運輸省港湾技術研究所

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港湾技術研究所報告　(REPORT OF P.H.R.I.)

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REPORT OF THE PORT AND
HARBOUR RESARCH INSTITUTE
Vol. 7 No. 1 (Mar. 1968)

1. The Effect of Overconsolidation on the Undrained Strength of Clays

Akio NAKASE*

Synopsis

In order to examine the possibility of applying the $\phi_u=0$ analysis to the long-term stability problem, experimental studies on the strength of overconsolidated clays were performed. Triaxial compression tests were conducted on four saturated marine clays, in which the specimen was allowed to rebound under an isotropic or anisotropic confining pressure before shear.

In spite of a considerable difference in index properties, three undisturbed clays showed similar patterns of strength reduction due to the isotropic rebound. The stress anisotropy was found to have some influence on the strength reduction due to rebound. It was also shown that the secant modulus of stress-strain curve decreases with the overconsolidation ratio.

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港湾技術研究所報告
第7巻第1号（1968年3月）

1. 粘土の非排水セン断強さに及ぼす過圧密の影響

中　瀬　明　男*

要　旨

大規模な掘削の場合のような長期安定問題に $\phi_u=0$ 法を適用する可能性を調べるため，過圧密粘土のセン断試験を行なった。四種類の飽和した海成粘土の三軸圧縮試験において，セン断前に等方または異方圧力のもとで供試体の膨張を許して過圧密試料を調整した。

実験の結果，三種類の乱さない粘土は，分類特性がかなり異なるにもかかわらず，膨張による強度減少の特性が類似していた。また，膨張時の拘束圧力の異方性が，この強度減少の特性に影響を及ぼすことが分かった。応力ヒズミ曲線の正割係数の値は，過圧密比の増大と共に減少する。

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1. Introduction

In construction works on soft clays, the preloading method is often employed to increase soil strength. In some cases, however, part of the consolidation load has to be removed after consolidation is completed. This situation is often encountered in harbour works, where the assigned waterfront depth must always be maintained.

In general, a decrease in confining pressure leads to swelling or rebound of soils, hence the reduction in strength. In the above situation, therefore, the stability of the earth structure would decrease with time as the rebound proceeds. The stability analysis required in such a case is of long-term type, and the $c'-\phi'$ method has been recommended (Bishop and Bjerrum, 1960). The reason why the $\phi_u=0$ analysis has not been applied to such a case is an ambiguity in estimating the long-term value of the undrained strength c_u. Experimental studies to data on the strength of overconsolidated clays (Henkel, 1960; Simons, 1960) are mostly in terms of effective stress.

In harbour works in Japan, the $\phi_u=0$ analysis has been successfully used in practice, since most important ports are underlain by normally-consolidated clays and the extent of excavation or load reduction has been relatively small (Nakase, 1967). The $\phi_u=0$ analysis is simple in practice, and there is no reason to denounce its use for the long-term stability problem if the long-term value of undrained strength is estimated reasonably. The main subject of the present paper, therefore, is the strength of overconsolidated clays in terms of total stress.

Table—1 Index properties of clays

	w_L (%)	w_P (%)	I_P	Clay fraction <2 micron (%)	Activity
Yokohama clay (Remoulded)	112	43	69	35	1.98
Nagoya clay (Undisturbed)	64	33	31	25	1.24
Kuji clay (Undisturbed)	95	41	54	28	1.93
Kobe clay (Undisturbed)	88	36	52	37	1.41

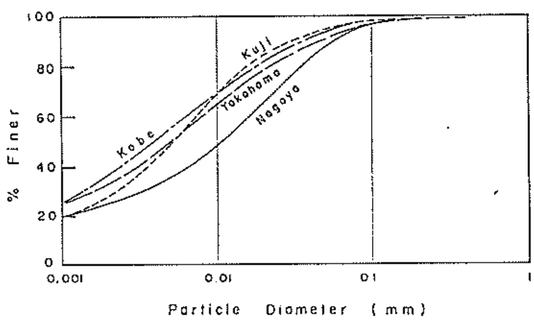

Fig. 1 Grain size distribution curves

2. Soils

Four types of saturated marine clays were used for the test. The Nagoya, Kuji and Kobe clays were tested as undisturbed samples. In the case of Yokohama clay, on the other hand, an excavated sample was thoroughly remoulded with a big meat grinder, and cylindrical specimens, 20 cm in both diameter and height, were prepared by consolidation under all-round pressure of 0.5 kg/cm^2. The remoulded clay slurry and reconsolidated specimens were stored in a fresh seawater tank. Index properties of the clays are listed in Table 1, and grain size distribution curves are shown in Fig. 1.

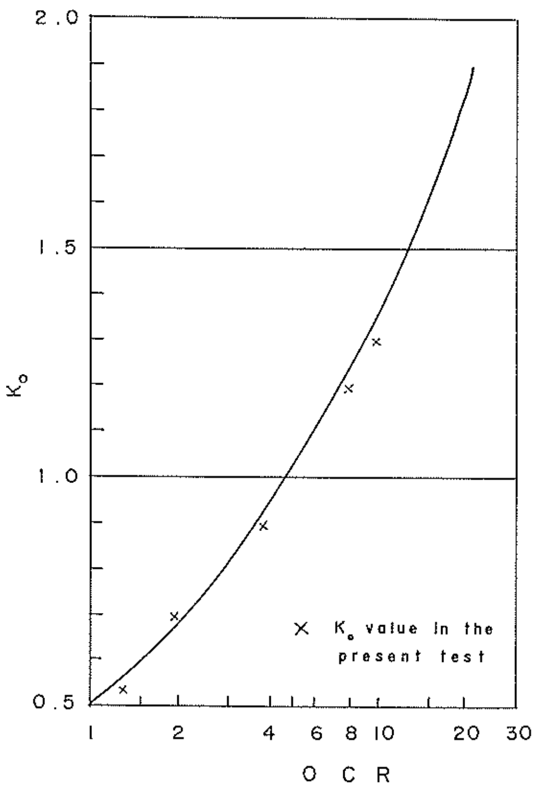

Fig. 2 Coefficient of earth pressure at rest K_0 and overconsolidation ratio OCR

3. Test Procedures

Three types of triaxial compression tests were carried out. In the CIU test, the specimen was consolidated under isotropic pressure and then sheared in the undrained condition. In the CIRIU test, the specimen was first consolidated under isotropic pressure, allowed to rebound under reduced isotropic pressure, and then subjected to undrained shear. In the CIRAU test, the specimen was isotropically consolidated, allowed to rebound under reduced anisotropic pressures, and subjected to undrained shear.

