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STRESS-STRAIN BEHAVIOR OF ANISOTROPICALLY CONSOLIDATED CLAYS DURING UNDRAINED SHEAR

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by

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ABSTRACT

The behavior of anisotropically and isotropically consolidated samples of six normally consolidated clays is compared in consolidated-undrained triaxial compression tests. Anisotropic consolidation had little effect on the ratio of undrained shear strength s_u to major principal stress at consolidation, while both $\bar{\phi}$ and A_f at failure were generally decreased. It is shown that the strength parameters of anisotropically consolidated samples can not be predicted from test results on isotropically consolidated samples.

The in situ undrained strength of clay is not an unique function of water content or consolidation pressure, and hence is not independent of total stress path, as commonly assumed. Triaxial data on two normally consolidated clays starting with K_o stresses show a two fold variation in s_u depending on whether the axial pressure is increased or decreased. Possible implications of such effects on current practice in selecting values of s_u for $\phi = 0$ stability analyses are discussed.

ABSTRACT

On compare l'action des échantillons d'argile anisotropiques ainsi que isotropiques, consolidés normalement, au moyen de l'essai de compression triaxiale sans drainage. L'effet de la consolidation anisotropique, sur le rapport entre la résistance au cisaillement à teneur en eau constante, s_u, et la contrainte principale à la fin de la consolidation, ne fut pas considérable, tandis que $\overline{\phi}$ et A_f, à la rupture, ont généralement diminué. On trouve, que l'on ne peut pas prédire les paramètres de résistance au cisaillement des échantillons consolidés anisotropiquement, à partir des essais effectués sur des échantillons consolidés isotropiquement.

La résistance in situ sans drainage de l'argiles n'est pas une relation unique de la teneur en eau ni de la pression de consolidation. C'est pour cette raison que la résistance n'est pas, contrairement aux croyances populaires, indépendante de la trace des contraintes totales. Les essais triaxiaux commencant avec une pression K_o de deux échantillons d'argile normalement consolidés, prouvent que la valeur de s_u peut doubler, soit qu'il s'agisse d'une augmentation ou dimintion de la pression axiale. On discute les conséquences de ces effects sur les habitudes du métier dans la choix des valeurs de s_u pour l'analyse des stabilités employant $\phi = 0$.

LIST OF FIGURES

I. INTRODUCTION

Consolidated-Undrained triaxial compression tests with pore pressure measurements are run on samples of saturated clays: (1) to obtain values of undrained shear strength for total stress (ϕ = 0) stability analyses; (2) to measure effective stress envelopes at failure for use in effective stress stability analyses; (3) to obtain values of stress-strain modulus for estimates of "immediate" (shear) settlements; and (4) to obtain values of the pore pressure parameter A (Skempton, 1954) for stability and/or settlement analyses. Isotropic consolidation stresses are frequently used in these undrained tests (called $\overline{\text{CIU}}$ tests). However the stress system acting on in situ clays is generally anisotropic, i.e. the horizontal and vertical stresses are not equal. This paper compares the stress-strain behavior of anisotropically consolidated and isotropically consolidated samples of six saturated normally consolidated clays as measured in undrained triaxial tests. The effects of rotation of principal planes during undrained shear are discussed.

The ratio of horizontal to vertical effective stress in one dimensionally consolidated horizontal soil deposits is called "coefficient of earth pressure at rest" and is denoted by $K_o = \bar{\sigma}_{hc} / \bar{\sigma}_{vc}$. The variation in K_o with overconsolidation ratio (OCR) is plotted in Fig. 1 for several clays. K_o for normally consolidated clays (OCR = 1) generally equals 0.6 ± 0.2 and has been found to be related to the slope of the effective stress envelope ($\bar{\phi}$ = friction angle) at maximum obliquity by $K_o = 1 - \sin\bar{\phi}$ (Jaky, 1948; Bishop, 1958; Simons, 1958). As a clay deposit is rebounded, K_o increases and becomes greater than unity for OCR values exceeding about 3.5 ± 1; in other words the horizontal pressure becomes larger than the vertical pressure. Thus the deviation of in situ stresses from an isotropic condition is most pronounced in normally consolidated and in highly overconsolidated clays, whereas in slightly overconsolidated clays the stresses are approximately isotropic.

II. TRIAXIAL COMPRESSION TESTS ON NORMALLY CONSOLIDATED CLAYS

The ratio of undrained shear strength s_u to major principal consolidation stress $\bar{\sigma}_{1c}$ from consolidated-undrained triaxial compression tests on normally consolidated clays with zero cohesion intercept is given by:*

$$\frac{s_u}{\bar{\sigma}_{1c}} = \frac{[K + A_f(1 - K)] \sin\bar{\phi}}{1 + (2A_f - 1)\sin\bar{\phi}} \tag{1}$$

where failure is defined at maximum stress difference and

$s_u = (\sigma_1 - \sigma_3)\ \text{max.}/2$; $K = \bar{\sigma}_{3c}/\bar{\sigma}_{1c}$; $\bar{\phi}$ = friction angle at failure

$A_f = (\Delta u - \Delta\sigma_3)/(\Delta\sigma_1 - \Delta\sigma_3)$ at failure.

