Static and Cyclic Triaxial Loading Tests of K_0-consolidated Clay

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Abstract

A new automatic control triaxial apparatus and testing methods are explained for monotonic and repeated loading tests after isotropic, K_0 or constant stress ratio consolidation. The double cell type lateral strain measuring system with new ideas is adopted to achieve the condition of K_0-consolidation. The test results proved that this new system worked very well in comparison with the conventional tests. Shear behaviour after K_0-consolidation was quite different from those of isotropically consolidated clays. The influence of K_0-consolidation can be observed, particularly in the compression side, in the existence of a phase change line and the increase of the critical state line slope. In repeated loading after K_0-consolidation, the amount of plastic strains differ, depending on whether the deviatoric stress is above or below the stress difference q_{K_0} or the stress ratio η_{K_0} after consolidation. In a region larger than q_{K_0}, soil behaviours indicating effects of K_0-consolidation were observable, such as the leaning of the effective stress path to the right and the fact that the axial strain during unloading scarecely changed. In the lower regions, however, there was no such difference between isotropically and K_0-consolidated clays.

1. Introduction

Natural grounds often consolidate in K_0-condition, namely without lateral strain, and consolidation occurs in anisotropic stress conditions. Natural grounds are subject to random stresses, in addition to cyclic stresses due to loading and unloading in earthquakes, traffic vibrations, underground water level changes, sea wave forces, etc..

Many approaches have been taken to establish constitutive equations of clay soils, considering such complicated stress histories. Researches on overconsolidation, anisotropic consolidation, repeated loading and other related constitutive equations have been reported in the past, but these are only considered separately. There is no research, for example, on coupled anisotropic consolidation and repeated loading.

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One reason for this is that there are still many questions to be cleared in each of these studies. Another reason must be the lack of an appropriate laboratory apparatus capable of testing these two phenomena—anisotropic consolidation and repeated shearing—successively. A triaxial apparatus was thus developed that could perform repeated loading after various consolidation tests.

This paper describes a new apparatus and testing methods, including discussions about the test results for repeated and monotonic shear loading tests after K_0 or constant stress ratio consolidation.

2. Experimental Apparatus and Methods of Testing

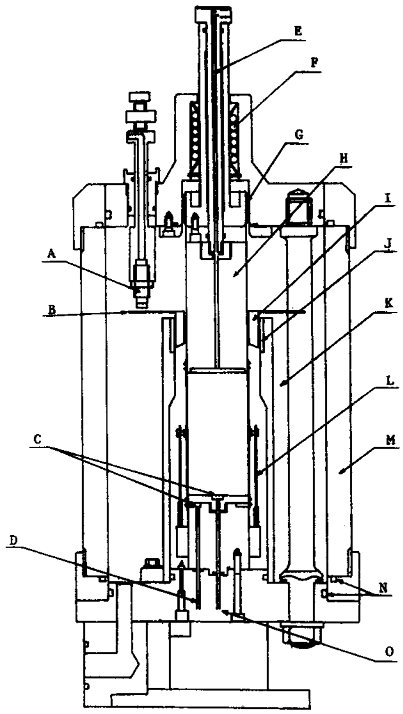
There are many methods of carrying out K_0-consolidation,[1] but in order to perform accurate tests and to obtain stress-strain relationships, the following conditions should be satisfied: 1) K_0-consolidation and shear test should be done successively. 2) Lateral frictions should be eliminated. 3) The K_0-value should be measured. 4) Back pressure could be applied.

In order to fulfill such conditions, the shear test after K_0-consolidation is carried out in a triaxial cell. Due to the complexity of the K_0-consolidation test operation, and due to the fact that the operation is extended over a period of time, automatic control is recommended. It is desirable for the shear test to be carried out consecutively after consolidation without stress release. Also a constant mean principal stress condition is preferred for accurately handling dilatancy characteristics.

Modifications were therefore introduced in a constant mean principal stress repeated loading shear test apparatus to carry out repeated shear loading tests after K_0-consolidation or constant stress ratio ($\eta=q/p$) consolidation with automatic control.[2] The main features of this new apparatus are: 1) Automatic control by a microcomputer. 2) A double cell type of a lateral strain measuring system. 3) Various types of shear tests, such as the constant mean principal stress repeated loading shear test, can be done successively after consolidation. 4) Control programs are written in high speed languages.

The present apparatus uses an electric-oil pressure servo-controlled system for axial and lateral loading, allowing these to be applied separately. Any stress condition, therefore, can be achieved by operating the control commands from the microcomputer. Applying these features, constant mean principal stress repeated shear loading tests following isotropic, K_0 or constant stress ratio consolidations can be done.

The double cell type lateral strain measuring system to achieve the condition of K_0-consolidation is shown in Figure 1. Fluorine silicon oil (density 1.26) was used for the inner cell liquid for the following reasons: 1) It is heavier than water



A : Non-contact type displacement sensor
B : Target , C : Porous stone , D : Drainage
E : Drainage , F : Slide ball bearing
G : Bellofram , H : Cap , I : Float
J : Teflon bushing , K : Inner cell
L : Cantilever type displacement sensor
M : Outer cell , N : O-ring
O : Pore pressure transduser

Fig. 1. K_0-consolidation triaxial cell

and does not mix with it. 2) It does not corrode materials in the triaxial cell (such as rubber membrane, O-ring, etc.). 3) It hardly allows air to pass through. 4) It is not poisonous.

