EVALUATION OF TIME-DEPENDENT DEFORMATION PROPERTIES OF SEDIMENTARY SOFT ROCK AND THEIR CONSTITUTIVE MODELING

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ABSTRACT

A triaxial testing system with local axial strain measurements was developed to accurately evaluate the time-dependent deformation properties of sedimentary soft rock. A series of consolidated triaxial creep tests were conducted on undisturbed samples of sedimentary silt-sandstone and mudstone. Drained creep tests, each lasting for three days, were performed at several deviator stresses in the course of drained triaxial compression. Creep axial strains measured locally at the lateral surface of the specimen were always noticeably smaller than externally measured ones, indicating significant time-dependent effects of bedding error occurring at the specimen ends. In these triaxial tests, the axial strain rate was stepwise changed during monotonic shearing to evaluate the effects of strain rate and its change on the stress-strain behaviour. It is shown that instantaneous irreversible strain and its rate uniquely control the current stress state whether the strain rate is constant or continuously changing. This isotach property is modeled in a simple model to be applied to the limited test condition examined. It is shown that this model can predict adequately the measured stress-strain-time behaviour for various time histories of strain (or stress).

Key words: creep, (local strain measurement), soft rock, strain rate, stress-strain relation, time-effect, triaxial test (IGC: F6)

INTRODUCTION

To accurately evaluate the deformation of a sedimentary soft rock deposit, the precise stress-strain property at small and intermediate strains of soft rock becomes necessary. This topic has recently become one of the most important issues in geotechnical research, and extensive study has been performed by means of laboratory element tests, model tests and analysis of field full-scale behaviour. Tatsuoka and Kohata (1995) evaluated the pre-peak deformation characteristics of a wide variety of stiff geomaterials, including sedimentary soft rocks, for a strain range from less than 0.001% with triaxial compression (TC) tests. The deformation characteristics of several types of sedimentary soft rock have been evaluated by TC tests measuring axial strains locally (e.g., Jardine et al., 1985; Thompson et al., 1993; Clayton et al., 1994; Kim et al., 1994). These all showed that axial strains should be measured locally in this type of TC test, as large bedding errors could be involved in externally measured axial strains. Therefore, local axial strain measurement has become the standard triaxial compression test procedure for TC tests on soft rock in many laboratories.

It was also demonstrated by Tatsuoka and Kohata (1995), among others, that the non-linear stress and strain relationship at small and intermediate strains is one of the key design parameters, as strain levels operating in the ground of relatively stiff materials at working loads are small, typically lower than 0.5%.

In a number of case histories, including foundations for the Akashi Kaikyo (Strait) and Rainbow bridges, deep and/or large-volume excavations and so on, the long-term displacement of structures constructed on and in sedimentary soft rock was one of the design issues (Tatsuoka and Kohata, 1995; Tatsuoka et al., 1995, 1997, 1999a). In these cases, it is considered that the drained creep behaviour of soft rock is relevant. Despite the above, the time-dependency of the deformation property at small and intermediate strains of sedimentary soft rock has been investigated only to a limited depth (Tatsuoka et al., 2000a). In addition, most of the existing data as to the time-dependent deformation property of soft rock has been obtained by conventional type TC creep tests with external axial strain measurement. This test procedure is perhaps based on the idea that the effects of bedding error are essentially time-independent

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and therefore insignificant in creep tests because of constant axial stress. Moreover, in most conventional creep tests, the axial strain rate at the start of creep is not rigorously controlled. It will be shown in this paper, however, that the creep rate is significantly affected by this value. In addition, effects of constant strain rate on the stress-strain behaviour and the creep property are usually separately investigated by using different specimens. Study of the effects of step changes in the constant strain rate and creep deformation on the subsequent stress-strain behaviour has also been limited. For these reasons, a unified picture of the time- (or rate-) dependency of the deformation property of sedimentary soft rock, even for a fixed stress path, has not been obtained.

In view of the above, a new triaxial testing system with local axial strain measurement was developed, and the importance and inevitability were examined of local axial strain measurement in triaxial tests for the accurate evaluation of the time-dependent properties of sedimentary soft rock, including creep property. The test results showed that the effects of bedding error could be significant even in creep tests and very different among monotonic loading tests at different constant axial strain rates. Effects of axial strain rate on the instantaneous stress-strain state and effects of strain rate changes and creep on the subsequent stress-strain behaviour were then investigated. Based on these observations, a simple constitutive model is proposed to simulate the observed time-dependent stress-strain behaviour, including creep deformation as one step towards the final goal of developing a model that can be applied to general stress states.

APPARATUS

The newly developed triaxial testing system in Fig. 1 (Hayano et al., 1997, 1999b) consists of a triaxial cell, a mechanical axial loading system, a pneumatic cell pressure loading system and a data-acquisition system. The specimen is cylindrical (50 mm in diameter and 125 mm

in height in the present study).

Pneumatic Cell Pressure System

An air-driven pump amplifies the supplied air pressure up to 3 MPa while an air-servo regulates the amplified air pressure according to signals from a D-A card of a computer. The regulated air pressure is applied to the triaxial cell through an air/water interface of neoprene rubber balloon installed in a high-pressure water tank. This arrangement is necessary to avoid the migration of air into the specimen. A High Capacity Differential Pressure Transducer (H.C.D.P.T.) directly measures the effective confining pressure σ'_c , while a Low Capacity Differential Pressure Transducer (L.C.D.P.T.) is used to measure the amount of water that is expelled from or soaked into the specimen. Volumetric strains of a specimen obtained by this method are not reliable due to the effects of bedding error at the top and bottom ends of the specimen and extra volume changes at the lateral surface in case the effective confining pressure changes, including that of filter paper (Hayano et al., 1997). Therefore, these volumetric strains were used only as reference values in the present study. A load cell installed between the loading piston and the specimen cap directly measures the axial deviator load.