*This ratio is often denoted by c/p. (See Skempton and Bishop, 1954).

If A_f and $\bar{\phi}$ are independent of K, it follows from Eqn. (1) that the influence of K on the value of $s_u/\bar{\sigma}_{1c}$ is:

$$R < 1 \quad \text{for } A_f < 1$$
$$R = 1 \quad \text{for } A_f = 1$$
$$R > 1 \quad \text{for } A_f > 1$$

where

$$R = \frac{s_u/\bar{\sigma}_{1c} \text{ for anisotropic consolidation } (K < 1)}{s_u/\bar{\sigma}_{1c} \text{ for isotropic consolidation } (K = 1)}$$

However A_f and $\bar{\phi}$ are not generally independent of the value of K as shown by the data in Table I from consolidated-undrained triaxial compression tests with pore pressure measurements on six remolded and undisturbed clays of varying plasticity. The $\overline{CIU}$ tests (K = 1) on the six clays gave $s_u/\bar{\sigma}_{1c} = 0.28 - 0.45$, $A_f = 0.80 - 1.10$ and $\bar{\phi} = 24 - 37^{\circ}$, all at $(\sigma_1 - \sigma_3)$ max. The $\overline{CAU}$ tests, for which K was approximately equal to K_o, showed the following effects of anisotropic consolidation (see Col. 6-11 in Table I);

(1) The change in $s_u/\bar{\sigma}_{1c}$ was generally small with a maximum increase of 10 per cent and a maximum decrease of 15 per cent.

(2) The value of A_f decreased by a significant amount except for the remolded Weald and Vicksburg Buckshot clays.

(3) The friction angle $\bar{\phi}$ decreased by 0 to 4°.

(4) The strain at failure was considerably smaller, being generally less than 1 per cent versus 2.5 to 15 per cent for the $\overline{CIU}$ tests.

For the condition of maximum obliquity (Col. 12 and 13), the $\overline{CAU}$ tests showed lower values of shear stress but essentially the same effective stress envelope.

Effective stress paths from $\overline{CIU}$ and $\overline{CAU}$ tests on tube samples of Kawasaki clay are compared in Fig. 2 in which half the stress difference $q = (\sigma_1 - \sigma_3)/2$ is plotted against the average effective stress $\overline{p} = (\overline{\sigma}_1 + \overline{\sigma}_3)/2$. These data again illustrate that anisotropic consolidation, relative to isotropic consolidation, lowers the friction angle at $(\sigma_1 - \sigma_3)$ max., although the effective stress envelope at maximum obliquity is practically unchanged.

The strength behavior of anisotropically consolidated samples is sometimes derived from $\overline{CIU}$ test data (Taylor, 1948, p. 387; Henkel, 1960; Lowe and Karafiath, 1960) by assuming that effective stress paths from $\overline{CIU}$ tests represent an unique relationship between shear stress and effective stress. If such were true, a $\overline{CAU}$ test consolidated under stresses falling on the $q - \overline{p}$ path of a $\overline{CIU}$ test would have an effective stress path during subsequent undrained shear that coincided with the path of the $\overline{CIU}$ test. The test data in Fig. 2 show that the strength behavior of anisotropically consolidated samples can not be predicted from $\overline{CIU}$ test data (compare the measured $\overline{CAU}$ test having $\overline{\sigma}_{1c} = 3.20$ with the extension of the $\overline{CIU}$ test shown with a dashed line). Similar data on the other clays* in Table I also show that effective stress paths

*Whitman, et al (1960) presented stress paths from $\overline{CIU}$ and $\overline{CAU}$ tests on Vicksburg Buckshot clay.

from $\overline{\text{CIU}}$ tests are not unique. Values of $s_u/\bar{\sigma}_{1c}$ for $\overline{\text{CAU}}$ tests with $K = K_o$ predicted from $\overline{\text{CIU}}$ stress paths will generally be less than the measured values (the Weald clay in Table I is an exception).

Fig. 3 compares stress-strain curves in a dimensionless form from $\overline{\text{CIU}}$ and $\overline{\text{CAU}}$ tests on normally consolidated samples of the Kawasaki and Boston Blue clays. The anisotropically consolidated samples show a much lower strain at failure, a larger drop off in stress difference at strains beyond failure, and a lower A parameter at small strains but a higher one at the larger strains. The excess pore pressure Δu is reduced since a large portion of the stress difference [equal to $(\bar{\sigma}_{1c} - \bar{\sigma}_{3c}) = \bar{\sigma}_{1c}(1 - K)$] was applied prior to undrained shear. Bjerrum and Lo (1963) suggest use of the relationship $\Delta u/\bar{\sigma}_{1c} + (1 - K)$ to correlate pore pressures from $\overline{\text{CIU}}$ and $\overline{\text{CAU}}$ tests at the condition of maximum obliquity. The tests in Fig. 3 give the following values at maximum obliquity:

Soil	Type of Test	$\Delta u/\bar{\sigma}_{1c} + (1 - K)$
Boston Blue Clay	$\overline{\text{CIU}}$	0.74
	$\overline{\text{CAU}}$	0.79
Kawasaki Clay	$\overline{\text{CIU}}$	0.75
	$\overline{\text{CAU}}$	0.80

By comparison Bjerrum and Lo (1963) observed that $\overline{\text{CAU}}$ tests yielded values of adjusted pore pressure equal to or slightly higher than those from $\overline{\text{CIU}}$ tests.