De-aired distilled water was used in the outer cell. The float in Figure 1 has an intermediate density between that of water and silicon oil. Attached to it is an aluminum target which measures the changes in the levels of the inner cell liquid by recording its up and down movements by a non-contact type displacement sensor (resolution 1/1000 mm).

Fig. 2 shows the entire system. The changes in the float position detected by the displacement sensor are collected by the microcomputer, and after some calculation, control signals are transmitted to the lateral servo-controller. Control and data collection programs are written mainly in an Assembler language that enables high speed processing. Even in the worst case, the fluctuation of the float was kept

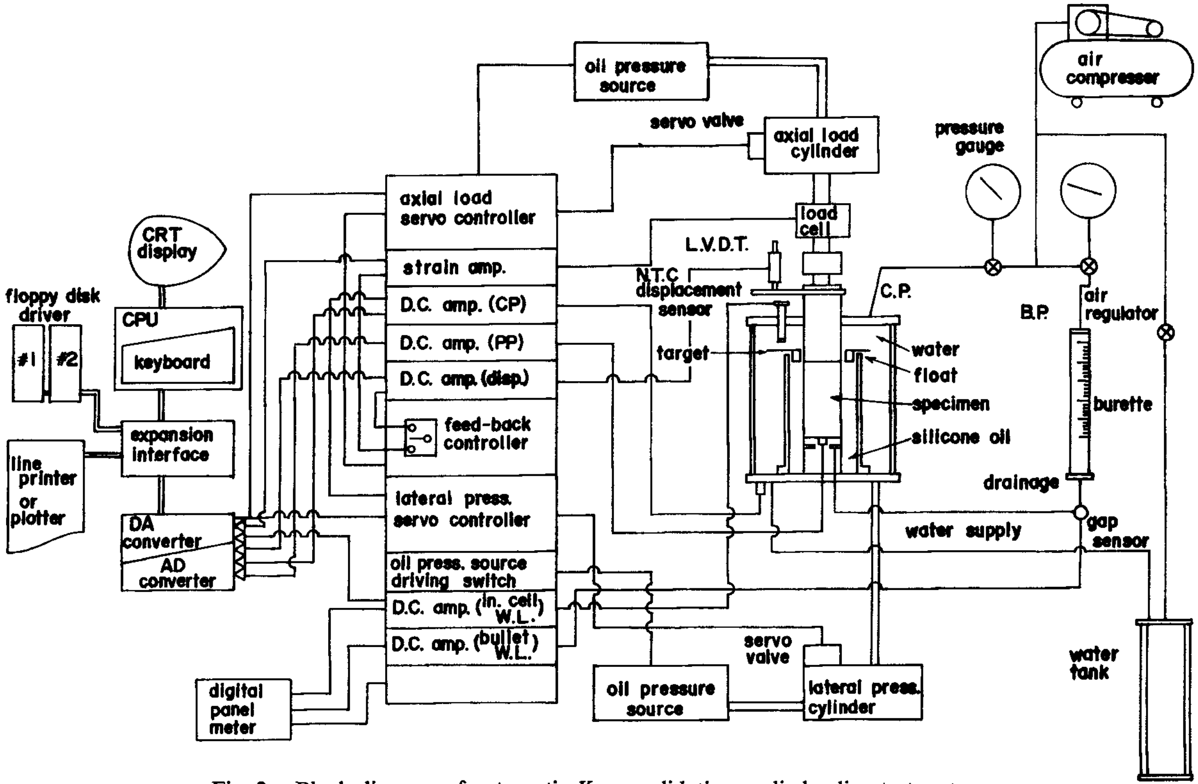


Fig. 2. Block diagram of automatic K_0-consolidation-cyclic loading test system

within a margin of a few thousandths of mm.

The present apparatus, as shown in Figure 1 can be extended to measure lateral strains directly by using cantilever-type deformation gauges. Constant stress ratio consolidation is achieved by transmitting constantly increasing or decreasing signals from the microcomputer to the axial and lateral pressure servo-controllers. After consolidation, the axial load is changed from a stress rate control to a strain rate control, and shearing was carried out maintaining a constant mean principal stress condition.

In the present system, the turnback of stresses in loading and unloading can be carried out at any moment by a microcomputer keyboard operation. Stress amplitude controlled tests, strain amplitude controlled tests, and stress ratio amplitude controlled tests can also be done, since all data are indicated in the display and recorded on floppy disks. Computed results are given by means of a line printer and an X-Y plotter.

Remolded Fukakusa clay with a preconsolidation pressure 69 kPa was used for the laboratory experiments. Index properties and mechanical constants are shown in Table 1. Specimen sizes are 5 cm in diameter and 10 cm in height.

Tables 2 and 3 show the test conditions of each K_0-consolidation. The KM test series represent monotionic shear loading after K_0-consolidation, and KC represents repeated shear loading. In K_0-consolidation, one step's loading time was about 2 hours, and each loading step follows the value shown in Tables 2 and 3. In this case, the load increment was given in undrained conditions since, in the field, initial stress changes due to external loads are sustained by pore water pressure. The axial load and the lateral load were increased isotropically, and the lateral control in the system was started as drainage was initiated.

