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THE STRENGTH OF "UNDISTURBED" CLAY DETERMINED FROM UNDRAINED TESTS

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SYNOPSIS

Test data from unconsolidated-undrained (*UU*) and consolidated-undrained (*CU*) tests on tube samples of several saturated clays are compared, and the generally large discrepancies in undrained shear strength, s_u , are analyzed in terms of changes in effective stress during the sampling process. Data are presented to show that the residual effective stress of laboratory specimens of normally consolidated clay is typically less than one third of the value corresponding to perfect sampling, where only the *in situ* shear stresses are released.

Methods are proposed for adjusting the values of s_u , measured from *UU* and *CU* tests, to that corresponding to an undrained triaxial compression test on a specimen after perfect sampling. The *UU* specimens are treated as overconsolidated specimens, and the strength values are corrected accordingly. The Hvorslev parameters are used to correct *CU* strength data for the volume changes attendant with reconsolidation in the laboratory. An alternate approach for obtaining an adjusted s_u from *CU* tests employs test data on specimens consolidated to pressures much greater than the *in situ* overburden pressure.

Two case studies are presented in which the results of *CU* and *UU* tests, which had shown a 50 per cent discrepancy in strength, are analyzed in accordance with the procedures proposed in this paper. Following correction of the test data for sample disturbance, there was good agreement (within 5 to 10 per cent) between the strengths obtained from *UU* and *CU* tests.

This paper considers the determination of undrained shear strength, s_u of saturated "undisturbed" samples of clay from two types of laboratory tests, namely (1) one in which the soil specimen is not consolidated prior to testing but sheared at the existing moisture content, that is, unconsolidated-undrained test (*UU*), and (2) one in which the soil specimen is consolidated prior to testing, that is, consolidated-undrained test (*CU*).[2] A comparison of results from the two types of tests is given, and methods of adjusting the results from each, to put them on an equal basis, are described.

A determination of the factor of safety against a shear failure of a foundation, embankment, natural slope, retaining wall, etc., must usually consider

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[2] The letter symbols employed in this paper are defined where they first appear and given in Appendix I.

a failure in which the soil does not undergo consolidation during shear. The undrained shear strength is the soil parameter needed for this determination. The selection of the proper value of this soil parameter for a clay can be the step in the investigation which is the most difficult and the one which is most subject to large error.

To obtain the undrained shear strength of an element of clay in a field problem, the engineer would like to test in the laboratory a specimen of clay having the same moisture content and the same effective stress system that exist in the field element. Considerable experience has shown that, unfortunately, both the moisture content and the effective stresses on a field element cannot be duplicated simultaneously on a laboratory specimen. The engineer must therefore choose between the following approaches: (1) keep the moisture content of the laboratory specimen equal to that desired and run an undrained test (*UU*), or (2) make the effective stresses on the laboratory specimen equal to those desired and run an undrained test (*CU*). Because the moisture content and initial effective stress for the two tests are unequal, the strengths measured by the two tests are normally different. This paper presents test data from a wide variety of clays which show that strength values from *UU* tests are only 40 to 97 per cent of the values from *CU* tests.

The essential cause of the usually large and significant difference between *UU* and *CU* test results is the change in soil structure which occurs during the process of removing a chunk of soil from the ground, transporting it to the laboratory, trimming a test specimen, and mounting the specimen in the equipment for shearing. The next section of this paper considers the changes in soil structure as evidenced by changes in effective stress resulting from this entire process, termed here sampling.

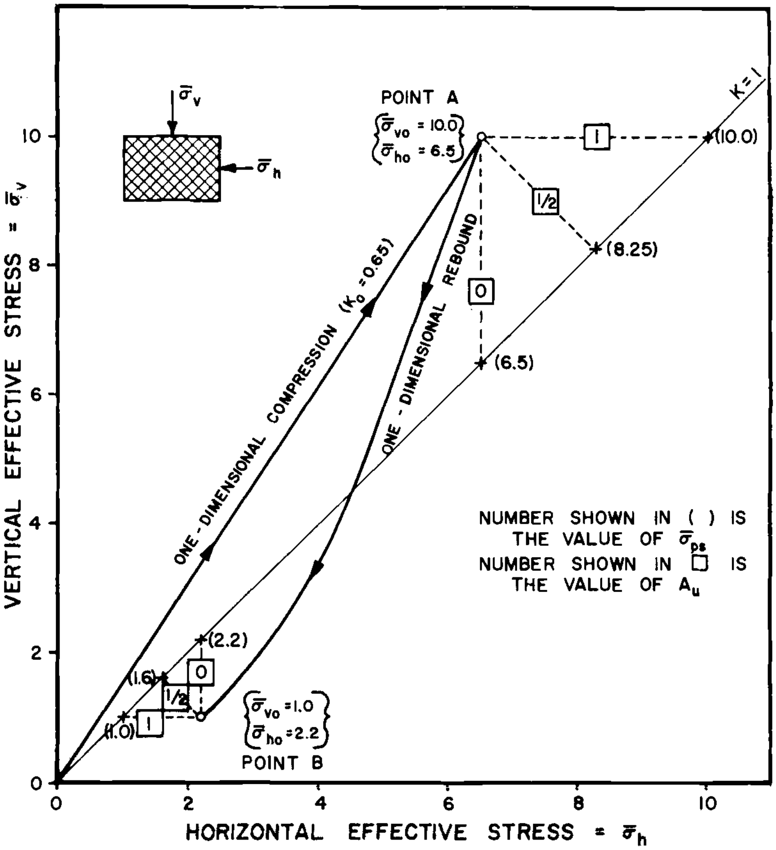
Methods are proposed for adjusting values of s_u measured from *UU* and *CU* tests to that corresponding to an undrained triaxial compression test on a specimen after perfect sampling, for which only the *in situ* shear stresses were released. It is emphasized that clays with a very sensitive structure, such as the quick clays and clays with a significant amount of natural cementation, are excluded from consideration. Furthermore it is not proposed that the adjusted value of s_u, which only corresponds to triaxial compression at normal strain rates, is necessarily the appropriate strength for a $\phi = 0$ analysis, since the effects of the intermediate principal stress, the possible reorientation of principal planes, different rates of strain, etc. are not taken into consideration. However, the methods suggested for evaluating the effects of sampling should enable the development, for many types of clay, of a more rational approach than now exists for predicting the strength in the field.

SAMPLING

Perfect Sampling:

The *in situ* stresses on a clay element are usually anisotropic, in that the horizontal and vertical stresses are not equal. Prior to running a laboratory triaxial shear test, the clay must of course be removed from the ground, taken to the lab, trimmed, and finally mounted in the test apparatus. The term perfect sampling denotes this process where no disturbance has been given to the specimen other than that involved with the release of the *in situ* shear stresses.

The ratio of horizontal to vertical effective stress in a one dimensionally consolidated horizontal clay deposit is denoted by $K_o = \bar{\sigma}_{hc}/\bar{\sigma}_{vc}$. For normally consolidated clays, K_o has been found by Bishop (1)[3] and Simons (2) to agree substantially with Jaky's (3) expression for the K_o of granular systems, that is, pressures). Skempton (4) found for the highly plastic London clay that K_o reached unity at an *OCR* of about 2.2 and attained values of 2 to 3 at an *OCR* exceeding about 8.



The one-dimensional compression and rebound curves simulate K_o data for the London clay Skempton (4).

FIG. 1—Perfect Sampling of a Normally Consolidated Clay and an Over-consolidated Clay.

$K_o = 1 - \sin \bar{\phi}$, where $\bar{\phi}$ equals the friction angle at maximum obliquity. For overconsolidated clays, K_o increases with increasing overconsolidation ratio ($OCR = \bar{\sigma}_{vm}/\bar{\sigma}_{vc}$ = ratio of maximum past to existing vertical consolidation

The isotropic effective stress after perfect sampling, $\bar{\sigma}_{ps}$, of a saturated clay which had *in situ* vertical and horizontal consolidation pressures of $\bar{\sigma}_{vo}$ and $K_o\bar{\sigma}_{vo}$, respectively, is given by

$$\bar{\sigma}_{ps} = \bar{\sigma}_{vo}[K_o + A_u(1 - K_o)] \quad \ldots\ldots(1)$$

where $A_u = (\Delta u - \Delta\sigma_h)/(\Delta\sigma_v - \Delta\sigma_h) =$

[3] The boldface numbers in parentheses refer to the list of references appended to this paper.

