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ANISOTROPY OF UNDRAINED SHEAR STRENGTH OF AN OVER-CONSOLIDATED SOIL BY TRIAXIAL AND PLANE STRAIN TESTS

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ABSTRACT

In order to find out the real cause leading to the anisotropy of undrained shear strength of clayey soils, two types of undrained shear tests were carried out; 1) undrained plane strain tests on lightly over-consolidated kaolin clay, and 2) undrained triaxial tests on heavily over-consolidated kaolin clay. Analysis of the data has shown that the anisotropy of undrained shear strength of over-consolidated kaolin clay is primarily caused by the direction-dependent (anisotropic) shear strength parameters c' and ϕ' in terms of effective stresses. The anisotropy of pore water pressure parameter A_f is artificially generated as a result of the anisotropic change of the undrained shear strength itself, rather than of the anisotropic excess pore-pressure development during shearing, and is of secondary, if any, importance to interpret why the undrained shear strength is anisotropic. Intensity of the anisotropy in the undrained shear strength is not altered much by either changing the boundary conditions, such as triaxial and plane strain or increasing the over-consolidation ratio up to 32. The minimum values of cohesion c' and friction angle ϕ' appear when a specimen is compressed with the major principal stress inclined at about 30° to the consolidation plane. When a specimen is compressed either normal or parallel to the consolidation plane, values of cohesion c' and friction angle ϕ' are high with no significant difference between them. The difference between the maximum and the minimum friction angle ϕ' is almost the same as that observed in the triaxial tests of sands. In the case of sands, however, a specimen compressed normal to the bedding plane yields the maximum friction angle ϕ', while a specimen compressed parallel to the bedding plane yields the minimum value in the triaxial tests and a value close to the minimum in the plane strain tests. These observations suggest that the micro-deformation mechanism leading to the strength anisotropy is quite similar in both clay and sand, but not exactly the same.

Key words: inherent and induced anisotropy, micro-structure, over consolidated clay, plane strain test, triaxial test, undrained shear strength (**IGC**: D6)

INTRODUCTION

It is well known that soil particles, especially platy clay minerals, tend to align their faces perpendicular to the direction of K_0-consolidation so that the micro-structure of the soil skeleton becomes inherently anisotropic (e.g., Mitchell, 1956; Martin, 1962; Kazama, 1996). Here, this is tentatively called the inherent anisotropy. As a result of the inherent anisotropy, the K_0-consolidated soil is expected to be anisotropic in the mechanical behavior. The problem is not so simple, however, since the inherent anisotropy can be easily modified during the successive stages of shearing to reconstruct a different anisotropic micro-structure, which is called the induced anisotropy. Casagrande and Carillo (1944) first distinguished two types of anisotropy, i.e., the inherent anisotropy and the induced anisotropy. From a micro-structural point of view, however, there is no difference between them. Micro-structure formed during sedimentation changes during K_0-consolidation, so that the micro-structural change during K_0-consolidation can be said to be the induced anisotropy. In this paper, however, the inherent anisotropy is tentatively referred to as the micro-structural anisotropy developed at the end of K_0-consolidation, and any micro-structural changes as a result of application of shear deformation after that are called the induced anisotropy. Besides, even isotropic soils, in the sense of micro-structure, respond anisotropically to stress change under an undrained condition (e.g., Hansen and Gibson, 1949; Duncan and Seed, 1966a and b; Ohta and Nishihara, 1985). This anisotropy was called the stress induced anisotropy by Ohta and Nishihara (1985). The stress induced anisotropy takes place only when the initial stress prevailing during K_0-consolidation

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changes by increasing shear stress up to failure under the undrained condition, in parallel with the principal stress rotation. It can be said, accordingly, that anisotropy of soils must be carefully discussed by taking these three independent sources of anisotropy into account, i.e., inherent anisotropy, induced anisotropy and stress induced anisotropy.

Many experimental studies have shown that the undrained shear strength of clayey soils changes depending on the direction of shear with respect to the major axis of consolidation (e.g., Aas, 1965; Duncan and Seed, 1966a and b; Bishop, 1966; Nakase and Kamei, 1983; Kazama, 1996). Discussing such anisotropic undrained shear strength, for example, Duncan and Seed (1966a) differentiated two types of anisotropy: 1) anisotropy with respect to the values of the shear strength parameters, c' and ϕ' in terms of effective stresses, and 2) anisotropy with respect to the excess pore-pressure developed while shearing up to failure, which is known as anisotropy induced by the pore-pressure coefficient A_f. In spite of such a separation, however, both types of anisotropy are basically caused by the same origin, i.e., anisotropic micro-structure developed during K_0-consolidation. The second type of anisotropy by Duncan and Seed should not be confused with the stress induced anisotropy although both commonly appear, in parallel, during shear under undrained condition. Furthermore, Duncan et al. (1966a) have shown that the anisotropy by the pore-pressure coefficient is primarily responsible for the observed difference in the undrained strengths of horizontal and vertical specimens, depending only slightly, if any, on anisotropy of the shear strength parameters in effective stresses. Bishop (1966) drew the same conclusion from undrained tests on London clay, but also pointed out the possibility that the strength parameters could be anisotropic even in terms of effective stresses.