III. STRENGTH BEHAVIOR AS INFLUENCED BY TOTAL STRESS PATH DURING UNDRAINED SHEAR

In selecting a value of undrained shear strength s_u for use in a "$\phi = 0$" stability analysis, it is commonly assumed that the value of s_u is not affected by different total stress paths during undrained shear. Data are presented in Fig. 4 to show that the undrained strength of clays is not necessarily "unique" but can indeed vary several fold depending on the stress system applied during shear.

A large chunk of Boston Blue clay, prepared by consolidating a slurry to 1.5 kg/cm^2, was cut up for subsequent strain controlled triaxial tests. Three specimens (the $\overline{CAU}$, CA-$\overline{UU}$ and $\overline{CAU}$-RE tests) were consolidated in small increments under approximate K_o stresses to $\bar{\sigma}_{1c} = \bar{\sigma}_{ac} = 4\ kg/cm^2$ and $\bar{\sigma}_{3c} = \bar{\sigma}_{rc} = 2.16\ kg/cm^2$. The effective stress paths during undrained shear in terms of the axial stress $\bar{\sigma}_a$ and radial stress $\bar{\sigma}_r$ are plotted in Fig. 4. The $\overline{CAU}$ test was sheared by increasing the axial total stress σ_a with $\Delta\sigma_r = 0$. This total stress path is representative of the stress conditions under the centerline of a circular loaded area (note that the value of $\Delta\sigma_r$ is inconsequential as long as $\Delta\sigma_a > \Delta\sigma_r$). In the CA-$\overline{UU}$ test the sample was sheared undrained by first decreasing σ_a until $\sigma_a = \sigma_r$ and then increasing σ_a until failure occurred. This test represents an Unconsolidated-Undrained compression test on a "perfect" sample, wherein "perfect" sampling means that no disturbance is given to the sample other than that involved with the release of the consolidation shear stresses (see Ladd and Lambe, 1963, and Skempton and Sowa, 1963). The $\overline{CAU}$-RE test, which is representative of the stress conditions under the centerline of a circular excavation (i.e. unloaded area), was sheared undrained by decreasing the axial stress*. The direction of the major and minor principal planes rotated 90^{o} when σ_a became less than σ_r and the sample failed in extension since the intermediate principal stress σ_2 equalled the major principal stress σ_1 (both equalling σ_r).

At failure, i. e. $(\sigma_1 - \sigma_3)$ max., the specimens showed:

$\overline{CAU}$ test	$s_u/\bar{\sigma}_{1c}$	= 0.33 ;	s_u	= 1.35 kg/cm^2
CA-$\overline{UU}$ test	"	= 0.275 ;	"	= 1.12 kg/cm^2
$\overline{CAU}$-RE test	"	= 0.165 ;	"	= 0.66 kg/cm^2

Thus the value of s_u for three specimens consolidated under essentially identical stresses, and hence with identical water contents at failure, varied by a factor of two because of the different stress systems applied during shear. A similar test series on normally consolidated samples of Kawasaki clay II yielded $s_u/\bar{\sigma}_{1c}$ = 0.44, 0.37 and 0.22 for the $\overline{CAU}$, CA-$\overline{UU}$ and $\overline{CAU}$-RE tests respectively. The range in strengths for identical samples was also two-fold. Data on variations in stress-strain curves with different stress systems were presented elsewhere (Ladd, 1964).

*The difference in stress paths from the CA-$\overline{UU}$ and $\overline{CAU}$-RE tests between K = K_o and K = 1 is due to experimental scatter.

IV. CONCLUSIONS

One dimensional, as contrasted to isotropic three dimensional, consolidation of normally consolidated clays of low to moderate sensitivity causes the following effects with undrained triaxial compression tests:

(1) $s_u/\bar{\sigma}_{1c}$ is practically unchanged (± 15%).

(2) Both $\bar{\phi}$ and A_f at $(\sigma_1 - \sigma_3)$ max. are generally decreased; the axial strain is much smaller.

(3) At maximum obliquity, $\bar{\phi}$ is essentially unchanged, the A parameter is often greatly increased, and $(\sigma_1 - \sigma_3)/\bar{\sigma}_{1c}$ is usually decreased.

These effects can not be predicted from the results of $\overline{CIU}$ tests.

Although $s_u/\bar{\sigma}_{1c}$ is approximately independent of K for failure in triaxial compression, its value can be highly dependent on the type of total stress system applied during shear. For example, triaxial specimens of a clay normally consolidated with K_o stresses and failed by increasing the radial pressure may have only half the undrained strength of identical specimens failed by increasing the axial stress. Simple shear tests have also produced much lower values of $s_u/\bar{\sigma}_{1c}$ than triaxial compression tests (Bjerrum, 1961). The most important variables are probably the values of K at consolidation, σ_2 at failure and the direction of σ_1 at failure relative to its direction after consolidation (i.e. rotation of principal stresses). The magnitude of the change in $s_u / \bar{\sigma}_{1c}$ with different stress systems would generally increase with the sensitivity of the clay.