CLAY	SYMBOL		DEPTH (m)	$\bar{\sigma}_{vo}$	$\bar{\sigma}_{1c}$	$\bar{\sigma}_{3c}$	A_u	$\bar{\sigma}_{ps}$	$\bar{\sigma}_{ps}/\bar{\sigma}_{1c}$
	$\overline{CAU}$	CA-$\overline{UU}$							
I	○	●	21	1.6	3.20	1.50	+.065	1.61	0.505
II	□	■	35	2.6	5.40	2.50	+.275	3.30	0.610
III	△	▲	45	3.5	7.55	3.50	+.115	3.97	0.525

ALL STRESSES IN kg/cm^2

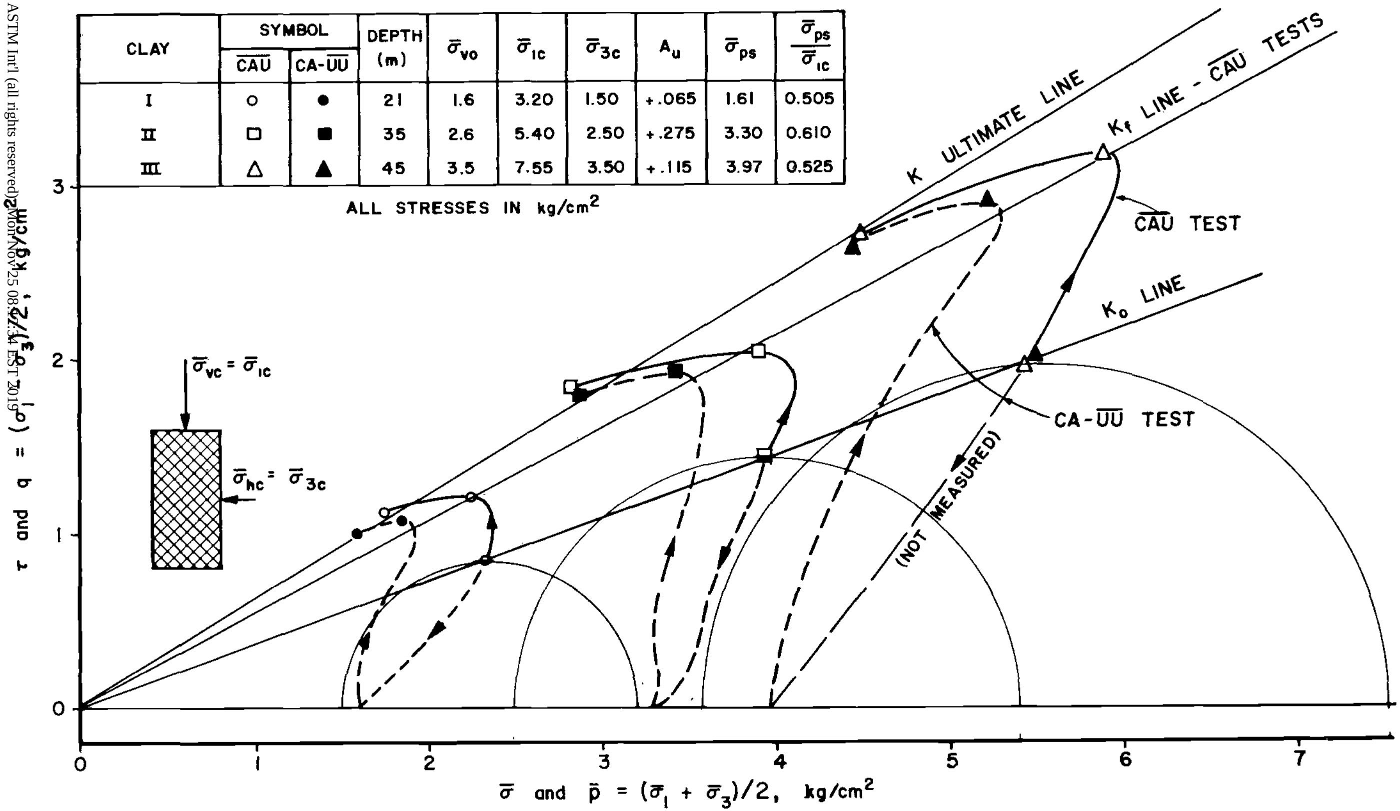


FIG. 2.—Effect of Perfect Sampling on Stress Paths for Normally Consolidated Kawasaki Clays.

TABLE 1—STRESS RATIOS FOR PERFECT SAMPLING.

Type of Specimen	K_o	A_u	$\bar{\sigma}_{ps}/\bar{\sigma}_{vo}$
Normally consolidated			
Clayey Silt	0.4 to 0.5	−0.1 to 0	0.35 to 0.5
Lean Clay	0.5 to 0.6	0.1 to 0.2	0.55 to 0.7
Plastic Clay	0.6 to 0.7	0.2 to 0.3	0.65 to 0.8
Heavily overconsolidated			
Plastic Clay	~2.5	~0.3	~2

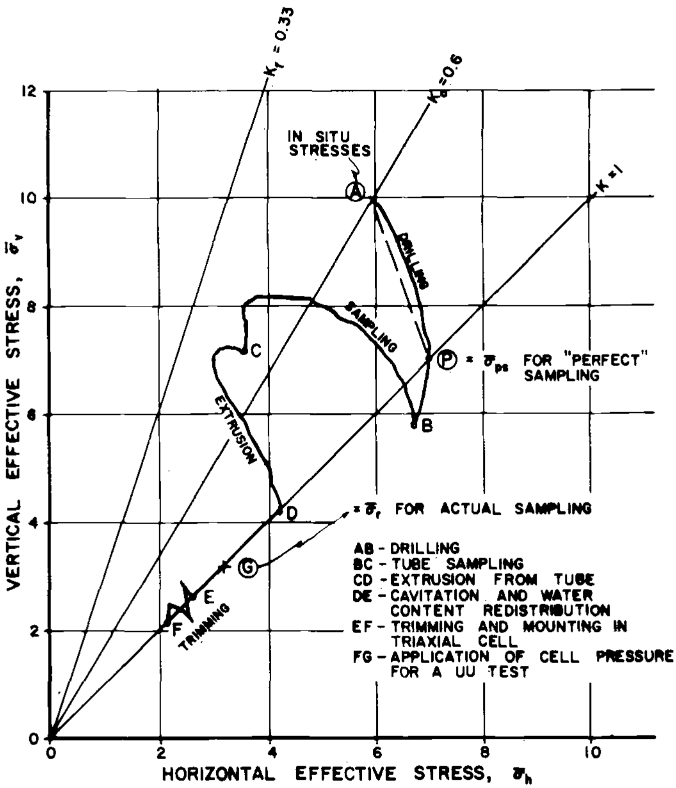


FIG. 3—Hypothetical Stress Path for a Normally Consolidated Clay Element During Tube Sampling.

an A parameter[4] for the undrained release of the shear stresses which existed at the K_o condition by changing the horizontal and vertical stresses in order to achieve an isotropic stress system.

[4] A_u is defined in terms of horizontal and vertical stresses in order to be consistent with the definition of K_o. Skempton's B parameter (5) is taken equal to unity.

The relationship between $\bar{\sigma}_{ps}$ and $\bar{\sigma}_{vo}$ is illustrated in Fig. 1 for normally consolidated (Point A) and highly overconsolidated (Point B) specimens of a hypothetical clay for three different values of A_u. The straight line from the origin to Point A indicates a constant K_o of 0.65 for normally consolidated clay; the curved line from point A to Point B

shows an increasing value of K_o as the OCR increases (K_o = 2.2 for OCR = 10 at Point B). The figure shows that $\bar{\sigma}_{ps}/\bar{\sigma}_{vo}$ for normally consolidated clay will always be less than unity for A_u values less than one; the reverse is true for overconsolidated specimens with $K_o \geqq 1$, that is, $\bar{\sigma}_{ps}/\bar{\sigma}_{vo}$ will be greater than unity for A_u values less than one.

