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ANISOTROPY OF SOILS AND ITS MODELLING
L'ANISOTROPIE DE SOLS ET MODELISATION

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SYNOPSIS: Anisotropy of clay was investigated on the basis of undrained triaxial tests and direct shear tests on natural clay and one-dimensionally consolidated clay. The observed anisotropic behaviour was uniquely analysed using a kinematic hardening model for clay in which stress induced anisotropy was taken into consideration. On the other hand, it was experimentally shown that anisotropy of deposited sand was different from that of clay, and the main cause of anisotropy was inherent anisotropy. An elastoplastic model for sand incorporating the fabric anisotropic tensor was able to describe well the anisotropic stress-strain relationship and strength in isotropic compression, triaxial compression and plane strain tests on deposited sand.

INTRODUCTION

It is well known that natural clay and one-dimensionally consolidated clay exhibit anisotropic behaviour in their deformation and strength characteristics. In particular, the influence of anisotropy on undrained strength of clay (c_u/p) has been often investigated experimentally. Furthermore, deposited sand also shows anisotropy, and experimental investigation on its stress-strain behaviour and strength has been carried out systematically by various researchers. On the other hand, many constitutive models for clay and sand have also been presented. Most of them, however, have not taken anisotropy into consideration in their modelling.

In the present study, we discuss firstly about the difference of anisotropy between clay and sand on the basis of experimental data already reported. Then we show that these anisotropies can be divided mainly into stress induced anisotropy and inherent anisotropy, respectively. Next, these anisotropic behaviours are analysed by the authors' elastoplastic models for clay and sand in which both stress induced and inherent anisotropy can be properly taken into consideration. In these models, stress induced anisotropy is introduced using kinematic hardening rule in the stress ratio space, and inherent anisotropy by fabric anisotropy tensor.

OBSERVED ANISOTROPY OF SOILS

Clay

Many experimental studies on anisotropy of clay (particularly, undrained strength) have been performed. We will discuss here about anisotropy of natural clay and one-dimensionally consolidated clay on the basis of some data reported before. Adachi et al (1991) carried out several undrained triaxial compression tests on undisturbed natural clay (Eastern Osaka clay), in which the angle β_y between the major principal stress plane and the bedding plane is different in each test (see Fig. 1), e.g., the angle β_y=0 implies that the direction of major principal stress is orthogonal to the bedding plane, and β_y=90° that the

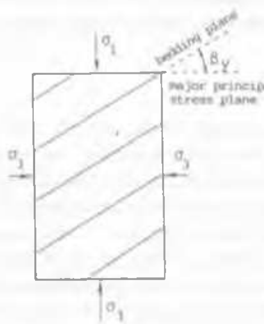


Fig.1 Angle β_y between major principal stress plane and bedding plane

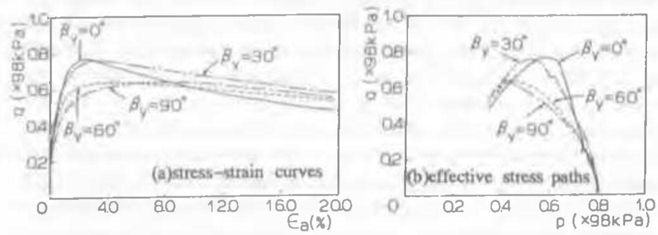


Fig.2 Results of undrained triaxial compression tests on clay (after Adachi et al (1991))

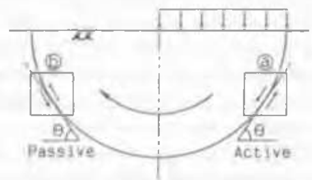
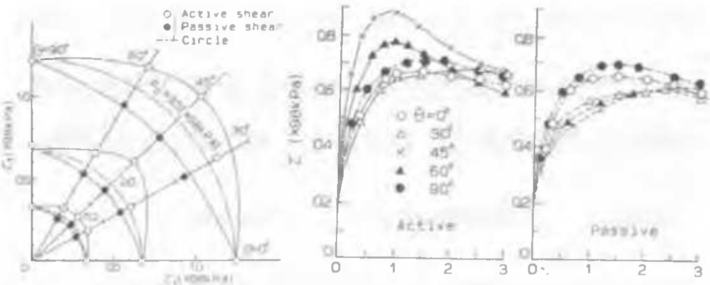