The CIRAU test was carried out in order to reproduce natural ground conditions. It has been recognized that the stress in the ground is not isotropic, and that the principal stress ratio (or the coefficient of earth pressure at rest K_0)

Table—2 Pressures and coefficient of earth pressure at rest K_0 in anisotropic rebound

Axial pressure σ_a (kg/cm^2)	2.00	1.50	1.00	0.50	0.25	0.20	0.10
Radial pressure σ_r (kg/cm^2)	2.00	0.80	0.70	0.45	0.30	0.26	0.18
Mean principal stress J (kg/cm^2)	2.00	1.03	0.80	0.47	0.28	0.24	0.15
Overconsolidation ratio in terms of J	1.00	1.94	2.50	4.28	7.07	8.33	13.07
Overconsolidation ratio in terms of σ_a	1.00	1.33	2.00	4.00	18.00	10.00	20.00
Coefficient of earth pressure at rest K_0	0.50	0.53	0.70	0.90	1.20	1.30	1.80

increases with the overconsolidation ratio (Skempton, 1961). The overconsolidation ratio OCR will be defined here as the ratio of the mean principal stress in a normally-consolidated state to that in the subsequent overconsolidated state. In the CIRIU test, the overconsolidation ratio can be expressed as the ratio of the axial pressures.

K_0 values for the CIRAU test can be obtained from the rebound test under condition of no lateral strain, i.e. the K_0 condition. At the time of the experiment, however, the measurement of K_0 values had not been made for the above

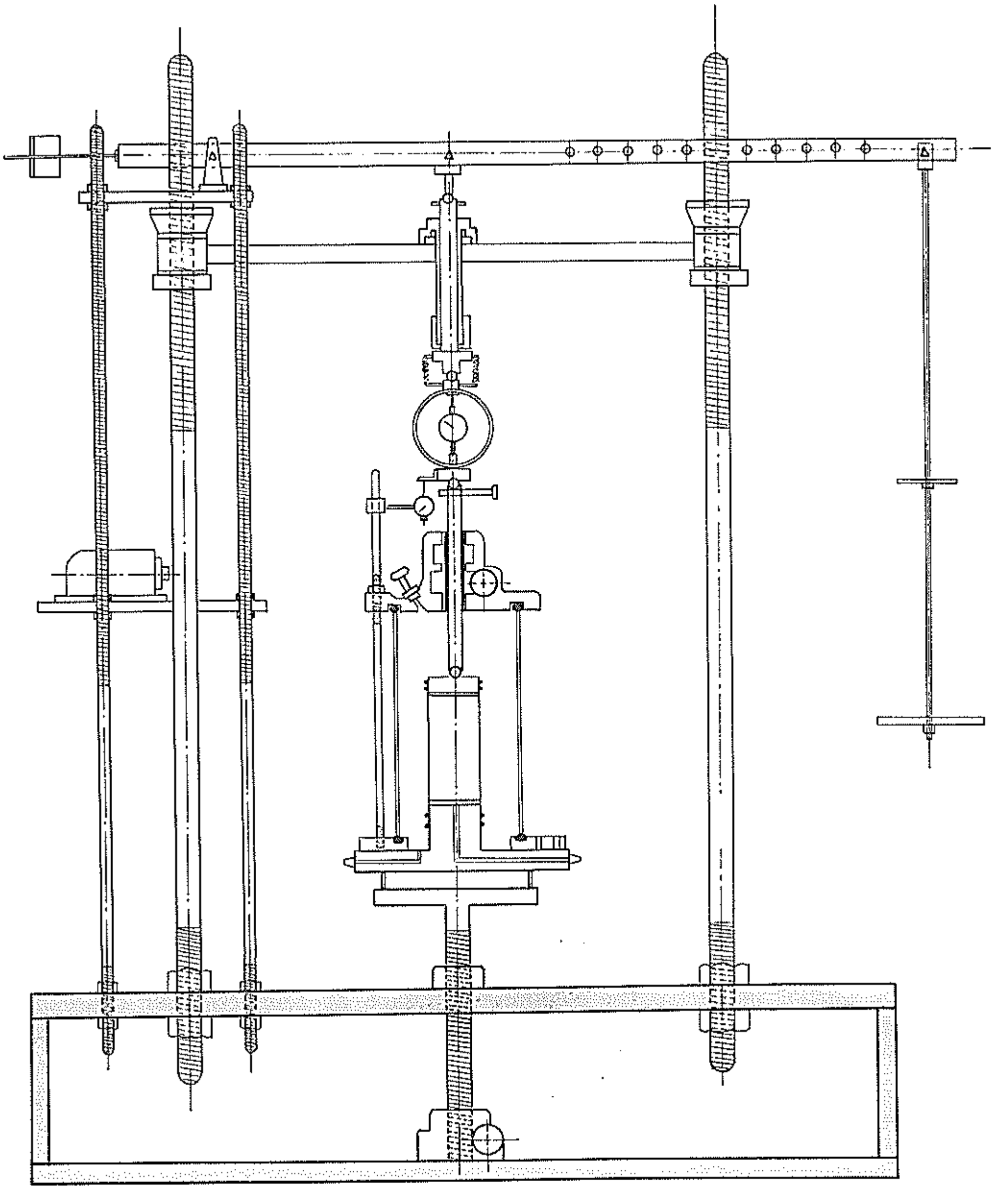

Fig. 3(a) Loading device

clays. Therefore, the K_0 data for the present test were taken as an average of values reported to date (Ladd, 1965). Fig. 2 shows a relationship between average value of K_0 and overconsolidation ratio in terms of the effective overburden pressure. Values of K_0 and of the axial and radial pressure employed in the CIRAU test are listed in Table 2.

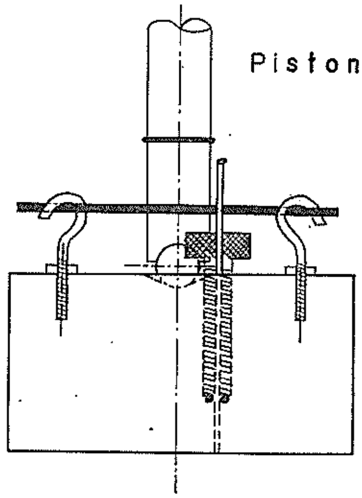

Fig. 3(b) Loading device

Triaxial compression apparatus used was similar to that described by Bishop and Henkel (1957). Where K_0 exceeds unity in the anisotropic rebound, the axial pressure was reduced by pulling up the upper cap of the specimen. The loading device for the anisotropic rebound is shown in Fig. 3.

Undisturbed clays were subjected to the CIU and CIRIU tests, using all-round consolidation pressure exceeding the in-situ effective overburden pressure by at least 50%. The Yokohama clay was tested by the CIRIU and CIRAU methods, with all specimens consolidated under all-round pressure of 2 kg/cm^2.

The cylindrical specimens, 35 mm in diameter and 80 mm in height, were lined vertically with filter-paper strips in order to accelerate consolidation and rebound. The rate of axial strain in compression was 0.06 %/min for the undisturbed clays and 0.1 %/min for the Yokohama clay respectively. Pore pressure was measured at the bottom of the specimen.