Figure 2 illustrates the effects of perfect sampling on stress paths for a group of clays from Kawasaki, Japan. Pairs of specimens were normally consolidated in triaxial cells under approximately the K_o condition. One specimen of each pair was sheared undrained with pore pressure measurements by increasing the axial (vertical) pressure ($C\overline{AU}$ test). The second specimen was first unloaded by decreasing the axial pressure (that is, perfect sampling), and then loaded by increasing the axial pressure with both steps being done under undrained conditions with pore pressure measurements (CA-$\overline{UU}$ test). The resulting values of the ratio $\bar{\sigma}_{ps}/\bar{\sigma}_{1c}$ were 0.56 ± 0.05 with

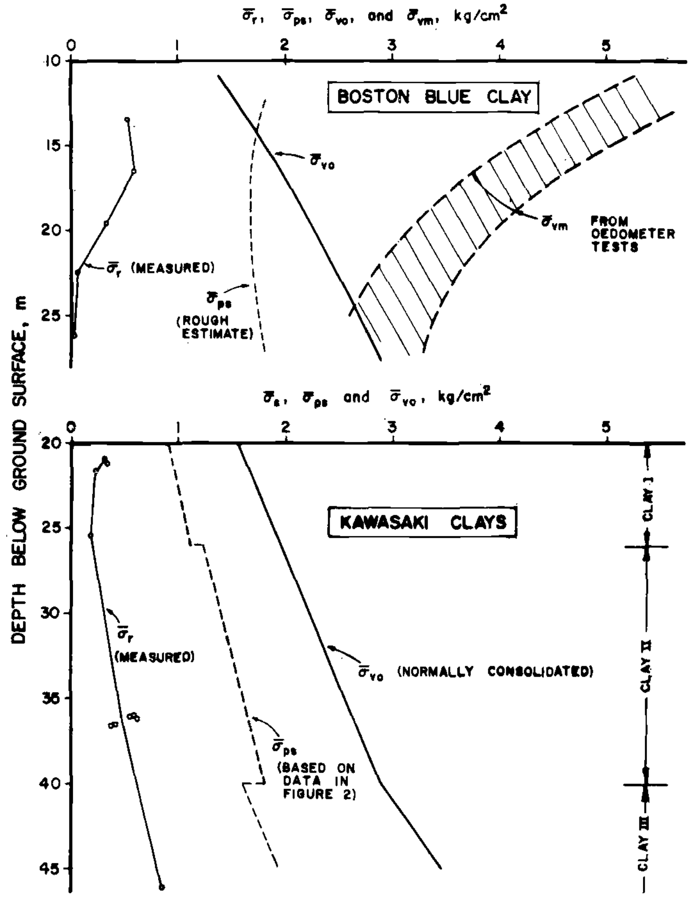


FIG. 4—Effect of Tube Sampling on Effective Stresses for Boston Blue Clay and the Kawasaki Clays.

TABLE 2—COMPARISON OF UNDRAINED STRENGTH FROM *UU* AND *CU* TESTS.

Case	Location (1)	Description of Clay (2)	Depth, m (3)	From *U* or *UU* Tests s_u, kg/cm^2 (4)	From *U* or *UU* Tests $s_u/\bar{\sigma}_{vo}$ (5)	$\frac{s_u\ (UU)}{s_u(CIU,\ \bar{\sigma}_c = \bar{\sigma}_{vo})}$ (6)	Remarks (7)	Source (8)
A	M.I.T., Cambridge, Mass.	o.c.[a] Boston blue clay o.c. Boston blue clay (slightly) n.c.[a] Boston blue clay	12.2 18.3 27.4	0.80 0.68 0.50	0.59 0.37 0.185	0.74 0.77 0.58	3 in. diameter shelby fixed piston samples	M.I.T.
B	Lagunillas, Venezuela	n.c. plastic clay $w_L \cong 61\%$ $P.I. \cong 37\%$	6.2	0.18	0.30	0.75	3 in. diameter shelby samples. s_u (*UU*) based on average of *U* and *UU* data	M.I.T.
C	Kawasaki, Japan	normally consolidated plastic clay Clay I, $P.I.^c \cong 31\%$ Clay I, $P.I. \cong 36\%$ Clay II, $P.I. \cong 43\%$	20.5 25 35	0.50 0.49 0.69	0.31 0.26 0.27	0.64 0.58 0.57	3 in. diameter shelby samples. s_u (*UU*) from top one-third of *U* and from *UU* data corrected to t_f = 5 hr	M.I.T.
D	Gulf of Mexico	o.c. soft plastic clay $P.I. \cong 80\%$ $L.I. \cong 50\%$	0 to 6	avg. $\cong$ 0.27	...	~0.85	Shelby samples	Fenske (**11**)
		firm plastic clay with silt and sand $P.L. \cong 60\%$ $L.I. \cong 35\%$	18 to 50	avg. $\cong$ 0.50	...	~0.5		
E	Skabo, Oslo, Norway	n.c. plastic clay with a high salt content $P.I. \cong 30\%$ $L.I. \cong 65\%$ $S_t \cong 5$	10.6 to 16	avg. = 0.32	0.31	0.74	2.1 in. diameter thin walled fixed piston samples	N.G.I.Internal Report No. F175 (1962)
F	Göta Valley, Sweden	Lilla Edet Clay highly o.c. $P.I. \cong 30\%$ slightly o.c. $P.I. \cong 33\%$ slightly o.c. $P.I. \cong 29\%$ slightly o.c. $P.I. \cong 37\%$	4 to 6.8 10 to 12.3 16.2 to 18 10.8 to 12.8	avg. $\cong$ 0.4 avg. = 0.34 avg. = 0.46 avg. = 0.13	~1.20 0.45 0.42 0.13	~0.9 0.60 0.66 0.40	2.1 in. diameter thin walled fixed piston samples. Clay believed to have a significant amount of natural cementation	Bjerrum and Wu (**12**)

G	Mexico City	Mexico City Clay, slightly o.c.	...	...	...	0.74	...	Marsal (**13**)
H	Drammen, Norway	normally consolidated soft silty clay with thin seams of silt and fine sand $P.I. \cong 8\%$ $S_t = 10$ $P.I. \cong 16\%$ $S_t = 9$ $P.I. \cong 14\%$ $S_t = 9$	 5 12 18	 0.25 0.25 0.24	 0.37 0.19 0.13	 0.97 0.57 0.40	2.1 in. diameter thin walled, fixed piston samples. s_u (UU) based on U samples with lowest strain at failure	Simons (**14**)
I	Sault Ste. Marie, Mich.	n.c. varved clay $P.I. \cong 28\%$ $S_t \cong 8$	~9	~0.3	~0.15	~0.60	3.5 in. diameter thin walled piston. $s_u(CIU)$ based on $s_u/\bar{\sigma}_c$ for $\bar{\sigma}_c > \bar{\sigma}_{vo}$	Wu (**15**); Wu *et al* (**16**)

[a] O.C. = overconsolidated.
[b] N.C. = normally consolidated.
[c] *P.I.* = plasticity index.

corresponding A_u values of 0.17 ± 0.10. Similar test data on normally consolidated Boston Blue Clay (plasticity index = 14 per cent; $K_o \approx 0.54$) yielded $\bar{\sigma}_{ps}/\bar{\sigma}_{1c}$ = 0.59 and A_u = 0.11.

What are thought to be typical values of K_o, A_u, and $\bar{\sigma}_{ps}/\bar{\sigma}_{vo}$ are suggested in Table I based on the above and other rather limited data by Bishop and Henkel (6) and by Skempton (4). As shown in Table I, the effective stress of perfect samples will be only 35 to 80 per cent of the *in situ* vertical effective stress $\bar{\sigma}_{vo}$ for normally consolidated clays, but may be double $\bar{\sigma}_{vo}$ for a highly overconsolidated plastic clay.

Actual Sampling:

The ideal sampling process would involve neither a change in moisture content nor a change in magnitude and distribution of effective stresses upon removing a specimen of clay from the field and placing it in the shear test equipment. Such sampling is essentially impossible and the best that can be hoped for is what has been termed in this paper perfect sampling, that is, the removal of the shear stresses acting on the element in the field to achieve the isotropic stress $\bar{\sigma}_{ps}$. The effects of perfect sampling on the undrained shear behavior in triaxial compression of a normally consolidated clay were illustrated in Fig. 2.