Fig.3 Active and passive shear of one-dimensionally consolidated clay (after Mikasa et al (1984))

direction of major principal stress is parallel to the bedding plane. Figure 2 shows test results (figure (a) is the deviatoric stress q vs. axial strain ε_a, and figure (b) the effective stress paths in deviatoric stress q– mean principal stress p plane). We can see from these figures that as the angle β_y increases, the stiffness ($dq/d\varepsilon_a$) decreases, the strength (q_{max}) becomes lower, and the pore water pressure developed by negative dilatancy is higher. It can also be seen that the internal friction angle ϕ' is independent of the angle β_y.

Performing undrained direct shear tests on one-dimensionally consolidated clay, Mikasa et al (1984) discussed the effect of anisotropy of clay on strength. It seems from Fig.3 that elements ⓐ and ⓑ, lying on a slip surface in one-dimensionally consolidated clay foundation ($\sigma_1=\sigma_v=p_o$, $\sigma_3=\sigma_h=K_o p_o$) and subjected to a strip load, are under stress states similar to direct shear tests (or simple shear tests) in which angle θ between shear plane and horizontal plane are the same but direction of shear stress are opposite in each elements respectively. They named the stress state of ⓐ active and ⓑ passive, respectively, and carried out several direct shear tests with different angle θ. Figure 4 (a) shows polar diagram of the relation between shear strength τ_f and angle θ, and Fig. 4 (b) the shear stress – shear deformation curves under active and passive states. It can be seen from these figures that shear strength and stiffness are higher in the active state than passive state, and these are the highest at θ=45° in active state and the smallest at θ=45° in passive state. Furthermore, it is known that if anisotropic clay is consolidated isotropically at a pressure much higher than in-situ pressure, the anisotropic structure will be destroyed and the clay becomes isotropic.



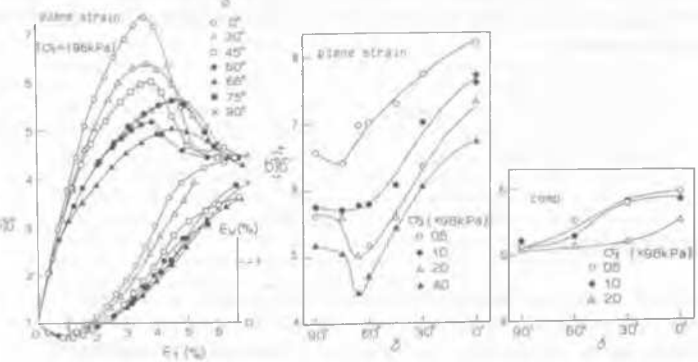
(a)polar diagram of relation between shear strength τ_f and angle θ

(b)shear stress–shear displacement curves in active and passive states

Fig.4 Results of undrained direct shear tests on clay (after Mikasa et al (1984))

Sand

Oda et al (1978) performed systematically plane strain tests and triaxial compression tests on artificially deposited Toyoura sand. Samples were prepared by pouring sand vertically. Various angles δ between major principal stress plane and bedding plane were employed for sample preparation (the definition of δ is the same as that of β_y in Fig.1). The observed stress–strain curves of plane strain tests are shown in Fig.5 (a). The observed relation between principal stress ratio at failure $(\sigma_1/\sigma_3)_f$ and the angle δ in plane strain and triaxial compression tests are shown in Fig.5 (b). As shown in these figures, the strength, stiffness and dilatancy are the highest when major principal stress is perpendicular to the bedding plane (i.e. δ=0°) in the same way as those of Fig. 2. Here, it should be noted that though the tests on sand in Fig. 5 were carried out under drained condition, the tests on clay in Fig. 2 and 4 were under undrained condition. Namely, the maximum deviatoric stress of anisotropic clay depends on the angle β_y, but its maximum stress ratio is independent of β_y. On the other hand, the maximum stress ratio of sand varies with the angle δ. It can also be seen in Fig.5 that at the same confining pressure and the same angle δ, the strength of sand is higher under plane strain condition than under triaxial compression condition, and the difference of strength due to anisotropy is more substantial under plane strain condition.



