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Undrained shear strength and interrelationships among CIUC, CKoUC, CIUE, and CKoUE tests

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Abstract

Because of simplicity and economy, to determine the undrained shear strength of cohesive soils most commercial laboratories perform isotropically consolidated undrained triaxial compression test using a conventional triaxial system, under an effective confining pressure equal to the estimated in–situ vertical stress. In–situ state of stress for most cohesive soils, however, is anisotropic. How different are the undrained shear strengths measured under in–situ conditions as opposed to routine laboratory conditions? Isotropically consolidated undrained triaxial compression (CIUC), Ko–consolidated undrained triaxial compression (CKoUC), isotropically consolidated undrained triaxial extension (CIUE), and Ko–consolidated undrained triaxial extension (CKoUE) tests were carried out on a single material to explore interrelationships of the undrained shear behaviour obtained from these test types.

The comparison of data obtained from four different types of triaxial test shows significant differences in undrained shear behaviour evaluations. A simple procedure has been developed to estimate the undrained shear strength of normally consolidated clay from the results of ordinary CIUC tests and plasticity index. The resulting correlations are shown and these provide an effective method for determination of meaningful c_u/p values.

Key words: cohesive soil, consolidated undrained shear, earth pressure at rest, laboratory test, plasticity, shear strength, stress path.

Introduction

The stresses in field deposits are commonly anisotropic; the major principal effective stress, σ_v', acts in a vertical direction and the minor principal stress, $Ko\sigma_v'$, acts in a horizontal direction as shown in Fig. 1 (a), where Ko is a coefficient of earth pressure at rest. Testing of soil samples in the laboratory plays an important role in geotechnical engineering practice. Figure 1 (b) shows typical consolidation tests commonly found in commercial and research laboratories. Most commercial laboratories, however, conduct routine isotropically–consolidated undrained triaxial compression test (CIUC) because of convenience and simplicity of testing procedure. There is no theoretical reason why values of undrained shear strength (c_u) from field vane and unconsolidated undrained triaxial tests would prove valid for Ko–consolidated clay deposits but not for the case of sloping deposits, and a closer look into the methods of obtaining c_u is warranted. The undrained shear strength is one of the most important soil parameter for evaluating the undrained shear behaviour of cohesive soil strata. Ideally, the soil parameters should be determined for the actual in–situ

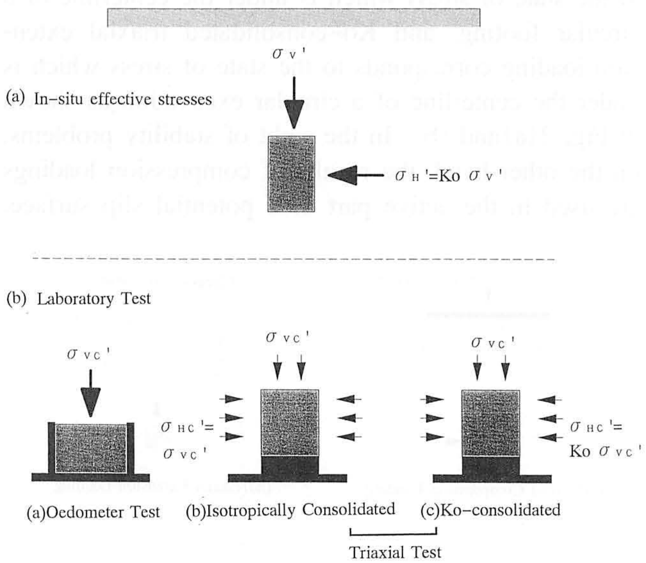

Fig. 1 In–situ effective stresses and stress conditions during consolidation process in laboratory tests.

soil conditions. Once a soil sample has been consolidated under a given state of stresses, the question now arises as to whether or not the type of stress system applied during undrained shear has an effect on the undrained shear behaviour.

Anisotropy in cohesive soils can be divided into two categories. One is an inherent anisotropy which results in the anisotropic stresses in field deposits, and the

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other is a stress system induced anisotropy which results in reorientation of the principal stress directions. The soil particles tend to become oriented in the horizontal direction during one–dimensional deposition. This preferred particle orientation causes an inherent anisotropy which can lead to changes in strength parameters. Measurement of inherent anisotropy effects have been inferred from results of unconsolidated undrained triaxial tests, or unconfined compression tests where performed on specimens cut at different orientations to the vertical direction. Construction of the embankment results in reorientation of the principal stress directions. This results from the fact that different increments of shear stress are required to produce foundation failure, as the major principal stress at failure varies between the vertical and the horizontal direction.

