated in Fig. 4. There is still no general agreement on whether there is a combination of primary and secondary compression during the process of pore pressure dissipation (Duncan et al. 1986). The disagreements appear clearly by following the formulation from Leroueil et al. (1985):

“... the experimental evidence is almost non-existent or not convincing; the consolidation test results obtained by Berre and Iversen (1972) on specimens of different heights which are often used to validate method B were also used by Leonards (1977) to justify theory A.”

The conclusion of this discussion is that the real soil behavior is somewhere in the middle since the two hypotheses correspond to two extreme cases. This has been confirmed by, e.g., Aboshi (1973) who made an experimental investigation of the problem of similitude in normally consolidated clays. In all approaches by the different writers mentioned above, whether hypothesis A or B is adopted, a logarithmic function is used to fit the oedometer test data of vertical strain (or void ratio) against time after the primary (method A) or instantaneous (method B) compression (Liingaard et al. 2004). It should be noted that there is no distinction between methods A and B in permeable soil, such as sand.

Drained and Undrained Creep

In connection with a creep process performed in a triaxial apparatus, two definitions of creep are found in literature. In drained creep, the effective stresses (i.e., the mean effective stress \( p' \) and the deviatoric stress \( q \)) are kept constant, i.e., the creep process corresponds to a single stress point. In undrained creep, the drains are closed, and this causes a pore pressure buildup, and the mean effective stress \( p' \) decreases, while the deviatoric stress \( q \), which is independent of pore pressure, remains constant. According to the definition of creep (development of strains over time at con-

\[ \text{Thick sample} \]

\[ \text{Thin sample} \]

Hypothesis B

Hypothesis A

\[ \log \text{time} \]

Fig. 4. Predicted strain–time curves for hypotheses A and B
stant effective stresses), it can be concluded that undrained creep does not represent a pure creep process, whereas drained creep does. In undrained creep deformations consist of plastic (due to changes in effective stresses) and inelastic (creep) deformations. However, in the literature, both processes are described as creep and, in the following, the processes will be denoted as drained and undrained creep.

Creep at Constant Load or Constant Stress
The requirement of constant effective stresses during a creep test is not fulfilled, in general. There are several occasions in literature where the authors refer to triaxial creep tests without explaining whether or not the effective stresses were kept constant. It is not always clear whether the writers refer to a triaxial creep test as a one where the effective stresses are kept constant or a one where the load is kept constant. There are clear differences. This is illustrated in Fig. 5.

Creep at constant stress corresponds to a point in the triaxial plane, as illustrated in Fig. 5. On the other hand, the constant load creep represents creep under stresses decreasing with time. This is due to the fact that the sample area increases, thereby resulting in a continuous decrease in creep stress. In fact, it is only the constant stress creep that can be taken as true creep, because the effective state of stress is maintained constant.

Stress Relaxation
A stress relaxation test (stress path $A \rightarrow B$) is illustrated in Fig. 6. Consider a soil sheared to the stress–strain state at point $A$. At this point, a stress relaxation process is initiated by letting the total strain be constant over time. As times goes by, the stress–strain state moves toward $B$. During this process, the stress is gradually decreasing—it relaxes. Therefore, it can be concluded that during a stress relaxation test, which is characterized by constant total strain, the stress decreases.

Constant Rate of Strain
In the CRS test, a total strain rate $\dot{\varepsilon} = d\varepsilon/dt$ is enforced and kept constant throughout the experiment. The stress response is then measured in order to obtain a stress–strain relationship.

In Fig. 7, the results of three CRS tests are shown. It appears that the larger the strain rate, the stiffer the soil. In connection with rate-independent elastoplasticity, the three curves will coincide.

Observations from One-Dimensional Tests
Creep
The observed behavior of creep in clays and sands is described in the following section. The descriptions are focused on the creep behavior with relation to factors, such as stress dependency and strain–time relations.

Clay
It is commonly accepted that clayey soils exhibit creep at constant effective stresses, especially normally consolidated clays. The phenomenon of drained creep of clays has been investigated extensively under one-dimensional conditions, where it is referred to as secondary compression. Secondary compression is often depicted as an approximately linear relationship between the vertical strain $\varepsilon_z$ or void ratio $\varepsilon$ and the logarithm of time $t$. This relation is given by the coefficient of secondary compression, as seen in Fig. 8. This coefficient can be defined in different ways, with the most commonly used definitions given by
The effects of the effective vertical stress on creep behavior sketched in Fig. 8. That is, the secondary compression or creep follows a linear relation in a $e_{s} - \log(t)$ diagram. This observation may be valid for several log cycles of time, but it does not hold true in general.

A general nonlinear strain–time behavior with respect to an $e_{s} - \log(t)$ diagram has been observed by Leonards and Girault (1961), Bjerrum (1967), Berre and Iversen (1972), Leroueil et al. (1985), and Yin (1999). Leroueil et al. (1985) reported a general nonlinear strain–time behavior based on long-term creep tests (duration of 140 days). The strain-time curves for overconsolidated specimens, type I in Fig. 9, showed a continuously increasing slope with the logarithm of time after end of primary consolidation (EOP). On the other hand, specimens in the normally consolidated range, type III in Fig. 9, showed a continuously decreasing slope with the logarithm of time. The above states that the logarithmic relation does not hold true, in general.

