Deformation and Strength Behaviors of Soil under Delayed Consolidation

by

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Abstract

In this paper, the deformation and strength behaviors of soil under delayed consolidation are investigated through the concept of a state boundary surface, with respect to quasi-overconsolidation characteristics. The mechanism of change in state during delayed consolidation is made clear, considering the restrictive conditions in terms of stress and strain for the K_0-secondary compression. Moreover, a brief discussion on the mechanical properties of soil after delayed consolidation is made, using the result of the K_0-triaxial loading test and undrained shear test for Kaolin.

1. Introduction

In the light of the recent tendency to erect large structures, it is often the case that the bearing capacity of alluvial ground is not sufficient to support such heavy structures, so that the terrace or upper diluvial strata situated at deeper levels must be regarded as the bearing layers. However, this kind of strata is still rather young in geological age compared with deeper diluvium; thus it has a small over-consolidation ratio (OCR). In the geological sense, it often happens that the loading histories of these strata do not exceed the present effective overburden load, resulting in the occurrence of problems regarding settlement behavior of structures upon them.

Bjerrum[1,2] turned his attention to the phenomenon that the preconsolidation pressure, p_c, is sometimes larger than the effective overburden pressure, p_o, due to the time effect, even in normally consolidated clay. He called such characteristics of the soil under delayed consolidation the precompression effect (p_c-effect). The p_c-effect increases in proportion with p_o, and the ratio p_c/p_o of a layer in the same sedimental age increases with the amount of secondary compression occurring under the actual surcharge. Since the secondary compression increases with the plasticity of clay, it is known that the ratio p_c/p_o increases with the plasticity index, I_p.

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Thompson[3] reported the experimental results on Cambridge Gault clay, measuring the lateral pressure change in K_0-consolidation throughout the primary and secondary compressions. The lateral pressure (total stress) during the primary consolidation decreased linearly with the degree of consolidation until the final K_0-value was reached. During the secondary compression, the lateral pressure increased remarkably at first, and thereafter kept a constant value. A similar research was performed by Tan[4], using the gauge oedometer under various stepwise loadings. His test results were summarized by three types of changes in the K_0-value during the secondary compression.

Recently, Schmertmann[5] presented an interesting note on the stress change during delayed consolidation of normally consolidated cohesive soils. In answer to the question in his survey about the variation in the coefficient of earth pressure at rest, $K_0=\sigma'_3/\sigma'_1$, as shown in Table 1, approximately one half of the responses said " increase ", one quarter " remains the same ", and one eighth each for " decrease " and " don't know ". (The total responses amounted to 32 from 40 geotechnical engineers.)

In this paper, the deformation and strength behaviors of soil under delayed consolidation are investigated through the concept of state boundary surface, with respect to quasi-overconsolidation characteristics. The mechanism of change in state during delayed consolidation is made clear, considering the restrictive conditions in terms of stress and strain for K_0-secondary compression. Moreover, a brief discussion on the mechanical properties of soil after delayed consolidation is made, using the results of the K_0-triaxial loading test and undrained shear test for Kaolin.

Table 1. Summary of response to question: Will $K_0=\sigma_3'/\sigma_1'$ of a normally consolidated cohesive soil increase or decrease during secondary aging in 1-D compression ?[5]

Categories	Number of Responses				
	Increase	Same	Decrease	Don't know	Total
Written	11	8	2	2	23
Verbal	5	1	2	1	9
	16	9	4	3	32
USA	11	3	4	2	20
Canada	2	4	0	0	6
Europe	3	2	0	1	6
Research/teaching	11	6	2	1	20
Consulting/practice	5	3	2	2	12
Estimated age>50	9	6	2	1	18
Estimated age≤50	7	3	2	2	14

2. State Boundary Surface and State Path

In order to investigate the state of soil during a secondary compression, let us express the deformation behavior of soil under delayed consolidation by using the concept of state boundary surface (Fig. 1)[6].