The experimental conditions of the constant stress ratio consolidation test are

Table 1. Physical properties of Fukakusa clay

Specific gravity	2.71
Liquid limit	45.5
Plastic limit	22.4
Plasticity index	23.1
Sand fraction	12.0
Silt fraction	64.0
Clay fraction ($<5\mu$)	24.0
λ	0.0900
κ	0.0133
M (compression)	1.40

shown in Table 4. The test series number indicates the stress ratio value during consolidation. First, after carrying out isotropic consolidation of 49 kPa for one

Table 2. Test conditions for monotonic loading after K_0-consolidation

Test No.	Loading step (σ_1' kPa)	Monotonic loading side
KM1	69, 167, 294 51 hours const. at last step	Comp.
KM2	69, 167, 294	Comp.
KM3	20, 39, 69, 98, 147, 196, 245, 294	Comp.
KM4	294	Comp.
KM5	5 (kPa/hr) Continuous loading	Comp.
KM6	20, 39, 69, 147, 294	Ext.
KM7	20, 49, 98, 196, 392	Comp.
KM8	20, 49, 98, 196, 392	Ext.
KM9	20, 39, 69, 147	Comp.
KM10	20, 39, 69, 147	Ext.
K1	69, 98, 147, 196, 245, 294, 245 196, 147, 98, 69, 39, 69, 98, 147 196, 245	—

Table 3. Test conditions for cyclic loading after K_0-consolidation

Deviatoric stress amplitude controlled tests

Test No.	Loading step (σ_1 kPa)	q_{min} (kPa)	q_{max} (kPa)	Number of cycles	q_{K0} (kPa)
KC1	69, 167, 294	172	196	10	172
KC2	20, 39, 69, 147, 294	167	216	10	167
KC3	20, 39, 69, 147, 294	196	147	10	196
KC4	20, 39, 69, 147, 294	216	137	5	176

Effective stress ratio amplitude controlled tests

Test No.	Loading step (σ_1 kPa)	η_{min}	η_{max}	Number of cycles	η_{K0}
KC5	20, 39, 69, 147, 294	0.85	1.13	10	0.85
KC6	20, 39, 69, 147, 294	−0.27	1.23	5	0.90

Table 4. Test conditions for cyclic loading after anisotropic consolidation

Test No.	η at consol.	q_{min} (kPa)	q_{max} (kPa)
AN00	0.00	0	49
AN25	0.25	49	98
AN50	0.50	98	147
AN75	0.75	147	196

day, drained shearing with an axial load increment of 4.9 kPa/hr was done up to the prescribed stress ratio. The constant stress ratio consolidation was then carried out up to $p=\sigma_m=196$ kPa increasing the axial load by the same rate.

After consolidation, the constant mean principal stress undrained shearing with 0.06%/min strain rate control was carried out. Shearing conditions of K_0-consolidated clays are shown in Tables 2 and 3. With respect to constant stress ratio consolidated clays, tests are carried out with a stress amplitude $q=49$ kPa and 10 times of repeated loading in the compression side. In the case of repeated shearing, monotonic shearing was carried out consecutively. In all tests, a back pressure of 49 kPa and a lateral drainage method by paper drain were used.

3. Test Results and Discussions

3.1 K_0-consolidation

Figure 3 shows the K_0-value-σ_1 relationship of each test. The K_0-value ap-

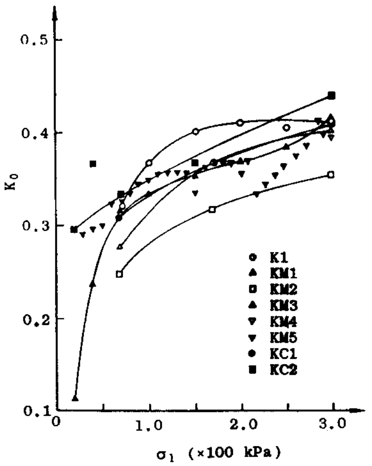


Fig. 3. Dependence of K_0-value on axial stress

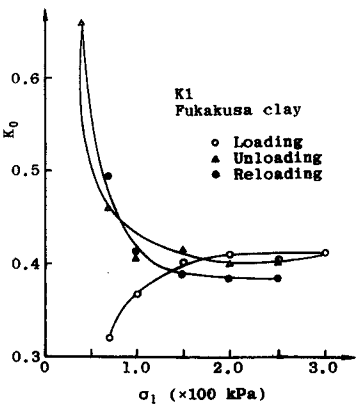


Fig. 4. K_0-Value in repeated loading

proaches a constant value for axial loads sufficiently greater than the preconsolidation load (69 kPa). In other words, it is constant in the normally consolidated region. Due to various load increment methods, a small amount of scattering may be observed in K_0-values, but it converges at about 0.40. Past research using Fukakusa clay reported a K_0-value of 0.42.[3] A satisfactory K_0-value of 0.38 was obtained by calculating the K_0-value from the Brooker and Ireland formula.[4] In the case of swelling and reconsolidation, the K_0-value increased with an increase in the over-consolidation ratio, as shown in Figure 4.