The common assumption that the in situ undrained strength is a constant is open to question since most field cases involve stress systems which vary from point to point (an example being different amounts of rotation of principal stresses along a curved failure surface). With footings on normally consolidated clays, the average value of s_u along a potential failure surface may be far less than that corresponding to triaxial compression. If such is true, the reported successful use of unconfined compression tests for $\phi = 0$ analyses (Bishop and Bjerrum, 1960) must be partly due to compensating errors. A decrease in the value of the in situ s_u (relative to s_u for shear in compression) caused by different total stress paths and a decrease in the s_u of unconfined compression tests caused by disturbance during sampling (Ladd and Lambe, 1963) are possible compensating errors.

Additional research on the effects of sample disturbance and effects of total stress path is required before the engineer can select with confidence the proper values of undrained strength for $\phi = 0$ stability analyses in situations where experience has not yet developed an empirical set of guidelines. In particular, programs of field and laboratory strength tests should give thought to the total stress path followed by different clay elements in the field.

V. ACKNOWLEDGEMENTS

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VI. REFERENCES

(1) Bishop, A. W. (1958). "Test Requirements for Measuring the Coefficient of Earth Pressure at Rest". Proceedings Brussels Conference on Earth Pressure Problems. Vol. 1, pp. 2-24.

(2) ———— and Bjerrum, L. (1960). "The Relevance of the Triaxial Test to the Solution of Stability Problems". American Society of Civil Engineers, Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado. pp. 437-501.

(3) Bjerrum, L. (1961). "The Effective Shear Strength Parameters of Sensitive Clays". Proceedings Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris. Vol. 1, pp. 23-28.

(4) ———— and Lo, K. Y. (1963). "Effect of Aging on the Shear-Strength Properties of a Normally Consolidated Clay". Geotechnique, Vol. XIII, No. 2, pp. 147-157.

(5) Henkel, D. J. (1960). "The Shear Strength of Saturated Remolded Clays". American Society of Civil Engineers, Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado. pp. 533-554.

(6) ———— and Sowa, V. A. (1963). "The Influence of Stress History on the Stress Paths Followed in Undrained Triaxial Tests". American Society for Testing and Materials and National Research Council of Canada, Symposium on Laboratory Shear Testing of Soils, Ottawa, Canada. Preprint.

(7) Jaky, J. (1948). "Pressure in Silos". Proceedings Second International Conference on Soil Mechanics and Foundation Engineering, Rotterdam. Vol. 1, pp. 103-107.

(8) Ladd, C. C. (1964). "Stress-Strain Modulus of Clay from Undrained Triaxial Tests". American Society of Civil Engineers, Technical Conference on Design of Foundations for Control of Settlements, Evanston, Illinois. Preprint.

(9) Ladd, C. C. and Lambe, T. W. (1963). "The Strength of "Undisturbed" Clay Determined from Undrained Tests". American Society for Testing and Materials and National Research Council of Canada, Symposium on Laboratory Shear Testing of Soils, Ottawa, Canada, Preprint.

(10) Landva, A. (1962). Norwegian Geotechnical Institute. Internal Report F. 175-4.

(11) Lowe, III, J. and Karafiath, L. (1960). "Effect of Anisotropic Consolidation on the Undrained Shear Strength of Compacted Clays". American Society of Civil Engineers, Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado. pp. 837-858.

(12) Simons, N. E. (1958). Discussion on: General Theory of Earth Pressure. Proceedings Brussels Conference on Earth Pressure Problems. Vol. 3, pp. 50-53.

(13) ———— (1960). "The Effect of Overconsolidation on the Shear Strength Characteristics of an Undisturbed Oslo Clay". American Society of Civil Engineers, Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado. pp. 747-763.

(14) Skempton, A. W. (1954). "The Pore Pressure Coefficients A and B". Geotechnique, Vol. IV, No. 4, pp. 143-147.

(15) ———— (1961). "Horizontal Stresses in Overconsolidated Eocene Clay". Proceedings Fifth International Conference on Soil Mechanics and Foundation Engineering, Paris. Vol. 1, pp. 351-357.

(16) ———— and Bishop, A. W. (1954). "Soils", Chapter X of Building Materials, their Elasticity and Inelasticity. Editor, M. Reiner. New York: Interscience Publishers.

(17) ———— and Sowa, V. A. (1963). "The Behavior of Saturated Clays During Sampling and Testing". Geotechnique, Vol. XIII, No. 4, pp. 269-290.

(18) Taylor, D. W. (1948). Fundamentals of Soil Mechanics. John Wiley and Sons, New York, 700 p.

(19) Whitman, R. V., Ladd, C. C. and da Cruz, P. (1960). Discussion to Session 3. American Society of Civil Engineers, Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado. pp. 1049-1056.