The applied pressures, pore pressure and volume change patterns in the CIRIU and CIRAU tests are shown schematically in Fig. 4.

4. Test Results

4.1 Consolidation and rebound processes

In both consolidation and rebound processes, the assigned axial and radial pressures were kept constant for 24 hours. Completion of both processes were established by complete dissipation of the pore pressure. Compared in the initial stage of the process, the rate of rebound was lower than that of consolidation, however, the time required for complete rebound was shorter than that for consolidation. Examples of pore pressure dissipation with time are shown in Fig. 5. No substantial difference in the pore pressure dissipation pattern was found between the isotropic and anisotropic rebounds.

On the partial release of the confining pressure at completion of consolidation, negative pore pressure was developed in the specimen. The negative pore pressure at the time of pressure release is due to the tendency of soil to swell, and may serve as measure of the elastic properties of the soil skeleton. The development of this negative pore pressure may be reflected on the volume change behaviour in the subsequent rebound process. Fig. 6 shows relationship between the negative pore pressure and the overconsolidation ratio. In Fig. 7 the ratio of volume change in the rebound process to that in the preceding consolidation

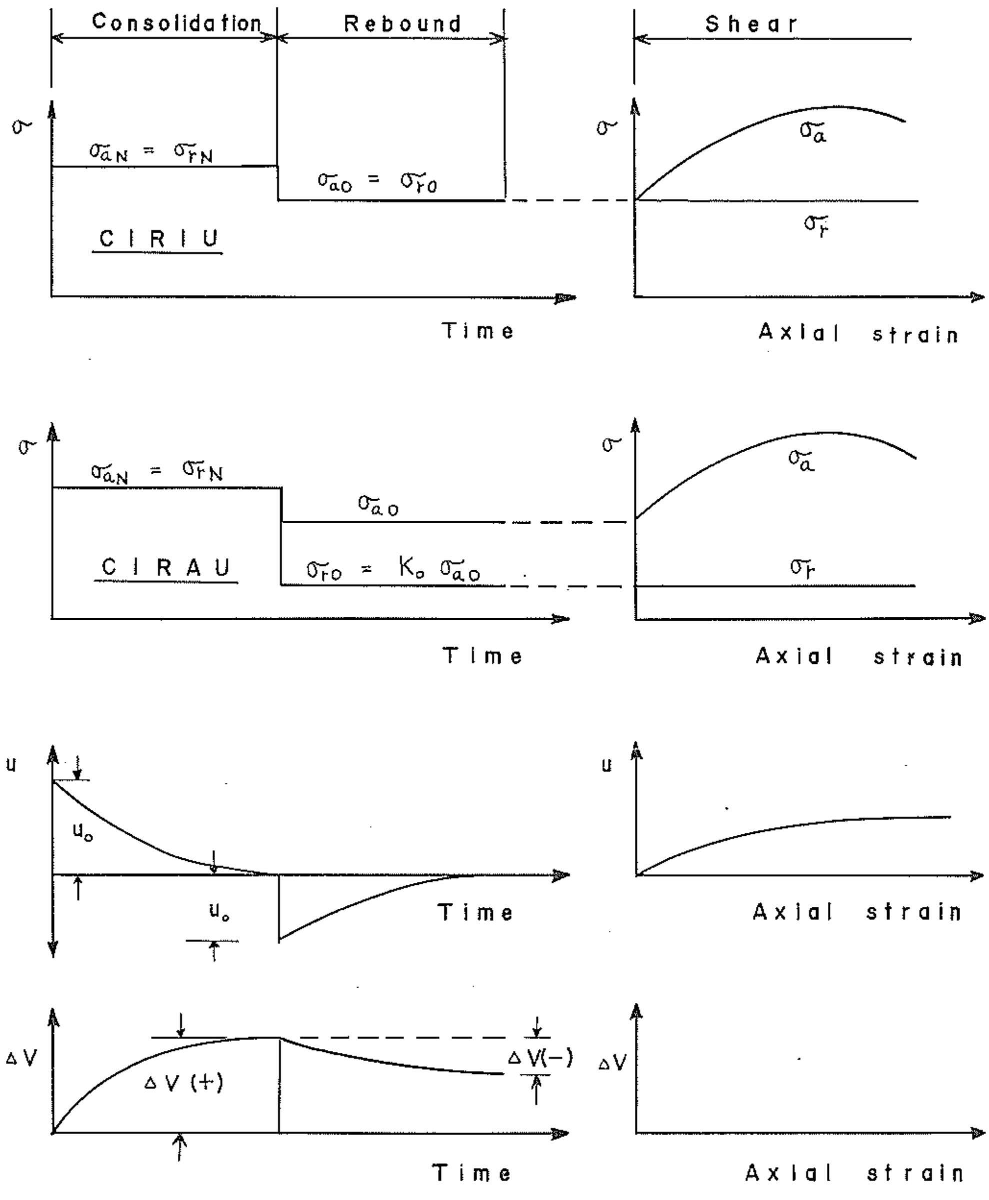

Fig. 4 Pressure and volume change in consolidation, rebound and shear processes (schematised)

process is plotted against the overconsolidation ratio. As seen in these figures, both the negative pore pressure and volume change ratio are, to some extent, in proportion to logarithm of the overconsolidation ratio. As far as the present test data are concerned, the effect of stress anisotropy can not be found in both the negative pore pressure and volume change behaviours.