The actual sampling process offers many opportunities for additional disturbance of the soil structure according to Rutledge (7), Hansen and Gibson (8), and Hvorslev (9). Some of these effects are illustrated in Fig. 3, which presents a hypothetical stress path for an element of normally consolidated clay during tube sampling. The Point P with an effective stress of $\bar{\sigma}_{ps}$ corresponds to perfect sampling, whereas Point G with an effective stress of $\bar{\sigma}_r$ represents the effective stress of an actual specimen at the start of a $\overline{UU}$ shear test. It is proposed that the ratio $\bar{\sigma}_r/\bar{\sigma}_{ps}$ is a measure of the amount of additional disturbance caused by actual sampling

Measured values of the residual effective stress after sampling, $\bar{\sigma}_r$, are presented in Fig. 4 for an overconsolidated deposit of Boston Blue Clay and for normally consolidated strata of Kawasaki clays (see Table 2 for further data on these clay deposits). The apparatus for $\overline{UU}$ tests described in the next section was used to measure $\bar{\sigma}_r$ with a total chamber pressure generally between 1 and 3 kg/cm^2, for which the B parameter became essentially equal to unity. Values of the maximum past pressure, $\bar{\sigma}_{vm}$, *in situ* vertical effective stress, $\bar{\sigma}_{vo}$, and effective stress for perfect sampling, $\bar{\sigma}_{ps}$, are shown for comparison. The ratio $\bar{\sigma}_r/\bar{\sigma}_{ps}$, which is proposed as a measure of the degree of excessive disturbance, ranged from 0.11 to 0.43 with an average value of 0.28 for the Kawasaki clays. Values of $\bar{\sigma}_r/\bar{\sigma}_{ps}$ for the deposit of Boston Blue Clay, which ranged from 0.01 to 0.34, showed a marked decrease with increasing depth and decreasing overconsolidation ratio. Partial drying of the deepest specimen of Boston Blue Clay (depth = 26.2m) revealed a grossly distorted structure which corroborates the very low value of $\bar{\sigma}_r$ of only 0.02 kg/cm^2. Data from two $\overline{UU}$ tests on normally consolidated Lagunillas clay (case B in Table 2) yielded values of $\bar{\sigma}_r/\bar{\sigma}_{ps}$ equal to 0.36 ± 0.07.

These limited data show that the effective stress of laboratory specimens of normally consolidated and slightly overconsolidated clays after tube sampling may be lowered by excessive disturbance to a value of only 20 ± 20 per cent of the theoretical value for perfect sampling. It is therefore suggested that the measurement of the residual effective stress, $\bar{\sigma}_r$, of laboratory specimens should become a standard test for jobs

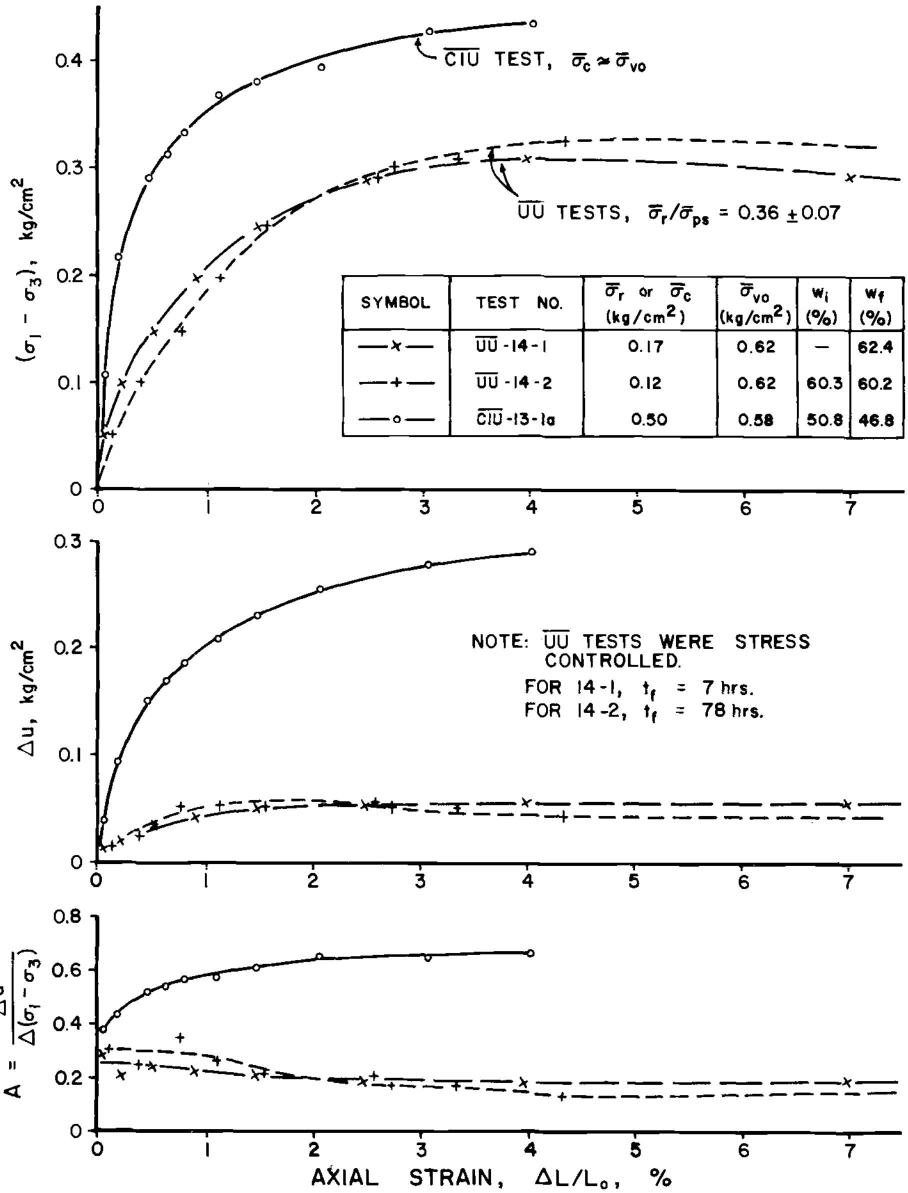


FIG. 5—Stress-Strain Data from Triaxial Tests on Lagunillas Clay.

requiring a rational interpretation of lab strength data, especially for deep tube samples of normally consolidated clay. The value of $\bar{\sigma}_r$ can be obtained from a direct measurement of the residual pore pressure u_r, (that is, u at zero confining pressure; Lambe (10) describes several methods for measuring u_r) provided that $\bar{\sigma}_r$ is less than one atmosphere, although the use of a confining pressure of several atmospheres is preferable since the B parameter is generally some-