(a)stress–strain curves in plane strain tests

(b)relation between stress ratio at failure and angle δ

Fig.5 Results of plane strain and triaxial compression tests on sand (after Oda et al (1978))

Figure 6 shows the results of cyclic isotropic compression tests on anisotropic Toyoura sand whose bedding plane is horizontal (δ=0°), arranged in terms of the relation between axial strain ε_a and volumetric strain ε_v. Confining pressure is varied between 49kPa and 784kPa. It can be seen from this figure that the observed relation does not satisfy $\varepsilon_v=3\varepsilon_a$ (dash–dotted line) under virgin loading because of anisotropy, and the degree of anisotropy is not altered very much even if confining pressure becomes higher. On the other hand, the sand deforms isotropically under unloading and reloading ($d\varepsilon_v \approx 3d\varepsilon_a$), so the elastic deformation of sand is not influenced greatly by anisotropy.

Table 1 Values of soil parameters of Fujinomori clay

$\lambda/(1+e_o)$	5.08×10^{-2}
$\kappa/(1+e_o)$	1.12×10^{-2}
$\phi'_{(comp)}$	33.7°
α	0.7
ξ	0.2

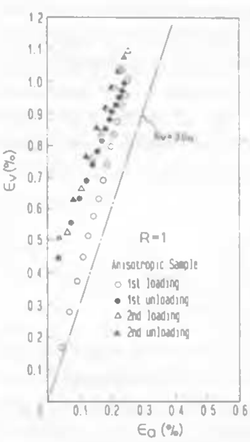


Fig.6
Observed volumetric strain vs. axial strain in isotropic compression test on sand

Table 2 Values of soil parameters of Toyoura sand

C_t	0.84×10^{-2}
C_e	0.60×10^{-2}
m	0.3
$R_f=(\sigma_1/\sigma_3)_{f(comp)}$	4.7
$D_f=(d\varepsilon_v/d\varepsilon_1)_{f(comp)}$	−0.6
α	0.85
ξ	0.3
b_1/b_3	0.9,0.8

MODELLING OF SOIL ANISOTROPY

As described in the previous section, deposited clay and sand show anisotropic characteristics in nature. In clay, even though anisotropy is observed in deformation behaviour, the internal friction angle is almost independent of anisotropy. Furthermore, the degree of anisotropy decreases under isotropic compression with increasing confining pressure. On the other hand, the internal friction angle of sand varies with anisotropy, and the degree is not altered under isotropic compression. From these experimental evidence, it can be postulated that clay and sand differ from each other in characteristics of anisotropy; i.e., anisotropy of natural clay and one-dimensionally consolidated clay is mainly characterized by stress induced anisotropy, and deposited sand by inherent anisotropy.

The authors have developed elastoplastic models for clay and sand in which these anisotropies are taken into consideration (Nakai and Hoshikawa, 1991; Nakai and Funada, 1992). Since the detailed formulation of these models has been described in these papers, we will show here only the ways to introduce these anisotropies into the elastoplastic models and will simulate the various tests on clay and sand in the previous section.

Modelling of Stress Induced Anisotropy in Clay

Since stress induced anisotropy of soils is caused not by change in shear stress but by change in stress ratio, anisotropy of clay is described using a kinematic hardening rule in stress ratio space. Isotropic hardening model for clay using t_{ij} concept gives its yield function f as a function of stress invariants (Nakai and Matsuoka, 1986)

$$f=f(t_N, X\equiv t_S/t_N)=0 \qquad (1)$$

Here, t_N, t_S and $X\equiv t_S/t_N$ are the mean stress, the deviatoric stress and the stress ratio based on t_{ij} concept (detail of t_{ij} concept and definition of stress quantities are described in the papers by Nakai and Matsuoka (1986) and Nakai and Hoshikawa (1991)). In the kinematic hardening model named kinematic t_{ij}–clay model (Nakai and Hoshikawa, 1991), replacing X in Eq.(1) by X^*+n, we give the yield function by the form of

$$f=f(t_N, X^*+n)=0 \qquad (2)$$

where, X^* and n are expressed as

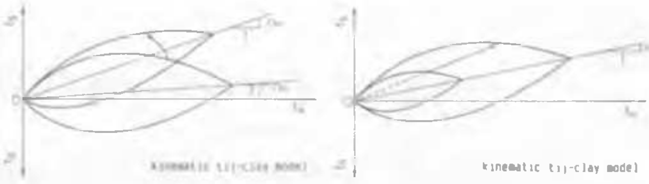
$$X^*=\sqrt{(x_{ij}-n_{ij})(x_{ij}-n_{ij})} \qquad (3)$$