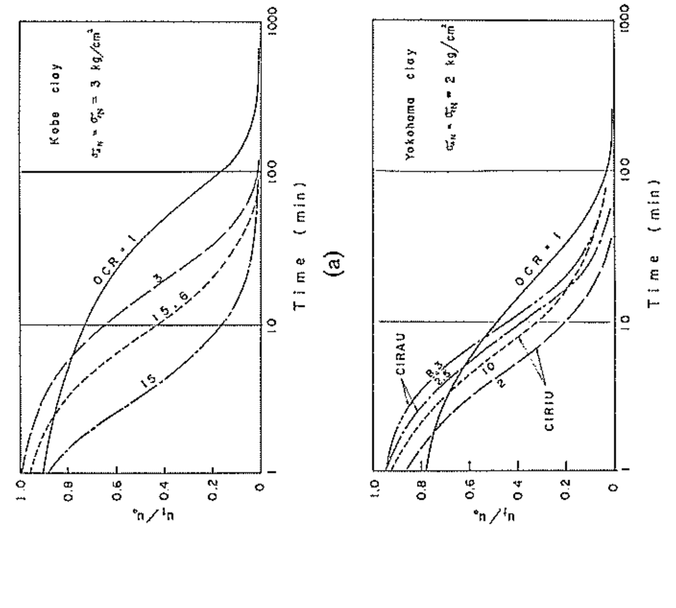

Fig. 5 Pore pressure ratio and time

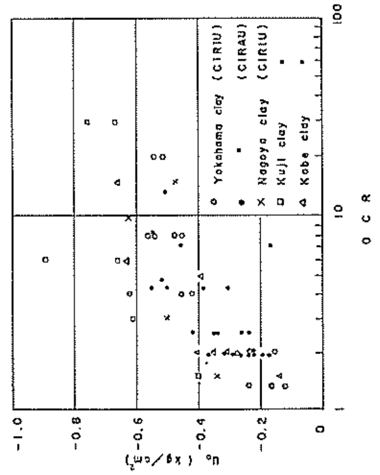

Fig. 6 Initial negative pore pressure and overconsolidation ratio

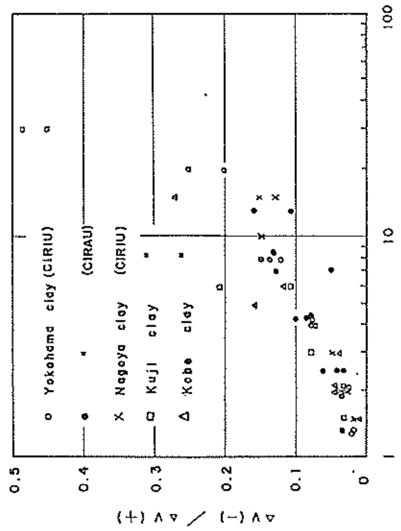

Fig. 7 Volume change ratio and overconsolidation ration

4.2 Undrained shear process

In the undrained shear process the failure criterion of maximum principal stress difference was used. In most cases, the maximum principal stress difference appeared at the axial strain of 8% to 12%, and any marked drop in stress was not observed after the peak had been reached. In the case where the confining pressure was reduced to zero in the rebound process, however, the stress-strain curve was very much similar to that in an ordinary unconfined compression test, showing marked drop in stress after the peak had been reached. Results of triaxial compression tests on the four clays are shown in Figs. 8 through 20.

Figs. 8, 11, 14, 17 and 18 show the plot of $1/2\cdot(\sigma_a'+\sigma_r')$ against $1/2\cdot(\sigma_a'-\sigma_r')$, where σ_a' and σ_r' are the axial and radial effective stresses at failure respectively. $1/2\cdot(\sigma_a'-\sigma_r')$ is the undrained strength c_u. Each plot in these figures corresponds to the top point of the Mohr's effective stress circle at failure. If a and α are the cohesion intercept and the slope angle for the straight line drawn through such points, the effective strength parameters c' and ϕ' are expressed

$$\sin\phi'=\tan\alpha \quad \text{and} \quad c'=a/\cos\phi'$$

It has been known that for normally-consolidated clays $c'=0$, but for overconsolidated clays the Mohr's failure envelopes do not in general pass throgh the origin, and c' and ϕ' are required to define these lines. In the present tests, this situation was found on the Kuji and Yokohama clays. For other clays, however, the test results on overconsolidated specimen were not enough to define a different failure envelope. As seen in Fig. 18, the effective strength parameters seem to be independent on the principal stress ratio in the rebound process.

Relationships between the undrained strength c_u and the mean principal stress J in the consolidation and rebound processes are shown in Figs. 9, 12, 15 and 19. In the figures, J_N and J_0 are the mean principal stresses in the consolidation and rebound processes respectively. In the CIU and CIRIU tests, the mean principal stress is equal to the confining pressure. As seen in the figures, the undrained strength of normally-consolidated clay is proportional to the consolidation pressure. In the case of overconsolidated clays, on the other hand, this relationship is not linear. Fig. 19 indicates that the relationship between the undrained strength and mean principal stress on overconsolidated clays is influenced by the principal stress ratio.

Pore pressure set up during undrained shear was largely dependent on the overconsolidation ratio. Figs. 10, 13, 16 and 20 show the change in the pore pressure coefficient at failure A_f (Skempton, 1954) with the overconsolidation ratio. These figures indicate that the pore pressure at failure would be close to zero in the overconsolidated specimen of $OCR \fallingdotseq 6$. In the present tests, the radial pressure in the shear process was kept equal to that in the preceding consolidation or rebound processes as shown in Fig. 4. Then the pore pressure coefficient at failure A_f is determined that

$$A_f=\Delta u/\Delta\sigma_{a\,\max}$$

As is seen in Fig. 20, the relationship between A_f and the overconsolidation ratio is influenced by the stress anisotropy. If the overconsolidation ratio is ex-

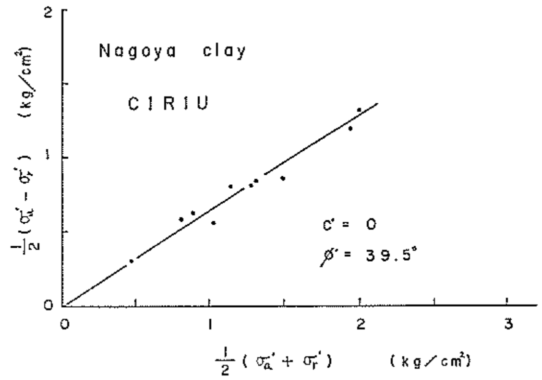

Fig. 8 $1/2\cdot(\sigma_a'+\sigma_r')$ and $1/2\cdot(\sigma_a'-\sigma_r')$ at failure on Nagoya clay

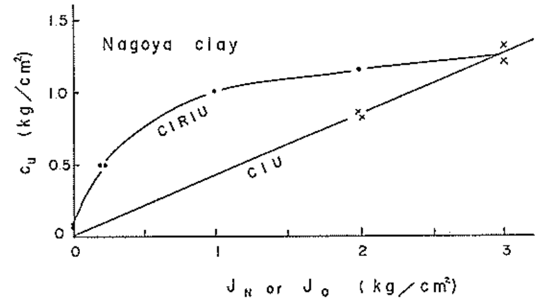

Fig. 9 Undrained strength and mean principal stress on Nagoya clay

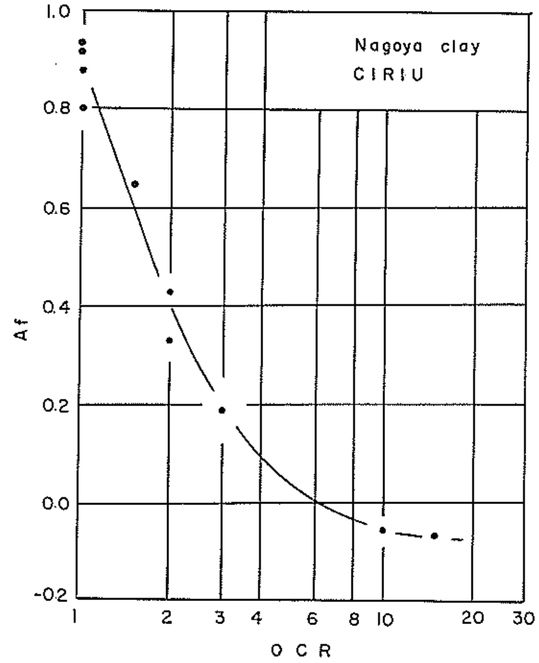

Fig. 10 Pore pressure coefficient at failure and overconsolidation ratio on Nagoya clay