Axial Loading System

A unique gear-type axial loading device (Fig. 2), precisely controls time histories of axial strain or stress. This system is a modified version of the original one described by Tatsuoka et al. (1994). The modification includes the introduction of an AC servo motor and an electro-magnetic brake (EMB). The latter precisely sustains the shaft during very short intervals between the end of working of either electro-magnetic clutch (EMC) and the start of working of the other. This system can precisely control the axial displacement of the loading piston with a resolution of less than one μm without backlash, and can also vary the axial strain rate stepwise or continuously by a

Fig. 1. Schematic diagram of newly developed triaxial testing system

Fig. 2. Gear-type axial loading device driven by A-C servo motor (Tatsuoka et al., 2000a; Santucci de Magistris et al., 1999)

factor of up to 3,000. Signals from the D-A card automatically control the on-off operation of the AC-motor, and the loading direction of the shaft is reversed through the on-off control of the two EMCs. The capacity of axial load is 50 kN. To keep deviator stress q constant during isotropic consolidation and creep stages, or more generally to control the time history of q , a simple feedback system was developed, which controlled the axial displacement according to the monitored output from the load cell.

TEST MATERIALS

Two kinds of sedimentary soft rock (the Kazusa group) were used in the present study. Core samples of sedimentary silt-sandstone were retrieved by rotary core tube drilling at a depth of about 100–130 m below the seabed at an offshore site near the mouth of Tokyo Bay. The geological age is about one million years. The mean diameter D_{50} (after thorough crushing) was 0.041–0.045 mm, and the initial water content was 24.1–24.4%. In addition, core samples of mudstone were obtained by hand-operated direct core sampling using a 5 cm diameter diamond core barrel at a depth of about 50 m inside a tunnel at the Sagami-hara experimental site (Tatsuoka et al., 1997). The geological age is about 1.5 million years. The D_{50} value was 0.0035–0.017 mm, and the initial water content was around 24%.

TESTING PROCEDURE

Each specimen was fully saturated by vacuuming and back-pressurizing to 196 kPa in a triaxial cell filled with de-aired water (Ampadu and Tatsuoka, 1993). Both ends of the specimen were capped with gypsum on-site in the triaxial cell to ensure a good contact with the stainless platens attached to the specimen cap and pedestal. This procedure has been found necessary to ensure a uniform boundary stress distribution for the reliable measurement of the stress-strain properties of soft rock. To facilitate drainage of the specimen, a side drain of filter paper was used, whose end was connected to the side drain of the cap and pedestal. From an initial confining pressure equal to 29.4 kPa, the specimen was isotropically re-consolidated to the in-situ effective overburden pressure.

Triaxial Compression Loading with Stepwise Changes in Constant Axial Strain Rate

After ageing for about 24 hours at the isotropic re-consolidation stress state, the specimen was subjected to drained triaxial compression (TC) loading, including several creep stages. The axial strain rate $\dot{\epsilon}_v^t$ was changed stepwise by a factor of 10 or 100 several times to investigate effects on the subsequent stress-strain behaviour. The super-script 't' of $\dot{\epsilon}_v^t$ denotes "total axial strain", to distinguish it from elastic (reversible) and inelastic (irreversible) strains.

Figure 3(a) shows the deviator stress q and axial strain ϵ_v^t (measured locally by using a pair of LDTs as described later) relationship typical for the silt-sandstone. Figure 3(b) shows the relationship at a value of ϵ_v^t less than 0.6%. Creep tests were performed when q became 0.98 and 2.45 MPa. The axial strain rates denoted by $\dot{\epsilon}_0$, $\dot{\epsilon}_0/10$ and $\dot{\epsilon}_0/100$ ($\dot{\epsilon}_0$ is the reference value equal to 0.01%/min) are nominal values, which are different from locally measured values (as shown below). Figure 3(c) shows the time history of ϵ_v^t . Figure 3(d) shows the time history of locally measured axial strain rate $\dot{\epsilon}_v^t$ (LDT), together with that of the externally measured $\dot{\epsilon}_v^t$ (from the axial displacement of the specimen cap), and nominal ones. It may be seen from Fig. 3(d) that the locally measured axial strain rate was always smaller than the externally measured one. The difference between the nominal axial strain rate and the externally measured one was due to the deformation of the apparatus (i.e., system compliance). Figure 3(e) compares the nominal and externally measured axial strain rates with the locally measured ones. In the analysis of time-effects described later, the locally measured axial strain rates were used.

Creep Tests

Figure 3(f) shows the time histories of q and ϵ_v^t (LDT) during a creep test at $q=2.45$ MPa. It may be seen that the q value is kept nearly constant within a small range of variation. A relatively large variation of q at the start of the creep was by small cyclic loading applied to evaluate the instantaneous elastic deformation property (dis-

Fig. 3. Typical stress-strain-time relationship from drained TC test on sedimentary soft silt-sandstone: (a) overall stress-strain relationship, (b) behaviour at small strains, (c) overall time history of axial strain, (d) time histories of nominal and measured axial strain rates, (e) comparison of axial strain rates, (f) time histories of deviator stress and total axial strain during creep and (g) relationship between axial and volumetric strains

cussed later in this paper). Relatively small but noticeable fluctuations of q seen during the whole creep stage are those that inevitably occurred because the axial displacement was controlled so as to keep the value of q constant according to the reading of the load cell. The amplitude of these axial strain fluctuations was less than 0.001%. Within this strain range, the behaviour of sedimentary soft rock is nearly elastic (Tatsuoka et al., 1994). Therefore, the behaviour during these small stress and strain fluctuations was essentially recoverable as seen from the figure inset in Fig. 3(b). It was considered, therefore that their effects on overall stress-strain behaviour during and after each creep stage were negligible.

Drained Conditions

All the tests performed with the drainage valve opened in the present study were considered essentially fully drained for the following reasons:

- 1) Figure 3(g) shows the relationships between the volumetric strain ϵ_{vol} (from burette measurements) and the local axial or external strains (from specimen cap displacement). It may be seen that the specimen is similarly "drained" at the different strain rates. A trend of increase in $d\epsilon_{vol}/d\epsilon_v$ with a decrease in $\dot{\epsilon}_v^l$, which would be expected if the specimen was partially drained, is not seen. Similar results were obtained from the other "drained" tests performed in the present study.
- 2) For the silt-sandstone reconsolidated to the field pressure level, the drained TC strength was noticeably larger than the undrained one under otherwise the same conditions (see Fig. 5 of Tatsuoka et al., 1997). It may be seen from Fig. 3(a), however, that the peak strength at the slowest strain rate $\dot{\epsilon}_v^l(\text{nominal}) = \dot{\epsilon}_0/100$, where the fully drained conditions is most likely, is considerably lower than that at the fastest one $\dot{\epsilon}_v^l(\text{nominal}) = \dot{\epsilon}_0$, where the fully drained condition is least likely. Therefore, the observed effects of strain rate on the stress-strain behaviour can be considered to be due mostly to the intrinsic time- (or rate-) dependency of material property.
- 3) The axial strain rate required to satisfy the drained conditions was analyzed based on the consolidation property for the test material (Hayano et al., 1997). It was found that due to the high stiffness of the test material, the coefficient of consolidation $C_v = k/m_v \gamma_w$ was not very low, despite a low coefficient of hydraulic conductivity. Based on the analysis, an axial strain rate of 0.01%/min ($= \dot{\epsilon}_0$) is enough to achieve satisfactorily drained conditions.