During a secondary compression, the initial void ratio, e_b, shown at point B_3 on the virgin compression line (K_0^{NC}), which corresponds to the state of normal consolidation under the present effective overburden pressure, has gradually decreased

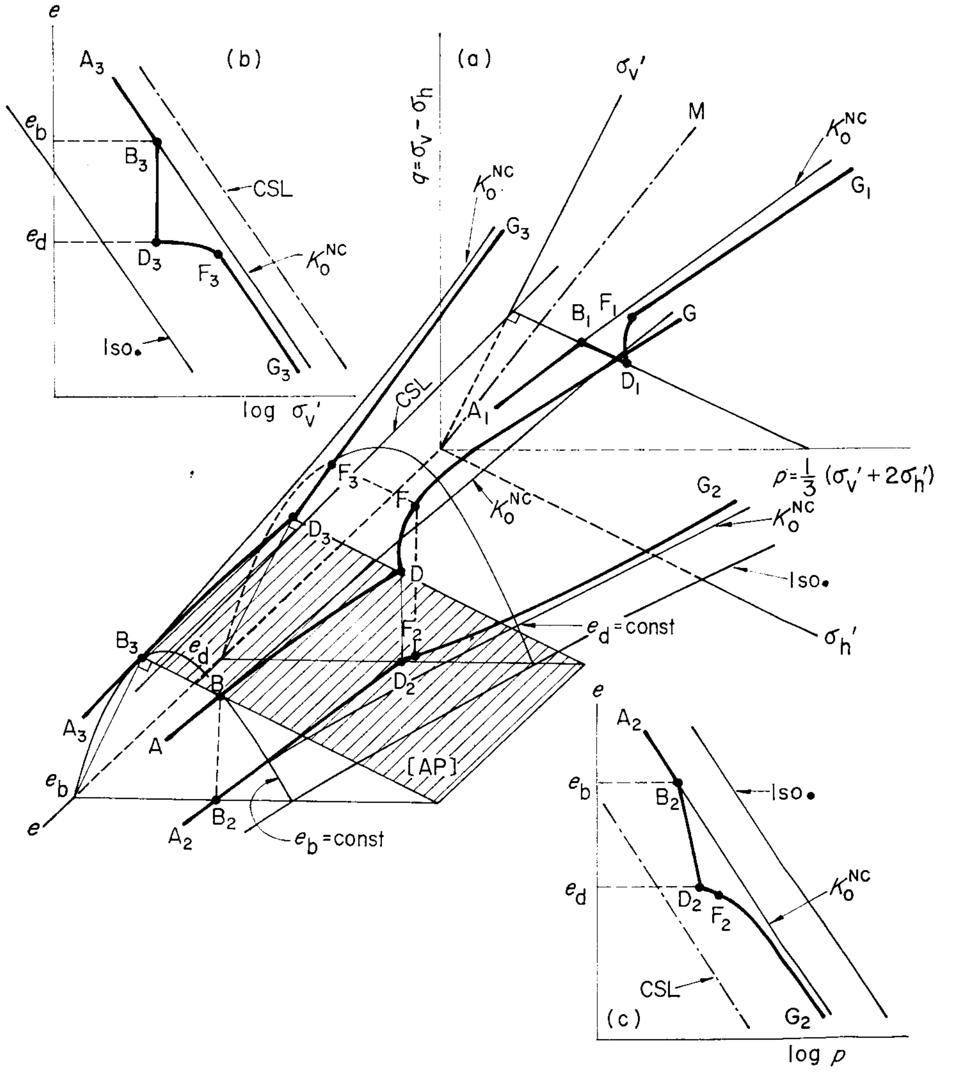

Fig. 1. State path in the e-p-q space and their projections for quasi-overconsolidated soil.

with time until e_d at point D_3. (See Fig. 1 (b).)

Fig. 1 (a) indicates the state path of soil under delayed consolidation in the pore-effective stress space (e-p-q space). Here, we introduce the σ_v'-axis, which is the major effective principal stress axis, beside the epq-axes, and utilize the process shown in Fig. 1 (b) where the path B_3D_3 during the secondary compression on the e-σ_v' plane is parallel to the e-axis. In Fig. 1 (a), the state path starts from point B on the state boundary surface (SBS) and moves into SBS on inclined plane (so-called aging plane [AP]). This plane is perpendicular to both e-σ_v' plane and p-q plane, as explained elsewhere[7].