In the case of cohesionless soils, non-spherical particles lie on the horizontal plane (called the bedding plane) with their long axes parallel to it when deposited under the action of gravity. As a result of the anisotropic particle alignment, not only friction angle ϕ' but also a deformation characteristic such as dilatancy change considerably depending on the major principal stress direction with respect to the gravitational direction during deposition (e.g., Arthur and Menzies, 1972; Arthur et al., 1977; Oda, 1972a; Oda, Koishikawa and Higuchi, 1978; Mahmood and Mitchell, 1974; Tatsuoka et al., 1990). Oda et al. (1978) have also found that such anisotropy in the shear strength parameters is more pronounced in the plane strain test than in the triaxial test. To explain this fact, they pointed out the importance of the change of particle orientation during shearing (induced anisotropy). During a triaxial test, particles roll to some extent, so that the initial micro-structure is considerably altered (Oda, 1972b). In this case, the inherent anisotropy by particle orientation gives only a minor, but still appreciable, effect on the friction angle ϕ'. In a plane strain test, on the other hand, the inherent anisotropy is well preserved until failure due to the condition that no strain is allowed in the intermediate principal stress direction, and the inherent anisotropy therefore has a large effect on the friction angle ϕ'. This may be supported by the fact that the shear strain at failure is about two times larger in the triaxial tests than in the plane strain tests.

In this paper, the following two topics will be discussed, in detail, by carrying out undrained shear tests using plane strain and triaxial test apparatuses:
1) How can the anisotropy of undrained shear strength be interpreted in terms of effective strength parameters c' and ϕ', rather than the pore-pressure coefficient A_f?
2) Is there any basis to deal with, in general, the strength anisotropy of both clay and sand from a micro-structural point of view?

It seems rather strange to the present authors that the anisotropy of cohesive clay is discussed almost independently of the anisotropy of cohesionless sand. The second topic is, therefore, the first trial to reorganize the knowledge on anisotropy of clay and sand from a micro-structural point of view. For the sake of simplicity, however, the stress induced anisotropy is omitted from the scope of the present study. Instead, the inherent and induced anisotropy of micro-structure will be mainly taken into account, with special interest in their relation to the anisotropy with respect to undrained shear strength.

TESTING MATERIAL AND PROCEDURE

Testing Material and Preparation

Only a kaolin clay was used in the present experimental study. It consists of a pure kaolinite mineral having good crystallinity and well-defined particle shape. The physical properties are summarized in Table 1.

Powder of the kaolin clay, which passed through a sieve of 0.85 mm, was fully mixed with de-aired water to prepare a slurry having a water content of 200%. The slurry was placed in a vacuum chamber to remove air bubbles that might have entered during mixing and was then poured into a large steel mold with an internal diameter of 25 cm and a height of 30 cm. The slurry was then consolidated one-dimensionally under a maximum pressure of 150 kPa to give a water content of 57.7% (void ratio, $e=1.52$) with a standard deviation of 0.77%. The large mold was used to prepare specimens for plane strain tests. To prepare specimens for triaxial tests, a small steel mold with an internal diameter of 15 cm and a height of 35 cm was used instead, which made it possible to apply a ten times higher one-dimensional consolidation pressure (1569 kPa) than that for the large mold. The water content was 45.6% after consolidation. To reduce wall friction, a thin film of vaseline was pasted on the wall of each mold.

Table 1. Physical properties of kaolin clay

LL (%)	*PL* (%)	*PI* (%)	G_s
83.8	32.6	51.2	2.633

Each cylindrical block was taken out of the mold, wrapped in a polyvinyl sheet, and then stored in a temperature-humidity controlled box until used. The whole process from specimen preparation to shear test was carried out under an ambient temperature between 19°C and 21°C.

Figure 1 shows two photographs which were taken using a scanning electron microscope on vertical and horizontal sections of a specimen consolidated under 150 kPa. It is clearly seen that kaolin clay particles are oriented, as usual, with their faces perpendicular to the direction of K_0-consolidation.