The e-ln$p$ relationships shown in Figure 5 become linear in the normally consolidated region, and their slopes λ are close to the value indicated in Table 1 due to isotropic consolidation. In other words, normal consolidation lines, due to the K_0 and isotropic consolidation, become parallel. These results agree with former research results.[5] The slopes of the swelling lines κ, due to the K_0-consolidation and isotropic consolidation, also become very close in value. The axial strain-volumetric strain relationship agrees closely with the unit slope line, and it indicates that lateral strains are almost negligible (Figure 6). Calculating the lateral strains from the amount of drained water and axial strains, the resulting value is less than 0.5%, even in the worst case. From these results it can be concluded that suitable conditions were achieved to examine soil behaviour in shearing after consolidation, using the present apparatus and system.

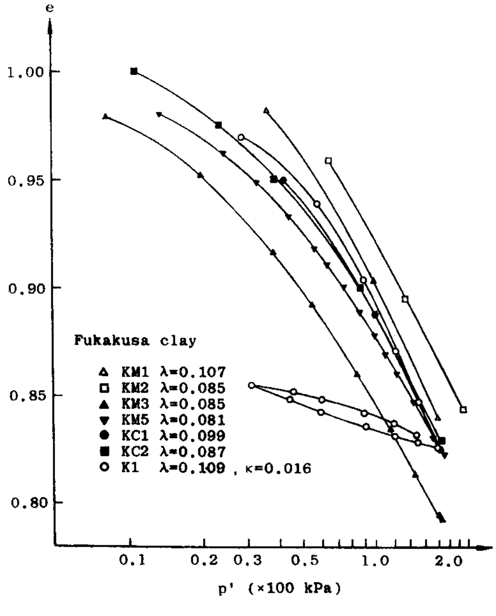


Fig. 5. e-ln p curves in K_0-consolidation

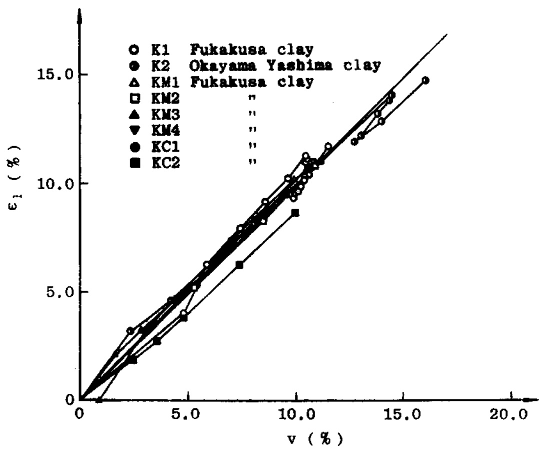


Fig. 6. Relationship between axial strain and volumetric strain in K_0-consolidation

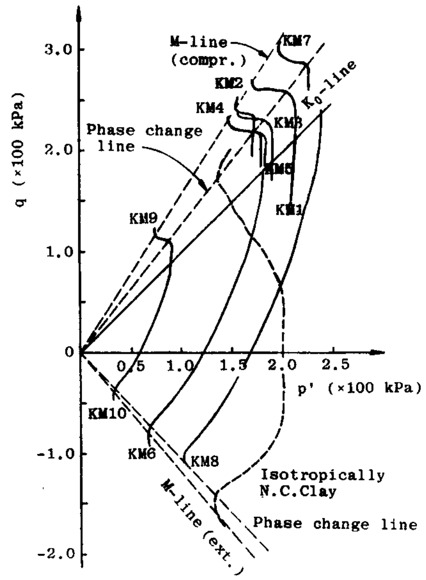
Incidentally, the axial loads of $\sigma_0=19.6$, 39.2 kPa are lower than the preconsolidation load, therefore the conditions can be considered to be overconsolidated. In the swelling and reconsolidation results, the K_0-value increased as the overconsolidation ratio increased. But after setting the specimens and applying $\sigma_1=19.6$ and 39.2 kPa, the K_0-value becomes less than 0.40, even with the same overconsolidated conditions (Figure 3). Three possible causes for this could be: 1) Influence of disturbance due to sampling. 2) The specimens receive isotropic stress release after being extracted from samplers. However, in the cell, stress conditions differ when unloading is carried out avoiding lateral strains. 3) In the case of low stresses, control is difficult giving rise to control errors. This phenomenon occurred in all experiments using remolded Fukakusa clay.

3.2 K_0-consolidated shear tests

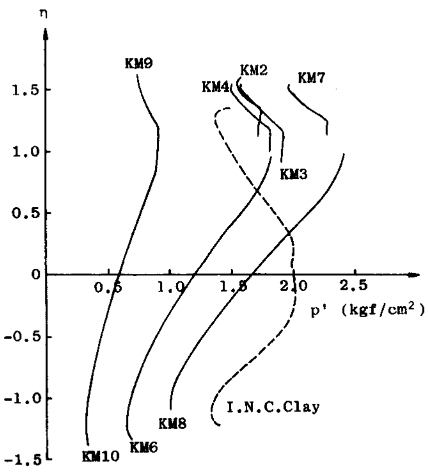
(1) Monotonic shear tests

Figures 7(a), (b), (c), (d) and (e) show the effective stress path and relationships between stress-strain and strain-pore pressure. The test results of isotropic normally consolidated clay are shown in the figure by dashed lines. The shear tests starting with an axial pressure of 294 kPa were carried out with five different kinds of loading steps and rates (KM1–KM5). Due to various types of loading during consolidation, the stress conditions and void ratio at the shearing start point are not uniform, but the stress path shapes are very similar, except for the long-term consolidated test KM1. Therefore, influences to monotonic shear behaviour due to different loading methods were not observed. Tests were carried out with three different shearing start loads ($\sigma_1=147$, 294, 392 kPa), resulting in very similar path shapes and stress-strain relationships.