When dealing with clays it is important to remember that the excess pore pressure during consolidation may hide the actual viscous effects in the soil. This is the fact for oedometer creep tests performed on samples in the vicinity of the preconsolidation
stress, type II in Fig. 9. At first, the specimen reacts as an overconsolidated soil, but after a certain time the strain rate remains momentarily constant. Finally, the sample reacts as a normally consolidated soil. This indicates some kind of creep delay when the sample passes from the overconsolidated range into the normally consolidated state. This behavior does not become obvious in Fig. 9. To visualize this kind of behavior, the creep test data are plotted in a log $\dot{\varepsilon}_z$–log $t$ diagram, as shown in Fig. 10. The main feature of this figure is that the slope of the straight line in the log $\dot{\varepsilon}_z$–log $t$ diagram is characterized by the $m$ parameter by Singh and Mitchell (1968):

$$m = -\frac{\Delta \log \dot{\varepsilon}}{\Delta \log t} \quad (4)$$

In one-dimensional tests, $\dot{\varepsilon}$ should be taken as the vertical strain rate $\dot{\varepsilon}_z$. In the log $\dot{\varepsilon}_z$–log $t$ diagram in Fig. 10(a), a straight line with a downward slope $m=0.5$ corresponds to Terzaghi’s solution for primary consolidation. This slope is observed in the beginning of the type III curve that corresponds to a normally consolidated soil. Later, the type III soil shows a slope that corresponds to an $m$ value greater than 1.0. The type I soil that is overconsolidated throughout the test shows an $m$ value below 1.0 during the test period. The type II soil starts out with an $m$ value corresponding to the overconsolidated soil (type I), but then the strain rate stops decreasing for a while. Thereafter, the type II soil shows the behavior of a normally consolidated soil. In the log $\dot{\varepsilon}_z$–log $t$ diagram in Fig. 10(b), the creep delay becomes clear. The S-shaped curve in Fig. 10(b) (type II) was observed by Bishop and Lovenbury (1969) and Tavenas et al. (1978), but no explanations were provided. Kabba et al. (1986) suggest that the S-shape corresponds to the creep delay, i.e., the transition from overly consolidated to normally consolidated creep state.

The above discussion only concerns the evolution of strains during secondary compression. As mentioned in the section entitled, “Basic Descriptions of Time Effects,” there is a third oedometer phase, tertiary compression, which has been observed after secondary compression. This phenomenon has been reported by Fodil et al. (1997) for oedometer creep tests on soft natural clay. The strain–time relation for creep started out as a linear relation in an $\varepsilon$–log $t$ diagram. After several log cycles of time, the creep behavior for the normally consolidated specimens showed a tendency for tertiary compression. Similar observations have been reported by den Haan and Edil (1994). They reported steepening of the compression—log $t$ curves in oedometer tests on Portage (Wisconsin) peat, and defined a tertiary compression phase after the secondary compression phase. The tertiary compression occurred at all stress levels.

The phase of tertiary compression is not well documented in literature. It appears that it is only a well-accepted phenomenon in literature concerning peat.

Sand

There are only a few reported laboratory studies of one-dimensional creep tests performed on sand. For that reason, relevant observations from triaxial tests under isotropic stress states are included in this section, in order to support the few investigations performed in connection with oedometer tests on sand. To model the creep behavior of sand in confined conditions, there are two important observation aspects that will be described: Stress dependency and strain–time behavior.

Stress dependency. The deformations of granular soils occur almost instantaneously upon load application at low confining stresses but may continue, at a decreasing rate, for long periods of time at high confining stresses. The behavior of granular soils at high confining pressures is qualitatively similar to the behavior of normally consolidated clays. Thus, time-dependent behavior of granular materials can be divided into: (1) Behavior at low confining stresses (analogous to overconsolidated clay) where the deformations are caused by rearrangement over time due to sliding and rolling between the sand particles and (2) behavior at high confining stresses (analogous to normally consolidated clay) where the deformations are associated with continuous fracturing (crushing) and deformation of the grains.

The transition between the low stress and high stress regime depends on the mineralogical composition of the particles and the initial void ratio. The transition is referred to as the “critical pressure.” For detailed studies of the critical pressure and breakage of grains, see Yamamuro et al. (1996). The two regimes seem to have different creep characteristics. At low stresses, the creep strains are generally of small magnitudes but not negligible. Creep strains in one-dimensional conditions at low confining stresses have been reported by Mejia et al. (1988). They performed oedometer tests on two types of loose sands (Ottawa sand and Brenda Mine Tailings sand) at different vertical effective stresses. The time-dependent creep strains observed by Mejia et al. (1988) seem to be caused by sliding and rolling and not

![Fig. 10. (a) Illustration of the characteristic $m$ values in log $\dot{\varepsilon}_z$–log $t$ diagram and (b) Types I, II, and III visualized in log $\dot{\varepsilon}_z$–log $t$ diagram](image-url)
crushing because the stress levels were relatively low (<800 kPa).

Similar tests have been reported by Colliat-Dangus et al. (1988). They performed isotropic creep tests at both low stresses and high stresses. They found that the compression of sand is not instantaneous but continues at an ever-decreasing rate over a long period of time, in a way similar to the phenomenon of secondary compression observed in clays. The results are shown in Fig. 11.

Despite some scatter in the test results, it is observed that the volumetric creep strains increase with the confining stress. For the Hostun sand (siliceous) the compression may be considered instantaneous as long as the confining stress is lower than 2 MPa, whereas the time effects are visible for the calcareous sand even at very low confining stresses. When the stresses reach a certain level, a sharp increase in the volumetric creep strain is observed. For the calcareous sand, it occurs at about 0.8 MPa, while the sudden increase for the Hostun sand occurs at 5–6 MPa. Colliat-Dangus et al. (1988) define this threshold as a “creep stress” below which the strains due to creep are insignificant. They analyzed the grain size distributions before and after each test, and the analyses showed that the time effects were correlated to grain crushing. This implies that the propagation of rupture of particles may be the physical factor responsible for the effects of compression of granular materials under high confining stresses. The creep stress is closely related to the “critical pressure” that describes the transition between low and high stress domain.

Strain-Time Behavior. The evolution of creep strains over time is described with respect to the observations at low stresses and high stresses. In the low stress regime, the strain–time relation seems to be linear when plotted as strain versus logarithm of time. This is observed by Colliat-Dangus et al. (1988). The same tendency is observed by Mejia et al. (1988). They reported that the strain–time behavior for sand under \( K_0 \) conditions far from failure converged toward a logarithmic relation after some high initial strain rates in the first few seconds of the creep tests.

In the high stress regime, the evolution of creep strains over time is approximately linear when plotted in strain–log (time) diagrams as well. This has been observed by Leung et al. (1996) who performed one-dimensional oedometer creep tests on sands at high stresses.