IMPORTANCE OF LOCAL STRAIN MEASUREMENTS

Local and External Strain Measurements

Axial strains were measured locally and externally by the following four methods (Fig. 4).

Fig. 4. Local and external axial strain measurements

(1) Local strain measurements

(a) ① LDT_{ave}; Local Deformation Transducers (see Goto et al., 1991). A pair of LDTs was attached at opposite ends of the specimen diameter along the lateral surface of the membrane. The LDT is a sort of clip gage, which detects changes in the distance between the two ends of a lightly bent phosphor bronze strip with electrical resistance gages attached at the center of the strip. The ones used in the present study were 0.25 mm-thick and 2.5 mm-wide. Axial strains of the specimen were obtained from changes in the distance between a pair of targets (hinges) attached to the specimen surface. As shown in Fig. 5(a), gypsum was used to ensure perfect adhesion using glue between the specimen and the membrane (Fig. 5(a)). As the total length of the LDT was 105 mm, compared to the specimen length of 125 mm, there was a distance of 10 mm between the end of the LDT and the top and bottom ends of the specimen. Therefore, axial strains measured with LDTs could be essentially free from the effects of bedding error and disturbance by trimming occurring at and near the top and bottom ends of specimen. Axial strain reported herein is the average of the readings of the pair of LDTs (it is also the case with the other type of local measurement described below).

(b) ② Gap (local); proximeters (gap sensors). Two pairs of proximeters were set at opposite ends of the specimen diameter also to locally measure axial strains along the lateral surface of the membrane. Axial strains of the specimen were obtained from the relative movement between the two targets (Fig. 5(b)). As these targets were attached on the lateral surface of the specimen essentially at the same elevation as the attachments for LDTs, measured axial strains could also be essentially free from the effects of bedding error.

Fig. 5. Set up of local transducers: (a) LDT and (b) Gap sensor

(2) External strain measurements

(a) ③ Gap (cap); proximeter. Another gap sensor was used to measure the axial movement of a target attached to the lateral surface of the specimen cap. Axial strains of the specimen obtained by this method are not free from the effects of bedding error.

(b) ④ External; displacement transducer. A displacement transducer was placed outside the triaxial cell to measure the axial displacement of the loading piston. Axial strains of the specimen obtained by this method include the effects of bedding error and also errors due to the deformation of the loading shaft, load cell and others (i.e., system compliance).

Long-Term Stability of LDT

As the long-term stable measurement of axial strains is essential for the present study, the long-term stability of LDT was examined in advance under the same condition as the triaxial creep tests (Matsumoto et al., 1999). A single LDT was fixed between a pair of attachments that were glued to two pieces of latex rubber membrane glued

Fig. 6. Examination of long-term stability of LDT: (a) set up of LDT and (b) typical time histories of strain and temperature

to a steel platen (Fig. 6(a)). The tests were performed in a temperature and humidity-controlled room. Figure 6(b) shows typical time histories of axial strain and room temperature after setting the LDT. It may be seen that until the elapsed time became about 1,000 minutes, the LDT length increased at a higher rate compared with those at the later stages. This relatively large initial drift was considered due to the creep deformation of glue and the membrane. However, the creep strain rate was reduced to a rate of about $3 \times 10^{-7}\%$ /min after the initial stage, which was much smaller than the creep strain rates of sedimentary soft rock that were to be analyzed (shown later). As a period of one day is usually necessary for assembling the triaxial cell and other equipment, and for saturating the specimen after setting the LDTs, the initial drift in the reading of an LDT is not usually an actual problem. The long-term stability of LDTs has been confirmed in tests lasting more than two months (San-tucci de Magistris et al., 1999).

Comparison of Local and External Axial Strains

Figure 7 compares the axial strains measured by the four methods described above for the data presented in Fig. 3. A similar data set for sedimentary soft mudstone is presented in Fig. 8. The following trends of behaviour can be seen:

- 1) The effects of system compliance, as noted by the difference between the readings of methods ③ Gap (cap) and ④ External, increased with the axial stress. The difference was basically a function of axial stress, which was essentially constant during each creep loading stage.

Fig. 7. Axial strains measured by four methods during TC and creep loading on silt-sandstone; (a) overall stress-strain relationship, (b) stress-strain relationship at small strains, (c) and (d) time histories of axial strains during creep tests at $q = 0.98 \text{ MPa}$ and 2.45 MPa and (e) comparison of axial strains measured by different methods

- 2) The two local gauges, ① LDT_{ave} and ② Gap (local), exhibited essentially the same strains for the entire range of axial strain.
- 3) The effects of bedding error during TC loading were large, as noted by the difference between the readings of gauge ③ Gap (cap) and the local gauges ① LDT_{ave} and ② Gap (local). Importantly, the bedding error increased significantly during the creep tests as well as in the course of monotonic loading. The difference during monotonic loading at a constant strain rate became larger as the strain rate became lower. It is likely, therefore, that thin disturbed zones were formed at both ends of the specimen during trim-

ming, exhibiting extra time-dependent strains that were considerably larger than those occurring in the main body of the specimen. Similar results have been obtained, without exception, from other similar tests performed by the authors.

These test results indicate that the local axial strain measurement is essential for creep tests and very slow constant strain rate tests on sedimentary soft rocks. It seems that the accuracy of the existing data that were obtained from conventional type triaxial creep tests on soft rocks with external axial strain measurements may have to be re-evaluated. The axial strains locally measured with LDTs were generally slightly smaller than those locally measured

Fig. 8. Axial strains measured by four methods during TC and creep loading on mudstone: (a) overall stress-strain relationship, (b) stress-strain relationship at small strains, (c), (d) and (e) time histories of axial strains during creep tests at $q = 0.98 \text{ MPa}$, 2.45 MPa and 3.43 MPa and (f) comparison of axial strains measured by different methods

with the proximity transducer Gap (cap). The difference could be due partly to the creep displacements at the attachments for LDTs, as shown in Fig. 6(b). However, the recorded difference was not necessarily a function of time elapsed from the start of test. So, the true reason for this difference is not well understood. On the other hand, as a whole, the LDT reading has a higher resolution than the proximity transducer reading. In the following, therefore, local axial strains measured with LDTs will be analyzed.