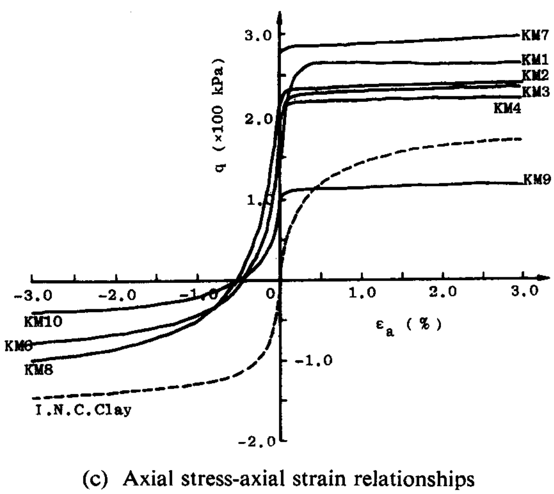
Shearing in the compression side does not generate plastic strains in the beginning. However, exceeding a certain stress ratio, both the plastic volumetric strain and the plastic shear strain increase abruptly. This stress point where the plastic strain increases abruptly will be called the "phase change" point, similar to the



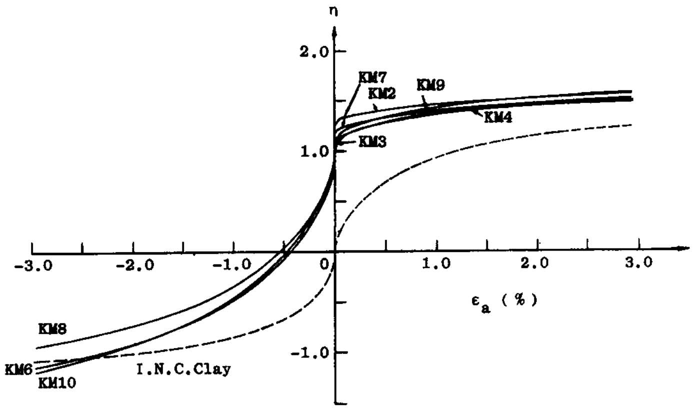
(a) Effective stress paths



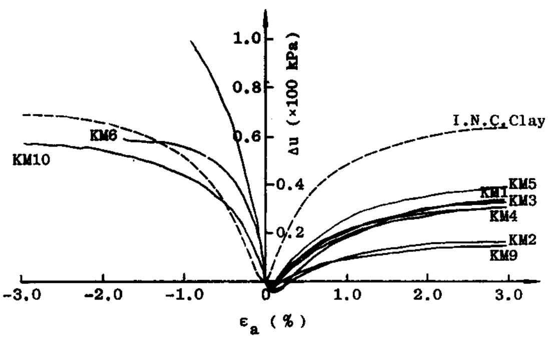
(b) Stress ratio-effective mean principal stress relationship



(c) Axial stress-axial strain relationships



(d) Stress ratio-axial strain relationships



(e) Exeess pore pressure-axial strain relationships

Fig. 7. Monotonic loading test after K_0-consolidation

"phase transition line" for sand. This phase change point is determined by a constant η value even for different consolidaton pressures (Figures 7(a) and (b)). Assuming that the critical state is the point where pore pressure increases change from negative to positive, the η value will be M. For K_0-consolidated clays, M-values are greater than those consolidated isotropically, as shown in Figures 7(a) and (b). Furthermore, the η value for the phase change line and the M-value of isotropically consolidated clays are very similar.

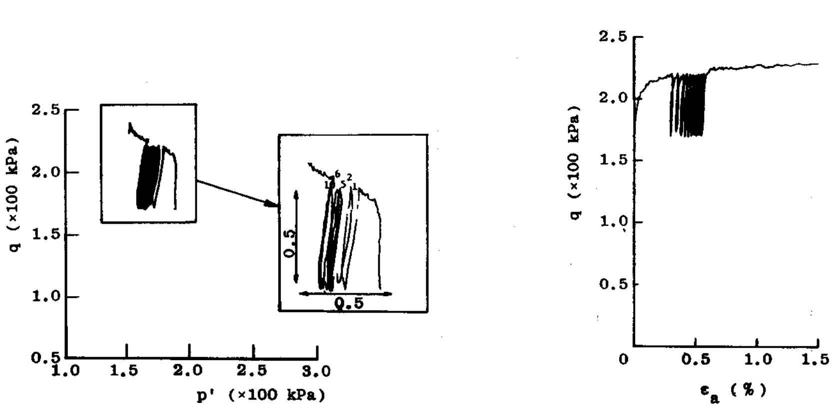
In the extension side, both the plastic volumetric strain and the plastic shearing strain occur immediately after the shearing starts, and the amounts increase gradually up to failure. The M-values are equal or greater than those of isotropically consolidated clays. In K_0-consolidated clays, completely different behaviours were observed, depending upon whether the shearing direction is at the compression side or the extension side. Influences due to consolidation history are stronger on the compression side ($q>0$) than on the extension side.