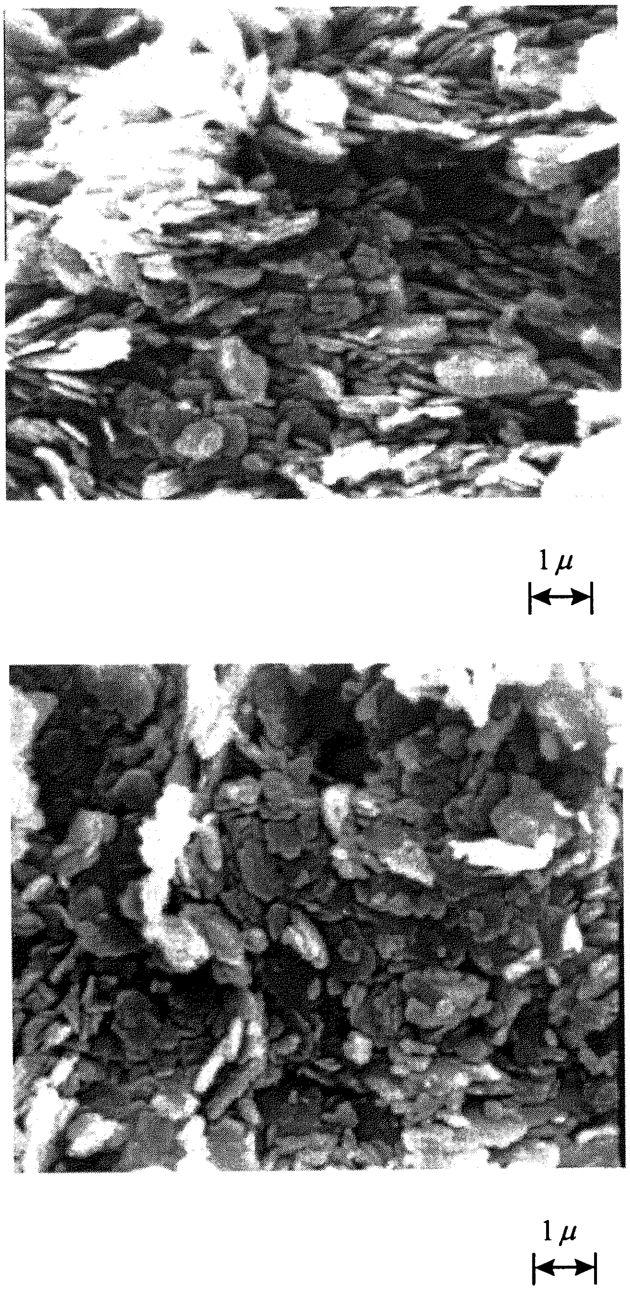


Fig. 1. Micro-structure of kaolin clay anisotropically consolidated to 150 kPa, obtained by a scanning electron microscope on a (a) vertical section, showing the alignment of particles with their faces perpendicular to the plane of paper, (b) horizontal section, showing the alignment of particles with their faces parallel to the plane of paper

Trimming of Specimens

Each cylindrical block (25 cm in diameter) was trimmed to make specimens of 10 cm long, 5 cm wide and 12.5 cm high for plane strain tests. A special device (Fig. 2) was needed to keep inclination angle θ at a desired value. Here, θ is the angle between the axis of the major principal stress in a shear test and the direction normal to the horizontal (consolidation) plane in the consolidation stage (Fig. 3(a)), and was chosen to be 0°, 30°, 45°, 60°, 75° or 90° in a plane strain test. (For simplicity, the term "consolidation plane" will sometimes be used to denote the plane on which the consolidation pressure, 150 or 1569 kPa, was applied.)

Each 15 cm diameter block was used to make specimens 3.5 cm in diameter and 7 cm in height for triaxial tests. Using a standard trimmer, the block was carefully trimmed such that θ was 0°, 30°, 60° or 90° (Fig. 3(b)).

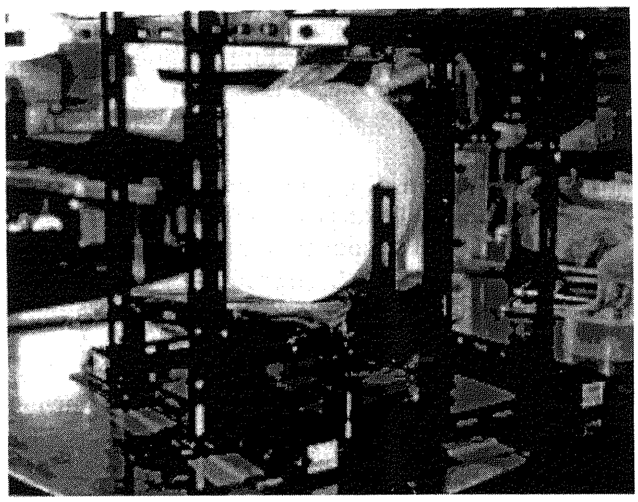


Fig. 2. A special frame to trim the sample with a desired inclination angle for the plane strain test

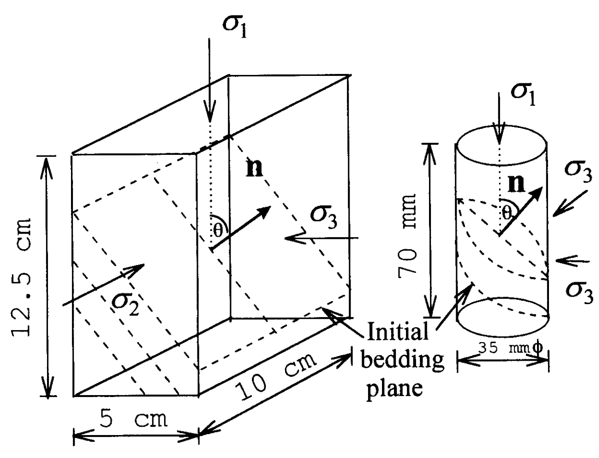


Fig. 3. Inclination angle θ of the major principal stress axis during shear with respect to the direction normal to the consolidation plane: (a) for the plane strain test and (b) for the triaxial test

Undrained Tests

A specimen with dimensions of 10 cm × 5 cm × 12.5 cm was set in a plane strain test apparatus, and was again consolidated under an isotropic pressure of 150, 100, 50 or 25 kPa, which corresponds to an over-consolidation ratio (OCR) of 1, 1.5, 3 or 6, respectively. (It should be noted here that the over-consolidation ratio (OCR) was tentatively defined as the ratio of the maximum anisotropic consolidation pressure (during K_0-consolidation) to the maximum isotropic consolidation pressure (prior to undrained shearing).) Dimensions of each specimen were corrected using the measurements of the volume change and the axial displacement which took place during the isotropic consolidation. To do this, two lateral strains in the horizontal directions were assumed to be equal. During undrained shearing tests, a back water pressure of 450 kPa was applied to facilitate saturation. The pore-pressure coefficient B was found to be within the range of 0.97 to 1.00 for all the specimens.

Top and bottom end platens were lubricated to reduce friction by two rubber membranes with silicone grease. To facilitate drainage from the upper and bottom ends, two strips of filter paper were placed along their central lines, with care not to introduce additional friction resistance. Two rubber membranes with silicone grease were also used between the side platens and the vertical sides of a specimen.