The creep test at high stresses performed by Colliat-Dangus et al. (1988) showed the same tendency. The creep behavior of the Hostun sand showed a linear relation between creep strain and log (time) in the range of confining stress from 2 MPa to 10 MPa. The calcareous sand showed the same behavior but after approximately one hour of creep. The creep behavior in the initial stage was characterized by high initial strain rates that converged toward the logarithmic relation in about an hour. This suggests that a power relation between the creep strain and time in the initial creep stage may be more appropriate. The bilinear creep behavior (power relation followed by logarithmic relation) at high stresses has also been observed by Lagioia (1998) in isotropic creep tests (2–5 MPa). The isotropic creep tests were part of an investigation of time-dependent behavior of carbonate sands.

Relaxation

It appears that there are few reported investigations that treat relaxation of clay or sand in one-dimensional conditions. The only relaxation investigations on sand in one-dimensional test conditions are uniaxial relaxation tests performed on frozen sand by Ladanyi and Benyamina (1995).

Rate Dependency

The rate dependency has been investigated extensively in the past 30–40 years and several writers have confirmed that rate effects have great influence on the stress–strain behavior of clay. There are no known CRS investigations concerning rate-dependent behavior of sand in one-dimensional tests.

Clay

Increasing attention has been paid to investigation of rate dependency of clays since the introduction of various continuous loading oedometer testing procedures. These testing methods have been used in order to reduce the time involved in performing a consolidation test and to obtain continuous stress–strain curves. Among these testing methods are CRS, constant rate of load and controlled hydraulic gradient tests. The testing methods are explained in detail by Hamilton and Crawford (1959), Smith and Wahls (1969), Lowe et al. (1969), Wissa et al. (1971), Gorman et al. (1978), and Janbu et al. (1981). To show the rate dependency of clays under one-dimensional testing conditions, two different situations are explained, constant rate of strain and change of rate of strain.

Constant Rate of Strain Tests. The characteristics of CRS tests are indicated by the influence of strain rate on the preconsolidation pressure and the stress–strain behavior. The general observation is that the faster the loading rate, the higher the effective stresses for a certain strain. A typical example showing the strain-rate dependency of natural clay is illustrated in Fig. 12. It is seen that the compression curves move to the right for higher strain rates. However, it should be noted that the compression curve for the slowest rate \((\dot{e}, \epsilon)\) deviates from the other tests. This deviation is explained in the section entitled, “Observations from Triaxial Tests.”

After analyzing results from a variety of oedometer tests on different natural clays, Leroueil et al. (1985) suggested that the behavior was controlled by a unique relationship between the vertical effective stress, strain, and strain rate \((\sigma^\prime \epsilon \dot{\epsilon})\). This unique relation is denoted “isotach behavior” and will be further discussed in the subsection entitled, “Strain-Rate Dependency.” Leroueil et al. (1985) showed that this unique relation could be described by two curves, one giving the normalized effective
stress–strain relationship and the other the variation of the preconsolidation pressure with strain rate. The normalized stress–strain relationship and the variation of preconsolidation with strain rate for Batiscan clay are shown in Figs. 13

From Fig. 13, it is observed that the normalized stress–strain curves coincide for different rates of strain. Furthermore, the variation of the preconsolidation pressure with the strain rate becomes clear. The experimental data for several other types of natural clays presented by Leroueil et al. (1985) are consistent with the tendencies shown in Fig. 13.

It should be noted that most of the investigations of rate dependency are based on data obtained in the normally consolidated range; the rate-dependent behavior in the overconsolidated range is not as clear. However, creep tests in the overconsolidated range indicate different constant rate of strain curves (Tavenas et al. 1978).

**Change of Rate of Strain.** The existence of the unique relationship between the vertical effective stress, strain, and strain rate has been confirmed by special CRS tests in which the strain rates were changed at various strains. Leroueil et al. (1985) performed two such tests on Batiscan clay, Fig. 14. Länivaara and Nordal (2000) have confirmed these special tests.

The results in Fig. 14 clearly show the unique stress–strain–strain-rate relationship (isotach behavior). An important feature is that the effects of change in rate are continuous, that is the soil “stays” on the same stress–strain curve until the strain rate is changed again. This isotach behavior is not observed for sand, as will be shown in the subsection entitled, “Strain-Rate Dependency.”

**Sand**

It appears that there are no reported investigations that deal with the rate dependency of sand in one-dimensional conditions.

**Accumulated Effects**

The apparent preconsolidation pressure increases during secondary compression. When a soil is subjected to a constant effective
stress for a long period of time, the void ratio and strain rate progressively decrease as shown in Fig. 15. If the soil is reloaded, there is an increase in strain rate and the compression curve moves to the curve of constant strain rate corresponding to the new strain rate. The compression curve then shows a preconsolidation pressure \( \sigma'_{c, pcl} \) associated with this new strain rate. The development of this “quasipreconsolidation pressure” was first observed by Leonards and Ramiah (1960) and explained in detail by Bjerrum (1967).

Oedometer tests performed by Leonards and Altshaeffl (1964) showed otherwise. They reported that the tested soil exhibited a preconsolidation pressure much higher than that due to its new void ratio only (after 90 days of secondary compression). Such a behavior is associated with the development of bonds between particles and aggregates and is referred to as “structuration.” The phenomenon of structuration has been reported by Leroueil et al. (1996) for oedometer tests on an artificially sedimented clay. In one of the tests, the soil was consolidated for 120 days under an applied vertical effective stress of 10 kPa. If the increase in the apparent preconsolidation pressure were to be determined from the concept of Bjerrum, as shown in Fig. 15, the preconsolidation pressure would have increased from 10 kPa to 11.5 kPa. However, due to the structuration effects, the preconsolidation pressure increased to 18.5 kPa during reloading. The compression curve for the oedometer test reported by Leroueil et al. (1996) is reproduced in Fig. 16.