(2) Repeated shear test

In KC1, shown in Table 3, repeated shearing was carried out with the stress level lower than the above-mentioned phase change line, but pore pressure and axial strain scarcely occurred. Hence, it was not convenient to verify plastic strains at repeated shearing. Figures 8(a) and (b) show the effective stress path and axial stress-strain relationship of KC2, whose stress amplitude exceeds the phase change line. During the first cycle, a large quantity of plastic strains in the normally consolidated region occurred. The increment of pore pressure decreases with the number of repetitions, and for K_0-consolidated clays, an equilibrium state for pore pressure seems to exist, as for isotropically consolidated clays.[6]

The pore pressure during shearing increases with unloading and reaches the maximum in the vicinity of the turnback point. The effective stress path migrates to a $p=0$ direction, leaning slightly to the right. Axial strain does not occur during unloading, but occurs abruptly immediately before reaching the maximum load q_{max}. Figures 9(a) and (b) show KC3 whose repeated shearing was carried out in the K_0-extension side. Notwithstanding the fact that the stress amplitude is equal to KC2, both pore pressure and axial strain scarcely occur. At the test KC4 in Figures 10(a) and (b), a load is applied with compression-extension cycling in reference to the K_0-line (stress ratio η_{K0}). The stress path is initially leaning somewhat to the right, but the accumulated amount of pore pressure at q_{max} increases with an advancing number of cycles, and the path assumes an upright position.

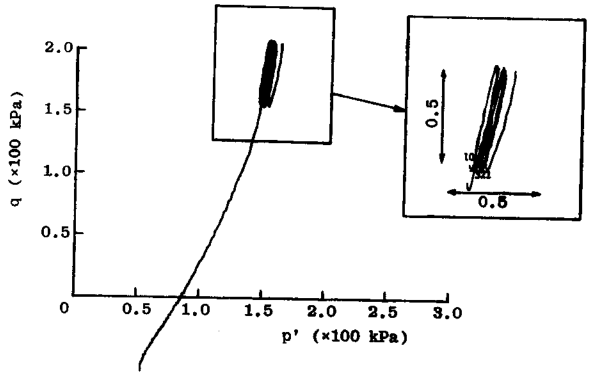
To clarify the relationship between the K_0-line, the critical state line and the turnback stress point at repeated shearing, constant stress ratio amplitude tests were carried out. Figures 11(a) and (b) shows the test results of KC6, in which repeated



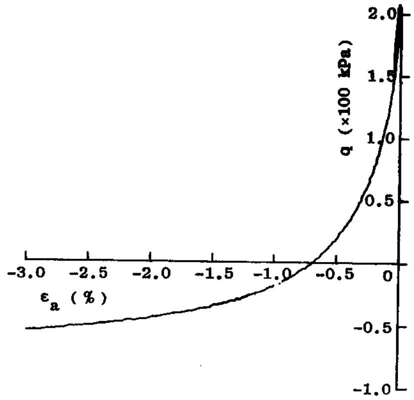
(a) Effective stress path

(b) Axial stress-axial strain relationship

Fig. 8. Cyclic loading test for KC2

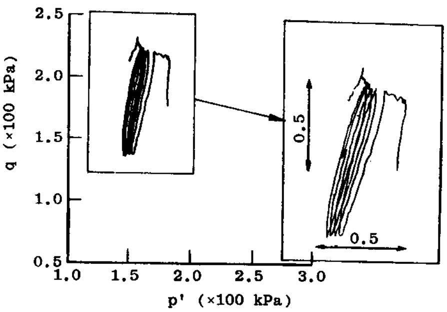


(a) Effective stress path

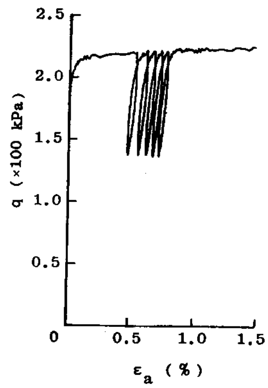


(b) Axial stress-axial strain relationship

Fig. 9. Cyclic loading test for KC3 (K_0-extension)

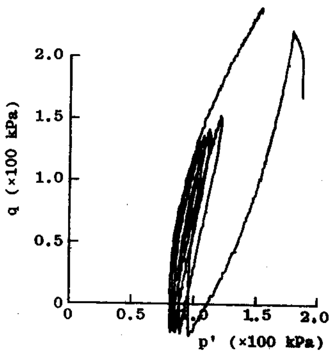


(a) Effective stress path

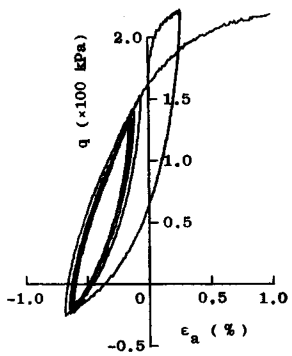


(b) Axial stress-axial strain relationship

Fig. 10. Cyclic loading test for KC4 (K_0-extension-compression cycling)



(a) Effective stress path



(b) Axial stress-axial strain relationship

Fig. 11. Cyclic loading test for KC6 (η constant)