Stress system induced anisotropy was first reported by Hansen and Gibson (1949). They developed theoretical expressions for the variation in undrained shear strength (c_u) with the direction of the major principal stress at failure. In practice, the combined anisotropy resulting from both the inherent and the stress system induced components is of more interest. In general, Ko–consolidated triaxial compression loading corresponds to the state of stress which is under the centerline of a circular footing, and Ko–consolidated triaxial extension loading corresponds to the state of stress which is under the centerline of a circular excavation as shown in Fig. 2(a) and (b). In the sight of stability problems, on the other hand, the results of compression loadings are used in the active part of a potential slip surface,

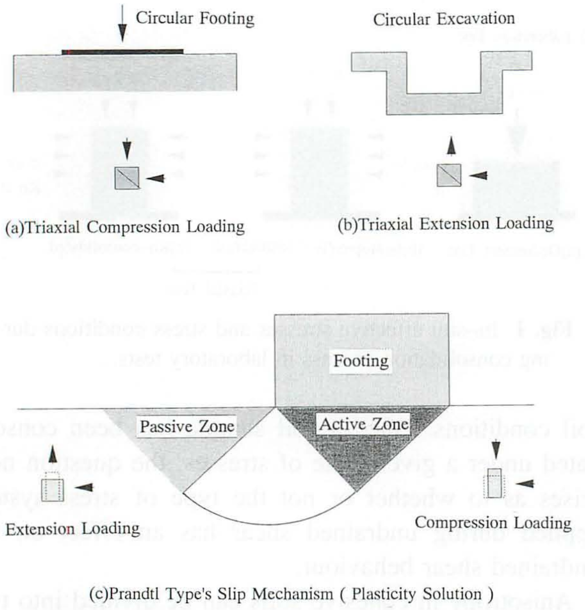

Fig. 2 Typical stress systems for a normally consolidated cohesive soil in the field.

and the results of extension loadings in the passive part, as shown in Fig. 2(c).

A number of studies have been performed on this topic (Bishop, 1966; Duncan and Seed, 1966; Berre and Bjerrum, 1973; Ladd and Foott, 1974; Vaid and Campanella, 1974). In their "state of the art" report, Ladd et al. (1977) summarized the results of studies on the undrained shear strength ansotropy in natural cohesive soils, concluding that the anisotropy becomes more marked as the plasticity index of cohesive soils decreases. The engineering significance of this result is that the undrained shear strength anisotropy should be taken into consideration in stability analyses. This is more important for soils with low plasticity index. Studies of the effects of anisotropy on bearing capacity can be seen elsewhere (Reddy and Srinivasan, 1970; Davis and Christian, 1971; Kinner and Ladd, 1973; Chen, 1975; Nakase and Kamei, 1983).

Typical stress systems which might occur in the field are shown in Fig. 2. It is well known that the undrained shear strength (c_u) is affected by the mode of testing, initial stress state, confining pressure level, boundary conditions, strain rate and other variables. It is expected, therefore, that different test types should produce different test results for c_u. Therefore, the mechanical behaviour of anisotropically cohesive soils have become of increasing importance as the number of embankments and other earth structures being designed and constructed on soft ground have increased.

For easy and economy, most commercial laboratories perform consolidated undrained triaxial shear tests using a conventional triaxial system as shown in Fig. 3 under an effective confining pressure equal to the estimated in–situ vertical stress. Consequently, it has become routine practice to consolidate the specimens isotropically (Kc=1) before shear to failure. As mentioned earlier, the in–situ state of stress for most cohesive soils, however, is anisotropic. How different are the undrained shear strengths measure under in–situ conditions as opposed to routine laboratory conditions?

Anisotropically consolidated tests are much more complicated and take much longer to run. Furthermore, if Ko–consolidation is desired, ensuring zero lateral strain throughout the consolidation process is not an easy task. Either a device to measure the radial dimension of the specimen is employed, or corresponding volumetric and axial deformations are measured and controlled so that the average radial strain is zero. Realistically, such testing requires that some type of automated or computer–controlled servomechanical system be employed as shown in Figs. 4 and 5 (Sugano and Masumi, 1982; Nakase and Kamei, 1983). This is certainly possible, but not yet common in most soil test-