3. Restrictive Conditions for K_0-Secondary Compression

In Fig. 1 (a) and (c), the state path during the secondary compression is conveniently drawn in such a manner that it starts from point B and moves on the aging planes [AP], so as to increase p and decrease q. In other words, the state path goes down on [AP]. We can consider another path, for instance, going up on [AP], or going parallel to the horizontal e-axis on [AP]. With all these paths, once projected on the e-σ_v' plane, we get a unique line B_3D_3 which is parallel to e-axis in Fig. 1 (b).

Which way should we take ? This is the main subject of the question proposed by Schmertmann[5].

Here, let us consider some conditions which are necessary to solve this problem. The restrictive conditions for the K_0-secondary compression are summarized by the following two parts.

A. Stress Condition: The stress condition during a K_0-secondary compression is only $\sigma_v=\sigma_v'=\text{const}$. This requires that the state path must move on the aging plane [AP]. The manner of moving on [AP] cannot be determined from this stress condition.

B. Strain Condition: In general, the axial strain $\varepsilon_v(=\varepsilon_1)$ of a cylindrical soil element is written as

$$\varepsilon_v=\frac{v}{3}+\frac{2}{3}(\varepsilon_v-\varepsilon_h)=\frac{v}{3}+\varepsilon \tag{1}$$

where, v, $v/3$ and ε denote the volumetric strain, the mean strain and the deviatoric strain, respectively.

The K_0-condition means no lateral strain $\varepsilon_h(=\varepsilon_3)$ during consolidation. Under this condition,

$$v=\varepsilon_v+2\varepsilon_h=\varepsilon_v+0=\varepsilon_v \tag{2}$$

Though the axial strain ε_v equal to the volumetric strain v in this case, it does not mean, of course, that there is no deviatoric strain ε under the K_0-condition.

Dividing ε_v into the strain component, the ratio of $v/3$ to $\varepsilon=2/3\cdot\varepsilon_v=2/3\cdot v$ is

$$v/3:\varepsilon=1:2 \tag{3}$$

Thus, the strain condition requires that the deviatoric strain ε must be as large as two-thirds of the volumetric strain v (=axial strain ε_v), at any time during consolidation (without regard to the primary and secondary compressions).

Directly, the mean strain $v/3$ is caused by the mean effective stress p while the deviatoric strain ε is caused by the principal stress difference $q=\sigma_v-\sigma_h$. Furthermore, we cannot neglect the volume change due to the deviatoric stress q. In conclusion, the correspondence between stress and strain is:

$$\left.\begin{tikzcd} p \arrow[r] & v/3 \\ q \arrow[r] \arrow[ur, dashed] & \varepsilon \end{tikzcd}\right\} \varepsilon_v\ (\Delta e) \tag{4}$$

where, the solid arrows mean " directly " and the dashed arrow means " indirectly ". Among them, it is supposed that only the correlation between p and $v/3$ does not have the time lag (*i. e.*, inviscid).

4. Drained Creep of K_0-Normally Consolidated Clay

As the first step to investigate the mechanism of a secondary compression of clay, a series of drained creep tests have been performed by means of a triaxial apparatus, taking Kaolin as the soil sample. The main physical properties of this Kaolin are $G_s=2.62$, $LL=89.3\%$ and $PI=57.6\%$.

The test procedure is as follows:

1) The specimen (5 cm in diameter and 10 cm in height) is allowed to consolidate anisotropically with lateral drainage under the effective stress ratio $\eta=q/p=0.706$, which corresponds to the coefficient of earth pressure at rest, $K_0^{NC}=0.52$. The final consolidation pressure is $\sigma_v=300$ kPa.

2) After the primary consolidation is over, the stress state is maintained as before. Thus, the vertical effective pressure $\sigma_v'=300$ kPa and the horizontal effective pressure $\sigma_h'=300\times0.52=156$ kPa. Under this condition, the sample is brought to the drained creep for about 7 weeks.