A specimen 3.5 cm × 7 cm was set in a triaxial cell, and was consolidated under an isotropic stress of 294, 196, 98 or 49 kPa, which corresponds to an over-consolidation ratio of 5.3, 8, 16 or 32, respectively. The other experimental conditions were the same as those of the plane strain tests.

The specimen thus prepared was sheared at a constant strain rate of 0.08%/min until a total axial strain of 17% for the plane strain test and 14% for the triaxial test was attained. During a test, the excess pore pressure and in the case of plane strain tests, the intermediate principal stress were also measured.

After testing, each sheared specimen was carefully sketched with special attention to the orientation of shear planes with respect to the consolidation plane.

TEST RESULTS

Deviator Stress-excess Pore Pressure-strain Relations: General Observation

Relations between deviator stress $(\sigma_1-\sigma_3)$, excess pore pressure Δu and axial strain ε_1 are shown in Fig. 4 for plane strain tests and in Fig. 5 for triaxial tests, in each of which data corresponding to four different over consolidation ratios are included. The following observations are worth noting:

1) Even in the plane strain tests on the normally consolidated kaolin clay with OCR=1, the deviator stress significantly drops immediately after reaching a peak (strain softening), and a clear shear band (or conjugate bands) is developed in parallel with the softening behavior. It should be noted that such a clear drop of the deviator stress was not observed in the triaxial tests on the normally consolidated kaolin clay (Kazama, 1996). It has also been observed in the tests using Toyoura sand that shear bands are clearly developed at a much smaller axial strain in the plane strain tests than that in the triaxial tests (Oda et al., 1978). Irrespective of the soil types, in general, the plane strain condition tends to facilitate strain softening behavior.

2) In the triaxial tests, there is a clear contrast in the softening behavior between the specimens with $\theta=90°$ and $\theta=0°$ (Fig. 5): The former specimens (i.e., $\theta=90°$), which were compressed parallel to the consolidation plane, show a similar softening behavior, as sharp as the plane strain tests, when the axial strain attains 7 to 9%. This is true irrespective of the values of OCR. The later specimens ($\theta=0°$), which were compressed normal to the consolidation plane, do not show such a sharp softening behavior. Only a slight decrease of the deviator stress appears when the axial strain increases over 14%. The other specimens with $\theta=30°$ and 60° are just intermediate between the above two cases. This suggests that the softening behavior is very sensitive to the inherent micro-structure, as well as boundary conditions such as $\sigma_2=\sigma_3$ and $\varepsilon_2=0$.

3) In the triaxial tests, the strain at failure decreases with any increase in the inclination angle θ. This is not the case in the plane strain tests. It can rather be said that the trend is the reverse, i.e., the failure strain tends to increase with increased inclination angle.

4) In the plane strain tests, the development of pore-pressure in the tests of $\theta=0°$, 90° is slightly, still generally, higher than the case of $\theta=60°$. In the triaxial tests, a slight decrease occurs in the development of pore-pressure with increased inclination angle θ. It should be emphasized here, that anisotropy with respect to the undrained shear strength cannot be attributed, as will be discussed later, to the difference in the development of pore-pressure.

Undrained Shear Strength

Undrained shear strengths c_u are summarized in Fig. 6 as the relations of c_u versus θ: (a) is for the plane strain tests and (b) is for the triaxial tests. In both cases, the undrained shear strength shows a clear minimum value at around $\theta=60°$, irrespective of values of OCR. Furthermore, the following three points are worth noting:

1) Intensity of the anisotropy in the undrained shear strength is quite similar in both tests. This seems to suggest that the boundary conditions, such as triaxial and plane strain, have little effect on the intensity of undrained shear strength anisotropy. This finding is in contrast with that of sands, in which the effect of anisotropy seems more significant in the plane strain tests than that in the triaxial tests (Oda et al., 1978). More importantly, this statement is qualitatively correct, as will be discussed later, even when such a comparison is made between the shear strength parameters in terms of effective stresses corresponding to undrained tests on clay and drained tests on sand. In order to make a quantitative compari-