ISOTACH BEHAVIOUR

Figure 9 shows results obtained from two other similar tests on silt-sandstone and mudstone. It is clearly seen from Figs. 7, 8 and 9 that the stress-strain relationships were significantly affected by instantaneous strain rates. In Fig. 9, immediately after the axial strain rate was suddenly increased by a factor of 10 or 100, the stress-strain relationship exhibited a high tangent stiffness. This was also the case immediately after loading at a constant axial strain rate was restarted following a creep stage. On the

Fig. 9. Stress-strain relationships from drained TC tests including stepwise changes in constant axial strain rate and creep stages: (a) sedimentary silt-sandstone and (b) sedimentary mudstone

Fig. 10. Comparison of results from two drained TC tests performed at different constant axial strain rates and drained TC test including stepwise changes in constant axial strain rate and creep stages: (a) overall stress-strain relationship and (b) stress-strain relationship at small strains (Tatsuoka et al., 2000a)

Fig. 11. Comparison of results from two drained TC tests performed at constant axial strain rates and drained TC test including stepwise changes in constant axial strain rate and creep stages: (a) overall stress-strain relationship and (b) stress-strain relationship at small strains (Tatsuoka et al., 2000a)

Stress State-Dependency of the Quasi-Elastic Young's Modulus

When the horizontal stress σ'_h remains constant under the drained triaxial test condition, the elastic axial strain rate $\dot{\epsilon}_v^e$ can be obtained as;

$$\dot{\epsilon}_v^e = \frac{\dot{\sigma}_v}{E_v} \quad (3)$$

where E_v is the instantaneous vertical quasi-elastic Young's modulus. In the present study, the values of E_v were evaluated by applying three very small unload/reload cycles with a single amplitude axial strain of about 0.001% at several states as shown in Figs. 13(a) and 14(a). Tatsuoka et al. (1999a) showed that for uncemented geomaterials, the quasi-elastic Young's modulus E_A defined for a major principal strain increment in a certain direction A is basically a unique function of the instantaneous normal stress σ'_A in that direction, based on the results from triaxial tests by Hoque and Tatsuoka (1998) on air-dried sands and by Jiang et al. (1997) on partially saturated gravels. Hayano et al. (1999d) also obtained similar experimental results from true triaxial tests on saturated sedimentary soft mudstone (Hayano et al., 1999b). Based on the above, the vertical quasi-elastic Young's modulus

E_v of sedimentary soft rock will be expressed by a function of the vertical stress σ'_v as;

$$E_v = E_{v0} \left(\frac{\sigma'_v}{\sigma'_{v0}} \right)^m \quad (4)$$

where E_{v0} is the E_v value when the vertical stress σ'_v equals σ'_{v0} . The power m represents the stress state-dependency of E_v , which can be obtained by log-log plotting of E_v and σ'_v as Figs. 13(b) and 14(b). The estimated quasi-elastic and irreversible strains obtained based on Eqs. (2) through (4) are plotted against q in Figs. 13(a) and 14(a). The following remarks are relevant:

- 1) E_v at $\sigma'_v = 0$ is not zero (Kim et al., 1994). Equation (4) should therefore be modified when applied to stress states with low values of σ'_v .
- 2) The value of E_{v0} is not perfectly independent of strain rate (Tatsuoka et al., 1999a, 2000a). For the sake of simplicity, however, this rate effect will be ignored in the analysis shown below.

Dependency of Stress on Irreversible Strain and Its Rate

As the quasi-elastic strains are essentially rate-independent, the "isotach" behaviour should be described in terms of irreversible strain ϵ_v^{ir} and its rate $\dot{\epsilon}_v^{ir}$. Moreover,

Fig. 13. (a) Relationships between q and total, elastic and irreversible axial strains of silt-sandstone and (b) stress state-dependency of E_v

Fig. 14. (a) Relationships between q and total, elastic and irreversible axial strains of mudstone and (b) stress state-dependency of E_v

Eq. (1) is not able to predict the following behaviour;

- 1) Intermediate stress-strain behaviour after a step change in the strain rate and before rejoining the new behaviour corresponding to the new constant strain rate, such as behaviour $f \rightarrow c$ by loading at $\dot{\epsilon}_v^t = \dot{\epsilon}_0$ after i) a creep stage ($b \rightarrow f$), or ii) constant strain rate loading at $\dot{\epsilon}_v^t = \dot{\epsilon}_0/100$ ($a \rightarrow f$) in Fig. 12. When following Eq. (1), the strain increment $\Delta \epsilon_v^t$ between states f and c should be zero, which is however not realistic.
- 2) Stress relaxation behaviour: when following Eq. (1), immediately after a stress relaxation test with $\dot{\epsilon}_v^t = 0$ starts, the q value should suddenly drop to a certain lower value of q corresponding to $\dot{\epsilon}_v^t = 0$. This is not realistic either. Therefore, the axial total strain ϵ_v^t and its rate $\dot{\epsilon}_v^t$ in Eq. (1) should be replaced by the irreversible axial strain ϵ_v^{ir} and its rate $\dot{\epsilon}_v^{ir}$ as:

$$q = q(\dot{\epsilon}_v^{ir}, \epsilon_v^{ir}). \quad (5)$$

If the irreversible strain at failure $(\epsilon_v^{ir})_f$ is independent of $\dot{\epsilon}_v^{ir}$ at failure, the maximum deviator stress $q_{\max}$ can be expressed as a unique function of $\dot{\epsilon}_v^{ir}$ at failure, according to Eq. (5). To examine this point, the relationship between the values of $(\epsilon_v^{ir})_f$ and the nominal total axial

Fig. 15. Relationship between irreversible axial strain at failure $(\epsilon_v^{ir})_f$ and nominal axial strain rate at failure for mudstone

strain rate $\dot{\epsilon}_v^t (= \dot{\epsilon}_v^{ir})$ at the peak stress state was summarized for mudstone (Fig. 15). It is seen that despite some scattering in the data, the values of $(\epsilon_v^{ir})_f$ are insensitive to changes in the axial strain rate. When creep failure takes place, such an analysis such as that shown in Fig. 15 is not possible, and another criterion is required. Moreover, for the modeling of post-peak stress-strain behaviour, the development of shear band should be taken into account (Hayano et al., 1999a). These issues are beyond the scope of the present study.