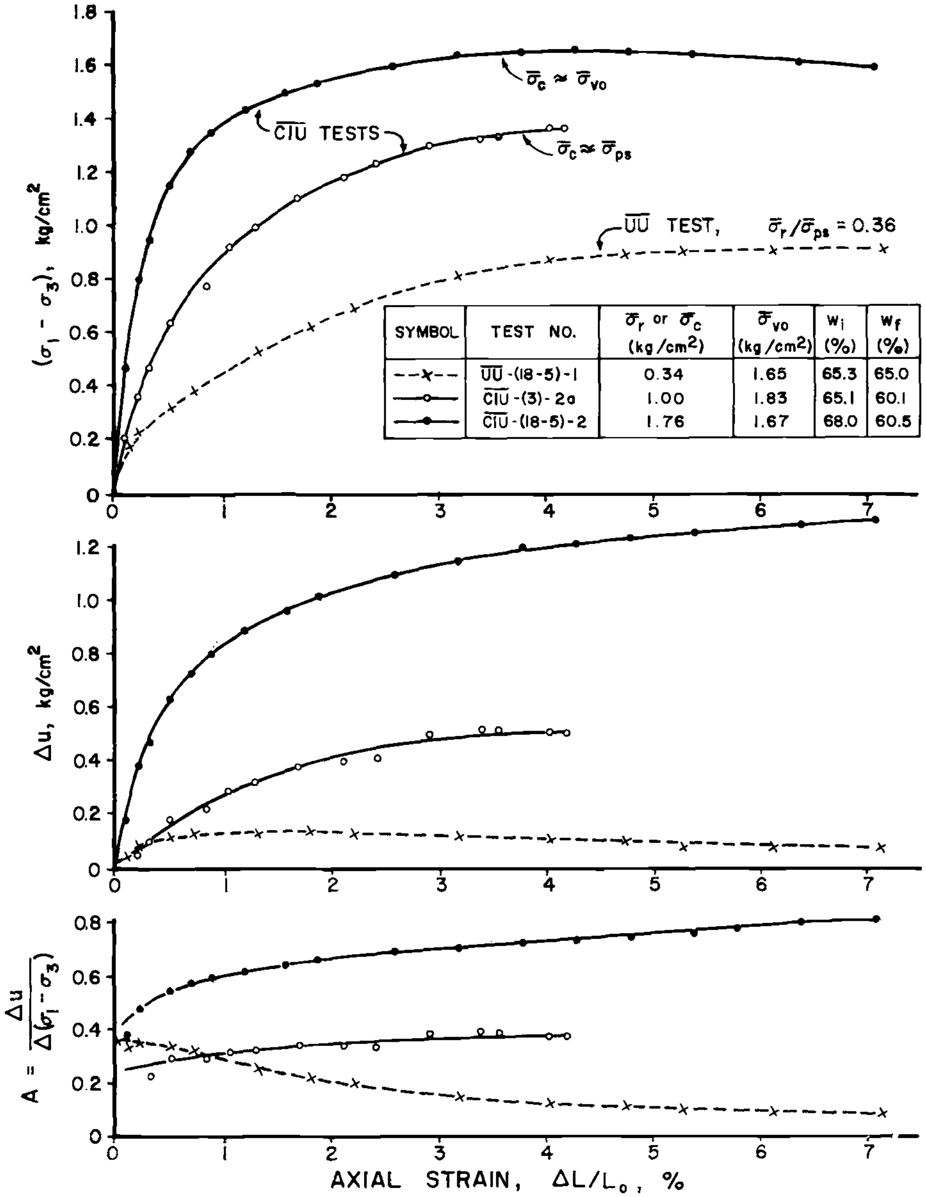


FIG. 6—Stress-Strain Data from Triaxial Tests on Kawasaki Clay I.

what less than unity.[5] Skempton (4) suggests some indirect methods for evaluating $\bar{\sigma}_r$.

[5] In this case, UU tests with confining pressure sufficient to make B equal unity are preferable to unconfined compression tests.

Experience at Massachusetts Institute of Technology has shown, for example, that the size of the specimen, the amount of trimming, and the placement of the specimen in an oedometer ring can affect the value of $\bar{\sigma}_r$. In other words, the

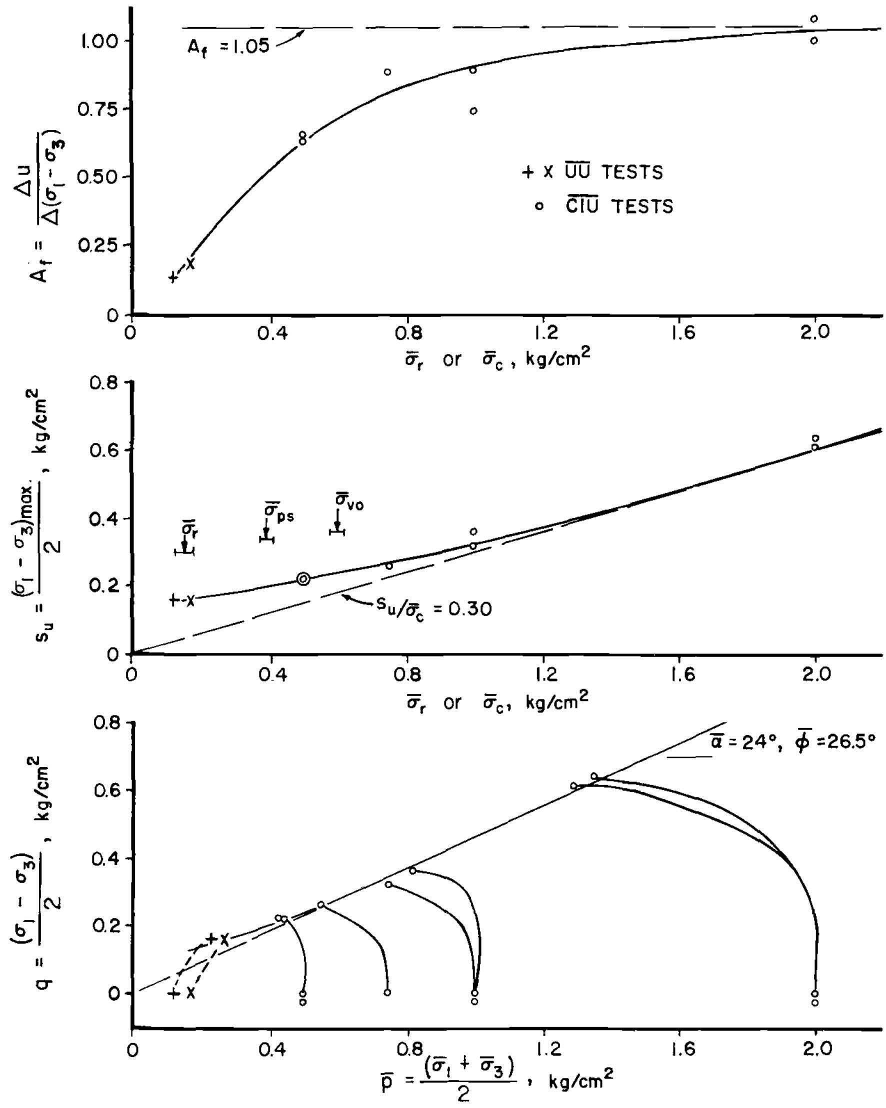


FIG. 7—Strength Data from Triaxial Tests on Lagunillas Clay.

value of $\bar{\sigma}_r$, and hence the degree of disturbance, are really variables.

COMPARISON OF UU AND CU TEST RESULTS

Table 2 presents test data on nine soils from various locations in the world. Most of the soils are normally consolidated or slightly overconsolidated, $\bar{\sigma}_{vm}/\bar{\sigma}_{vo}$ equal to less than 3, lean to plastic clays with sensitivities below ten. Specimen depths varied from 0 to 35 m below ground surface. The strength values in Columns 4 and 5 are the averages from unconfined (U tests) and UU triaxial compression tests except for Case C as noted. The results of vane tests were