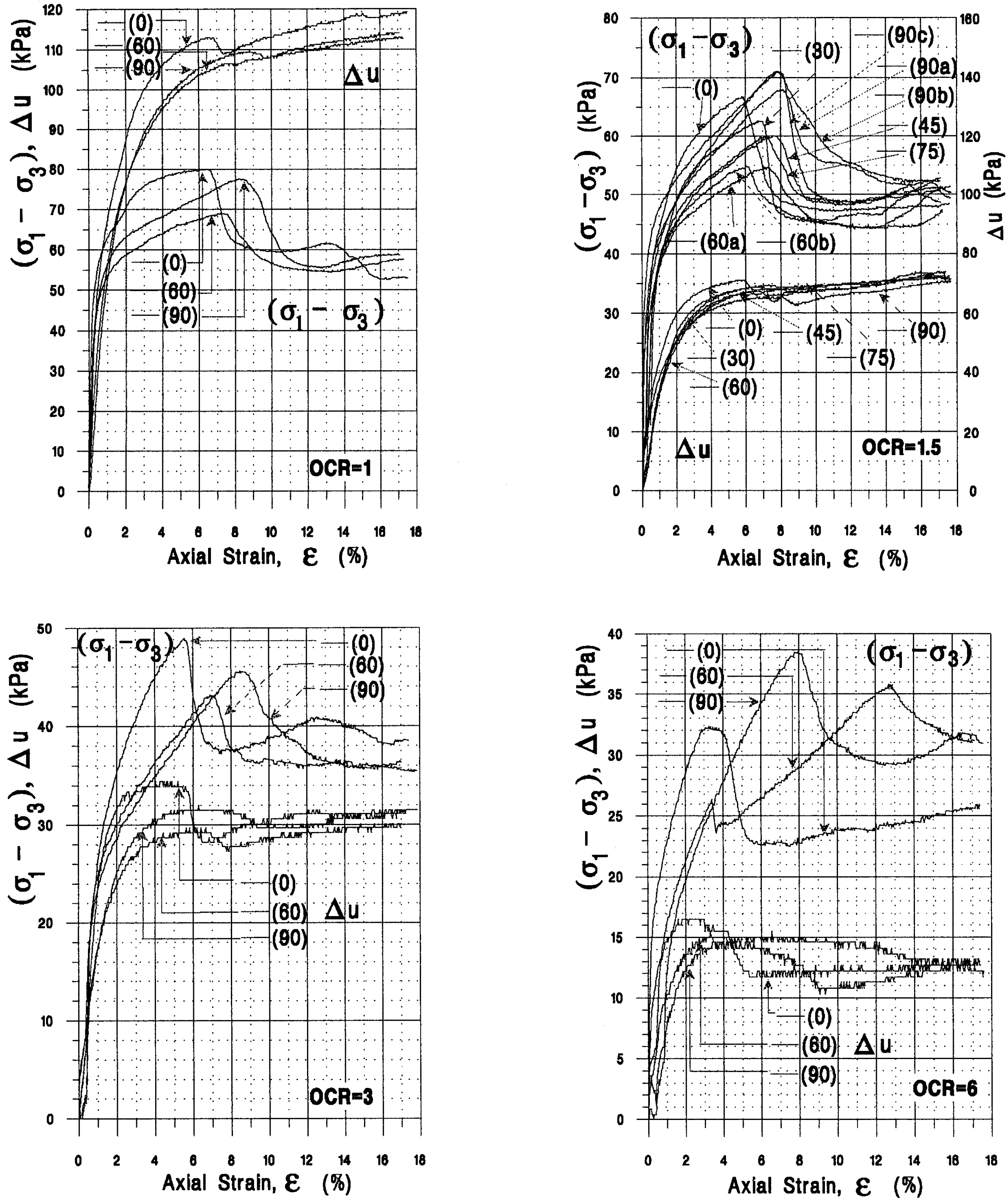


Fig. 4. Relations between stress, strain and excess pore-pressure for the plane strain tests: (a) OCR=1; (b) OCR=1.5; (c) OCR=3 and (e) OCR=6

son, however, shear strength parameters for both clay and sand should be determined under drained tests.

2) In both cases, there is no essential effect of OCR on the anisotropy of the undrained shear strength, at least within the range of OCR up to 32.

3) The undrained shear strength is largest at $\theta=90°$, typically in the triaxial tests of the kaolin clay. However, the smallest appears at $\theta=90°$ in the drained triaxial tests of sands (Arthur et al., 1972; Oda, 1972a; Tatsuoka et al., 1990), which arises from the fact that cohesionless particles cannot be stable when they are stressed parallel to their elongation direction. However, this interpretation is not valid for clayey soils. Cohesive particles can resist, rather stably, applied stress parallel to their elongation direction, probably due to electric forces among them.

In order to see the effect of the pore-pressure coefficient A_f on the undrained shear strength, Fig. 7 is