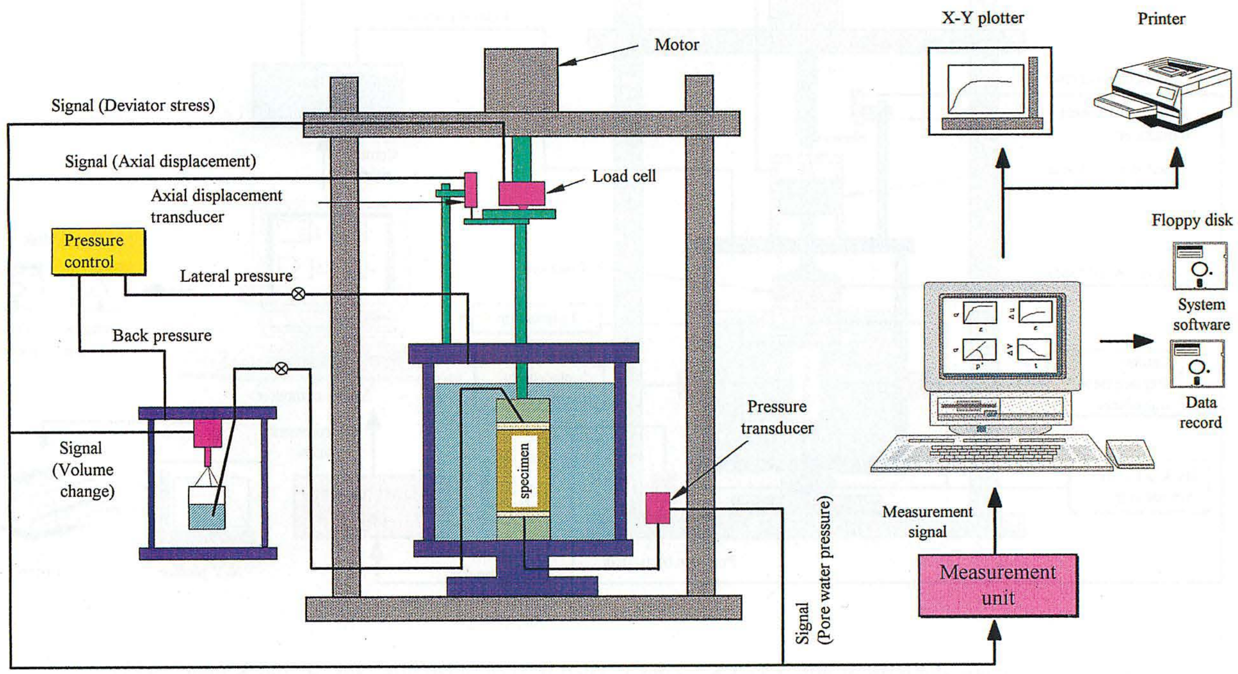

Fig. 3 Schematic diagram of conventional triaxial testing system.

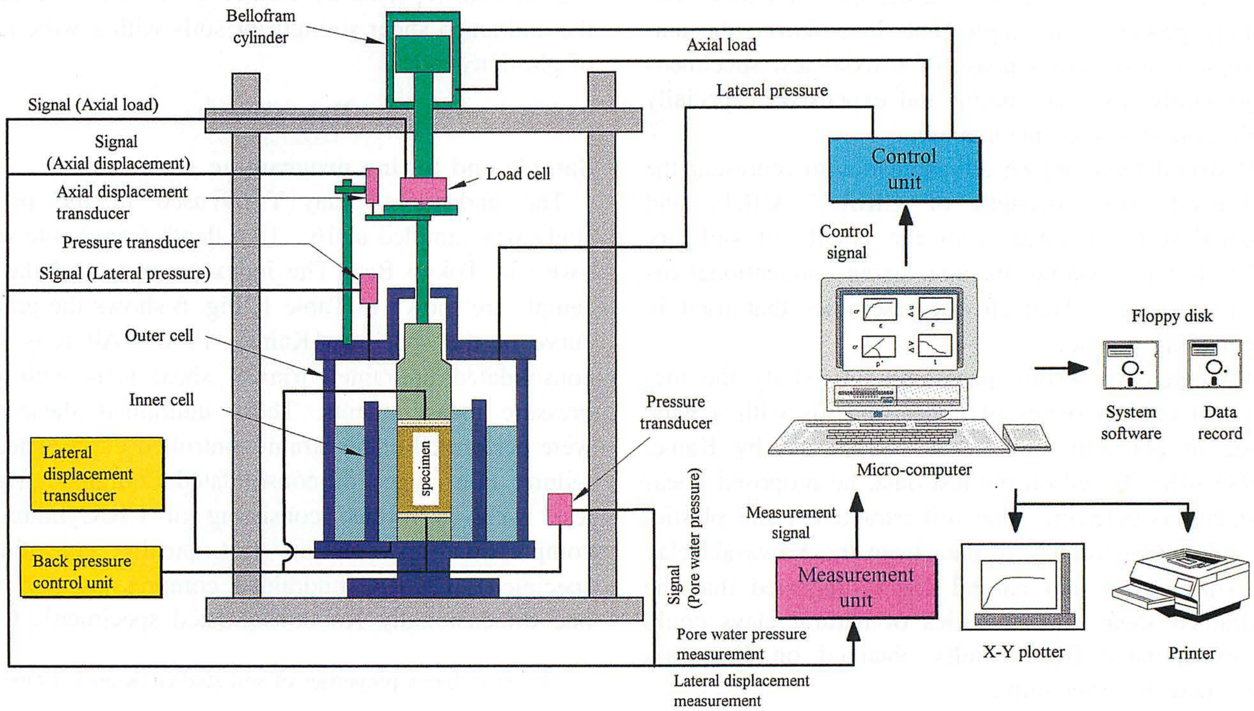

Fig. 4 Schematic diagram of automated Ko–consolidated triaxial apparatus (Sugano and Masumi, 1982).