Referring to most of the existing experimental data showing that the maximum deviator stress $q_{\max}$ proportionally increases with the power of the strain rate (e.g., Nishi et al., 1983; Hayano et al., 1999c), the $q_{\max}$ value will be expressed as follows;

$$q_{\max} = q_{\max}(\dot{\epsilon}_v^{ir}) = a \cdot (\dot{\epsilon}_v^{ir})^b \quad (6)$$

where a and b are the material parameters representing the strain rate-dependency of $q_{\max}$, which can be obtained from experimental results (as shown later). According to Eq. (6), as $\dot{\epsilon}_v^{ir}$ approaches zero and infinitive, the value of $q_{\max}$ becomes respectively zero and infinitive. Therefore, Eq. (6) should be considered as an approximated one that is relevant only for a limited range of $\dot{\epsilon}_v^{ir}$. This issue is discussed and a modification to alleviate this drawback is proposed in Tatsuoka et al. (1999b). More research will be necessary to understand the strain rate-dependency of $q_{\max}$.

Normalized Stress-Strain Relationship

The relationships between the normalized stress and strain, $y = q/q_{\max}$ and $x = \epsilon_v^{ir}/(\epsilon_v^{ir})_f$, of silt-sandstone and mudstone obtained from the data shown in Figs. 13(a) and 14(a) are plotted in Figs. 16(a) and (b). The data points represent stress-strain states immediately before the axial strain rate was stepwise changed, where the effects of previous histories of strain rate can be considered to have been nearly erased. It can be seen that each y - x relationship can be assumed to be unique for different strain rates. The y - x relationship will herein be expressed by the following hyperbolic equation:

Fig. 16. y - x relationships from stress-strain relationships shown in Figs. 13(a) and 14(a): (a) silt-sandstone and (b) mudstone

$$y=f(x)=\frac{x}{Q_1+Q_2x} \quad (7)$$

where Q_1 and Q_2 are constants depending on the material property and test conditions. For rigorous fitting of the data, Eq. (7) should have the infinite and zero values of the tangent stiffness ($\partial y/\partial x$) at $x=0$ (for the initial elastic zone) and $x=1.0$ (for smooth transition of $\partial y/\partial x$ at failure between pre- and post-peak curves). Despite the fact that Eq. (7) does not satisfy these conditions, these limitations have no effect on the analysis that is presented below.

By substituting Eqs. (6) and (7) into Eq. (5), the following equation (for the concerned test conditions with a constant σ_h) is obtained:

$$q=q(\dot{\epsilon}_v^{\text{ir}}, \epsilon_v^{\text{ir}})=a \cdot (\dot{\epsilon}_v^{\text{ir}})^b \cdot f(x)=a \cdot \left(\dot{\epsilon}_v^{\text{ir}} - \frac{\dot{q}}{E_v} \right)^b \cdot f\left(\frac{\epsilon_v^{\text{ir}} - \epsilon_v^c}{(\epsilon_v^{\text{ir}})_f} \right) \quad (8)$$

Strain Rate-Dependency (Determination of Parameter b)

Points A and B in Fig. 17 denote two different typical states where the ϵ_v^{ir} value is the same, but the $\dot{\epsilon}_v^{\text{ir}}$ value is different (i.e., $\dot{\epsilon}_{v1}^{\text{ir}} < \dot{\epsilon}_{v2}^{\text{ir}}$). According to Eq. (8), the following relationship is obtained:

Figures 18(a) and (b) show a summary of the q_2/q_1 and $\dot{\epsilon}_2/\dot{\epsilon}_1$ relationship for the silt-sandstone and mudstone, respectively. Rigorously, the ratio $\dot{\epsilon}_2/\dot{\epsilon}_1$ of the nominal axial strain rates before and after a step change in the constant axial strain rate is different from the corresponding ratio $\dot{\epsilon}_{v2}^{\text{ir}}/\dot{\epsilon}_{v1}^{\text{ir}}$ of the irreversible axial strain rate. For the data presented in Fig. 18, however, the elastic axial strain increment is much smaller than the corresponding irreversible axial strain increment and therefore, differences between the two types of strain rate ratio are negligible to obtain the values of b . Moreover, it may be more relevant to describe the strain rate-dependency of stress-strain behaviour in TC tests in terms of irreversible shear strain rate $\dot{\gamma}^{\text{ir}} = \dot{\epsilon}_v^{\text{ir}} - \dot{\epsilon}_h^{\text{ir}}$. This value could not be obtained in the present study as reliable values of $\dot{\epsilon}_h^{\text{ir}}$ were not obtained. On the other hand, the results from drained TC tests on rectangular prismatic specimens of sedimentary soft rock measuring locally both axial and lateral strains (Hayano et al., 1997) showed that the ratio of irreversible lateral to axial strain increments $v_{\text{tan}}^{\text{ir}} = -d\epsilon_h^{\text{ir}}/d\epsilon_v^{\text{ir}}$ increased only slightly with the decrease in the axial strain rate. It was confirmed by using the data of Hayano et al. (1997) that, for the present test conditions, at a given

Fig. 17. Typical q - ϵ_v^{ir} relations to obtain q_2/q_1 and $\dot{\epsilon}_2/\dot{\epsilon}_1$ relations: (a) strain rate increased and (b) strain rate decreased