TABLE I Consolidated-Undrained Triaxial Compression Tests on Normally Consolidated Clays (All stresses in kg/cm^2)

1	2	3	4	5	6	7	8	9	10	11	12	13
Soil	Classification Data				Consolidation		At $(\sigma_1 - \sigma_3)_{max.}$				At $(\bar{\sigma}_1/\bar{\sigma}_3)_{max.}$	
	ω_L, %	I_p, %	I_L, %	Activity	$\bar{\sigma}_{1c}$	$\bar{\sigma}_{3c}/\bar{\sigma}_{1c}$	$s_u/\bar{\sigma}_{1c}$	A_f	$\bar{\phi}°$	$\mathcal{E}_f$, %	$q/\bar{\sigma}_{1c}$	$\bar{\phi}°$
Remolded Boston Blue Clay (Fresh water) (s_t = 5-10)	33	15	375	0.5	4 - 6	1.00	0.30	1.10	27.5	2.5	0.29	32.5
					3 - 6	0.54	0.33	0.60	26.5	0.4	0.27	33
Remolded Weald Clay*	46	24	48	0.63	2 - 7.5	1.00	0.32	0.92	26	15	—	—
					2 - 7	0.61	0.27	1.80	26	7	—	—
Remolded Vicksburg Buckshot Clay	63	39	230	0.75	3 - 6	1.00	0.28	1.05	24	6	0.27	25
					3 - 6	0.54	0.28	1.05	23.5	0.7	0.25	25
Undisturbed Kawasaki Clays I and II (s_t = 10)	80 (50-100)	38 (20-50)	80 (60-100)	~1 (0.7-1.7)	1.5 - 6	1.00	0.45	0.80	37	4.5	0.44	38.5
					2 - 5.5	0.52	0.42	0.50	33	0.9	0.40	39
Undisturbed Brobekkveien, Oslo Clay** (s_t = 5)	39	18	72	0.5	1.5 - 4	1.00	0.35	0.95	30.5	3.5	0.35	31
					2 - 12	0.47	0.32	0.75	27	0.6	0.28	31.5
Undisturbed Skabo Clay*** (s_t = 2-6)	52	29	65	0.6	2 - 6	1.00	0.32	1.05	30	5.3	—	—
					2 - 5	0.47	0.32	0.75	26.5	0.6	—	—

* From Skempton and Sowa (1963) ** From Simons (1960) *** From Landva (1962)

Notes:

Col. 1: s_t = sensitivity

Col. 4: I_L values at remolding and in situ for remolded and undisturbed samples respectively

Col. 5: Activity = I_p/(% − 2 microns)