Further investigations of structuration effects were performed by Leroueil et al. (1996). They made a comparison between the following three kinds of tests, see Fig. 17: (1) Conventional oedometer test (the applied load was increased in steps every 24 h), (2) CRS oedometer test performed at a strain rate of \( 1.27 \times 10^{-3} \) s\(^{-1}\), and (3) CRS oedometer test performed at a strain rate of \( 1.00 \times 10^{-2} \) s\(^{-1}\).

The three tests were compared and a unique relationship between the effective stress, strain, and strain rate \( (\sigma'_{e} - \varepsilon_{e} - \dot{\varepsilon}_{e}) \) was presumed. Good agreement was observed between the results from the conventional test and the “rapid” CRS test performed at a strain rate of \( 1.27 \times 10^{-5} \) s\(^{-1}\). The compression curve constructed from the conventional oedometer test data corresponded to a strain rate of approximately \( 1 \times 10^{-7} \) s\(^{-1}\), i.e., two orders of magnitude slower than the rapid CRS test at \( 1.27 \times 10^{-5} \) s\(^{-1}\). It was expected that the compression curve for the “slow” CRS test at a strain rate of \( 1.00 \times 10^{-7} \) s\(^{-1}\) should coincide, or at least be close to the compression curve obtained by the conventional oedometer test.

The slow CRS test turned out to be very different from the conventional oedometer test. Furthermore, the compression curve for the slow CRS test was located above the rapid CRS test as shown in Fig. 17. This behavior did not agree with the unique \( \sigma'_{e} - \varepsilon_{e} - \dot{\varepsilon}_{e} \) relation and the disagreement was explained by the effects of structuration developing with time. It should be noted that the deviation of the compression curve for the slowest rate \( (\dot{\varepsilon}_{e, b}) \) in Fig. 12 might be due to structuration effects as well.

The agreement between the conventional test and the rapid CRS test was explained as follows: In the rapid CRS test, the strain rate was so high that the structuration had no time to develop; in the conventional oedometer test, the structure which might have developed during the previous loading step was destroyed after the new load was applied. In other words, the structuration in the rapid CRS test and the conventional oedometer test did not develop and the difference between the compression curves could be explained by rate dependency alone. On the other hand, the slow CRS test allowed structuration to develop resulting

---

**Fig. 15.** Development of quasipreconsolidation pressure due to secondary compression. The preconsolidation pressure increases from \( \sigma'_{c, pcl} \) to \( \sigma'_{c, pcl} \). It should be noted that the lines of constant strain rate are approximately equal to the lines of constant time (after Bjerrum 1967).

**Fig. 16.** Compression curve with structuration

**Fig. 17.** Constant rate of strain and conventional 24 h oedometer tests on resedimented Jonquière clay (after Leroueil et al. 1996)
in a continuous strengthening. This strengthening over time is the reason that the compression curve for the slow CRS test is stiffer that the curve for the rapid CRS test.

The final comments are that the findings above show that the structuration phenomenon can be related to both time (the duration of secondary compression) and strain rates. The latter can be explained by the fact that structuration develops at slow rates. The distinction of slow and rapid rates has not yet been defined clearly but slow rates correspond to rates below $10^{-7}$ s$^{-1}$ (Leroueil et al. 1996). Furthermore, Leroueil et al. (1996) suggest that structuration in clays can be explained by strengthening of contacts between particles or aggregates due to thixotropy or cementation and may be influenced by the age of the clay.

Sand
Effects of apparent preconsolidation in sand in confined conditions have been reported by Lagioia (1998), as shown in Fig. 18. Carbonate sand specimens were isotropically or one-dimensionally compressed at a constant loading rate (100 kPa/h) and then left to creep under a constant effective stress (800 kPa). Upon further loading, the specimens exhibited a considerable increase in stiffness that appeared to depend on the creep time. This behavior is similar to the concept of apparent preconsolidation observed in normally consolidated clays by Bjerrum (1967). However, the compression curve during reloading was expected to rejoin the normal compression line obtained by continuous loading (point B in Fig. 18), but Lagioia (1998) observed that the yield point moved to the right of the normal compression line (point C in Fig. 18). When the yield point was reached, the compression increased and the compression curve rejoined the normal compression line. Lagioia (1998) suggested that the structuration effects are due to complex interparticle cementation.

Observations from Triaxial Tests

Creep
In the one-dimensional case the description of creep behavior was focused on the influence of vertical stress and strain evolution over time. The creep behavior in triaxial conditions will be focused on the stress-level dependency, i.e., proximity to failure.

Clay
There are relatively few reports of drained creep tests on clay, compared with the number of undrained tests. In the following, emphasis will be placed on the observations from drained conditions whenever it is possible. The observations from undrained conditions will also be mentioned. In light of the information in the section entitled “Basic Descriptions of Time Effects,” undrained creep is not a true time effect. The influence of shear stress will be shown for creep in the normally and overconsolidated range (Fig. 19). The term “limit state surface” is equivalent to the preconsolidation pressure in one-dimensional conditions, and it corresponds to the plastic yield surface under two- and three-dimensional conditions. Inside the limit state surface, the soil is overconsolidated and “outside” the soil is normally consolidated.

Creep in the Normally Consolidated Range. In order to visualize the observed creep behavior in triaxial conditions, the creep test data are again plotted in log $\dot{\varepsilon}$–log $t$ diagrams. When the data are presented in such diagrams, the creep behavior can be analyzed by the parameter $m$, which is the slope of a straight line in the log $\dot{\varepsilon}$–log $t$ diagram. However, at first sight, it may be quite complex to imagine the consequences of varying $m$ values. For that reason, the characteristics of three different $m$ values are illustrated in Fig. 20. $m$ is given by Eq. (4) and $\dot{\varepsilon}$ is taken as the axial strain rate $\dot{\varepsilon}_1$ in this section concerning triaxial conditions. In the section entitled, “Basic Descriptions of Time Effects,” $\dot{\varepsilon}$ is taken as the vertical strain rate $\dot{\varepsilon}_z$.