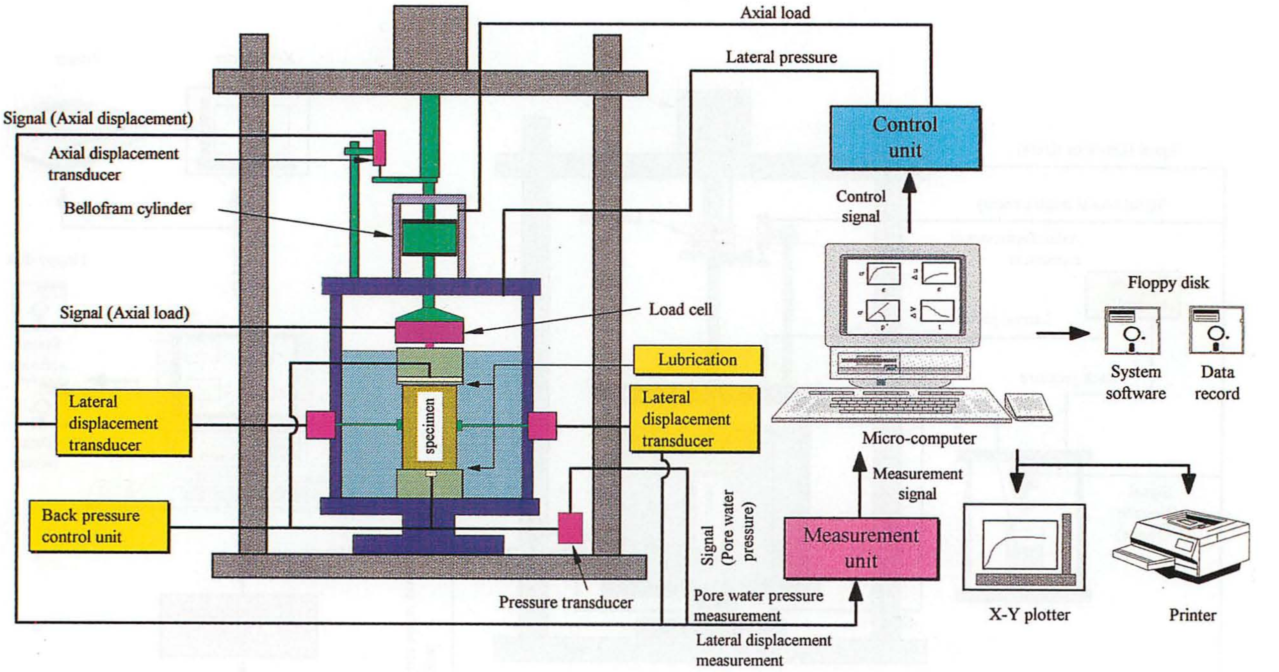

Fig. 5 Schematic diagram of automated Ko–consolidated triaxial apparatus (Nakase and Kamei, 1983).

ing laboratories. The consolidation stress ratio is necessarily equal to unity, although rarely are natural soil deposits isotropic, nor do they often have an in–situ Ko value equal to unity. Unless a device has specialized features as seen in Figs. 4 and 5, Ko–consolidation with no lateral strain is a difficult and time consuming procedure to implement. In addition, the anisotropic consolidation phase of triaxial test specimens is generally time consuming and expensive, especially if Ko conditions are maintained.

If would therefore be advantageous to represent the undrained shear strength of CKoUC, CIUE, and CKoUE tests obtained from the results of ordinary CIUC test in a simple manner, using conventional triaxial apparatus which closely resembles that used in engineering practice.

An extensive testing programme to study the mechanical characteristics of cohesive soils with a wide range of plasticity index was carried out by Kamei (1985 MS). Based on the test data, he proposed linear correlations between some soil parameters and plasticity index. Consistency of data from the Kawasaki clay –mixture series and natural clays suggested that the undrained shear characteristics of natural clays could be extrapolated from results obtained on Kawasaki clay–mixture series only.

The purpose of this paper is to quantitatively investigate the effects of consolidation and shear conditions on the undrained shear behaviour of normally consolidated undisturbed cohesive soil with high plasticity index. In addition, data are compared to assess the interrelationships for undrained shear strength among CIUC, CKoUC, CIUE, and CKoUE tests. The resulting correlations provide a useful procedure to determine c_u in a more meaningful manner, when combining the data reported by Kamei (1985 MS) to evaluate the undrained shear strength of soils with a wide range of plasticity index .

Experiments

Sample and testing programme

The undisturbed clay (Y–69) used in the present study was sampled at 16～18m depth from a site south –west of Tokyo Bay. The index properties of the soil sample are shown in Table 1. Fig. 6 shows the grading curve of the specimen (Kamei, 1996). All tests were consolidated undrained triaxial shear tests with pore pressure measurements. These undrained shear tests were performed under strain–controlled during shear to failure. Four types of consolidated undrained triaxial tests were performed consisting of CIUC (undrained compression test on an isotropically consolidated specimen), CKoUC (undrained compression test on a one–dimensionally Ko–consolidated specimen), CIUE

Table 1 Index properties of soil studied (Kamei, 1996).

Soil Sample	ρ_s(g/cm^3)	w_L(%)	w_P(%)	PI	Sand(%)	Silt(%)	Clay(%)
Yokohama Clay	2.701	110.0	41.1	68.9	3.0	43.0	54.0

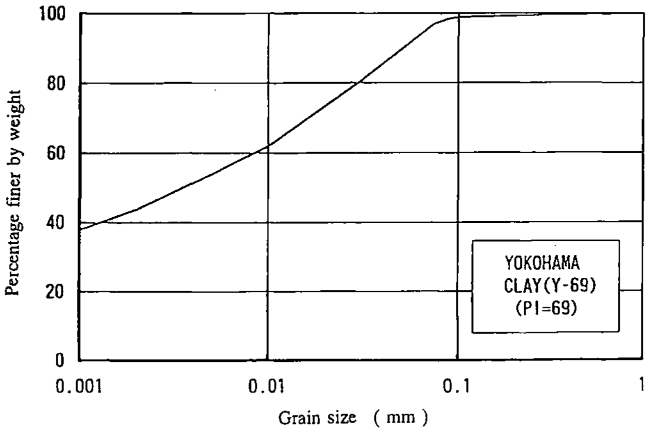

Fig. 6 Grain size distribution curve of Yokohama clay sample Y–69(Kamei, 1996).