Fig. 18. q_2/q_1 and $\dot{\epsilon}_2/\dot{\epsilon}_1$ relationships to obtain strain rate-dependency parameter b : (a) silt-sandstone and (b) mudstone

stress state, the ratio of irreversible shear strain rates for different axial strain rates is very similar to the corresponding ratio of irreversible axial strain rates; i.e., $\dot{\gamma}_1^{\text{ir}} / \dot{\gamma}_2^{\text{ir}} = (\dot{\epsilon}_{v2}^{\text{ir}} - \dot{\epsilon}_{h2}^{\text{ir}}) / (\dot{\epsilon}_{v1}^{\text{ir}} - \dot{\epsilon}_{h1}^{\text{ir}}) = (\dot{\epsilon}_{v2}^{\text{ir}}(1 + v_{\tan 2}^{\text{ir}}) / (\dot{\epsilon}_{v1}^{\text{ir}}(1 + v_{\tan 1}^{\text{ir}})) \approx \dot{\epsilon}_{v2}^{\text{ir}} / \dot{\epsilon}_{v1}^{\text{ir}}$. It is very likely therefore that the parameter b does not change significantly even when $\dot{\epsilon}_v^{\text{ir}}$ is replaced by $\dot{\gamma}^{\text{ir}}$ in Eq. (9). In fact, this was the case with poorly graded sands (Di Benedetto et al., 2000). Using these b values, the value a can be obtained from the measured $q_{\max}$ value for a given $(\dot{\epsilon}_v^{\text{ir}})_f$ value, as $a = q_{\max} / \{(\dot{\epsilon}_v^{\text{ir}})_f\}^b$ (Eq. (6)).

SIMULATION OF OBSERVED TIME-DEPENDENT BEHAVIOUR

Based on the simple isotach model described above, the observed stress-strain relationships were simulated for given time histories of $\dot{\epsilon}_v^t$ or q , as shown in Figs. 19, 20 and 21. Referring to Fig. 22, for strain-controlled simulation, the increment dq that satisfies Eq. (9) was sought for a given total axial strain increment $d\epsilon_v^t$ with known quantities of q and $\dot{\epsilon}_v^t$ at stage n . For stress-controlled simulation (including that of creep tests), the increment $d\epsilon_v^t$ that satisfies Eq. (9) was sought for a given dq with known quantities of q and $\dot{\epsilon}_v^t$ at stage n . The fol-

Fig. 19. Comparison between measured and simulated stress-strain relationships for silt-sandstone: (a) overall stress-strain relationships, (b) relationships at small strains, (c) relationships between tangent Young's modulus $E_{\tan}$ and q , (d) and (e) creep behaviour at $q = 0.98$ MPa and 1.96 MPa (see Fig. 9(a))

Fig. 20. Comparison between measured and simulated stress-strain relationships for mudstone: (a) overall stress-strain relationships, (b) relationships at small strains, (c) relationships between tangent Young's modulus E_{tan} and deviator stress q , (d) and (e) time histories of axial strain during creep behaviour at $q = 1.47 \text{ MPa}$ and 2.45 MPa (see Fig. 9(b))

lowing trends of behaviour can be seen from Figs. 19, 20 and 21:

- 1) The observed effects of the constant strain rate and its sudden change on the stress-strain relationship are well simulated. The observed variations in the tangent Young's modulus $E_{tan} = \partial q / \partial \epsilon_v'$ are therefore well simulated. Discrepancy becomes more significant approaching the failure state, which may be due partly to the fact that possible effects of damage to soil structure on the E_v values at large strains (Hayano et al., 1999b) are not considered in this simulation.
- 2) As seen for Figs. 19(d) and (e) and Figs. 20(c) and (d), the observed time histories of axial strain during creep tests are quite well simulated.

These two observations support the relevance of the proposed simple isotach model for the tested sedimentary soft rocks. It is also noted that all the parameters needed for the model can be obtained by changing stepwise the constant axial strain rate several times in a TC test using a single specimen (as presented in Figs. 13 and 19).

Creep Behaviour

According to the proposed model, creep deformation is a process with a continuous decrease in the irreversible strain rate at a constant stress, while satisfying Eq. (5)

(see Fig. 23). We obtain the following equation for creep tests from Eq. (8):

$$dq = \frac{\partial q}{\partial \epsilon_v^{ir}} \cdot d\epsilon_v^{ir} + \frac{\partial q}{\partial \dot{\epsilon}_v^{ir}} \cdot d\dot{\epsilon}_v^{ir} = a \cdot (\dot{\epsilon}_v^{ir})^{b-1} \cdot \left\{ b \cdot f(x) \cdot d\dot{\epsilon}_v^{ir} + \frac{\dot{\epsilon}_v^{ir}}{(\dot{\epsilon}_v^{ir})_f} \cdot \frac{\partial f}{\partial x} \cdot d\dot{\epsilon}_v^{ir} \right\} = 0. \quad (10)$$

From Eq. (11), the following equation is obtained:

$$d\epsilon_v^{ir} = -C_r \cdot \frac{d\dot{\epsilon}_v^{ir}}{\dot{\epsilon}_v^{ir}} \quad (11)$$

where C_r is given as:

$$C_r = \frac{b \cdot (\dot{\epsilon}_v^{ir})_f \cdot f}{\frac{\partial f}{\partial x}}. \quad (12)$$

Here f is the stress level (q/q_{max}) for a given $x = \epsilon_v^{ir}/(\dot{\epsilon}_v^{ir})_f$, and $\partial f/\partial x$ is the normalized tangent stiffness. The value of C_r is continuously changing during a given process of creep deformation. Equation (11) means that the creep strain increment $d\epsilon_v^{ir}$ becomes larger as the value of C_r becomes larger. Equation (12) means that the value of C_r becomes larger with the increase in b , $(\dot{\epsilon}_v^{ir})_f$, and f , and with the decrease in $\partial f/\partial x$. So, the creep strain rate could increase rapidly when approaching the failure state

Fig. 21. Comparison between measured and simulated relationships for silt-sandstone: (a) creep test (Fig. 3(a)) and for mudstone, (b) creep test1 (Fig. 10(a)) and (c) creep test2 (Fig. 11(a))

(because f increases and $\partial f / \partial x$ decreases). It is also seen that materials having a large strain rate-dependency of deformation with a large value of b and those having a large value of $(\dot{\epsilon}_v^{ir})_f$ exhibit larger creep rates.

By approximating the value of C_r to be constant for a given creep process, Eq. (11) can be integrated as follows (Tatsuoka et al., 2000a):

$$\Delta \epsilon_v^{ir} = C_r \cdot \ln \left\{ \frac{\Delta t \cdot (\dot{\epsilon}_v^{ir})_0}{C_r} + 1 \right\} \quad (13)$$

where $\Delta \epsilon_v^{ir}$ is the creep axial strain increment which is

Fig. 22. Integration scheme of Eq. (8)

Fig. 23. Creep behaviour according to simple isotach model

zero at the start of creep test, Δt equals 0, and $(\dot{\epsilon}_v^{ir})_0$ is the initial irreversible axial strain rate at the start of the creep test. The simulated results based on Eq. (13) using the initial value of C_r at the start of the creep stage are shown in Figs. 19(d) and (e) and Figs. 20(c) and (d). The difference between this approximated simulation and the simulation by direct integration of Eq. (8) is not so large in this case with relatively small creep strains.