In one of the first studies of creep under drained and undrained triaxial conditions on various normally consolidated clays, Singh and Mitchell (1968) found that the parameter $m$ varied between 0.75 and a value slightly greater than 1.0, with most values less than 1.0. They also suggested that the value of $m$ was independent of the deviatoric stress level for a given soil. In other words, the creep lines for different deviatoric stress levels have the same slope in the log $\dot{\varepsilon}_1$–log $t$ diagram. Singh and Mitchell (1968) also found that the creep strain rate increases with the applied deviator stress.

In Fig. 21, a graph from Bishop and Lowenbury (1969) depicts the strain rate against time for drained triaxial tests and two oedometer tests on normally consolidated Pancone clay. Consider the part of the graph between 1 and 20 days. It is seen that $m \approx 1$ for the oedometer tests, thus the strain is more or less logarithmic with time. In the triaxial tests, $m$ increases with deviator stress level $S$ ($S = q/q_{failue}$) from $m = 0.8$ at $S = 50\%$ to $m > 1.2$ for $S = 85\%$. The graph in Fig. 21 illustrates that $m$ is not always

---

Fig. 18. Observed and expected structuration of sand due to creep (after Lagioia 1998)

Fig. 19. Limit state surface in the triaxial plane. Inside the soil is overconsolidated and outside the soil is normally consolidated. The preconsolidation pressure in one-dimensional conditions corresponds to the intersection between the $K_0$ line and the limit state surface.
constant in time and independent of deviator stress level as assumed by Singh and Mitchell (1968). But, in general, the graph shows that the strain rates increase with increasing stress levels. In the part of Fig. 21 between 20 and 100 days, the "abrupt" change in strain rate was interpreted as a limited instability that reflects a fundamental modification in soil structure. Bishop and Lovenbury (1969) provided no further explanations. As mentioned in the section entitled, "Observations from One-Dimensional Tests," Kabbaj et al. (1986) suggested that the S-shape corresponds to a creep delay, occurring at the transition from an overconsolidated to normally consolidated creep state.

Tian et al. (1994) also reported that the m value appeared to be a function of the deviator stress level. Their analyses were based on drained triaxial creep tests on undisturbed marine sedimented clays from the Gulf of Mexico and the North-Central Pacific. The tests showed that the m value increased with increasing deviatoric stress levels for high-plasticity clay specimens from the Gulf of Mexico. The tests on clay specimens from the North-Central Pacific showed no significant variation in the m value. Variations in the m value as the deviator stress increases have also been reported by Feda (1992).

Furthermore, Zhu et al. (1999) acknowledged the change of m

---

**Fig. 20.** Creep characteristics for three different m values. The strain–time curves for the m values are shown to the right. m = 1 is corresponds to a straight line in the ε1–log t diagram. m ≠ 1 corresponds to curved lines in ε1–log t diagrams.

**Fig. 21.** Axial strain rate as a function of time for various stress levels (given as a percentage of deviator stress at failure). Results obtained from drained triaxial tests and oedometer tests on Pancone clay. Reproduced from Bishop and Lovenbury (1969). Guidelines showing m = 1.0 are inserted.
values with deviator stress in undrained triaxial creep tests. But they found that $m$ decreased with an increasing stress level ($m$ decreased from 0.91 to 0.57 for $q = 14\, \text{kPa}$ to $121\, \text{kPa}$). This tendency is opposite that observed by Tian et al. (1994). The work was based on soft Hong Kong Marine Deposits. Zhu et al. (1999) showed that the observed creep behavior in triaxial extension was somewhat similar to that in compression.

The $m$ parameter has also been studied for other low permeability soils. den Haan (1994) reported that when drained or undrained triaxial tests are performed on peat and clays, it is common to find values of $m \approx 0.7$–0.9, largely independent of deviator stress level. In particular, it is found that $m \approx 0.84$–0.90 for peat and $m \approx 0.81$–0.96 for organic mud. Such values are also found when the applied shear stress level is close to $K_0$ (one-dimensional) conditions.

**Creep in the Overconsolidated Range.** Tavenas et al. (1978) made a detailed study of the volumetric and shear creep behavior of a lightly overconsolidated natural clay. With reference to the work of Singh and Mitchell (1968) they concluded that the development of both volumetric and shear strains with time can be represented by means of the $m$ parameter given in Eq. (4).

Their triaxial creep tests were performed for several stress conditions inside the limit state surface of undisturbed Saint-Alban clay. The stress levels at which the creep processes were observed are shown in Fig. 22(a). Parallel to the triaxial tests, a series of long-term oedometer tests was performed.

The axial strain rates $\dot{e}_1$ are plotted against the time $t$ in a log–log diagram in Fig. 22(b) for triaxial and oedometer tests at various stress conditions. Note that two triaxial test specimen failed: One test corresponding to the stress state: $p' = 9.7; q = 23.2\, \text{kPa}$ and another one corresponding to the stress state: $p' = 13.4; q = 30.8\, \text{kPa}$. The specimens failed after approximately 300 and 8000 min, respectively. The failure mode was creep rupture. The failures are probably due to the fact that the stress states are close to the strength envelope, which is representing the limit state surface for the overconsolidated soil. Fig. 22 also shows that there is a temporary increase in $\dot{e}$ for the oedometer tests. It appears to be the same phenomenon as observed by Bishop and Lovenbury (1969).

Fig. 22(b) shows a linear decrease of $\log \dot{e}_1$ with $\log t$. The $m$ parameter with respect to the axial strains varies between 0.6 and 0.95 for all the triaxial creep tests. Tavenas et al. (1978) reported that the $m$ parameter increased slightly with the deviator stress, but if $m$ should be taken as a constant, the value would be of the order of 0.76 ± 0.16. Furthermore, there is a clear increase in axial strain rate as the deviator stress increases.

Tavenas et al. (1978) found that development of volumetric strain over time could be shown as a linear decrease of $\log \dot{e}_1$ with $\log t$, even for the triaxial test that exhibited dilation. The $m$ parameter with respect to the volumetric strains varied between 0.52 and 0.93 for all the triaxial creep tests, and was of the order of 0.8 for the oedometer tests. It was suggested that a constant $m$ of the order of 0.7–0.8 could be considered representative of both the axial and volumetric behavior of the overconsolidated clay. Further, it was observed that the volumetric strain rates increase slightly with the effective mean stress $p'$.