3) During the above mentioned two stages, the measured values are the axial strain, ε_v, the volumetric strain, v, and the excess porewater pressure, u. From the data of ε_v and v, one can calculate the mean strain, $v/3$, the deviatoric strain, ε $(=\varepsilon_v-v/3)$, and the horizontal strain, ε_h.

Fig. 2 indicates the test results. From the manner of pore pressure dissipation, it is known that the primary K_0-consolidation is arrived at $t_{100}\fallingdotseq 90$ min. During this period, the K_0-condition is kept well, because there exists no change in the lateral strain. During $t>t_{100}$, on the other hand, the horizontal strain, ε_h, tends to increase in expansion, that is, the soil specimen deforms under a non-K_0-condition

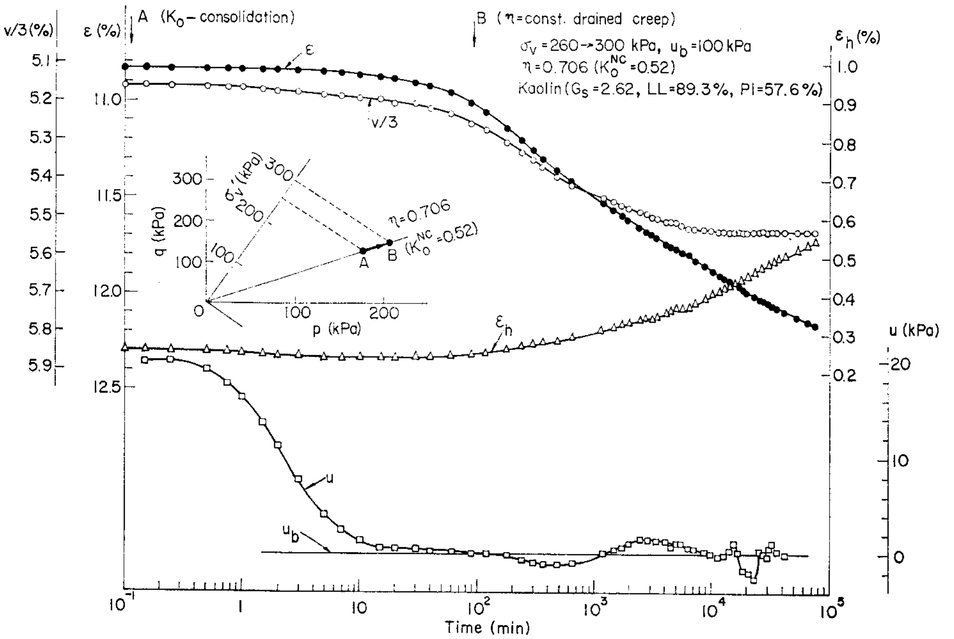

Fig. 2. K_0-consolidated-drained creep test (p, q : const.).

after the end of the primary consolidation, while both the mean strain, $v/3$, and the deviatoric strain, ε, continue to increase. The ratio of latter two is $v/3:\varepsilon=1:2$ during the K_0-consolidation, as explained theoretically before.

This ratio decreases gradually with time during the drained creep with the constant effective stress ratio, η. The deviatoric strain, ε, increases approximately in proportion to the logarithm of time for a long period, whereas the mean strain, $v/3$, ceases the incremental behavior when $t \fallingdotseq 10^4$min is reached. It can be concluded, therefore, that the time-dependency in the deformation of Koalin is superior in ε than in $v/3$.

5. Change in State of Soil during K_0-Delayed Consolidation

During a primary consolidation, it is well known that the mean effective stress p and the principal stress difference q increase gradually as the excess porewater pressure dissipates. At the end of a primary consolidation, these stresses reach their final values[8].

Suppose at first that the state of stress would not change at all during the K_0-delayed consolidation. This corresponds to the stress condition in the drained creep test mentioned before. Then, the response of strains is considered as follows:

1) Since p=const., the mean strain which can be assumed as inviscid $v/3=0$.
2) Though q=const., there occurs a deviatoric strain $\varepsilon(t)$ with a time lag,

called the viscid creep.