Equation (13) has a logarithmic form as some of the existing empirical logarithmic creep laws (Tatsuoka et al., 2000a). Equation (13) indicates that the creep rate increases with the increase in $(\dot{\epsilon}_v^{ir})_0$. Figure 24(a) compares the measured time histories of axial strain obtained from similar creep tests, shown in Fig. 8(a), together with their simulated relations by the proposed model. In this test, the creep rate at $q = 3.43 \text{ MPa}$ is smaller than that at $q = 2.45 \text{ MPa}$ despite a larger q for the former. According to the proposed model, this behaviour is due to the fact that the value of $(\dot{\epsilon}_v^{ir})_0$ was much smaller (by a factor of 100) at $q = 3.43 \text{ MPa}$ than at $q = 2.45 \text{ MPa}$. This feature is reasonably simulated by the proposed model. Figure 24(b) shows a similar comparison of creep behav-

Fig. 24. Comparison between measured and simulated time histories of axial strain during creep tests: (a) creep tests at $q=0.98$, 2.45 and 3.43 MPa (see Fig. 8(a)) and (b) creep tests at $q=2.45$ MPa (see Figs. 10 and 11)

our at the same $q=2.45$ MPa but with initial strain rates $(\dot{\epsilon}_v^{\text{ir}})_0$ differing by a factor of 100, presented in Figs. 10 and 11. It is readily seen that the creep rate is much smaller with a smaller $(\dot{\epsilon}_v^{\text{ir}})_0$. This feature is reasonably simulated by the proposed model. Now, we understand that it is crucial to control the initial strain rates when performing creep tests at a certain stress level and to take into account their effects when interpreting creep test results.

ON THE VARIABLE FOR MODELS

The experimental data shown above indicate that “time t ” could not be the basic variable to describe the time-dependent stress-strain behaviour of sedimentary soft rock (and other types of geomaterials; Tatsuoka et al., 1999b, 2000a). For example, in Fig. 12, when $t=0$ is defined at state a, it is readily seen that the elapsed time at state c is much larger when path $a \rightarrow b \rightarrow f \rightarrow c$ is traversed than when path $a \rightarrow b \rightarrow c$ is traversed. The behaviour $c \rightarrow d$ for the same strain rate is essentially independent of the time that has elapsed by state c. This point will be discussed again later in this paper. That is, referring to Fig. 25, the time period required to reach the same stress-

Fig. 25. Schematic diagram to explain Eq. (16)

strain state is not unique for the same origin of time defined at a certain initial stress-strain state. This feature is expressed mathematically as follows according to the simple isotach model. From Eq. (8), we obtain:

$$\dot{\epsilon}_v^{\text{ir}} = \left(\frac{q}{a \cdot f(x)} \right)^{1/b} \quad (14)$$

$$dt = \left(\frac{a \cdot f(x)}{q} \right)^{1/b} \cdot d\epsilon_v^{\text{ir}} = \left(\frac{a \cdot f(x)}{q} \right)^{1/b} \cdot (\epsilon_v^{\text{ir}})_f \cdot dx. \quad (15)$$

By integrating Eq. (15) from $t=0$ to $t=t_1$, we obtain,

$$t_1 = a^{1/b} \cdot (\epsilon_v^{\text{ir}})_f \cdot \int_{x_0}^{x_1} \left\{ \frac{f(x)}{q} \right\}^{1/b} dx. \quad (16)$$

Equation (16) means that for the same initial stress-strain state with $x=x_0$ at $t=0$, the time period t_1 that is needed to reach the target stress-strain state with $x=x_1$ is shorter when traversing an intermediate stress-strain path with higher deviator stresses q (Fig. 25). Indeed, “time t ” is paradoxically not a proper variable to model the time effects on the stress-strain behaviour of sedimentary soft rocks and also other types of geomaterials (Tatsuoka et al., 1999b, 2000a). Therefore, any relevant constitutive model to describe the intrinsic time-dependent stress-strain property of geomaterials cannot include “time” as an objective variable.

CONCLUSIONS

A series of consolidated drained triaxial compression tests was conducted to investigate the time- (or rate-) dependency of the stress-strain properties of sedimentary silt-sandstone and mudstone. The test results showed the importance of local strain measurement even during creep loading at constant stress states.

The constant axial strain rate was changed stepwise and creep tests were performed during monotonic shearing. The test results showed that the stress state is controlled uniquely by the instantaneous irreversible strain and its rate whether the strain rate is kept constant, or changing stepwise or gradually (as during creep tests) (i.e., the isotach property). Based on these findings, a sim-

ple isotach model was proposed to simulate the obtained test results. It was found that, although the analyzed test conditions were limited, the proposed model could simulate well many different aspects of the observed stress-strain-time behaviour. The model can also simulate the experimental fact that the creep rate increases with the increase in the initial strain rate. A testing procedure using a single specimen by which all the parameters needed for the model is also proposed.