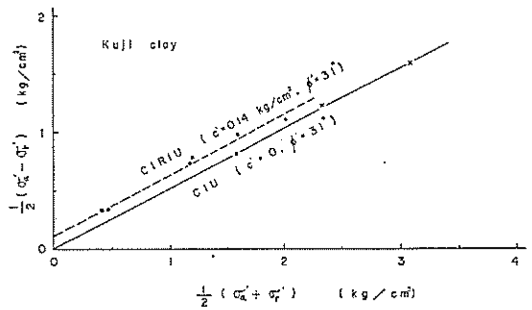

Fig. 11 $1/2 \cdot (\sigma_a' + \sigma_r')$ and $1/2 \cdot (\sigma_a' - \sigma_r')$ at failure on Kuji clay

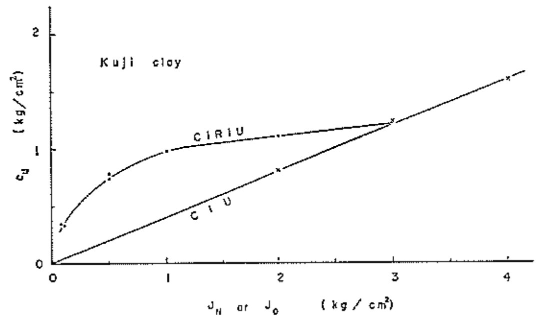

Fig. 12 Undrained strength and mean principal stress on Kuji clay

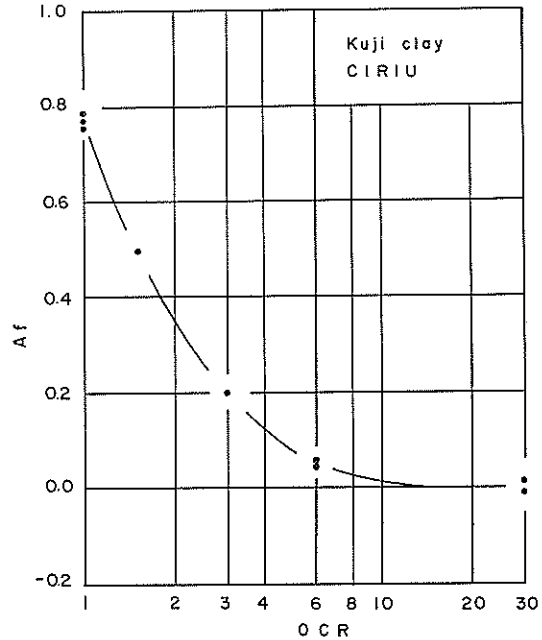

Fig. 13 Pore pressure coefficient at failure and overconsolidation ratio on Kuji clay

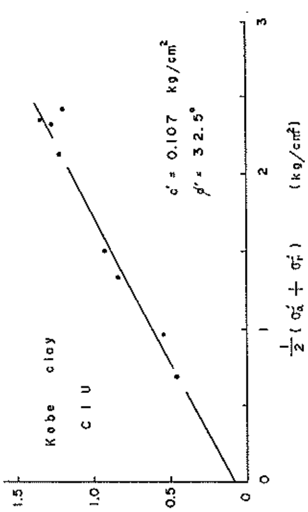

Fig. 14 $1/2 \cdot (\sigma_a' + \sigma_r')$ and $1/2 \cdot (\sigma_a' - \sigma_r')$ at failure on Kobe clay

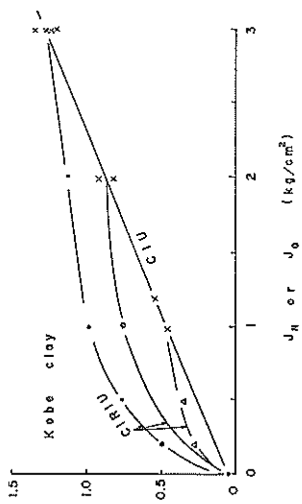

Fig. 15 Undrained strength and mean principal stress on Kobe clay

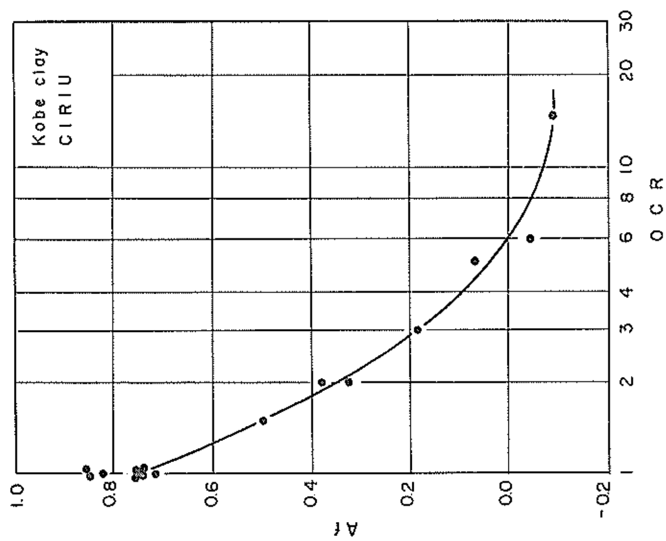

Fig. 16 Pore pressure coefficient at failure and overconsolidation ratio on Kobe clay

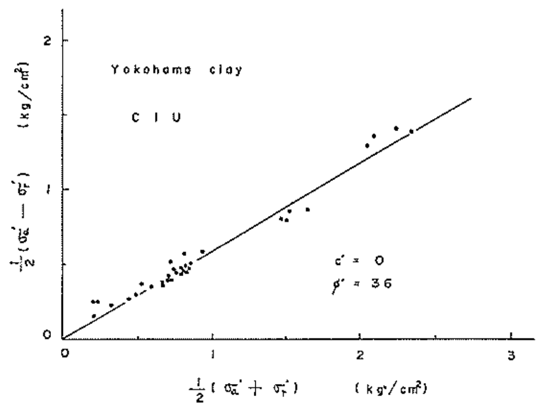

Fig. 17 $1/2 \cdot (\sigma_a' + \sigma_r')$ and $1/2 \cdot (\sigma_a' - \sigma_r')$ at failure in CIU test on Yokohama clay