Col. 6: Range of values of $\bar{\sigma}_{1c}$ prior to shear

Col. 7-13: Ave. values from tests within the pressure range in Col. 6

Col. 10, 13: Values of $\bar{\phi}$ assuming zero cohesion intercept

Col. 11 : $\mathcal{E}_f$ = axial strain at failure

Col. 12 : $q = (\sigma_1 - \sigma_3)/2$

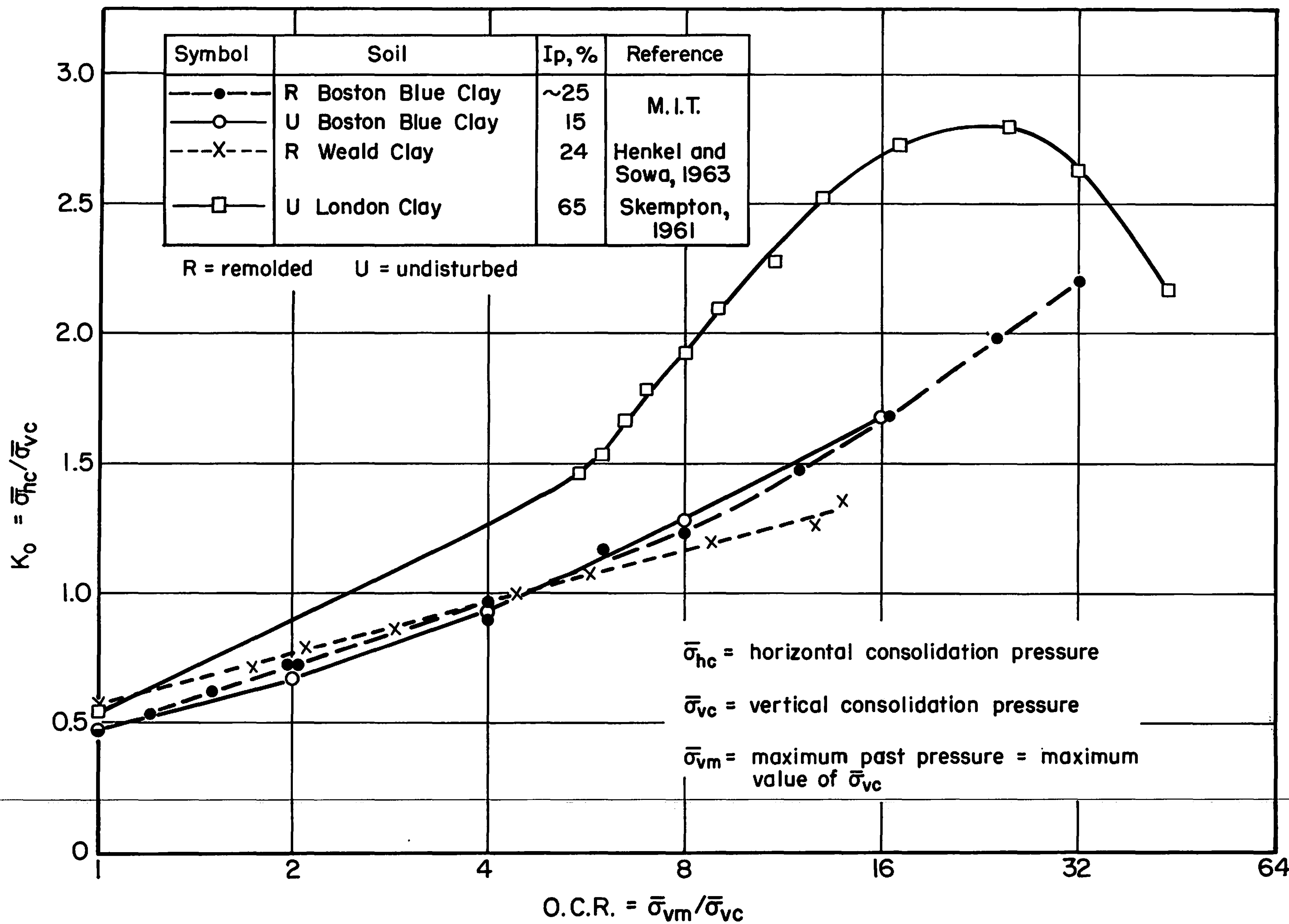

Symbol
Soil
Ip, %
Reference
R Boston Blue Clay
~25
M.I.T.
U Boston Blue Clay
15
R Weald Clay
24
Henkel and Sowa, 1963
U London Clay
65
Skempton, 1961
R = remolded
U = undisturbed
3.0
2.5
2.0
1.5
1.0
0.5
0
$K_o = \bar{\sigma}_{hc}/\bar{\sigma}_{vc}$
1
2
4
8
16
32
64
O.C.R. = $\bar{\sigma}_{vm}/\bar{\sigma}_{vc}$
$\bar{\sigma}_{hc}$ = horizontal consolidation pressure
$\bar{\sigma}_{vc}$ = vertical consolidation pressure
$\bar{\sigma}_{vm}$ = maximum past pressure = maximum value of $\bar{\sigma}_{vc}$