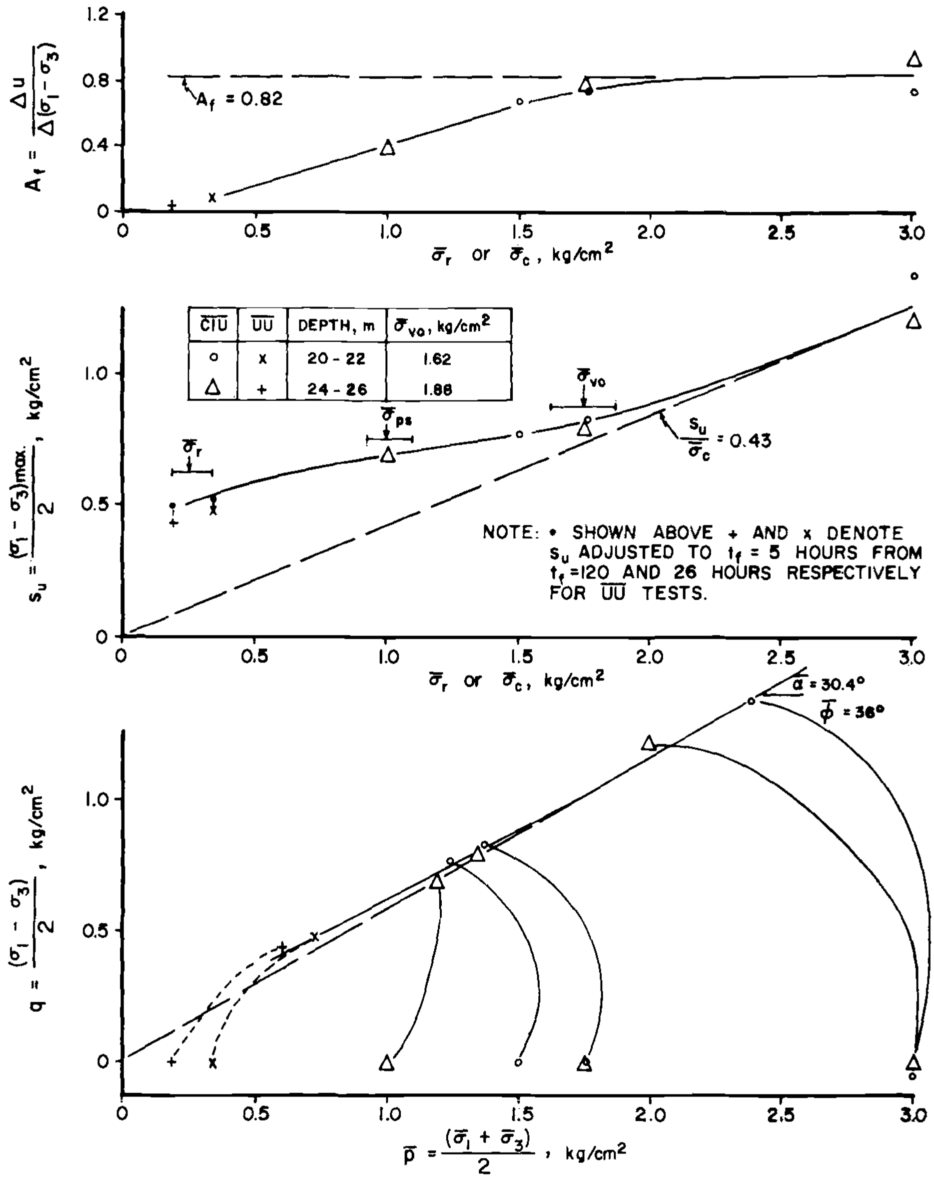


FIG. 8—Strength Data from Triaxial Tests on Kawasaki Clay I.

not included in Table 2 since the interpretation of vane data is open to question.[6]

As Column 6 of Table 2 shows, the undrained strength as obtained from *UU* tests varies from 40 to 97 per cent of the strength as obtained from *CIU* tests with $\bar{\sigma}_c = \bar{\sigma}_{vo}$, with an average value of

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[6] The ratio of s_u (*UU*) to vane strength averaged 0.85 (0.45 to 2.0) for Cases *B*, *C*, *D*, *F*, *H*, and *I* in Table 2.

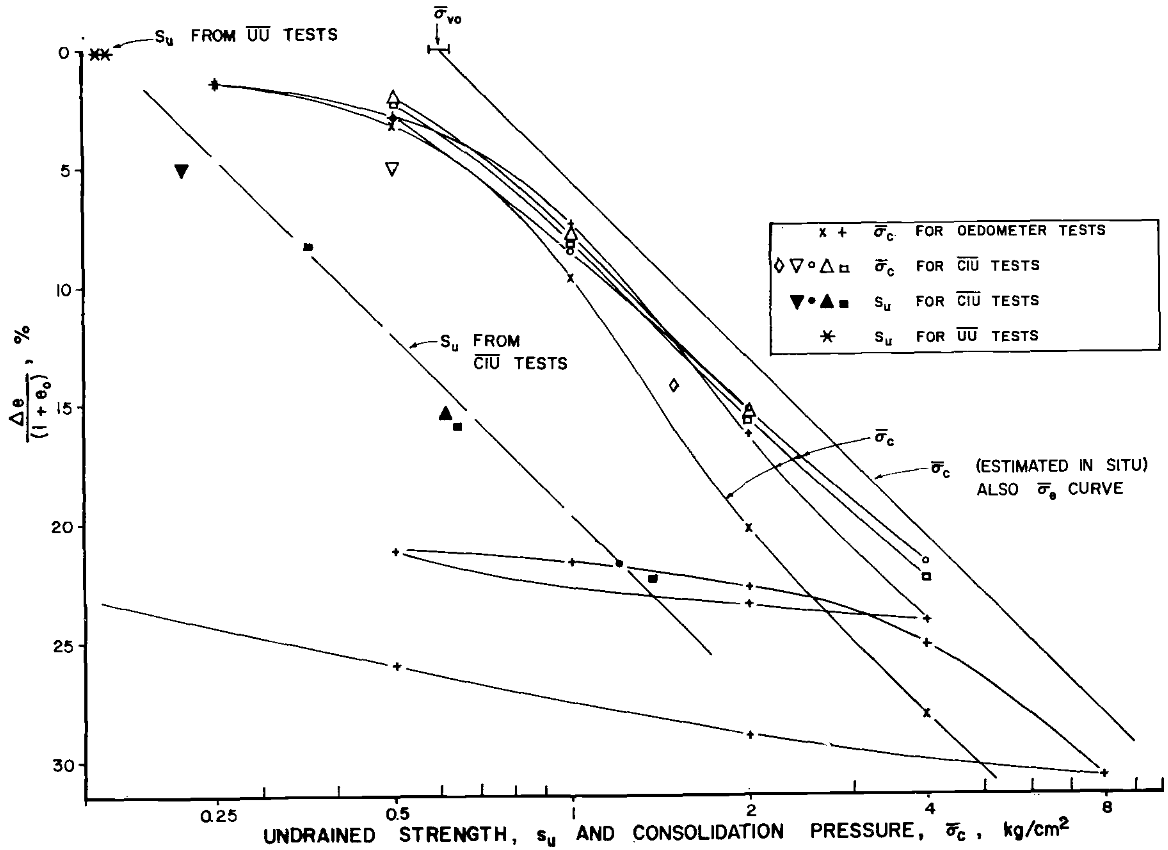


FIG. 9—Strength and Consolidation Pressure Versus Volumetric Strain on Lagunillas Clay.

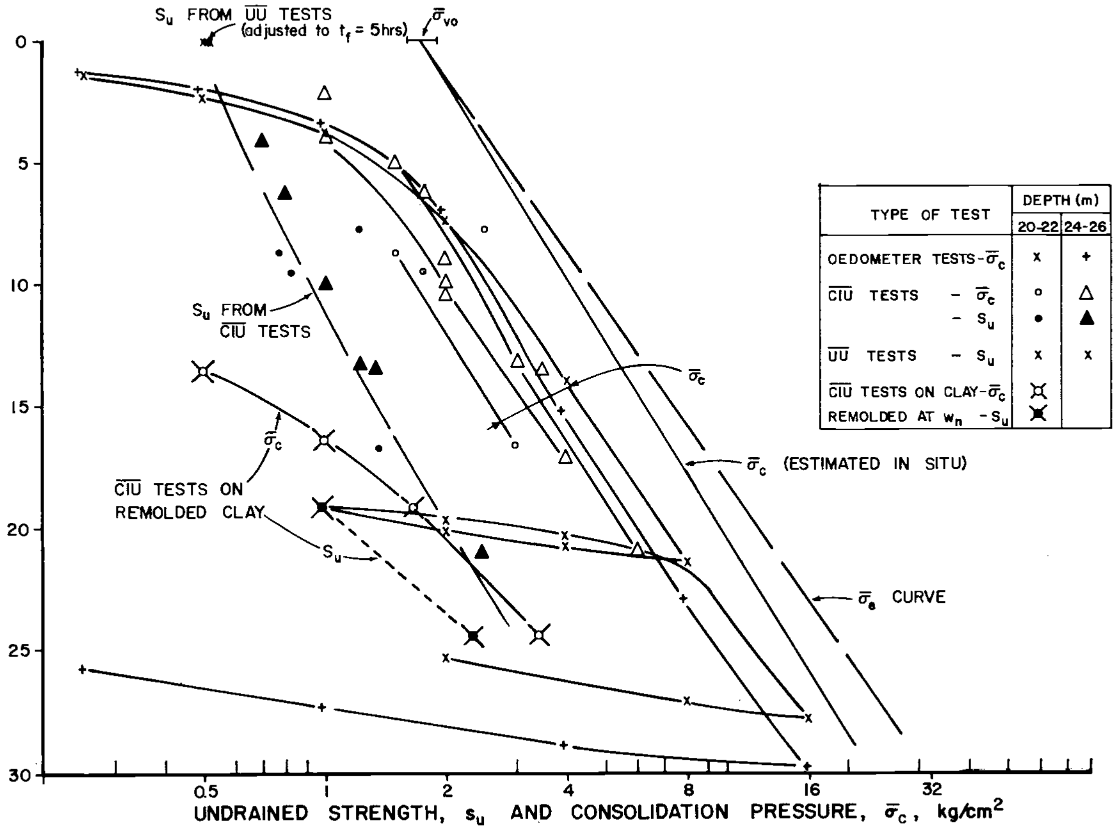


FIG. 10—Strength and Consolidation Pressure Versus Volumetric Strain on Kawasaki Clay I.