3) Though q=const. again, there occurs a mean strain $v(t)/3$ with a time lag, called the viscid diliatancy.

Owing to the restrictive condition in terms of strain in the K_0-condition, the ratio of the mean strain to the deviatoric strain should be $v(t)/3 : \varepsilon(t) = 1 : 2$ at any time during the delayed consolidation. If the former is less than half of the latter, just as in our test for Kaolin before mentioned, an increase in p is needed to compensate the shortage of the mean strain. In addition, a decrease in q is accompanied at the same time, owing to the restrictive condition in terms of stress; σ_v'= const. On the other hand, provided the former is larger than half of the latter, a decrease in p accompanied with an increase in q should occur.

Thus, we have possibilities for three cases of change in the K_0-value (increase, remain the same, and decrease) during delayed consolidation of a normally consolidated cohesive soil. It depends on the physical properties of soil under compression; namely, the creep and dilatancy characteristics, both of which indicate a high time-dependency. This is the answer to the question proposed by Schmertmann[5].

Fig. 3 shows the axial strain-time curve as well as the change in lateral pressure in the long-term K_0-consolidation test of Kaolin for about 7 weeks. The test was performed in a K_0-triaxial cell, controlling axial loading (σ_v=140→300 kPa) and zero lateral strain of the test specimen during consolidation. It is found from this figure that coefficient of earth pressure at rest, K_0^{NC}=0.52, during the primary consolidation increases gradually in the period of the secondary compression, reaching the final value of K_0^{QOC}=0.77. The coefficient of the secondary compression can be divided into two phases, corresponding to the manner of variation in the K_0-value

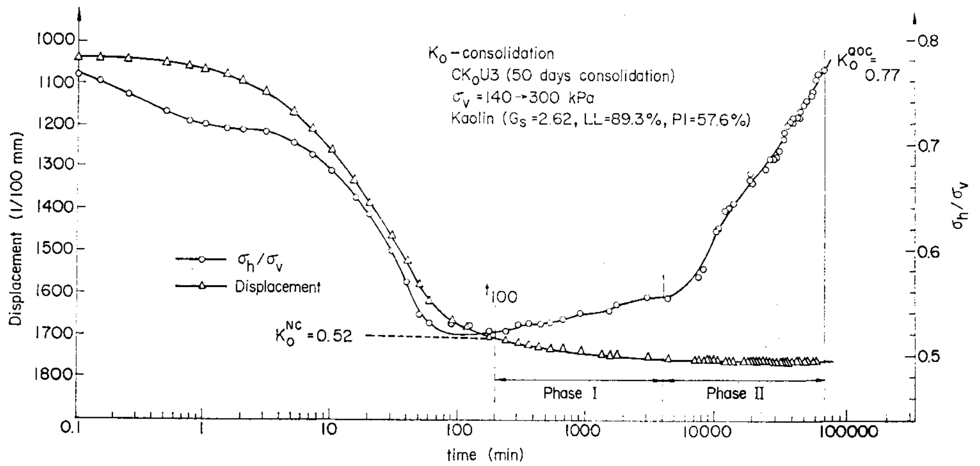

Fig. 3. Axial strain and lateral pressure behavior during K_0-consolidation.

during the long-term consolidation. After a very long time, the consolidation-time curve would turn more concave upwards and the secondary compression would tend to finish.

6. Mechanical Properties of Soil after Delayed Consolidation

6.1. K_0-triaxial loading test

The deformation behavior after loading on a soil under delayed consolidation is also of great interest, because the amount of settlement caused in young diluvial clay is now in a spot of light in analyzing the behavior of reclaimed on-shore or off-shore land.