(undrained extension test on an isotropically consolidated specimen), and CKoUE(undrained extension test on a one-dimensionally Ko-consolidated specimen). Two different values of vertical effective consolidation pressure of 147 kPa and 294 kPa were used in the consolidation process. A back pressure of 196 kPa was applied to all the test specimens throughout the consolidation and undrained shear. For each soil sample, triaxial compression and extension loadings were performed, with a constant rate of axial strain of 0.07%/min. (Kimura and Saitoh, 1983; Nakase and Kamei, 1986). Details of the test conditions and the test appartus used in the present study are presented elsewhere(Nakase and Kamei, 1983). In addition, the undrained shear characteristics of Ko-consolidated on Yokohama clay(Y–69)have been reported previously (Kamei, 1996)

Test results and discussions

Undrained shear behaviour

Once a sample has been consolidated under a given state of stresses, the question now arises if the type of stress system applied during undrained shear has an effect on the undrained shear behaviour.

Figure 7 shows typical stress–strain behaviour, with results obtained from four types of triaxial tests, illustrating how the shape of the stress–strain curve changes with changing consolidation and shear conditions, where the principal stress difference $q=\sigma_a-\sigma_r$ is normalized by dividing by the vertical effective consolidation pressure σ_{vc}'. Although it is common to use positive strain in compression loading and negative strain in extension loading, the axial strains are plotted in terms of absolute value in the present paper. The advantage of this fashion is that the differences in the stress at the same strain level can be readily compared. As seen in this figure, the shapes of the stress–strain curves of the CIUC and CIUE tests are almost the

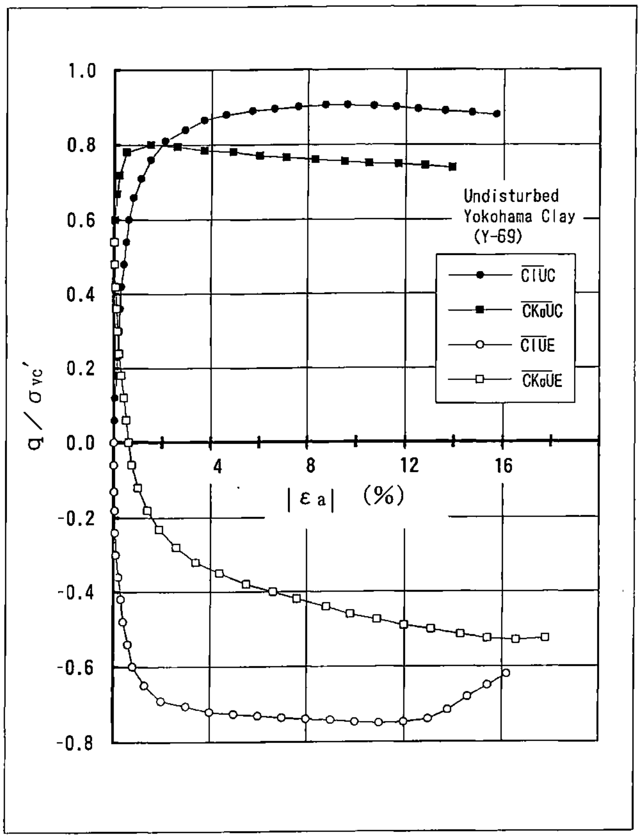

Fig. 7 Comparisons of normalized stress–strain behaviour obtained from four types of triaxial tests.

same, with a sharp bending of the stress–strain curves at around 2% of axial strain, and the peak stresses at around 10% of axial strain. It also can be seen from the figure that there is a distinct measure of symmetry about the q=0 line for compression and extension loadings for isotropically consolidated samples. The stress–strain curve of the CKoUC test, on the other hand, has a pronounced peak, whereas such a peak does not appear in the CKoUE test. In the CKoUC test, the axial strain at peak stress is about 2%. In the CKoUE test, on the other hand, the peak stress does not appear until axial strain is around 17%. It has also been found that the anisotropy in the sress–strain behaviour of Ko-consolidated clay have been summarized as anisotropy in strength, strength mobilization, and stress–strain behaviour(Kamei and Sakajo, 1995). To this end, failure strains are found to be affected by the stress system during consolidation and the mode of shear. This sort of difference in strains at failure between the CKoUC and CKoUE tests may correspond to the difference in strains required to mobilize the active and passive earth pressure(Nakase and Kamei, 1983). It is known that the principal stress differences obtained from the Ko-consolidated samples are smaller than those obtained from the isotropically consolidated samples, irrespective of compression or extension loadings. Con-