66 per cent. In general, this ratio decreased with increasing depth of specimen.

A more detailed comparison of the results of $\overline{UU}$ and $\overline{CIU}$ tests on the Lagunillas clay, Case *B* of Table 2, and the Kawasaki clays, Case *C* of Table 2, is given in Figs. 5 to 10. The tests on these normally consolidated clays were run on trimmed specimens 10 cm^2 in area by 8 cm high using either the Norwegian Geotechnical Inst. (NGI) equipment (**17**) or English equipment (**18**). The NGI null device was used for the $\overline{CIU}$ tests; filter strips were placed on the test specimens, and backpressures of 1 to 3 kg/cm^2 were employed. The rate of strain was about 1 per cent/hr. A filter strip correction of 0.10 kg/cm^2 was subtracted from measured values of $(\sigma_1 - \sigma_3)$ for axial strains exceeding 2 per cent. The $\overline{UU}$ tests employed a fine porous base stone and Dynisco pressure transducer (**19**), no filter strips, a cell pressure of several kg/cm^2, and a time to failure generally exceeding one day. All triaxial specimens were encased with two prophylactics with silicone grease; measured values of the *B* parameter before shear always exceeded 0.95 within 2 min.

The data in Figs. 5 through 10 show the following results:[7]

1. The undrained shear strength, s_u, from $\overline{UU}$ tests is only 70 to 85 per cent of the value from $\overline{CIU}$ tests consolidated to the theoretical pressure corresponding to perfect sampling, that is, $\bar{\sigma}_c = \bar{\sigma}_{ps}$ (Figs. 7 and 8). This difference in s_u arises from the fact that the residual effective stress $\bar{\sigma}_r$ of the laboratory shear specimens is considerably less than the theoretical value $\bar{\sigma}_{ps}$. Identical strengths would have resulted if $\bar{\sigma}_r$ had equalled $\bar{\sigma}_{ps}$ since the preshear $\bar{\sigma}$ and void ratios would therefore have been equal for both tests.

2. The slope of the stress-strain curve from $\overline{UU}$ tests is approximately one half of that from $\overline{CIU}$ tests with $\bar{\sigma}_c = \bar{\sigma}_{ps}$ (Figs. 5 and 6).

3. There is good agreement among $\overline{UU}$ and $\overline{CIU}$ strength data if the $\overline{CIU}$ data are extrapolated backward so that the consolidation pressure equals $\bar{\sigma}_r$ (Figs. 7 and 8) or the volumetric strain from the field condition is zero (Figs. 9 and 10). This is to be expected since as the $\bar{\sigma}_c$ of $\overline{CIU}$ tests increased beyond $\bar{\sigma}_r$, there are corresponding increases in preshear $\bar{\sigma}$ and volumetric strain.

4. As the consolidation pressure of $\overline{CIU}$ tests is increased from $\bar{\sigma}_{ps}$ to several times the *in situ* vertical effective stress $\bar{\sigma}_{vo}$, A_f shows a large increase and $s_u/\bar{\sigma}_c$ exhibits a marked decrease, whereas changes in the effective stress envelope are relatively minor (Figs. 7 and 8).

5. A relatively large volume decrease occurs when specimens are isotropically consolidated to pressures between $\bar{\sigma}_{ps}$ and $\bar{\sigma}_{vo}$ (Figs. 9 and 10). Moreover, the oedometer test data[8] show that low values of $\bar{\sigma}_r$, indicating high disturbance, rather than an isotropic stress system *per se*, are primarily responsible for the volume decrease.

These five trends are believed to be typical of tube specimens of normally consolidated, moderately sensitive, plastic clays which do not possess significant natural cementation.

In summary:

1. Disturbance during tube sampling of normally consolidated clays may com-

[7] Measured or estimated values of $\bar{\sigma}_r$, $\bar{\sigma}_{ps}$ and $\bar{\sigma}_{vo}$ are generally shown in the figures for reference.

[8] The fact that one of the oedometer test *e* versus log $\bar{\sigma}_c$ curves in Fig. 9 fell considerably below the curves for isotropic consolidation at high pressures is not considered typical.

monly decrease the effective stress of lab specimens by 80 ± 20 per cent compared to perfect sampling.

2. The resultant low values of $\bar{\sigma}_r$ cause the undrained strength s_u from *UU* tests to be too low, whereas s_u from *CIU* tests, which experience a significant volume decrease during consolidation, will most likely be too large.

3. The ratio of s_u from *UU* tests to that from *CIU* tests with $\bar{\sigma}_c = \bar{\sigma}_{ps}$ will typically be 75 ± 25 per cent.

EXAMINATION OF THE UNCONSOLIDATED-UNDRAINED TEST[9]

The test data in Figs. 5 through 8 indicate that soil tested in unconsolidated-undrained shear resembles, in several respects, the results of tests on overconsolidated clays. In particular, the $\overline{UU}$ data show:

1. Values of undrained shear strength for a given initial effective stress $\bar{\sigma}_r$ which are above the corresponding values for normally consolidated clay;

2. Effective stress paths during shear which move upward and to the right since the pore pressure parameter *A* is less than one half, a characteristic of overconsolidated clay;

The similarity between the results of $\overline{UU}$ tests and tests on overconsolidated specimens is presented in Fig. 11 for Kawasaki clays. In Fig. 11(*b*) the *CA*-$\overline{UU}$ tests represent the undrained strength of perfect specimens, preshear $\bar{\sigma} = \bar{\sigma}_{ps}$, whereas the $\overline{UU}$ tests show the strength of actual specimens, preshear $\bar{\sigma} = \bar{\sigma}_r$. This figure shows that the effective stress paths for the $\overline{UU}$ tests, Fig. 11(*b*), when normalized by dividing by $\bar{\sigma}_{ps}$, are similar in shape to the effective stress paths of $\overline{CIU}$ tests on overconsolidated clays shown in Fig. 11(*a*), when normalized by dividing by $\bar{\sigma}_{cm}$. In other words, the reduction in effective stress from $\bar{\sigma}_{ps}$ to $\bar{\sigma}_r$ caused by sample disturbance and the reduction in effective stress from $\bar{\sigma}_{cm}$ to $\bar{\sigma}_c$ caused by rebound appear to have comparable effects on undrained strength.

This analogy suggests that the strength from *UU* tests can be considered as test results on overconsolidated specimens with a maximum past pressure equal to $\bar{\sigma}_{ps}$, the value of effective stress existing before gross sample disturbance occurred (see Point *P* in Fig. 3). Thus the corrected value of s_u from *UU* tests, which will correspond to the strength of a specimen after perfect sampling, can be estimated by treating the ratio of $\bar{\sigma}_{ps}$ to $\bar{\sigma}_r$ as an overconsolidation ratio.

The proposed method of correcting the undrained shear strength for sample disturbance involves the following three steps:

1. Relate undrained shear strength to overconsolidation ratio using *CIU* tests on specimens with $\bar{\sigma}_{cm}$ much greater than $\bar{\sigma}_{vo}$ to reduce the effects of disturbance,

2. Find the equivalent overconsolidation ratio for the *UU* test being corrected by measuring $\bar{\sigma}_r$ and calculating $\bar{\sigma}_{ps}$ (use Eq 1 and *CA*-$\overline{UU}$ data or estimates from Table 1), and

3. Obtain the shear strength correction for the particular overconsolidation ratio as described below.

Figure 12 presents the ratio of undrained shear strength at $\bar{\sigma}_c$ to the strength at $\bar{\sigma}_{cm}$ versus overconsolidation ratio for *CIU* tests on several natural and remolded clays. As can be seen, the strength ratio decreases significantly with an increase in OCR. Considering the ratio of $\bar{\sigma}_{ps}$ to $\bar{\sigma}_r$ equivalent to OCR, one can read from such a figure the loss in shear strength from excessive sample

[9] The following two sections on *UU* and *CU* Tests are restricted to normally consolidated clays of moderate sensitivity, although the concepts can be extended to overconsolidated clays.