$$n=\sqrt{n_{ij}n_{ij}} \qquad (4)$$

The stress ratio tensor x_{ij} is given by

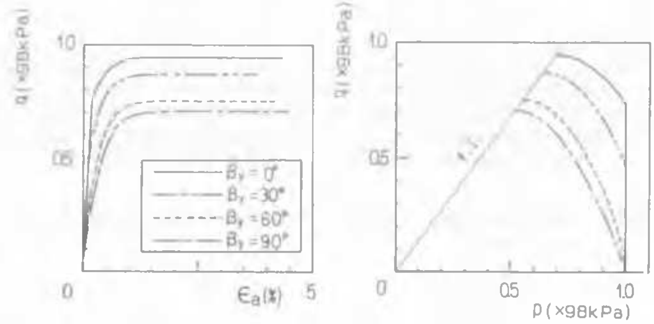
$$x_{ij}=\frac{t_{ij}}{t_N}-a_{ij} \qquad (5)$$

and n_{ij} is a tensor which represents the position of centre of yield surface in x_{ij} space. The tensor n_{ij} is then assumed to be movable while keeping constant distance $X^*=\xi$ between x_{ij} and n_{ij}. Under a monotonic loading without rotation of principal stress axes from isotropic stress condition, X^*+n is almost equal to X. Hence the results calculated by isotropic hardening model and kinematic hardening model become identical under such stress paths. Figure 7 illustrates schematically the change of yield surface in t_N–t_S space under shear loading and proportional loading. The yield surface rotates about the origin under



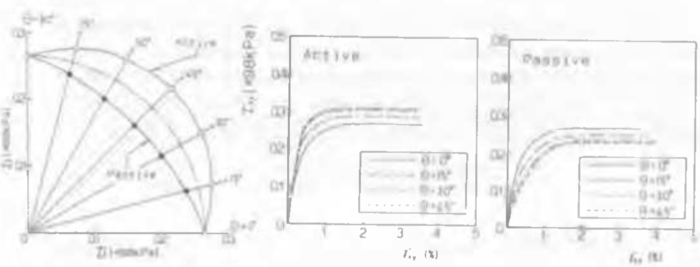
(a)under shear loading (b)under proportional loading

Fig.7 Evolution of yield surface of kinematic t_{ij}–clay model



(a)stress–strain curves (b)effective stress paths

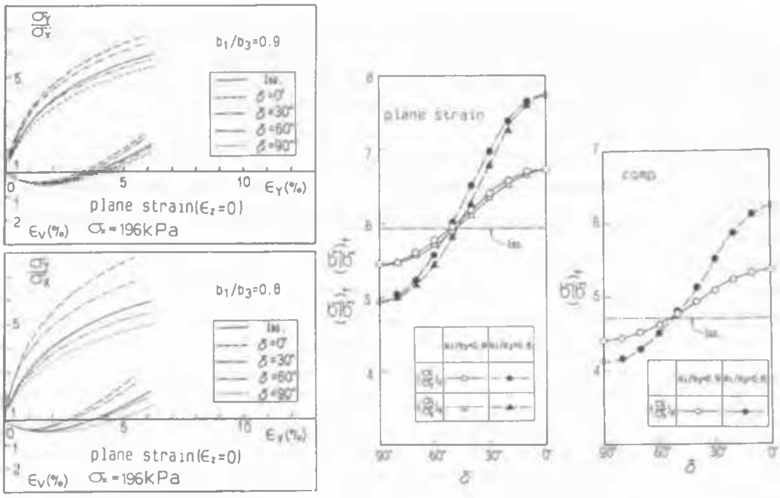
Fig.8 Analytical results of undrained triaxial compression tests on clay (p_o=98kPa)



(a)polar diagram of relation between shear strength τ_f and angle θ (b)shear stress–shear strain curves in active and passive states