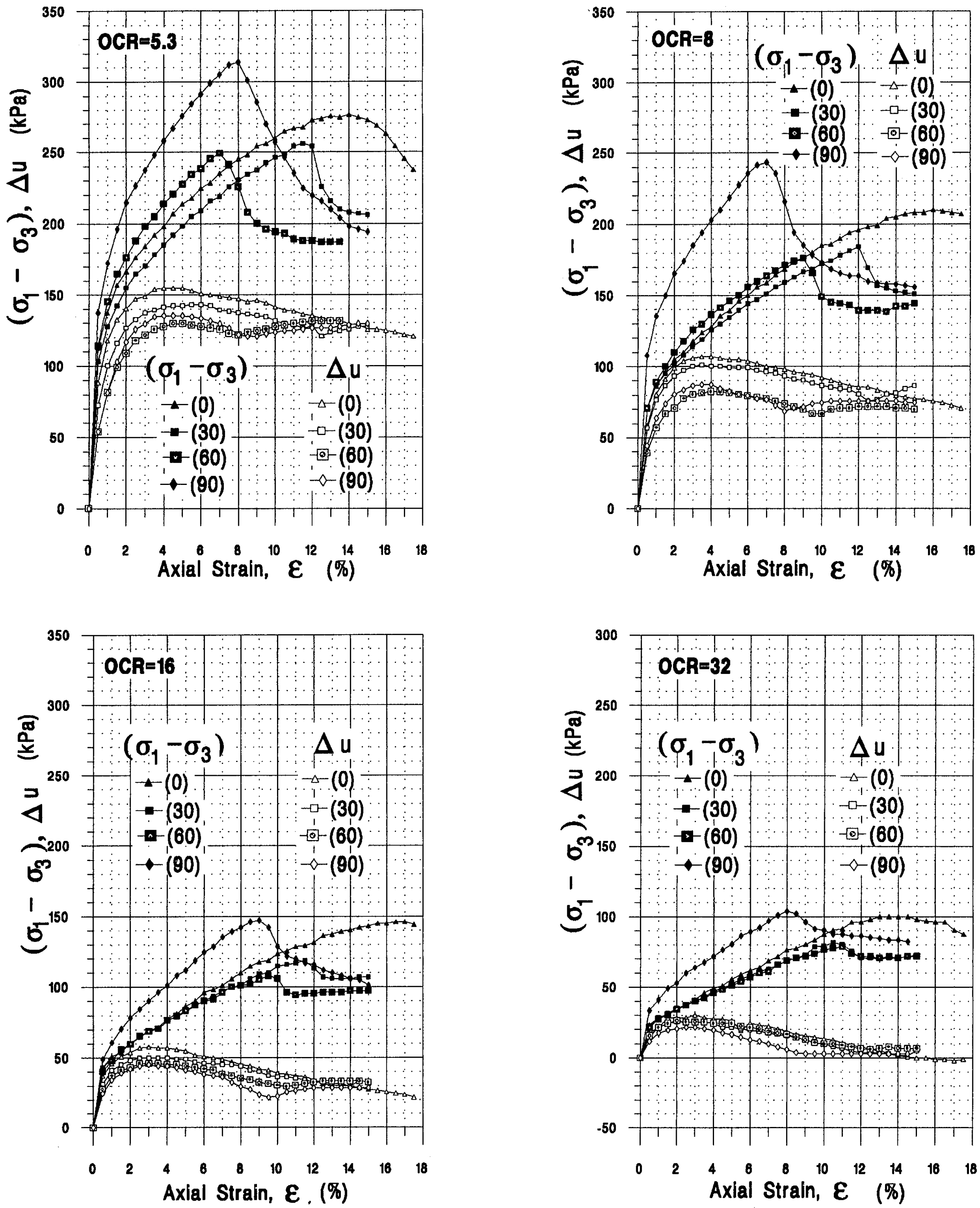


Fig. 5. Relations between stress, strain and excess pore-pressure for the triaxial test: (a) OCR=5.3; (b) OCR=8; (c) OCR=16 and (d) OCR=32

prepared: (a) is for the plane strain tests and (b) is for the triaxial tests (here, A_f is defined after Parry and Nadarajah (1973)). Except for a few cases, the coefficient A_f shows the maximum value at $\theta=60°$ with the minimum at $\theta=90°$ in both the plane strain and triaxial tests. In other words, the relation between A_f and θ is just the reverse of the relation between c_u and θ. Apparently, the relation of Fig. 7 seems to be in good agreement with a commonly accepted statement that anisotropy with respect to the value of the pore-pressure coefficient A_f is primarily responsible for anisotropy of the undrained shear strength of clayey soils.

It should be noted, however, that a high value of A_f does not necessarily mean a high development of excess pore-pressure during shear. In fact, as has already been shown, the excess pore-pressure at failure is rather small

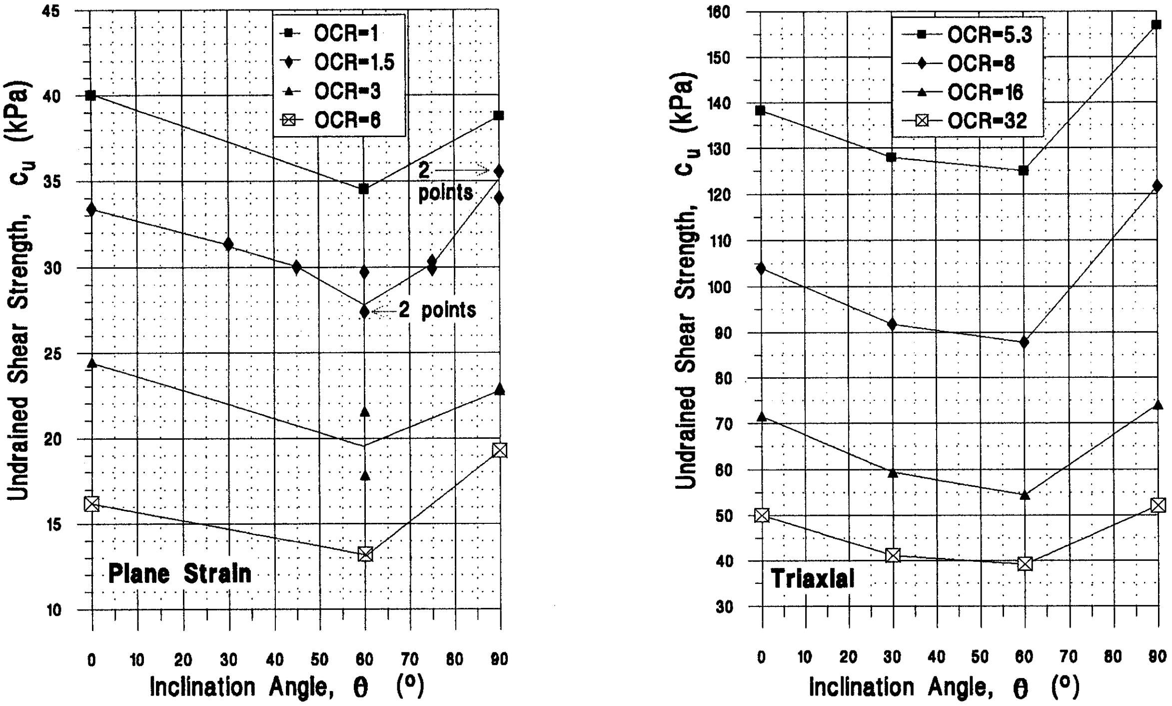


Fig. 6. Change of the undrained shear strength with the inclination angle θ; (a) for the plane strain test and (b) for the triaxial test