Figure 1

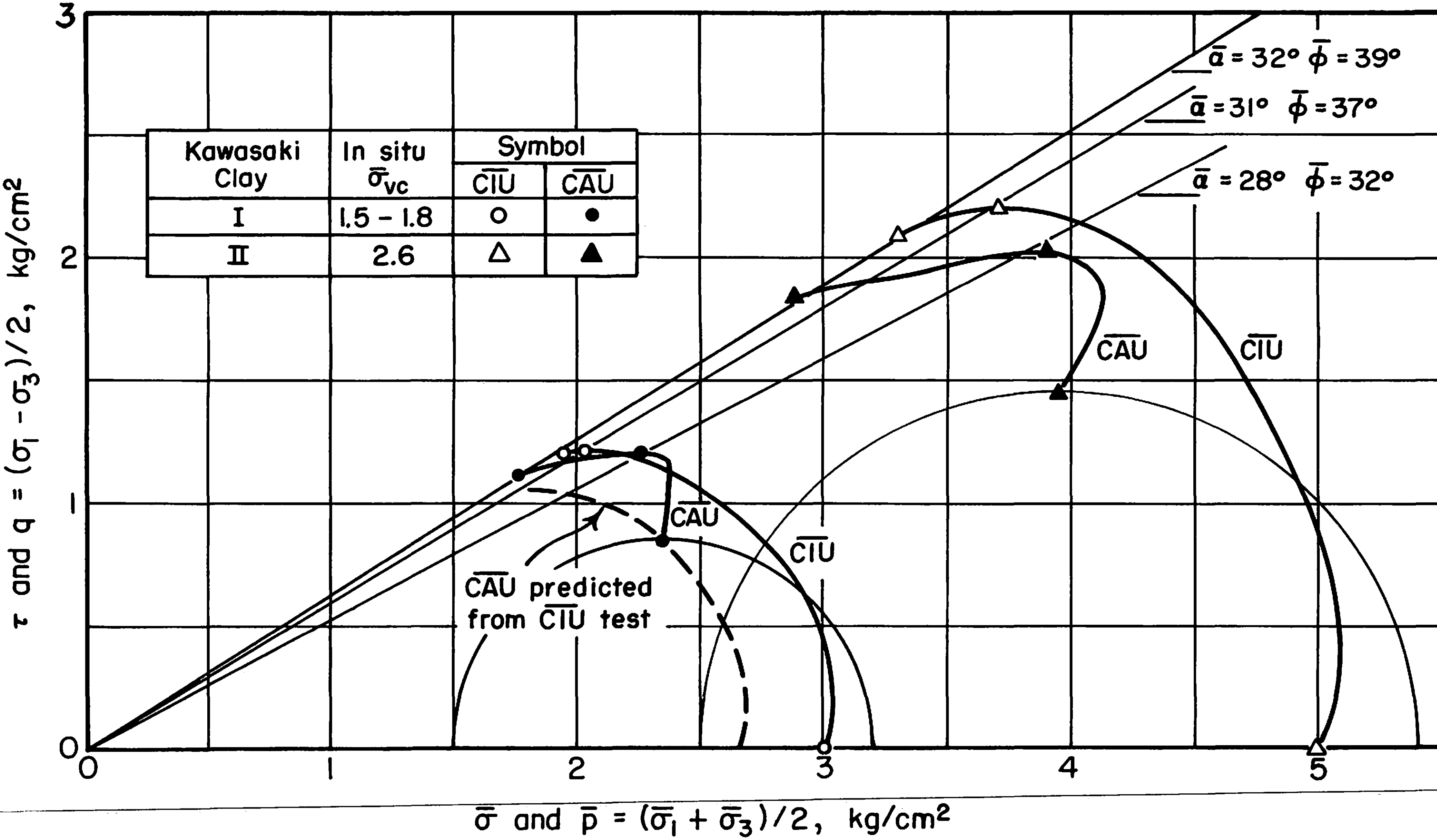
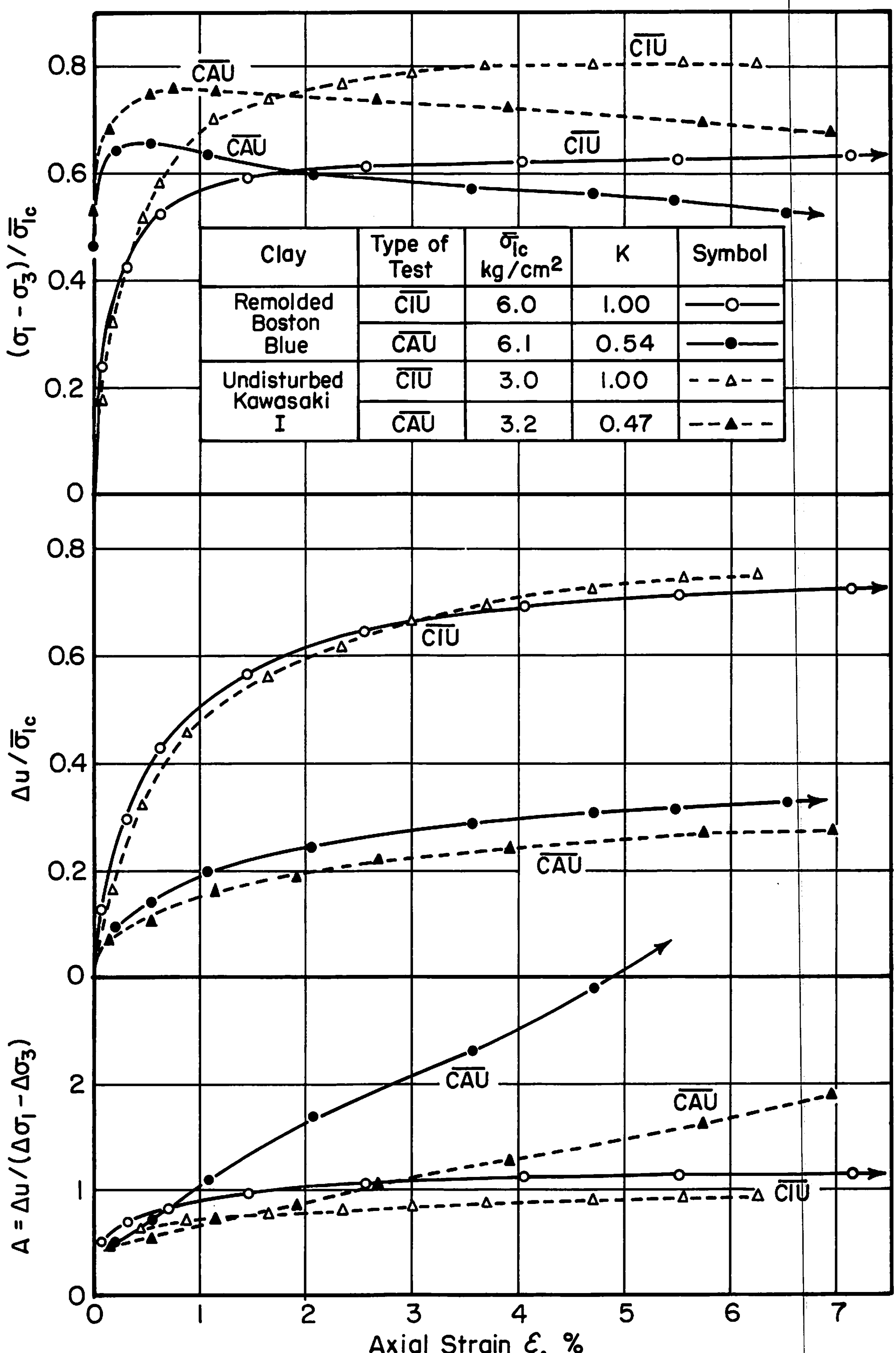
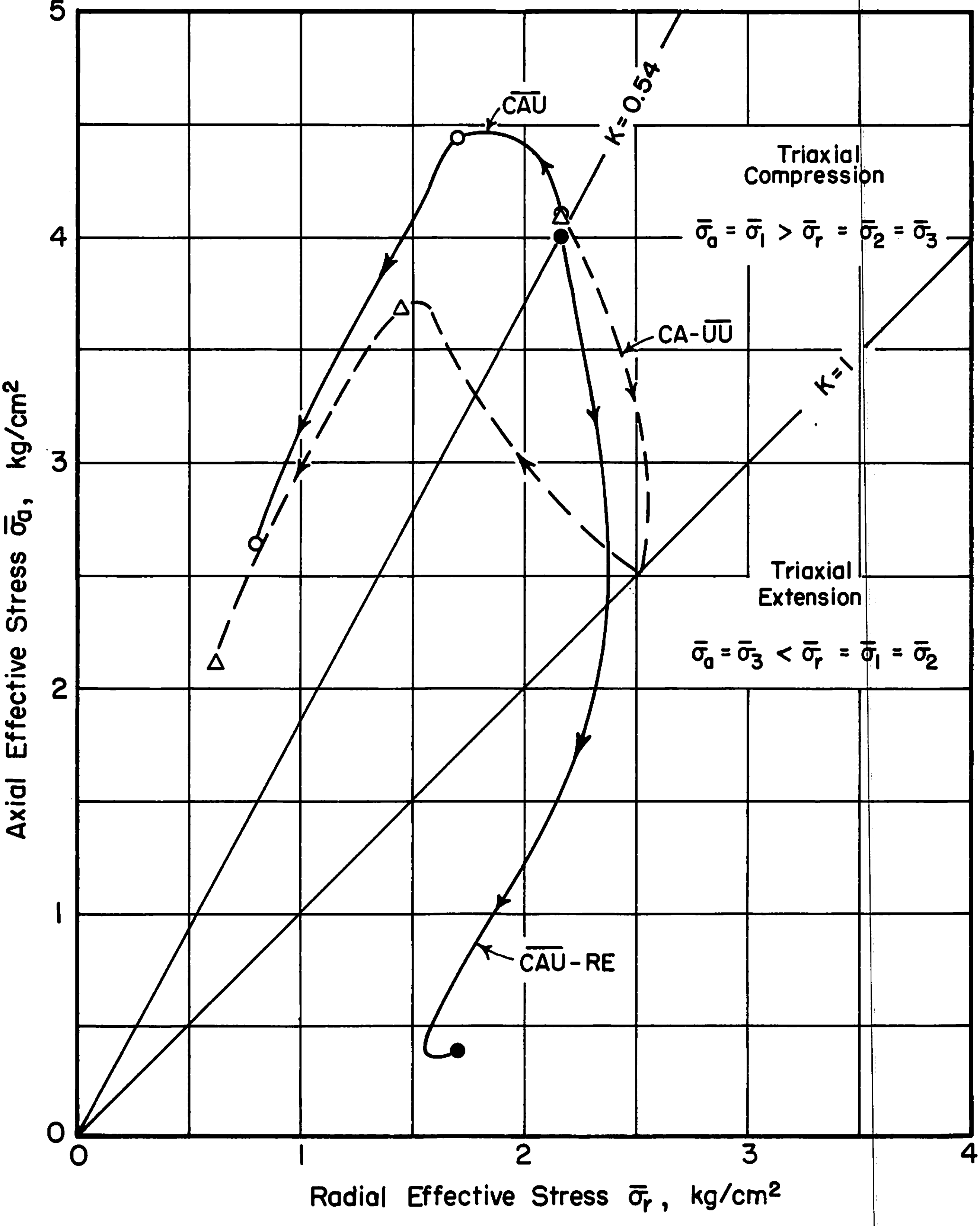