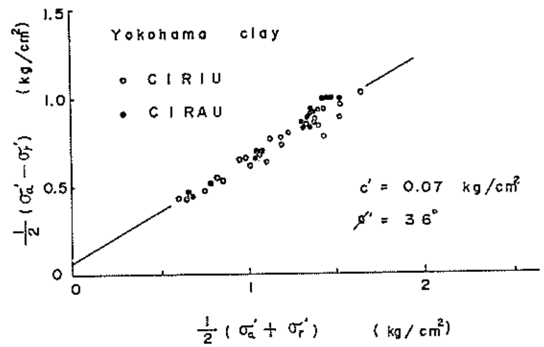

Fig. 18 $1/2 \cdot (\sigma_a' + \sigma_r')$ and $1/2 \cdot (\sigma_a' - \sigma_r')$ at failure in CIRIU and CIRAU tests on Yokohama clay

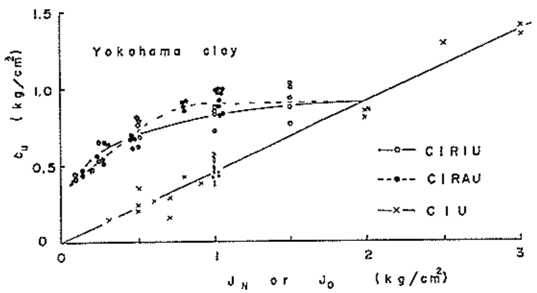

Fig. 19 Undrained strength and overconsolidation ratio on Yokohama clay

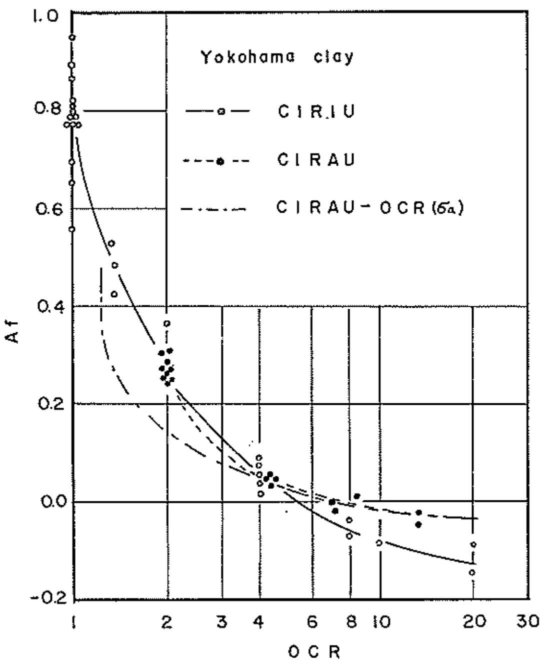

Fig. 20 Pore pressure coefficient at failure and overconsolidation ratio on Yokohama clay

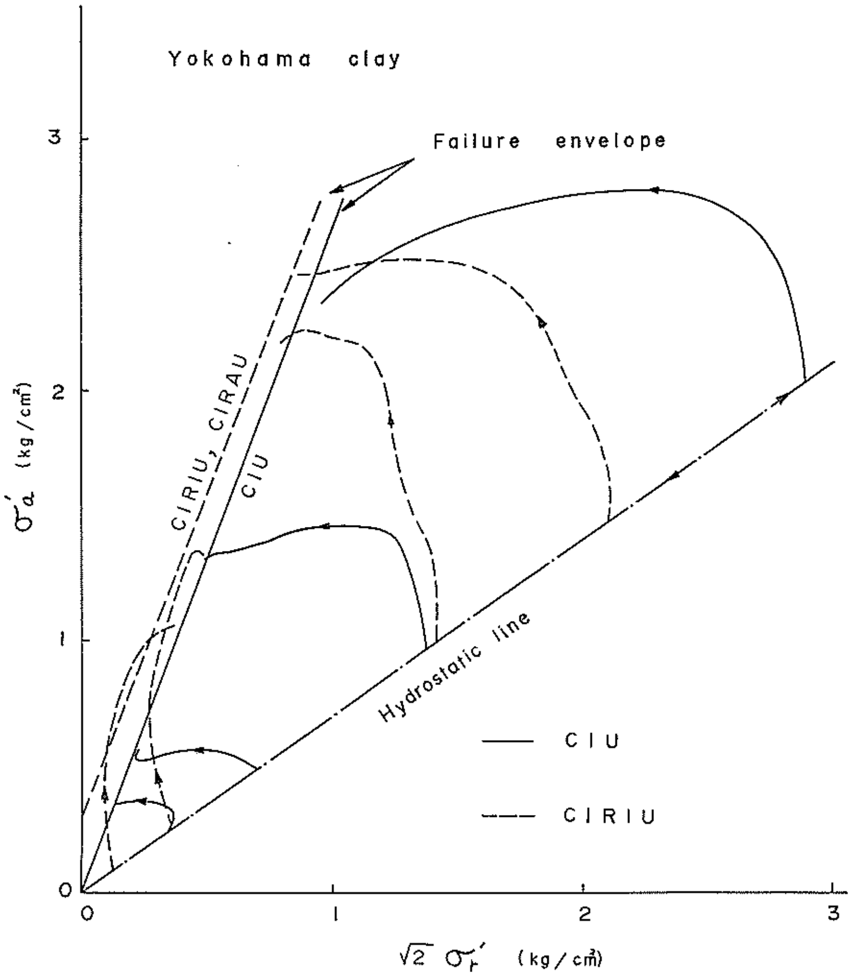

Fig. 21 Effective stress path in CIU and CIRIU tests on Yokohama clay

pressed in terms of axial pressure, the influence of stress anisotropy is much more pronounced as shown in the figure.

Change in effective stresses during shear process can be visualised by means of the effective stress path. Fig. 21 shows the Rendulic's stress diagram on the Yokohama clay in the CIU and CIRIU tests respectively. Difference in the shape of effective stress paths on the narmally- and over-consolidated specimens can readily be seen in this diagram. In the figure, two failure envelopes are drawn for the CIU and CIRIU tests respectively. In the Rendulic's stress diagram the failure envelope for the compression tests is defined by the axial stress intercept b and the slope angle β to the radial stress axis as follows;

$$b=\frac{2c'\cos\phi'}{1-\sin\phi'} \quad \text{and} \quad \tan\beta=\frac{1+\sin\phi'}{\sqrt{2}\,(1-\sin\phi')}$$

The strength parameters c' and ϕ' are given in Figs. 17 and 18.