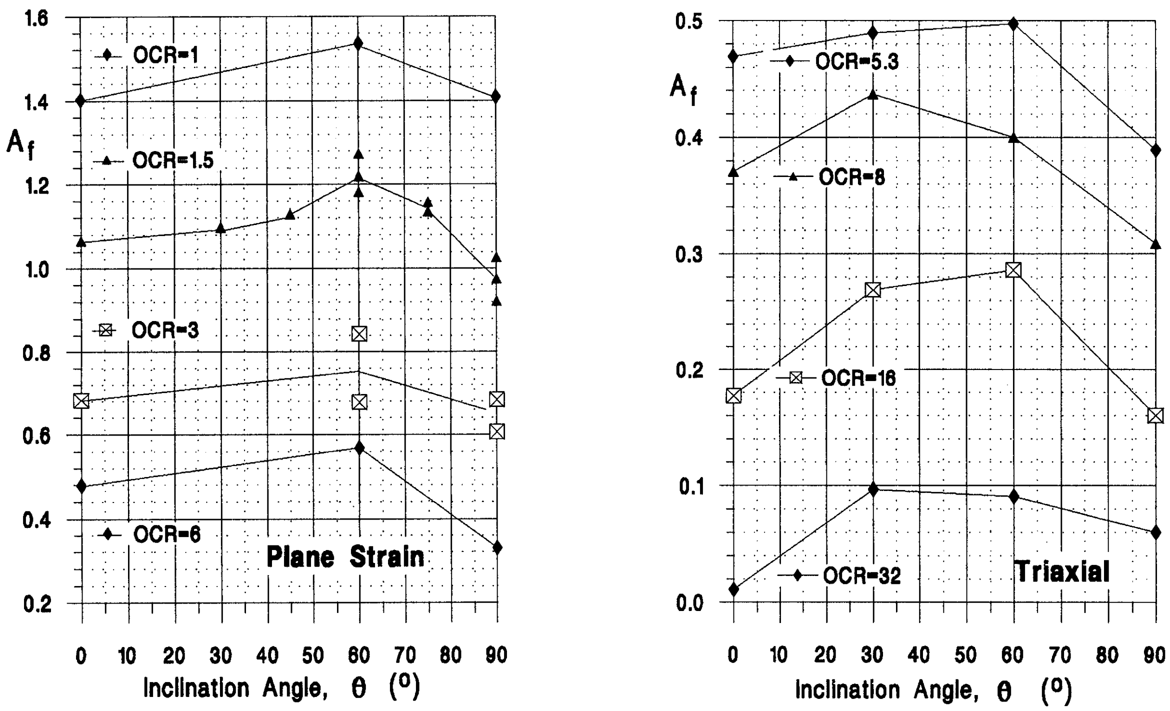
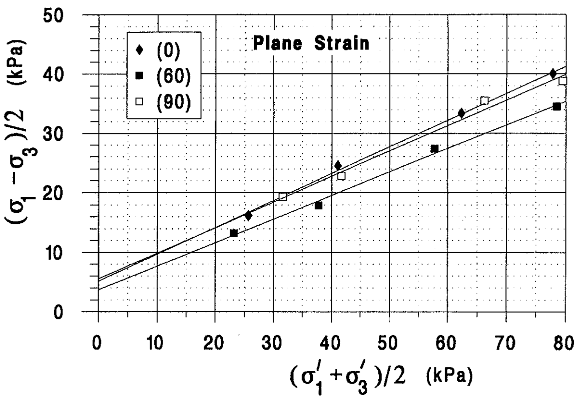


Fig. 7. Change of the pore-pressure coefficient at failure with the inclination angle θ: (a) for the plane strain test and (b) for the triaxial test

in the case of $\theta=60°$ where the coefficient A_f is the highest. In the triaxial tests, there is a slight decrease in the development of pore-pressure with any increase in the inclination angle θ. Still, such a slight change of pore pressure cannot explain the observed changes of c_u and A_f in Figs. 6 and 7. An important point, instead, is that the high value of A_f at $\theta=60°$ is simply because of the low shear strength c_u. It can never be the reason for the phenomenon whereby the undrained shear strength becomes minimum at $\theta=60°$.

Anisotropy of Shear Strength Parameters c' and ϕ' in Terms of Effective Stresses

In order to seek a more realistic interpretation of the



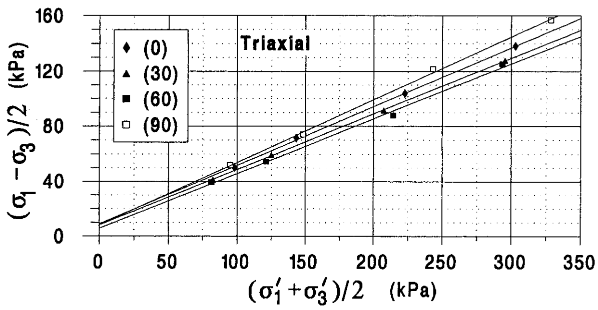


Fig. 8. Relation between the maximum shear stress at failure and the mean effective stress: (a) for the plane strain test and (b) for the triaxial test

Table 2. Shear strength parameters in terms of effective stresses: (a) for the plane strain test and (b) for the triaxial test

θ (°)	c' (kPa)	ϕ' (°)
0	5.8	26.8
60	4.1	23.3
90	6.2	25.4

(a)

θ (°)	c' (kPa)	ϕ' (°)
0	10.1	25.3
30	8.9	24.0
60	6.3	23.5
90	9.0	27.2

(b)

anisotropy of the undrained shear strength (Fig. 6), shear strength parameters c' and ϕ' in terms of effective stresses will next be discussed. To this end, the maximum shear stress at failure $(\sigma_1-\sigma_3)/2$ is plotted against the mean effective stress in Fig. 8: (a) is for the plane strain tests and (b) is for the triaxial tests. The shear strength parameters c' and ϕ' are listed in Table 2, the values of which were calculated using the least squared method on the data shown in Fig. 8.