Fig.9 Analytical results of undrained simple shear tests on clay (p_o=98kPa)

shear loading, because kinematic hardening rule is assumed in stress ratio space. On the other hand, the yield surface expands isotropically or anisotropically under proportional loading where stress ratio does not change. Now, this model considers not only stress induced anisotropy but also the influence of the intermediate principal stress on deformation and strength by using t_{ij} concept and the influence of stress path on the direction of plastic flow by dividing the plastic strain increment into two components. Figures 8 and 9 are the analytical results corresponding to the test results in Figs. 2 and 4 respectively. These analyses are done by assuming an one-dimensionally consolidated clay ($\sigma_1=\sigma_v$=98kPa, $\sigma_3=\sigma_h=K_o\sigma_1$=49kPa) and using soil parameters of Fujinomori clay in Table 1. The soil parameter added to extend isotropic hardening model to kinematic hardening model is ξ alone. It can be seen from the comparison between these figures that the kinematic hardening model describes well various observed distinct features of anisotropic clay.



(a)stress-strain curves in plane strain tests

(b)relation between stress ratio at failure and angle δ

Fig.10 Analytical results of plane strain and triaxial compression tests on sand

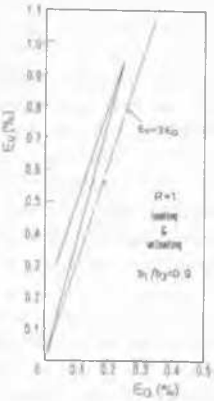


Fig.11
Analytical volumetric strain vs. axial strain in isotropic compression test on sand

Modelling of Inherent Anisotropy in Sand

Since anisotropy of sand which is formed during deposition process hardly changes with subsequent stress change, such anisotropy can be regarded as inherent anisotropy. In order to take into account inherent anisotropy, stress ratio tensor x_{ij} is modified by a fabric anisotropy tensor b_{ij}. Thus, stress ratio X^* in Eq. (2) is replaced by $\bar{X}^*$ as follows:

$$f=f(t_N, \bar{X}^*+n)=0 \qquad (6)$$

where $\bar{X}^*$ is expressed as

$$\bar{X}^*=\sqrt{(\bar{x}_{ij}-n_{ij})(\bar{x}_{ij}-n_{ij})} \qquad (7)$$

and $\bar{x}_{ij}$ is defined as follows using fabric anisotropy tensor b_{ij}

$$\bar{x}_{ij}=\frac{1}{2}\frac{t_{ik}b_{kj}+b_{ik}t_{kj}}{t_N} - a_{ij} \qquad (8)$$

The fabric anisotropy tensor b_{ij} is a symmetric tensor whose trace b_{ii} is 3. Furthermore, if $b_{ij}=\delta_{ij}$ (unit tensor), $\bar{x}_{ij}$ is identical to x_{ij}. For naturally deposited sand, since two of the three principal values of b_{ij} are identical (transverse isotropy), i.e. $b_2=b_3$, the tensor b_{ij} can be determined from the direction of deposition, the ratio of its principal values (b_1/b_3) and the condition of $b_{ii}=3$. Here, $b_1/b_3=1$ implies isotropic sand, and smaller values of b_1/b_3 means higher degree of anisotropy. The rest of formulations of the model are the same as those without inherent anisotropy. Figures 10 and 11 are the analytical results corresponding to the test results in Figs 5 and 6, respectively. Analyses are done for the cases of b_1/b_3=1.0 (isotropic), 0.9 and 0.8 using soil parameters of medium dense Toyoura sand in Table 2. In order to consider inherent anisotropy, only the soil parameter b_1/b_3 is added. The analytical results describe well the observed deformation and strength characteristics of anisotropic sand in Figs. 5 and 6.

CONCLUSIONS

The main results of the present study are summarized as follows:

(1) Based on undrained test results reported before it is indicated that anisotropy of natural clay and one-dimensionally consolidated clay has influences on deformation and maximum shear stress but does not affect the internal friction angle. This anisotropy can be analysed using an elastoplastic model for clay in which stress induced anisotropy only is considered with kinematic hardening rule in stress ratio space.

(2) Observed results of various tests on deposited sand show that the effect of anisotropy is not only on deformation but also on internal friction angle. This anisotropy can be analysed using an elastoplastic model for sand in which inherent anisotropy is considered by the introduction of a fabric anisotropic tensor.

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