Fig. 4 indicates the effective stress path and e-log σ_v' curve in the K_0-triaxial loading test for Kaolin aged for one week under the vertical effective stress $\sigma_v'=300$ kPa. At the first loading ($\sigma_v=300\rightarrow320$ kPa), one can recognize a "locking" response where the stress path goes on, maintaining the constant effective mean stress p. (See Fig. 4(a).) Owing to the confining condition of K_0-deformation, the deviatoric strain ε and the vertical strain ε_v (=the volumetric strain v) hardly occur, resulting in almost no decrease in the void ratio e (Fig. 4 (b)). By succeeding increments of loading, the stress path approaches gradually the K_0^{NC}-line ($K_0^{NC}=0.52$), reaching a rather high K_0-value in this experiment. The general feature of the state path DFG after the K_0-loading is additionally depicted in Fig. 1.

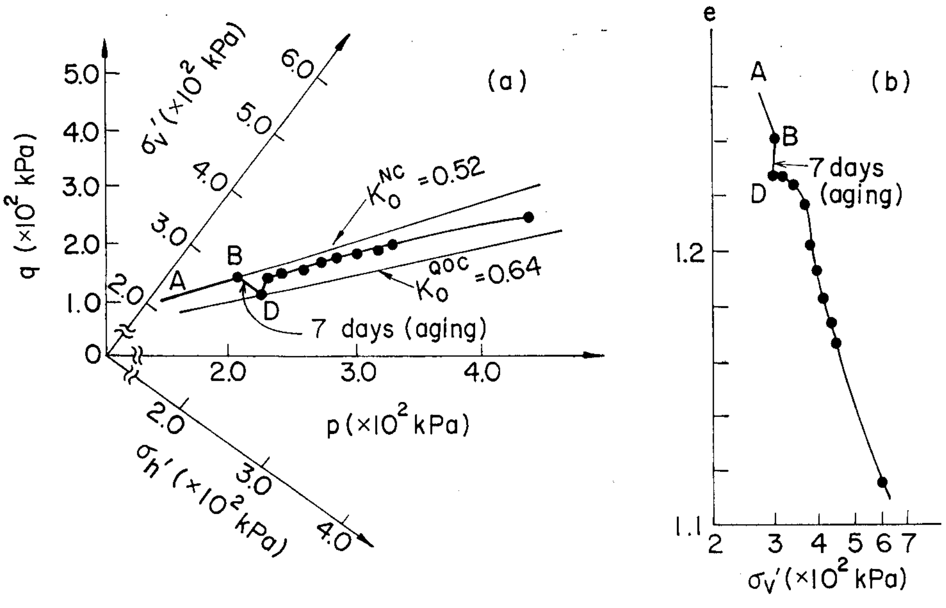

Fig. 4. Effective stress path and e-log σ_v' curve in K_0-triaxial loading test after aging.