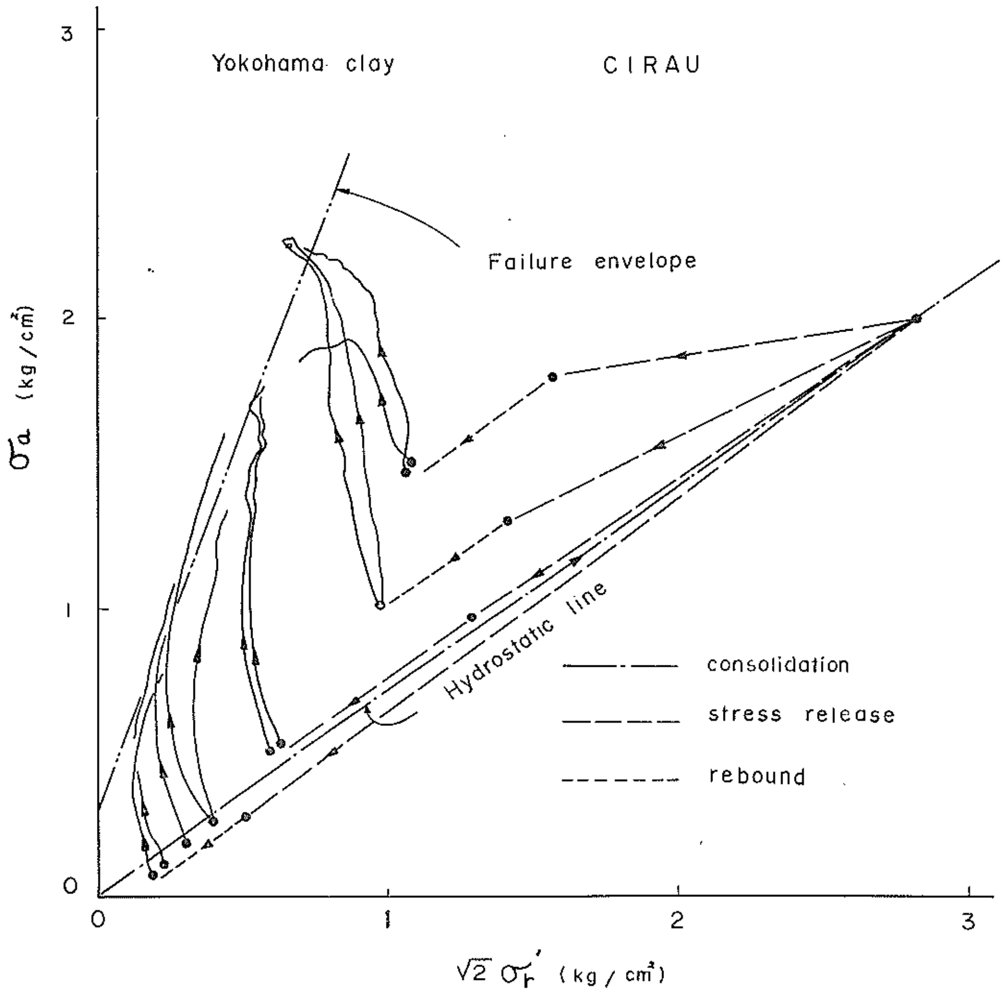

Fig. 22 Effective stress path in CIRAU test on Yokohama clay

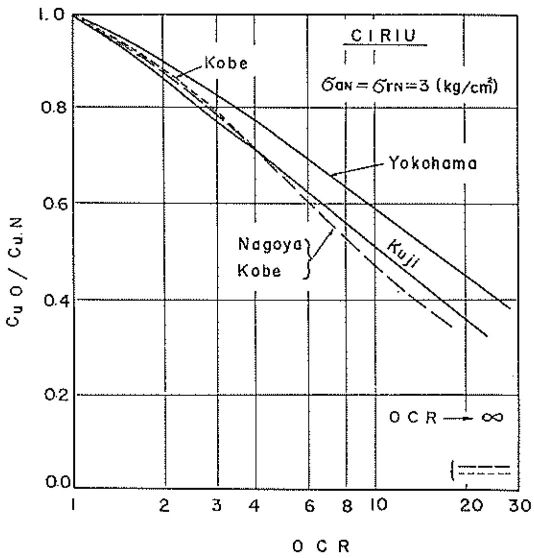

Fig. 23 Strength ratio and overconsolidation ratio in CIRIU test

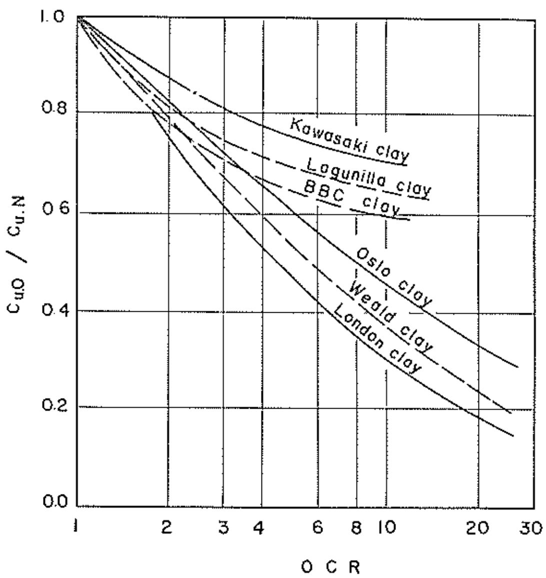

Fig. 24 Strength ratio and overconsolidation ratio in CIRIU test (summarised by Ladd and Lambe)