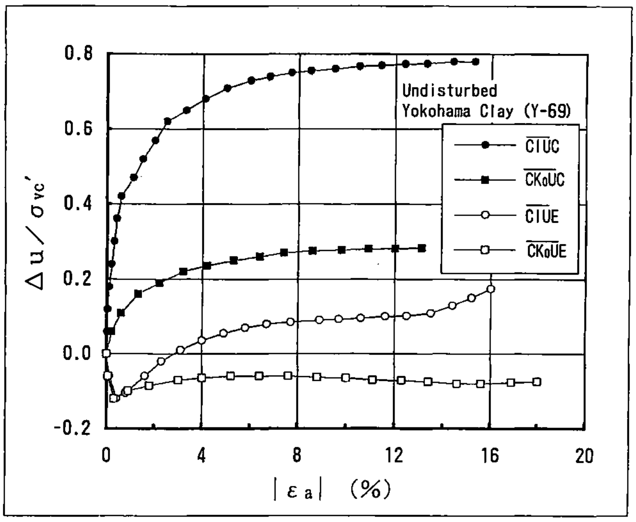

Fig. 8 Comparisons of normalized excess pore pressure Δu–strain behaviour obtained from four types of triaxial tests.

solidation and shear conditions clearly affect the stress –strain curves of the four different types of triaxial tests.

Comparison of excess pore pressure Δu–strain behaviour obtained from four types of triaxial tests is shown in Fig.8, where the excess pore pressure Δu is normalized by the vertical effective consolidation pressure σ_{vc}'. This figure shows that the normalized excess pore pressure Δu–strain behaviour of undrained triaxial compression and extension loadings for undisturbed Yokohama clay (Y–69) seem to depend on the consolidation and shear processes. The maximum excess pore pressure is seen at axial strain of about 10%, irrespective of the consolidation and shear processes used. The maximum normalized excess pore pressure decreases in order from CIUC, through CKoUC and CIUE, to CKoUE. The differences in the maximum normalized excess pore pressures are due to differences in the test conditions. The development of excess pore pressure in the foundation should be taken into account when the mechanical behaviour of foundation is evaluated.

Figure 9 compares the effective stress path obtained from four types of triaxial tests, where the principal stress difference $q=\sigma_a-\sigma_r$ and the mean effective stress $p'=(\sigma_a'+2\sigma_r')/3$ are normalized by dividing by the vertical effective consolidation pressure σ_{vc}'. As seen in this figure, the shape of effective stress paths is almost identical, with a distinct symmetry about the q=0 line for compression and extension loadings for isotropically consolidated samples. The effective stress paths are also characterized by sharp reversal as they approach the critical state line, which thereafter they tend to follows the single and unique line of failure points of both drained and undrained tests is defined

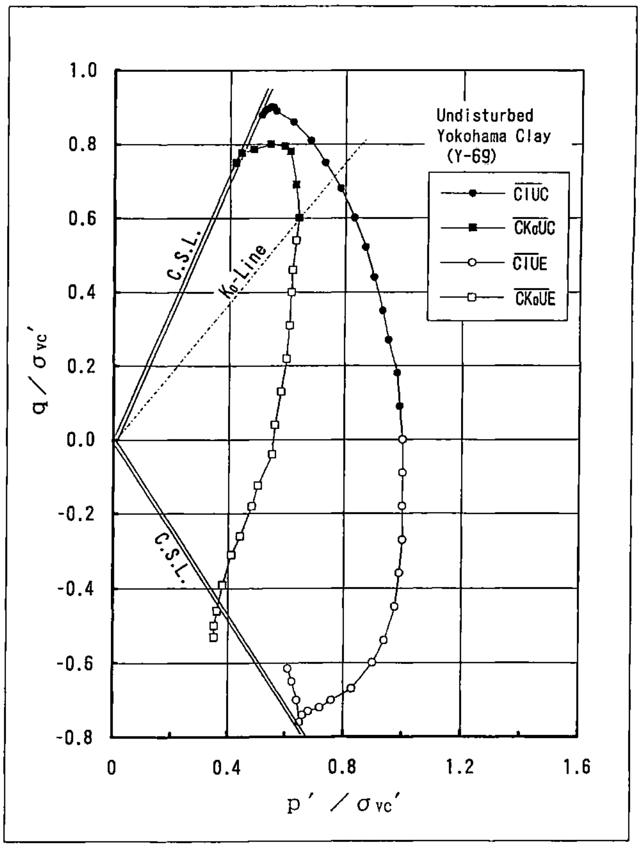

Fig. 9 Comparisons of normalized effective stress path obtained from four types of triaxial tests.

as the critical state line. Failure will be manifested as a state at which large shear distortions occur, until soils flow as a frictional fluid occur with no change in stress, or in specific volume (v). It is not synonymous with failure, and is determined by a particular combination of all three state parameters q, p′, and v (Schofield and Wroth, 1968). This surface has been called the Roscoe surface. The Roscoe surface coincides with the test path for the normally consolidated sample.

Marked change in the shape of the stress path appears between the tests for the isotropically consolidated sample and Ko–consolidated sample. This will be due to the previously mentioned change in excess pore pressure response. The test path for the CKoUC test rises almost vertically towards the Roscoe surface, and then moves along close the Roscoe surface to the critical state line. After the effective stress path has reached the extended yield surface, plastic strain is produced, and it progresses with further yielding to the critical state line in the CKoUC test. In CKoUE test, the effective stress path progresses almostly vertically at the initial stage, during shear from the completion point of the Ko–consolidation. Subsequently, the effective mean principal stress decreases with the approach toward the critical state line.