Kawasaki Clay
In situ $\bar{\sigma}_{vc}$
Symbol
$\overline{CIU}$
$\overline{CAU}$
I
1.5 - 1.8
II
2.6
$\bar{a}$ = 32° $\bar{\phi}$ = 39°
$\bar{a}$ = 31° $\bar{\phi}$ = 37°
$\bar{a}$ = 28° $\bar{\phi}$ = 32°
$\overline{CAU}$
$\overline{CIU}$
$\overline{CAU}$ predicted from $\overline{CIU}$ test
τ and q = $(\sigma_1 - \sigma_3)/2$, kg/cm²
$\bar{\sigma}$ and $\bar{p} = (\bar{\sigma}_1 + \bar{\sigma}_3)/2$, kg/cm²

Figure 2



Clay	Type of Test	$\bar{\sigma}_{1c}$ kg/cm^2	K	Symbol
Remolded Boston Blue	$\overline{CIU}$	6.0	1.00	—○—
	$\overline{CAU}$	6.1	0.54	—●—
Undisturbed Kawasaki I	$\overline{CIU}$	3.0	1.00	- -△- -
	$\overline{CAU}$	3.2	0.47	- -▲- -

Figure 3



Axial Effective Stress $\bar{\sigma}_a$, kg/cm^2
Radial Effective Stress $\bar{\sigma}_r$, kg/cm^2
0
1
2
3
4
5
$\overline{CAU}$
K=0.54
K=1
Triaxial Compression
$\bar{\sigma}_a = \bar{\sigma}_1 > \bar{\sigma}_r = \bar{\sigma}_2 = \bar{\sigma}_3$
CA-$\overline{UU}$
Triaxial Extension
$\bar{\sigma}_a = \bar{\sigma}_3 < \bar{\sigma}_r = \bar{\sigma}_1 = \bar{\sigma}_2$
$\overline{CAU}$-RE

Figure 4