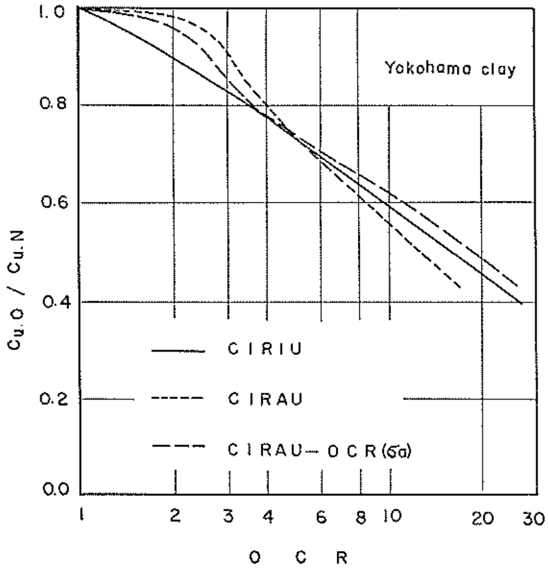

Fig. 25 Strength ratio and overconsolidation ratio on Yokohama clay

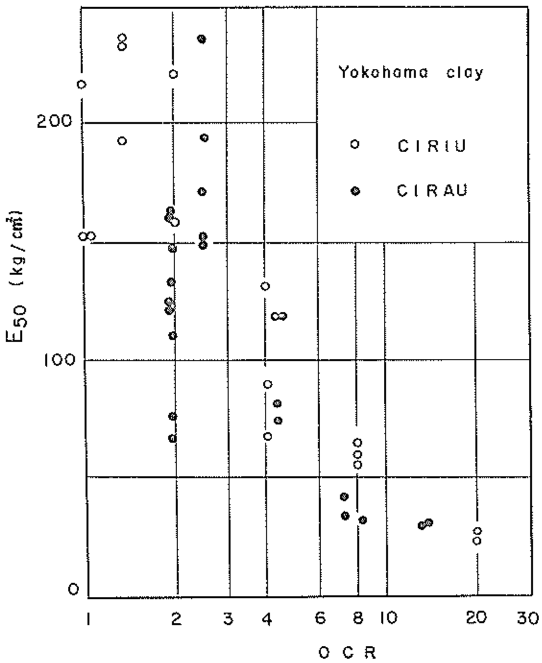

Fig. 26 Secant modulus and overconsolidation ratio on Yokohama clay

Effective stress paths in the CIRAU test are shown in Fig. 22. In the present test, the principal stress difference was kept constant during the rebound process as shown in Fig. 4. In the Rendulic's diagram, the principal stress difference is represented by the vertical distance between a stress point and the hydrostatic line, which represents the condition of $\sigma_a'=\sigma_r'$. Then the stress path in the anisotroic rebound is parellel to the hydrostatic line.

As shown in Figs. 9, 12, 15 and 19, the rebound of clay is accompanied by a decrease in strength. Degree of reduction in the undrained strength with the overconsolidation ratio is shown in Fig. 23, where c_{u0} is the undrained strength of overconsolidated clay and c_{uN} is the strength of the corresponding normally-consolidated state. As seen in the figure, the relationships between the strength ratio and the overconsolidation ratio on three undisturbed clays are similar, in spite of a considerable difference in index properties. In the case of the Nagoya and Kobe clays, several specimens were allowed to rebound under zero pressure. The strength ratio corresponding to infinite overconsolidation ratio are also shown in Fig. 23.

Fig. 24 shows the relationship between the strength ratio and the overconsolidation ratio in the CIRIU tests summarised by Ladd and Lambe (1963). The main difference between the results in Figs. 23 and 24 is in the value of strength ratio in heavily overconsolidated state. Looking at these results, a significant influence of index properties on the strength reduction can not be found out.

Effect of the stress anisotropy on the strength reduction is shown in Fig. 25. As shown in the figure, the effect of stress anisotropy varies with the overconsolidation ratio. If the overconsolidation ratio in terms of the axial pressure is considered, the strength reduction due to rebound becomes apparently small.

In addition to the change in the effective stress path and in the undrained strength, the rebound of soil seems to influence the shape of stress-strain curves. Fig. 26 shows the relationship between the secant modulus E_{50} (Skempton and Bishop, 1954) and the overconsolidation ratio. In spite of a considerable scatter in plots, it will be seen that the secant modulus decreases with the overconslidation ratio. In this figure, the anisotropy does not seem to have any marked influence on the above tendency.

5. Conclusions

Conclusions are as follows:

(a) The time required for rebound is shorter than that for the preceding consolidation.

(b) In spite of a considerable difference in index properties, three undisturbed clays showed similar patterns of strength reduction in the CIRIU test.

(c) The stress anisotropy has some influence on the strength reduction due to rebound.

(d) The secant modulus of stress-strain curve decreases with the overconsolidation ratio.

In the present test, all the specimens were consolidated under an isotropic pressure. In order to reproduce the natural ground condition, however, it will be necessary to consolidate under anisotropic pressures corresponding to the K_0 condition. It has already been reported that stress anisotropy in consolidation process has an appreciable influence on the undrained strength of clays (Henkel and Sowa, 1963; Akai and Adachi, 1965). Dependence of the anisotropic consolidation on the rebound characteristics of clays is open to question.

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References

Akai, K. and T. Adachi (1965) Study on the one-dimensional consolidation and the shear strength characters of fully-saturated clay, in terms of effective stress, Trans. Japan Soc. Civil Engrs., No. 113, pp. 11-27.

Bishop, A. W. and L. Bjerrum (1960) The relevance of the triaxial test to the solution of stability problems, Proc. Research Conf. Shear Strength of Cohesive Soils, ASCE, pp. 437-501.

Bishop, A. W. and D. J. Henkel (1957) The measurement of soil properties in the triaxial test, Edward Arnold, London, 228 pp.

Henkel, D. J. (1960) The shear strength of saturated remoulded clays, Proc. Research Conf. Shear Strength of Cohesive Soils, ASCE, pp. 533-554.

Henkel, D. J. and V. A. Sowa (1963) The influence of stress-history on stress path in undrained triaxial tests on clay, Laboratory Shear Testing of Soils, ASTM, STP No. 361, pp. 280-291.

Ladd, C. C. (1965) Stress-strain behaviour of anisotropically consolidated clays during undrained shear, Proc. 6th Int. Conf. SMFE, Vol. 1, pp. 282-286.

Ladd, C. C. and T. W. Lambe (1963) The strength of 'undisturbed' clay determined from undrained tests, Laboratory Shear Testing of Soils, ASTM, STP No. 361, pp. 342-371.

Nakase, A (1967) The $\phi_u=0$ analysis of stability and unconfined compression strength, Soil and Foundation, Vol. 7, No. 2, pp. 33-50.

Simons, N. E. (1960) The effect of overconsolidation on the shear characteristics of an undisturbed Oslo clay, Proc. Research Conf. Shear Strength of Cohesive Soils, ASCE, pp. 747-763.

Skempton, A. W. (1954) The pore pressure coefficients A and B, Geotechnique, Vol. 4, No. 4, pp. 153-173.

Skempton, A. W. (1961) Horizontal stresses in overconsolidated Eocene clay, Proc. 5th Int. Conf. SMFE, Vol. 1, pp. 351-357.

Skempton, A. W. and A. W. Bishop (1954) Soils, Chapter 10 of Building Materials, North Holland Publ. Co., Amsterdam, pp. 417-482.

List of Symbols

A_f : Pore pressure coefficient at failure
a : Cohesion intercept
b : Axial stress intercept
c' : Apparent cohesion in terms of effective stress
c_u : Undrained strength
c_{uN} : Undrained strength of normally-consolidated clay
c_{uo} : Undrained strength of overconsolidated clay
E_{50} : Secant modulus
J_N : Mean principal stress in consolidation
J_O : Mean principal stress in rebound
K_0 : Coefficient of earth pressure at rest
OCR : Overconsolidation ratio
u : Pore pressure
ΔV : Volume change
α, β : Slope angle
σ_a : Axial stress in terms of total stress
σ_a' : Axial stress in terms of effective stress
σ_r : Radial stress in terms of total stress
σ_r' : Radial stress in terms of effective stress
ϕ' : Angle of shearing resistance in terms of effective stress
ϕ_u : Angle of shearing resistance in terms of total stress (undrained test)