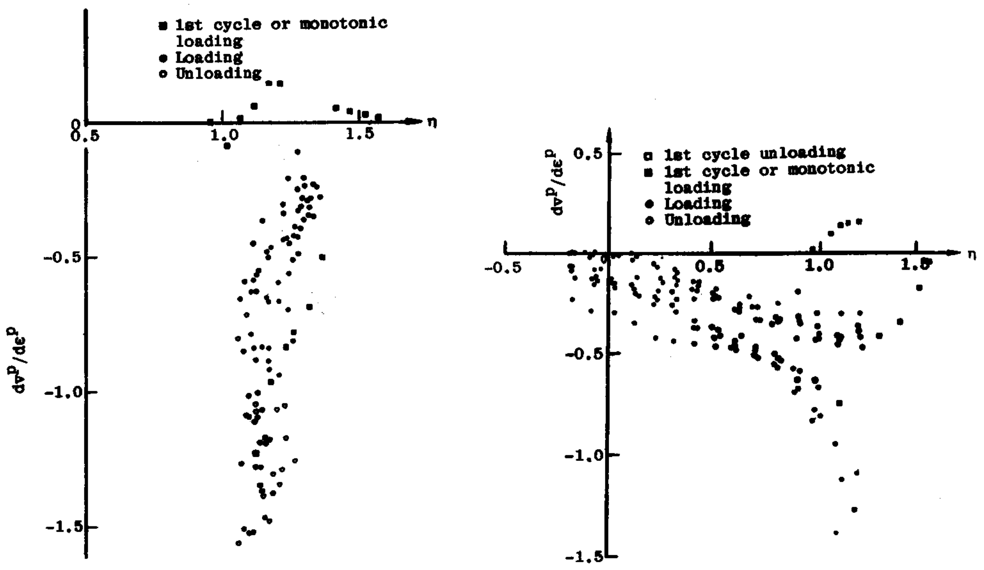
shearing was carried out in the region between $\eta_{max}=1.23$ and $\eta_{min}=-0.27$, whose stress ratio relation is given below:

$$\frac{\eta_{max}-\eta_{K_0}}{M_{comp}-\eta_{K_0}}=\frac{\eta_{K_0}-\eta_{min}}{\eta_{K_0}-M_{ext}}=\text{constant}$$

where M_{comp} is M-value at the compression side and M_{ext} is at the extension side.

The effective stress path leans slightly to the right for the large values region of deviatoric stress q, particularly for those above the K_0-line. For those below this region, however, the stress path is not much different from that of normally consolidated clay. In the stress-strain relationship, after the first turnback, there exist almost no axial strains during unloading up to q_{K0}. After that point, however, strains occur.

Summarizing the four types of test results, (KC2, KC3, KC4, KC6), the occurrence of plastic strains differ depending upon the deviatoric stress regions above or below the stress difference q_{K_0} during consolidation or the stress ratio η_{K_0}. For lower regions, there is not much difference between the K_0-consolidated clays and the isotropically consolidated clays. For larger regions, while pore pressure occurs together with unloading, axial strains do not change. This characteristic can be understood by investigating the relationship between the plastic strain increment rations and the stress ratios as shown in Figures 12(a) and (b). In the isotropically consolidated Fukakusa clay repeated shearing test results, this plastic strain increment ratio—stress ratio relationship was linear with a slope value of -1. However, in K_0-consolidated clay, when testing in large deviatoric stress regions only, such as KC2 as indicated in Figure 12(a), the stress ratio scarcely changes because of a small stress amplitude. However, the plastic strain increment ratio changes considerably



(a) Test No. KC2 (b) Test No. KC6

Fig. 12. Relationship between stress ratio and plastic strain increment ratio

and a near vertical linear relation was obtained. In the large amplitude test KC6, as indicated in Figure 12(b), for values lower than $\eta=1.0$, which can be considered to be close to K_0, the plastic strain increment ratio has a linear relation with a slope value close to -1, and the test results are similar to those of isotropically consolidated clays. For values larger than 1.0, however, the slope is nearly vertical, as in KC2, indicating that the conditions of the plastic strain production are different having K_0 as a boundary.

Given these KC6 results, notwithstanding its long history of repeated shearing, the effective stress path and the stress-strain relation kept almost the same shapes after two cycles, resulting only in a movement of the origin. It can be concluded that both the dilatancy characteristics and the stress-strain relationships did not undergo changes because the relationship between η and M_{comp}, M_{ext} at the turnback point were kept constant. It can be observed from each test result that even for K_0-consolidated clays, the effective stress path in repeated shearing migrates into the state boundary surface (SBS), adopting a relatively large stress amplitude loading. In monotonic shearing after repeated loading, the stress path in the SBS leans slightly to the right even exceeding the preset amplitude. However, as it reaches a point considered to be on the stress path for the normal consolidation region, it bends abruptly towards to the critical state line on the stress path for a clay which has not received any repeated loading. This change is sharper than that of normally consolidated clays.

3.3 Constant stress ratio consolidation and shear test

Figure 13 shows the e-ln p relationships in constant stress ratio consolidation tests. In each test, the λ-values approach those of isotropic or K_0-consolidation (indicated in Table 1 or Figure 5). Figure 14 shows the axial strain-volumetric strain relationships during consolidation. Lateral strains decrease as the curve approaches the line of slope 1.0 indicated by a dashed line in the figure. Due to a one day previous isotropic consolidation of 49 kPa, scattering occurs at low strain levels, but for a volumetric strain larger than 1.5%, it becomes almost linear. The shape of the slope indicates a clear direct proportional relationship between shear stress, namely stress ratio during consolidation and lateral strain.