It seems of particular importance to note the following observations. In the plane strain and triaxial tests, c' and ϕ' are both minimum at $\theta=60°$. For both plane strain and triaxial tests, the cohesion c' and the friction angle ϕ' at $\theta=60°$ are about 35% and 7 to 14% less than the corresponding values at $\theta=0°$, 90°, respectively. Such a quantitative change, especially of the friction angle, is comparable to the change observed in the triaxial tests of the Toyoura sand (e.g., Oda et al., 1978; Tatsuoka et al., 1990).

These differences in the strength parameters are substantially large, and can never be said to be negligible. More importantly, the variation of these strength parameters with θ is in good harmony with the variation of the undrained shear strength with θ in Fig. 6; i.e., the undrained shear strength in the triaxial and plane strain tests is minimum in the direction that the strength parameters are minimized.

It is commonly believed that the anisotropy by the pore-pressure coefficient is primarily responsible for the observed anisotropic variation of the undrained strengths, depending only slightly, if any, on anisotropy of the shear strength parameters in effective stresses. Duncan and Seed (1966b) reported two friction angles ϕ' by carrying out undrained plane strain tests of one-dimensionally consolidated San Francisco bay mud, 38° for the specimen with $\theta=0°$ (VPS; Vertical Plane Strain) and 35° for the specimen with $\theta=90°$ (HPS; Horizontal Plane Strain). They said that "this difference in the values of ϕ' has a negligible effect on the undrained strength, but a difference of this magnitude would have a considerable influence on the drained strength". Apparently, as has already been pointed out, the variation of the value of the pore pressure coefficient A_f in Fig. 7 seems to support this idea. It must be noted, however, that the variation did not arise from the variation of the excess pore-pressure at failure, but rather simply from the variation of the undrained shear strength itself. Our conclusion, therefore, is that the anisotropy of the undrained shear strength is primarily caused by the shear strength parameters depending on the inclination angle θ, i.e., $c'=c'(\theta)$ and $\phi'=\phi'(\theta)$.

To support this conclusion, the following observations might be important: A shear band (or conjugate shear bands) is always observed in the plane strain tests. The direction of, at least a major shear band, appears more or less parallel to the consolidation plane. The same result was also reported by Duncan and Seed (1966a). These results strongly suggest that inherent anisotropy by particle orientation, though partially disturbed by the plastic deformation during shear deformation, remains effective in characterizing the micro-structural anisotropy at failure.

Comparison between Clay and Sand

Anisotropic behavior of the kaolin clay is quite similar qualitatively, as well as quantitatively, to that of the Toyoura sand. It must be pointed out, at the same time, that there exist some differences between them.

Firstly, in the case of the kaolin clay, the degree of anisotropy is not altered much by changing the boundary conditions such as triaxial ($\sigma_2=\sigma_3$) and plane strain ($\sigma_2=0$). In the case of the Toyoura sand, however, anisotropy with respect to the friction angle ϕ' in terms of effective stresses becomes more pronounced in the plane strain tests than that in the triaxial tests. Secondly, the kaolin clay gains higher shear strength parameters c' and ϕ', irrespective of whether a specimen is compressed parallel or perpendicular to the plane on which particles lie with their long dimensions. In the case of Toyoura sand, however, a specimen compressed parallel to the plane gives a friction angle much less than that of a specimen compressed perpendicular to the plane.

The reason for these differences to appear is not fully understood yet. It seems of particular importance to clarify this reason, since it makes it possible to understand the essential difference between clay and sand in the deformation behavior. To do this, further study is needed. Especially, drained shear tests on clayey soils are of particular importance.

CONCLUSIONS

In order to find out the real cause of anisotropy in the undrained shear strength of clayey soils, two types of undrained tests were carried out; 1) undrained plane strain tests on lightly over-consolidated kaolin clay, and 2) undrained triaxial tests on heavily over-consolidated kaolin clay. The test data were analyzed from a micro-structural point of view and compared with those of similar tests on sands. The conclusions are summarized, as follows:

1) Anisotropy in undrained shear strength of over-consolidated kaolin clay is primarily caused by the anisotropy of shear strength parameters c' and ϕ' in terms of effective stresses, rather than the anisotropic pore-pressure development. This conclusion contradicts the commonly accepted statement that the anisotropy of undrained shear strength arises primarily from the anisotropy of pore water pressure parameter A_f.

2) Intensity of the anisotropy in the undrained shear strength of one-dimensionally consolidated kaolin clay is not altered much by changing the boundary conditions, such as triaxial and plane strain, and also by increasing the over-consolidation ratio up to at least 32.

3) Higher values of cohesion c' and friction angle ϕ' appear when a specimen is compressed either normal or parallel to the consolidation plane. The lowest appears when a specimen is compressed with the major principal stress direction inclined at about 30° to the consolidation plane. The difference between the maximum and the minimum is quite similar to that observed in triaxial tests of sands. In case of sands, however, specimens compressed normal to the bedding plane yield the maximum friction angle, while specimens compressed parallel to the bedding plane yield the minimum value in triaxial tests, and a value close to the minimum in plane strain tests. These observations suggest that the micro-deformation mechanism leading to the strength anisotropy is quite similar, although not exactly the same, in both clay and sand.

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