The observations of linear decrease of the logarithm of strain rate with the logarithm of time for overconsolidated clay have been confirmed in tests performed by, e.g., Bishop and Lovenbury (1969) and D’Elia (1991).

**Summary of the $m$-Parameter for Clay.** The reported values of $m$ in triaxial creep tests show that the strain–time relation, in general, differs from the classical logarithmic relation, discussed in the “Creep” subsection of the section entitled “Observations from One-Dimensional Tests.” Within the normally consolidated range, $m$ varies between 0.7 and 1.2 with most values less than

---

**Fig. 22.** (a) Stress levels at which the creep tests are carried out. Furthermore, stress levels are plotted in relation to the limit state surface and critical state line. (b) Axial strain rate–time relationship for drained tests under various stress conditions. The tests are carried out on undisturbed Saint-Alban clay (after Tavenas et al. 1978).
1.0. In the overconsolidated range the $m$ values seem to be less than 1.0.

Several writers report that $m$ is not always independent of deviator stress level as assumed by Singh and Mitchell (1968). In some cases, $m$ decreases with increasing deviator stress; other cases show the opposite. It is a general observation that the strain rate increases with increasing deviator stress or stress level.

Most of the reported studies of creep under triaxial conditions involve determination of $m$ for axial strains only. Some studies mention the $m$ value for the volumetric strains. In the work of Tavenas et al. (1978), it is suggested that $m$ may be taken as a constant, i.e., the same value for the volumetric and axial strain development. This is not the general opinion. Feda (1992) and Tian et al. (1994) found different values of $m$ for the axial and volumetric parts of the creep tests. Note that the determination of $m$ for the volumetric part is associated with some uncertainty compared with determination of $m$ for the axial part.

Sand

The general opinion is that the creep behavior in sand for various stress levels is similar to that of clay. For that reason some writers investigate the creep behavior by means of the $m$ parameter (defined in the section entitled, “Basic Descriptions of Time Effects”). Fig. 23(a) shows the creep behavior of Toyoura sand at different stress levels, as reported by Murayama et al. (1984), while Fig. 23(b) shows creep tests on Tailings sand for various stress levels, as presented by Mejia et al. (1988).

In the tests by Murayama et al. (1984), the axial effective stress was applied by a loading lever, which corresponds to constant load creep, while Mejia et al. (1988) applied a static load that was periodically adjusted to maintain constant stress. However, creep was allowed for only 20 min at each stress level in both test series.

Both tests series show that the strain rates increase with the applied deviator stress as expected. The strain–time relation in Fig. 23(a) seems to be semilogarithmic at low stress levels, indicated by $m$ values approximately equal to 1.0. The $m$ values at low stresses in Fig. 23(b) are approximately 0.9. The results by Mejia et al. (1988) indicate an initial low slope for deviator stresses at $q = 1,240$ kPa and $1,400$ kPa. After approximately 10 s, the slopes increase and become similar to the slopes for creep at lower stress levels. In both test series, creep failure is observed at high stress levels. Mejia et al. (1988) and Murayama et al. (1984) reported that the stress levels at which creep failure occurred corresponds to the stress level at failure for the usual triaxial compression tests at the same confining pressure, i.e., creep rupture was therefore inevitable.

Only few long-term creep tests on sand have been presented in literature. The above-mentioned tests series involved creep periods of only 20 min, which is relatively short to capture the strain–time evolution. On the other hand, Lade and Liu (1998) performed long-term creep tests on Antelope Valley Sand for different stress levels. Their tests results showed more or less logarithmic strain–time relations for creep periods up to 10,000 minutes (7 days).

Stress Relaxation

Relatively few investigators have studied stress relaxation in soils under triaxial conditions. Most of the relaxation tests have been performed on clays. That is, there are only few reports of relaxation in sands, but the section is still divided into descriptions of clay and sand.

Clay

There appears to be a kind of “reference relaxation behavior” presented by Lacerda and Houston (1973), which acts as a basis for other investigations of relaxation. They reviewed the few previous investigations, performed several stress relaxation tests (on N.C. San Francisco Bay Mud, kaolinite, Monterey sand, and Ygnacio Valley Clay), and made contributions to the modeling aspect of time-dependent behavior. Some of the important aspects of the work of Lacerda and Houston (1973) are as follows:

- As illustrated in Fig. 24(b), the ratio between the deviator stress $q$ at time $t$ and the deviator stress at the beginning of stress relaxation $q_0$ was found to be linear with the logarithm of time after an initial time period. Previous studies performed by Murayama and Shibata (1961) and Vialov and Skibitsky

---

Fig. 23. (a) Shear strain rate–time response for Toyoura sand at different deviator stresses (after Murayama et al. 1984). (b) Shear strain rate–time response for Tailings sand at different deviator stresses (after Mejia et al. 1988). Circles denote creep rupture in tests. The values are deviator stress, $q = \sigma_1 - \sigma_3$ (kPa).
(1961) showed similar patterns. However, Vialov and Skibitsky (1961) reported the existence of a final relaxed stress level. This was not observed by Lacerda and Houston (1973) for test durations up to 10,000 minutes (7 days).

- The strain rate prior to the relaxation phase influences the time at which stress relaxation begins. Lacerda and Houston (1973) observed that when slow strain rates were used to reach the relaxation strains, there was a time delay prior to the initiation of deviator stress decay. This is illustrated in Fig. 24.

- The test results were obtained in undrained stress relaxation tests. However, Lacerda and Houston (1973) observed that the variation of excess pore pressure during the undrained stress relaxation tests was practically zero, and Murayama and Shibata (1961) reported similar observations.