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REFERENCES

- 1) Ampadu, S. K. and Tatsuoka, F. (1993): Effects of setting method on the behavior of clays in triaxial compression from saturation to undrained shear, *Soils and Foundations*, **33** (2), 14–34.
- 2) Clayton, C. R. I., Gordon, M. A. and Matthews, M. C. (1994): Measurement of stiffness of soils and weak rocks using small strain laboratory testing and field geophysics, *Proc. 1st International Symposium on Pre-failure Deformation Characteristics of Geomaterials*, Sapporo, **1**, 229–234.
- 3) Di Benedetto, H., Tatsuoka, F. and Ishihara, M. (2000): Time-dependent deformation characteristics of sand and their constitutive modeling, *Soils and Foundations* (submitted).
- 4) Goto, S., Tatsuoka, F., Shibuya, S., Kim, Y. S. and Sato, T. (1991): A simple gauge for local small strain measurements in the laboratory, *Soils and Foundations*, **31** (1), 169–180.
- 5) Hayano, K., Sato, T. and Tatsuoka, F. (1997): Deformation characteristics of a sedimentary soft mudstone from triaxial compression tests using rectangular prism specimens, *Géotechnique*, **47** (3), 439–450.
- 6) Hayano, K., Maeshiro, T., Tatsuoka, F., Sato, T., Wang, L. and Kodaka, T. (1999a): Shear banding in a sedimentary soft mudstone subjected to plane strain compression, *Geotech. Testing J.*, **22** (1), 67–79.
- 7) Hayano, K., Tatsuoka, F. and Yoshiizumi, N. (1999b): Modeling pre-failure stress-strain properties of sedimentary softrocks based on very small strain stiffness, *ASTM STP 1350, Nondestructive and Automated Testing for Soil and Rock Properties*, 259–275.
- 8) Hayano, K., Koseki, J. and Tatsuoka, F. (1999c): Fundamental study on the time effects on non-linear stress-strain properties of sedimentary mudstone from triaxial tests, *Proc. 34th Japan National Conf. on Geotech. Engrg.*, JGS, Tokyo (in Japanese).
- 9) Hayano, K., Koseki, J. and Tatsuoka, F. (1999d): Stress-induced anisotropy in the quasi-elastic stiffness of sedimentary soft mudstone, *Poster Session Proc. of the 11th Asian Regional Conf. on SMFE*, Seoul, 11–12.
- 10) Hoque, E. and Tatsuoka, F. (1998): Anisotropy in the elastic deformation of materials, *Soils and Foundations*, **38** (1), 163–179.
- 11) Jardine, R. J., Brookes, N. J. and Smith, P. R. (1985): The use of electrical transducers for strain measurements in triaxial tests on weak rock, Technical note, *Int. J. of Rock Mech. Min. Sci. and Geomech. Abstr.* **22** (5), 331–337.
- 12) Jiang, G. L., Tatsuoka, F., Flora, A. and Koseki, J. (1997): Inherent and stress state-induced anisotropy in very small strain stiffness of a sandy gravel, *Géotechnique*, **47** (3), 509–521.
- 13) Kim, Y. S., Tatsuoka, F. and Ochi, K. (1994): Deformation characteristics at small strains of sedimentary softrocks by triaxial compression test, *Géotechnique*, **44** (3), 461–478.
- 14) Matsumoto, M., Hayano, K., Sato, T., Koseki, J. and Tatsuoka, F. (1999): Time effects on stress-strain properties at small strains of sedimentary soft mudstone, *Proc. 2nd Int. Symp. on Pre-failure Deformation Characteristics of Geomaterials*, Torino, **1**, 313–322.
- 15) Momoya, Y., Ishi, T. and Tatsuoka, F. (1998): Strain rate-dependency of deformation of NC clay and prediction of undrained creep, *Proc. 33rd Japan National Conf. on Geotech. Engrg.*, JGS, Yamaguchi, 615–616 (in Japanese).
- 16) Nishi, K., Okamoto, T. and Esashi, Y. (1983): Strength-deformation characteristics of mudstone under some kinds of loading conditions and its unified interpretation, *J. of Geotech. Engrg.*, JSCE, (338), 149–158 (in Japanese).
- 17) Santucci de Magistris, F., Koseki, J., Amaya, M., Hamaya, S., Sato, T. and Tatsuoka, F. (1999): A triaxial testing system to evaluate stress-strain behaviour of soils for wide range of strain and strain rate, *Geotech. Testing J.*, **22** (1), 44–60.
- 18) Suklje, L. (1969): *Rheological Aspects of Soil Mechanics*, Wiley-Interscience, London.
- 19) Tatsuoka, F., Sato, T., Park, C. S., Kim, Y. S., Mukabi, J. N. and Kohata, Y. (1994): Measurements of elastic properties of geomaterials in laboratory compression tests, *Geotech. Testing J.*, **17** (1), 80–94.
- 20) Tatsuoka, F. and Kohata, Y. (1995): Stiffness of hard soils and softrocks in engineering applications, *Proc. 1st Int. Symp. on Pre-failure Deformation Characteristics of Geomaterials*, Sapporo, **2**, 947–1063.
- 21) Tatsuoka, F., Kohata, Y., Tsubouchi, T. and Ochi, K. (1995): Stiffness of sedimentary soft rocks evaluated by triaxial compression tests, *Proc. of the 8th Int. Congress on Rock Mechanics*, Tokyo, **3**, 1201–1204.
- 22) Tatsuoka, F., Ochi, K., Tsubouchi, T., Kohata, Y. and Wang, L. (1997): Sagami-hara experimental underground excavations in sedimentary softrock, *Geotech. Engrg., Proc. Institution of Civil Engineers*, (125), 206–223.
- 23) Tatsuoka, F., Jardine, R. J., Lo Presti, D., Di Benedetto, H. and Kodaka, T. (1999a): Characterizing the pre-failure deformation properties of geomaterials, *Theme Lecture for the Plenary Session No. 1*, 14th ICSMFE, Hamburg, **4**, 2129–2164.
- 24) Tatsuoka, F., Uchimura, T., Hayano, K., Di Benedetto, H., Koseki, J. and Siddiquee, M. S. A. (1999b): Time-dependent deformation characteristics of stiff geomaterials in engineering practice, *Theme Lecture, Pre-print of the 2nd Int. Symp. on Pre-failure Deformation Characteristics of Geomaterials*, Torino.
- 25) Tatsuoka, F., Santucci de Magistris, F., Hayano, K., Momoya, Y. and Koseki, J. (2000a): Some new aspects of time effects on the stress-strain behaviour of stiff geomaterials, *The Geotechnics of Hard Soils-Soft Rocks (Evangelista, A. and Picarelli, L. eds.)*, 1285–1372.
- 26) Tatsuoka, F., Ishihara, M., Di Benedetto, H. and Kuwano, R., (2000b): Time-dependent deformation characteristics of geomaterials and their simulation, *Soils and Foundations* (submitted).
- 27) Thompson, R. P., Newman, R. L. and Davis, P. D. (1993): Advances in the in-situ testing of weak mudstone, *Proc. 26th Annual Conf. Engrg. Group of the Geological Society*, Leeds, 303–310.
- 28) Vaid, Y. P. and Campanella, R. G. (1977): Time-dependent behaviour of undisturbed clay, *J. of the Geotech. Engrg. Div., ASCE*, **103** (7), 693–709.