Undrained shear strength

To investigate the general applicability of the undrained shear strength of Yokohama clay (Y–69), the test results are compared with that of the Kawasaki clay–mixture series (Kamei, 1985 MS). The Kawasaki clay–mixture series consists of five soils named M–10, M–15, M–20, K–30, and K–50. The letter M stands for mixture and the numbers refer to plasticity index (PI). K–30 and K–50 were Kawasaki clay. Based on the test data, Kamei (1985 MS), Nakase and Kamei (1988), and Nakase et al. (1988) reported that the engineering properties of Kawasaki clay–mixture series correspond well with those of natural marine clays. It would be interesting to seek possible correlation of the undrained shear strength of Y–69 with Kawasaki clay–mixture series for extending the general applicability of the test results obtained from the series. Plasticity index (PI) is considered here to be a main parameter, as described in detail elesewhere (Kamei, 1996).

Direct comparison of c_u/p values from CIUC, CKoUC, CIUE, and CKoUE tests is shown in Fig.10. This shows the relationship between c_u/p values and the plasticity index (PI) of the soil samples for each test condition, where c_u is the undrained shear strength and p is the vertical effective consolidation pressure in the consolidation process. The results obtained from the Yokohama clay (Y–69) are very close to those obtained from the Kawasaki clay–mixture series. Values of c_u/p to decrease in order of the CIUC, CKoUC, CIUE, and CKoUE tests, irrespective of PI of the soil samples. Effect of stress conditions in the consolidation process and shear conditions on the value of c_u/p is more pronounced as the PI of the soil samples decreases. In light of the test results, it may be said that the difference in c_u/p value due to differing in the test condition is more marked than that due to a difference in the PI. This is consistent with general supposition (Nakase and Kamei, 1983; Mayne, 1985).

In the present study, anisotropy in undrained shear strength is defined as the ratio of the extensive strength to the compressive strength. This definition has been widely used by others (Duncan and Seed, 1966; Davis and Christian, 1971; Ladd et al., 1977; Nakase and Kamei, 1983). Figure 11 shows the relationship between the ratio of the c_u/p value in the extension test, $(c_u/p)_E$, to the c_u/p value in the compression test, $(c_u/p)_C$, and the PI. Strength anisotropy in terms of the ratio of extensive strength to compressive strength, decreases as the PI of soil decreases. No appreciable difference in this tendency is seen between the Kawasaki clay–mixture series and the Yokohama clay (Y–69). The undrained shear strength anisotropy is more marked in the CKoU tests. In this sense, it may be said that the dependence of undrained shear strength anisotropy on stress condition in the consolidation process is more pronounced as the plasticity index of cohesive soils increases. Figure 11 also shows that $(c_u/p)_E$ is smaller than $(c_u/p)_C$, and the ratio of $(c_u/p)_E$ to $(c_u/p)_C$ decreases with a decrease in PI. The undrained shear strength of the CKoUE test is about 0.6 times that of the CKoUC test, but for soil with lower plasticity index this ratio drops to about 0.5. The engineering significance of this result is that the undrained shear strength of a sample below the center line of a circular excavation may be only 50% of that for shear below the center line of a circular footing. It is emphasized that these two samples had identical consolidation stresses and water contents at failure.

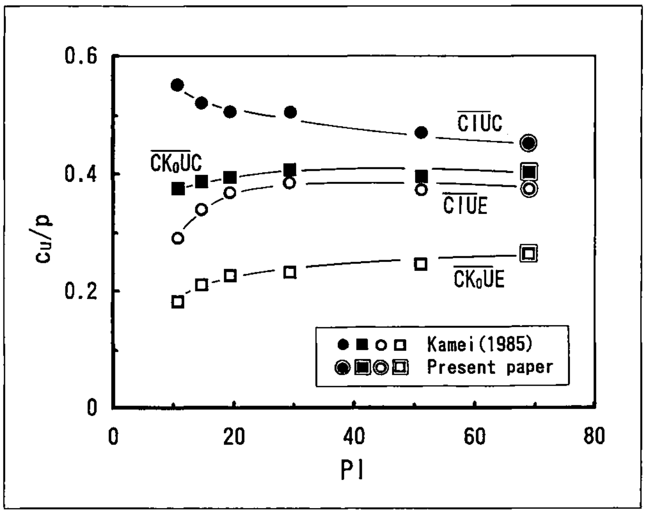

Fig. 10 Relationship between c_u/p values and the plasticity index.