More recently, relaxation behavior has been investigated by Silvestri et al. (1988), Sheahan et al. (1994), and Zhu et al. (1999). Silvestri et al. (1988) performed undrained triaxial relaxation tests on a soft overconsolidated clay (Louisville clay). They observed that the deviator stress reached a final relaxed level after a period of time of less than one day. Furthermore, they suggest that a curve joining these relaxed stress states would represent a “static” effective stress state curve. It is noteworthy that the “static stress curve” is similar to the term “static yield surface,” which is used in connection with modeling time-dependent behavior of soils by means of Perzyna’s overstress theory (Perzyna 1963, 1966). The static stress curve is illustrated in Fig. 25. Reaching final relaxed stress states after relaxation were reported by Sheahan et al. (1994), while Zhu et al. (1999) reported no relaxed level when the relaxation tests were stopped after approximately 1,000 minutes. In the investigations by Silvestri et al. (1988), Sheahan et al. (1994), and Zhu et al. (1999) only small pore-pressure developments were observed during undrained relaxation.

Finally, the time delay of initiation of deviator stress decay shown in Fig. 24(b) seems to be inversely proportional to the strain rate prior to the relaxation phase. This implies that there is a relation between shearing at different strain rates and the relaxation behavior, and this suggests a correspondence between relaxation and rate dependency of clays.

Sand

Only few studies of stress relaxation in sand have been reported. Stress relaxation for Monterey sand has been reported by Lacerda and Houston (1973). Matsushita et al. (1999) reported that a considerable amount of stress relaxation was observed in triaxial tests on Hostun and Toyoura sands. Mitchell (1993) states that the relaxation behavior of clays and sands are generally the same. Most of the observations of relaxation on sand under triaxial test conditions are relaxation tests performed on frozen sand by Ladanyi and Benyamina (1995).

Strain-Rate Dependency

Clay

The description of creep phenomena in clays in the “Creep” subsection of the section entitled “Observations from Triaxial Tests” was divided into those observed for over and normally consolidated clays. This is also convenient for describing the rate dependency of the strength envelope for clays. The strength envelope for clays is sketched in Fig. 26.

In Fig. 19 the limit state surface is taken purely as a generalization of the preconsolidation pressure in one-dimensional conditions. It turns out that the upper part of the limit state surface corresponds to the peak strength envelope of an overconsolidated soil (from A to B in Fig. 26). The strength envelope in the normally consolidated range coincides with the critical state line (from B to C in Fig. 26). In the following, the effect of strain rate on the peak strength and the strength in the normally consolidated range will be evaluated.

Tavenas and Lereoueil (1977) indicated that the effects of time and strain rate on the preconsolidation pressure can be generalized to the entire limit state surface of the soil. Based on this statement, the rate dependency of the peak strength envelope can be explained by the rate dependency of the preconsolidation pressure obtained from one-dimensional conditions. In other words,
this means that the strain-rate effects on the strength envelope and thus the entire limit state surface are similar to the strain rate effects on the preconsolidation pressure shown in Fig. 13.

Fig. 27 shows the effect of strain rate on the stress–strain relation for overconsolidated Leda (Saint-Jean-Vianney) clay. The tests were performed as undrained CRS tests by Vaid et al. (1979). The results are obvious: The greater the strain rates, the greater the peak strength. Otherwise, the stress–strain relations are similar in nature. Similar observations have been reported by Tavenas et al. (1978) for overconsolidated Saint-Alban clay, Lefebvre and Leboeuf (1987) for structured Grande-Baleine clay, and Zhu et al. (1999) for soft Hong Kong marine deposits. Fig. 28 shows schematically the influence of strain rate on peak strength and thereby the limit state surface.

Fig. 28(a) also shows a stress–strain curve obtained by a hypothetical test performed at a strain rate approximating zero. The peak strength is denoted $q_0$. The corresponding limit state surface in Fig. 28(b) is denoted “static yield surface,” also see the subsection entitled “Stress Relaxation.” The existence of this surface has been debated in literature because several constitutive models based on Perzyna’s overstress theory rely on the existence of a static yield surface (Perzyna 1963). Vaid and Campanella (1977) reported that there are indications of the existence of the static yield surface.

The part of the strength envelope denoted “critical state line” in Fig. 28 seems to be independent of rate effects. Several tests performed at different strain rates on normally consolidated clays did not show any significant effects of strain rate on the friction angle in the normally consolidated range (Vaid and Campanella 1977; Lefebvre and Leboeuf 1987; Sheahan et al. 1996).

**Sand**

Casagrande and Shannon (1948) and Lee et al. (1969) were among the first to study the strain rate effects on the strength of cohesionless soil. Yamamuro and Lade (1993) investigated the strain rate effects on dense Cambria sand in drained as well as undrained conditions performed at high pressure. The tests were performed at various constant strain rates but showed no significant rate effects on the stress–strain relations.

Matsushita et al. (1999) studied the rate dependency on the stress and strain behavior by performing drained plane strain compression tests and triaxial compression tests on Hostun sand and Toyoura sand. The rate dependency was investigated in two loading situations: (1) Tests where the strain rate for each test was kept constant, and (2) Tests where the strain rate was changed stepwise.

In tests where the axial strain rate was kept constant, the stress–strain relationships were essentially independent of the constant strain rates, which differed by a factor up to 500. This is illustrated in Fig. 29(a). In contrast, when the constant shear strain rate was changed stepwise, the shear stress increased and decreased temporarily, Fig. 29(b). The stress–strain relationship temporarily overshoots the unique relationship for the constant rate of strain curve when the strain rate is increased stepwise.

**Fig. 26.** Strength envelope for clays. AB is the peak strength envelope of an overconsolidated soil. BC is the strength envelope in the normally consolidated range.

**Fig. 27.** Stress–strain behavior of Saint-Jean-Vianney clay in undrained constant rate of strain tests (after Vaid et al. 1979)

**Fig. 28.** Schematics of strain rate effects on limit state surface: (a) drained stress–strain curves for different constant rates of strain. $q_A$, $q_B$, and $q_C$ are peak strengths, and (b) location of limit state surface for the different strain rates. The broken stress–strain curve in (a) corresponds to a test performed at a rate close to zero. The corresponding surface in (b) is denoted as static yield surface.
After having exhibited clear yielding, the stress–strain relationship gradually rejoins the unique relationship for the constant rate of strain curve. On the other hand, when the strain rate is decreased stepwise, the stress–strain relation undershoots temporarily and eventually rejoins the unique relationship for the constant rate of strain curve. Similar observations of the phenomena of temporary over- and undershooting have been reported by Tat-souka et al. (2000) and Santucci de Magistris and Tatsouka (1999).