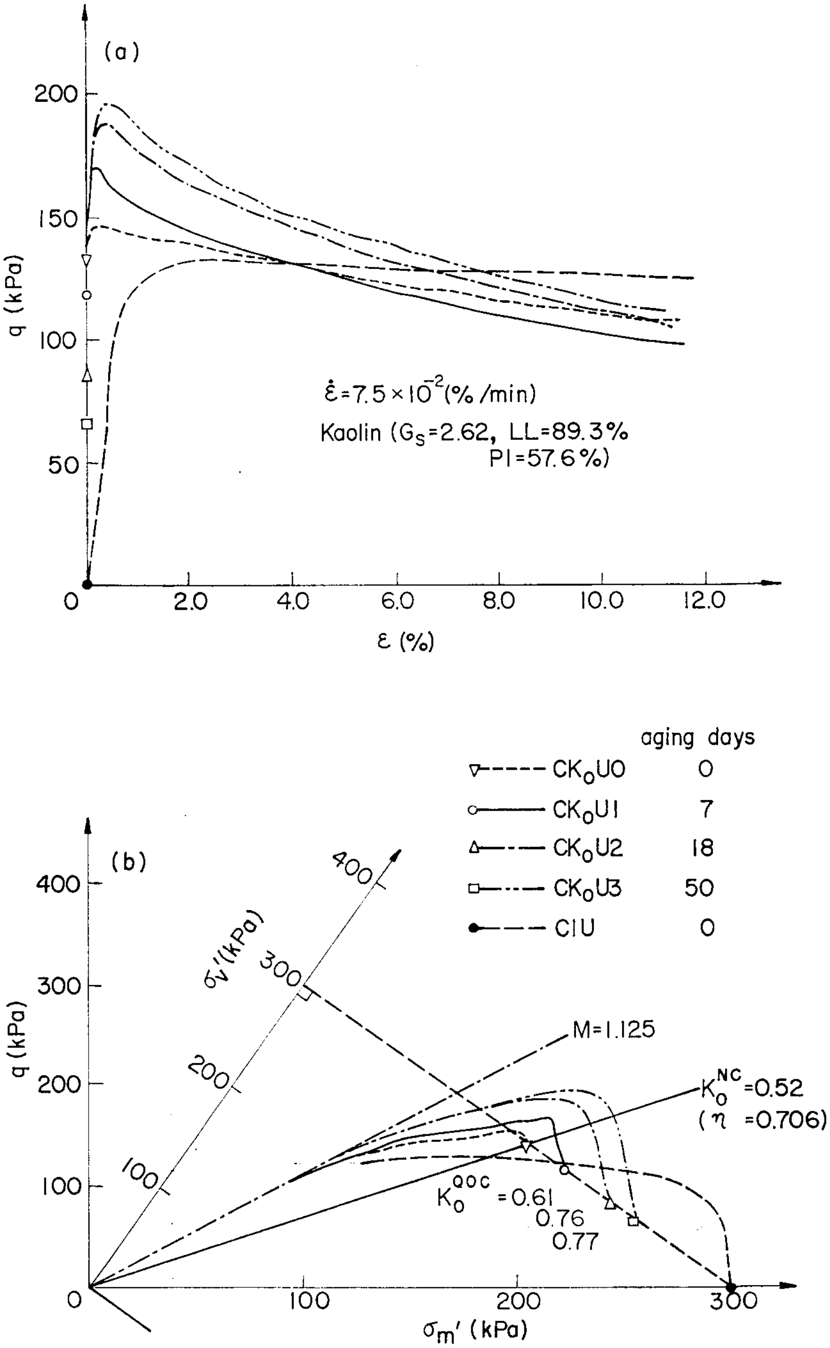

Fig. 5. Stress-strain curves and stress paths in a series of undrained shear tests after aging.

6.2. Undrained shear test (rate of strain $\dot{\varepsilon}=7.5\times10^{-4}$/min)

As explained in Fig. 1 (a), the state path during delayed consolidation starts from point B on the state boundary surface (SBS) and moves into SBS on the aging plane [AP] to the present point D. It can be recognized, therefore, that the soil under delayed consolidation has an elastic component with respect to deformation, likely as an overconsolidated soil.

Fig. 5 (a) shows the stress-strain curves in a series of undrained shear tests of such a soil aged for various periods under K_0-consolidation ($\sigma_v'=300$ kPa). For reference, the curve of isotropically consolidated clay, of course without aging, is also drawn. In Fig. 5 (b) are shown the corresponding stress paths during undrained shear.

From these figures, we can recognize the so-called quasi-overconsolidation behavior of soil after delayed consolidation. The longer the aging period is, the more the peak strength of clay increases. At the large strain, however, the stress-strain curves as well as the stress paths converge, which means that the residual strength of aged clay has a unique value independent of the period of aging.

7. Conclusion

From a site investigation on the overconsolidation characteristics of marine clay layers under the seabed of Osaka Bay, it has been known that the upper diluviums correspond to the soil under delayed consolidation where $\mathrm{OCR}=p_c/p_0$ is a little larger than unity. It should be noted that an increase in the horizontal stress σ_h during delayed consolidation would result in an increase of the K_0-value and OCR. The degree of increase depends on the physical properties of soil under compression; namely, the creep and dilatancy characteristics, both of which indicate a high time-dependency.

Using the concept of the state boundary surface, it is shown that such soil has an elastic component with respect to deformation. The feature of the state path during both consolidation and loading is investigated, based on the result of a series of long-term consolidation test, loading test and undrained shear test performed by a newly designed K_0-triaxial apparatus.

Acknowledgments

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