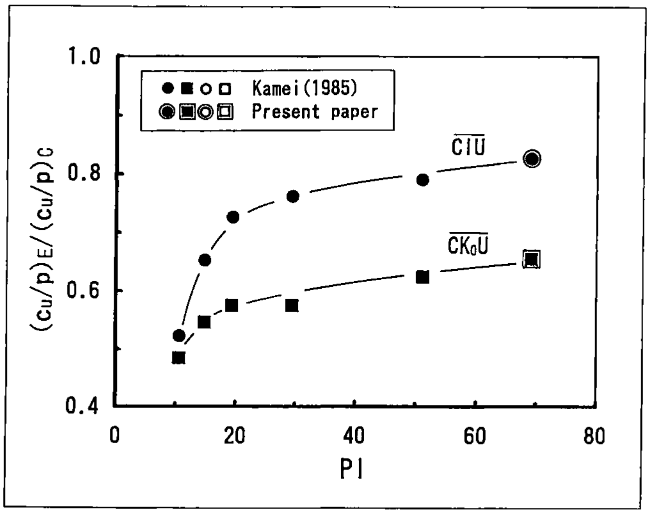

Fig. 11 Relationship between undrained shear strength anisotropy and PI.

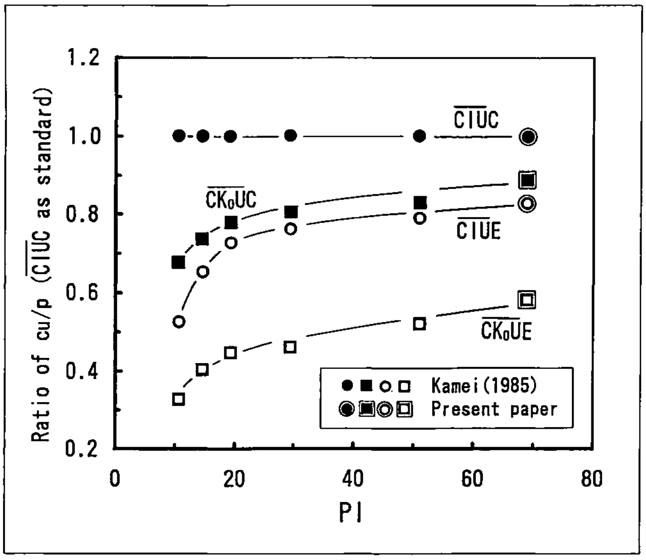

Fig. 12 Relationship between ratio of c_u/p (CIUC as standard) and PI.

However, the large differences in c_u are caused soley by the different modes of failure.

Figure 12 summarizes the c_u/p value obtained from the present experiment and data by Kamei (1985 MS). The CIUC test always yields higher strengths than the CKoUC, CIUE, and CKoUE tests, as would be expected because of the consolidation and shear processes in the triaxial tests. Undrained shear strengths from the latter tests generally range from 40% to 80% of the CIUC strengths. Values of c_u/p obtained from the CKoUC, CIUE, and CKoUE tests, in terms of the ratio to the c_u/p value from the CIUC test are plotted against PI of the soil samples. When the undrained shear strength by CIUC is known, the c_u/p values corresponding to other stress conditions either in consolidation and shear process can be evaluated by multiplying the correction factor determined from Fig. 12. The resulting correlations are shown and provide a useful procedure to determine meaningful c_u/p values. The combination of the undrained shear strength obtained from the CIUC test and the proposed correction factor provide an extremely rapid, easy, reliable and economic means of evaluating the c_u/p values corresponding to other stress conditions either in consolidation and shear process. Limitations do exist in the proposed testing procedure, and it should be regarded only as a first approximation. This study provides, however, a basis for quantitative evaluation of undrained shear strength. A larger data base may enable a more definitive relationship to be established.

Conclusions

The comparison of data obtained from different four types of triaxial test shows significant differences in undrained shear behaviour evaluations, which is consistent with general supposition. A simple procedure has been developed to estimate the undrained shear strength of normally consolidated clay without special triaxial tests. The proposed procedure for evauating the undrained shear strength for different consolidation and shear processes requires only the undrained shear strength obtained from the CIUC test and the plasticity index. It is, therefore, a simple, rapid and economical method. Tentative correction factors based on the CIUC test result are recommended for the other test procedures in triaxial tests. Although only limited data are available, the procedure appears to be sufficiently accurate for practical purposes. Much additional research in this area is required, however, to provide more comparative data.

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（要　旨）

亀井健史，1996，四種類の圧密非排水三軸試験から得られた非排水せん断強さとその相互関係．島根大学地球資源環境学研究報告，15, 137–145.

本研究では，東京湾より採取した塑性指数 70 程度の高塑性な粘土に対して三軸等方圧密・Ko 圧密後に非排水圧縮・伸張試験を行い，圧密・せん断過程の違いが粘土の非排水せん断試験結果に及ぼす影響を検討している．また，非排水せん断強さに関して上記試験結果と既往の試験結果との比較検討を行っている．

その結果，圧密・せん断過程の違いが粘土の非排水せん断試験結果に及ぼす影響を定量的な観点から明らかにしている．また，等方圧密非排水三軸圧縮試験から得られた強度増加率とその土試料の塑性指数から簡単に異なる圧密．せん断条件下における強度増加率が得られる補正図表を提案している．提案した補正図表は，塑性指数が 10～70 程度の幅広い粘性土に対して適用可能であり，実地盤において想定される異なる圧密・せん断過程における粘性土の非排水せん断強さを推定する際には，第一次近似における非排水せん断強さ評価法として有用であると考えられる．