Comparison of Rate Effects in Sand and Clay

The fact that the phenomena of creep, relaxation, and strain-rate effects are governed by the same basic time mechanism is denoted isotach behavior, i.e., there is a unique stress–strain–strain-rate relation for a given soil as shown to the left in Fig. 30. The isotach behavior corresponds to some extent to the observed behavior of clay. This means that creep and relaxation properties can be obtained by means of CRS tests and vice versa (Leroueil and Marques 1996). This kind of mechanism where creep, relaxation, and rate dependency are considered to be due to the same basic mechanism is also denoted the "correspondence principle" according to Sheahan and Kaliakin (1999).

The sand tested by Matsushita et al. (1999) exhibited noticeable amounts of creep and relaxation but no strain-rate effects. This led to one of the main conclusions: The phenomena of creep and relaxation cannot be predicted from results obtained in CRS loading tests on sand. This is because the changes of strain rate

Fig. 29. Schematic diagrams illustrating the rate dependency observed for sand by Matsushita et al. (1999): (a) The stress–strain relation for different constant strain rates coincide for the three strain rates and (b) temporary over- and undershooting due to stepwise change in strain rate

Fig. 30. Isotach behavior is observed in clay for (a) creep and relaxation and (b) stepwise change in rate. Nonisotach behavior is observed in sand for (c) creep and relaxation and (d) stepwise change in rate.
are temporary, as shown in Fig. 29. This behavior of sand does not correspond to the observed rate effects in clays. For sand, this behavior is labeled as "nonisotach." The nonisotach behavior is illustrated to the right in Fig. 30.

**Accumulated Effects**

As mentioned in the one-dimensional cases there are several observations of aging effects in both clay and sand. These effects are observed in the stress–strain relation subsequent to long periods of aging, due to drained or undrained creep. Patterns similar to the observations in one-dimensional conditions have been reported in triaxial tests. The phenomena have been summarized by Tatsuoka et al. (2000). They consider with three types of postaging stress–strain relationships, as shown in Fig. 31:

- **Type 1**, aging without structuration: The stress–strain relationship after aging rejoins the original primary loading relationship without exhibiting overshooting (B to C).
- **Type 2**, temporary structuration effects: The stress–strain relationship after aging rejoins the original primary loading relationship after having exhibited a temporary overshooting (B to D).
- **Type 3**, persistent structuration effects: The stress–strain relationship after aging does not rejoin the original primary loading relationship. There is a persistent overshooting with a noticeably larger peak strength than that obtained by the original primary loading (B to E).

**Clay**

The type 3 behavior has been observed for some clays, e.g. by Vaid and Campanella (1977) for undisturbed Haney clay and by Tatsouka et al. (2000) for Fujinomori clay. In Fig. 32, the observation by Vaid and Campanella (1977) is shown. The persistent overshooting is distinct.

As mentioned, it is assumed that the isotach behavior is adequate for describing time effects in clays. At first sight, it seems reasonable to confirm the isotach behavior for clays, but this holds true only with certain exceptions. When the soil is reloaded after periods of aging, structuration effects have been observed. The additional stiffness due to structuration cannot be explained by isotach behavior.

**Sand**

It appears that the type 2 behavior corresponds to the observations reported for sands. This type of behavior have been reported by Mejia et al. (1988) for Tailings and Ottawa sand and by Tatsuoka et al. (2000) for Hostun sand. The movement of the yield surface during drained creep was investigated by Lade (1994) on Sacramento River sand. The yield surface location was calculated by means of the single hardening model (Lade and Kim 1988) and compared with experiments. The type 1 behavior was expected but a considerable amount of structuration was observed, corresponding to type 3 behavior. The stress–strain behavior subsequent to various creep periods is shown in Fig. 33. It is seen that...
the stress–strain relation after creep shows structuration effects. The calculated yield points are below the observed yield points.

The disagreement with the isotach behavior for sand became evident in the previous section where the rate effects corresponded to nonisotach behavior. The nonisotach property is also observed on the stress–strain relationship after periods of aging.

As for clay, there are structuration effects that cannot be explained by isotach behavior.

**Conclusion**

The observed time-dependent soil behavior found in literature is described. The review has focused on the observed time-dependent behavior obtained from laboratory tests in one-dimensional oedometer tests and triaxial test conditions. The description has been separated into reported characteristics of creep, stress relaxation, rate dependency, and accumulated effects. The latter have been focused on the occurrence of the phenomenon of structuration. It has been shown that all of the above-mentioned phenomena are present in both sand and clay. The time-dependent phenomena are more pronounced in clay than sand; however, sand exhibits relatively large deformations at high confining pressures because of grain crushing.

The review revealed essential characteristic situations for different types of soils. That is, whether the time-dependent behavior can be characterized as isotach or nonisotach. It seems reasonable that the isotach behavior is adequate for describing the time effects in clays in most situations. However, there are exceptions, such as the time-dependent behavior at very low strain rates, where the effects of structuration play a role. The structuration effects cannot be explained by isotach behavior.

The isotach behavior is not valid for sands. The disagreement with the isotach behavior becomes evident in several situations. The rate dependency in sand appears to be small and follows different patterns of behavior than in clays. However, when a step change is made in the CSR in tests on sand, the stress–strain state deviates temporarily from the unique strain rate-independent reference stress-strain relationship. This behavior is the so-called over- and undershooting. It appears that rate dependency in sand is of a temporary nature, which does not fit into the framework of isotach behavior. This leads to the fact that the phenomena of creep and relaxation cannot be directly related and predicted based on results obtained by CRS loading tests with respect to sand. However, it should be noted that the reported investigations of time-related phenomena for sand are few compared to the investigations of clay. The above conclusions have been made on the basis of available experimental results.

**References**