Effective stress paths and stress-strain relationships of repeated shearing tests after constant stress ratio consolidation are shown in Figures 15(a) and (b). Four different types of test results are shown in the same figure, but except for η-values during consolidation in each test, it should be noted that the consolidation time and void ratio differ. By increasing the stress ratio at consolidation, the behaviour of clay resembles KC2 which has the same repeated loading conditions after K_0-consolidation. In other words, the following features become obvious: 1) The existence of a phase change point becomes observable. 2) The shape of the effective stress path leans slightly to the right and migrates toward a $p=0$ direction. 3) Pore pressure increases during unloading. 4) Axial strain during unloading scarcely

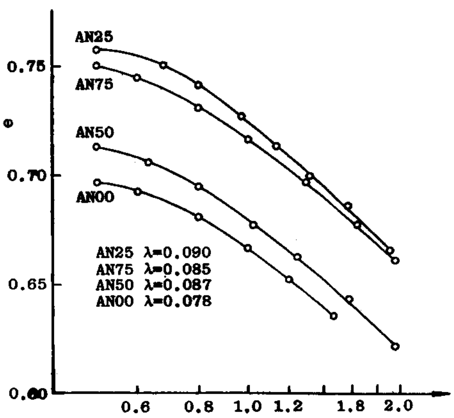


Fig. 13. e-ln p curves in anisotropic consolidation

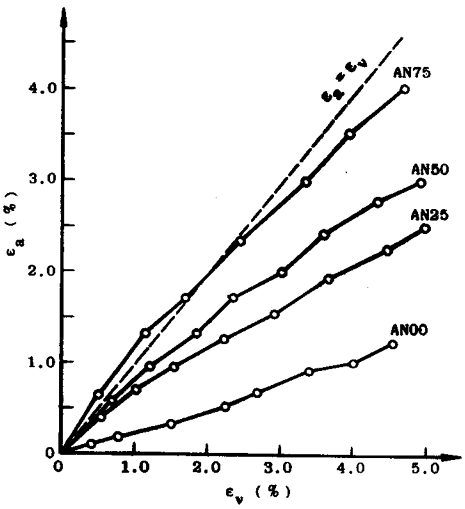
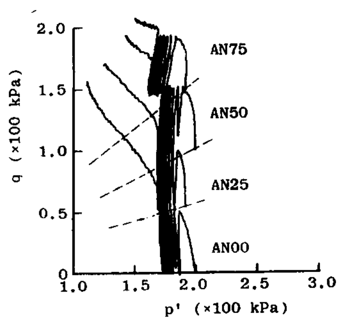
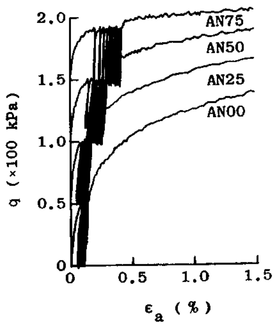


Fig. 14. Axial strain-volumetric strain relationships in anisotropic consolidation



(a) Effective stress paths



(b) Axial stress-axial strain relationships

Fig. 15. Cyclic loading tests after anisotropic consolidation

increases. 5) In monotonic shear after repeated loading, the path turns back abruptly toward a $p=0$ direction as soon as it reaches the original stress path.

These characteristics change according to, or in proportion to shear stresses, i.e., stress ratio during consolidation, or to the rate of lateral strain. Given that the K_0-value becomes constant in the normal consolidation region during K_0-consolidation; that the lateral strain during consolidation scarcely occurs at AN75 (Figure 14); and that the repeated test results of AN75 and KC2 are very similar, the shear behaviour of K_0-consolidated clays can be handled as a constant stress ratio consolidated clay when dealing with constitutive relationships considering consolidation history. The shear conditions of the four tests that were carried out were only of one type, but it will be necessary to carry on future tests with different conditions.

4. Conclusions

The following summarizes test results obtained in this research: 1) Using the newly developed automatic K_0-consolidation triaxial test apparatus, the behaviour of clay observed during K_0-consolidation was the same as observed in the previous research, therefore providing suitable conditions to verify the shear behaviour of K_0-consolidated clays. 2) The shear behaviour after K_0-consolidation was very different from those of isotropically consolidated clays. The influence of K_0-consolidation can be observed, particularly in the compression side, in the existence of a phase change line and the increase in the critical state line slope. 3) In repeated shearing after K_0-consolidation, the amount of plastic strains differ depending on whether the deviatoric stress is above or below the stress difference q_{K0} or the stress ratio η_{K0} after consolidation. In a region larger than q_{K0}, soil behaviours indicating effects of K_0-consolidation were observable, such as the leaning of the effective stress path to the right, and the fact that axial strain during unloading scarcely changed.

In the lower regions, however, there was no such difference between isotropically and K_0-consolidated clays. 4) When carrying out monotonic shear after repeated loading, K_0-consolidated clay behaviour returns to that of monotonic shear without repeating. 5) In the repeated shear behaviour of constant stress ratio consolidated clays, it is observed that as the η-value during consolidation approaches the final stress ratio η_{K_0} of K_0-consolidated clay, it can approximate the behaviour of K_0-consolidated clay.

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