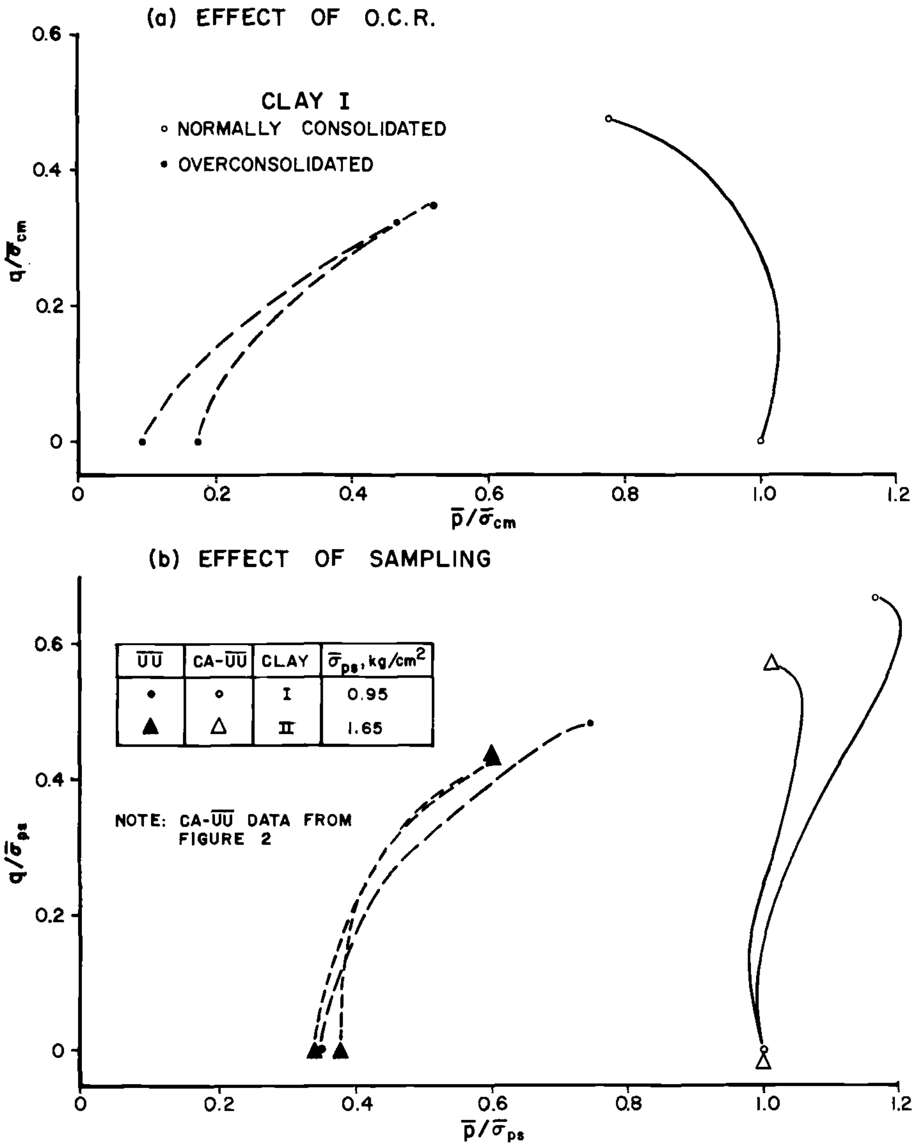
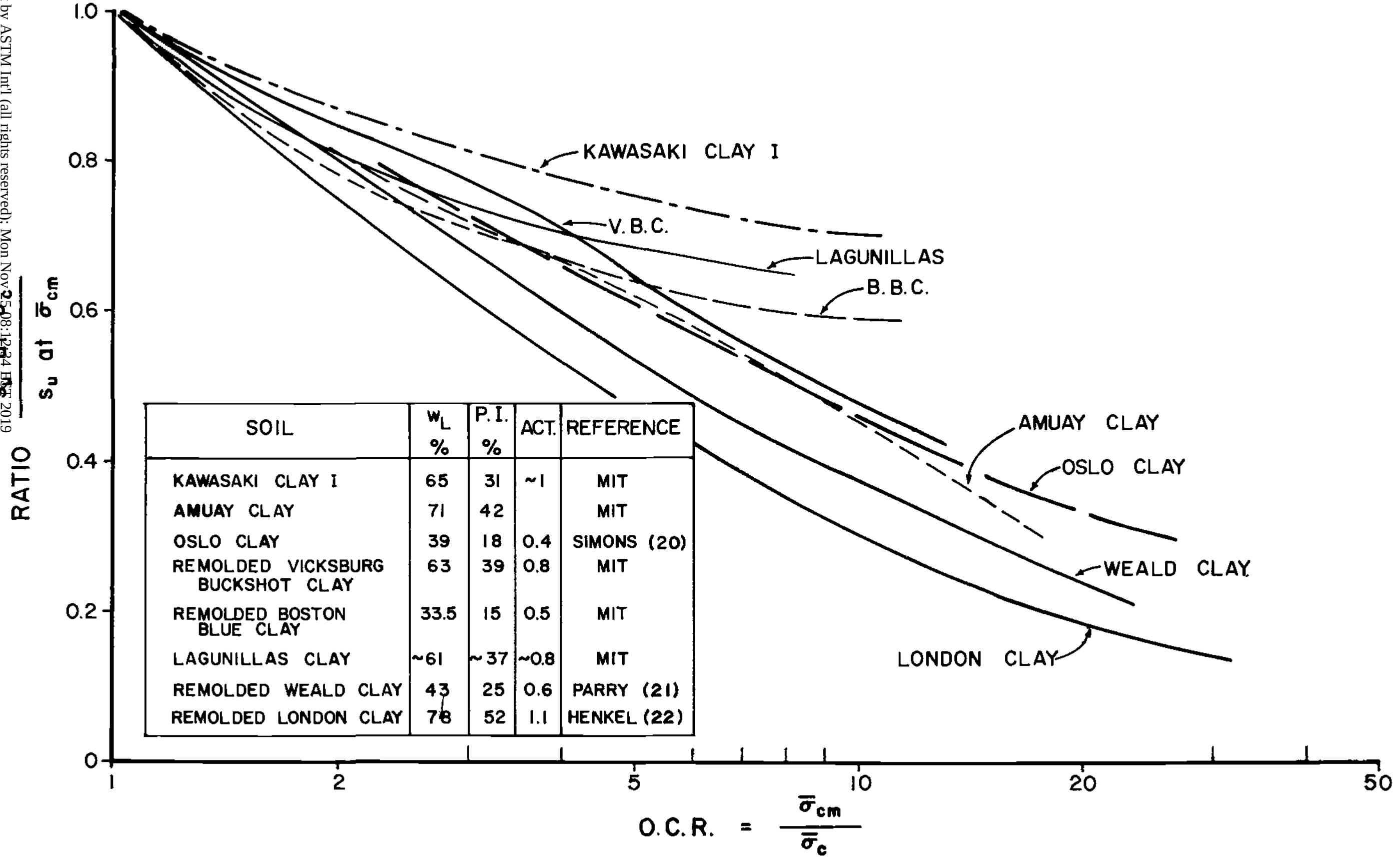


FIG. 11—Stress Paths for Kawasaki Clays.

disturbance which reduces the effective stress from $\bar{\sigma}_{ps}$ to $\bar{\sigma}_r$. From the residual effective stress data already discussed, a typical value of $\bar{\sigma}_r/\bar{\sigma}_{ps}$ might be $\frac{1}{3}$ to $\frac{1}{4}$; the equivalent OCR would then be 3 to 4. Using the data in Fig. 12, one sees that measured values of *UU* strength would therefore be 20 to 50 per cent too low depending on the type of clay.

Considering a *UU* test as a test on an overconsolidated specimen is thought to be an important concept. To correct the results of *UU* tests by use of an equivalent overconsolidation ratio is proposed



SOIL	w_L %	P.I. %	ACT.	REFERENCE
KAWASAKI CLAY I	65	31	~1	MIT
AMUAY CLAY	71	42		MIT
OSLO CLAY	39	18	0.4	SIMONS (20)
REMOLDED VICKSBURG BUCKSHOT CLAY	63	39	0.8	MIT
REMOLDED BOSTON BLUE CLAY	33.5	15	0.5	MIT
LAGUNILLAS CLAY	~61	~37	~0.8	MIT
REMOLDED WEALD CLAY	43	25	0.6	PARRY (21)
REMOLDED LONDON CLAY	78	52	1.1	HENKEL (22)

FIG. 12—Effect of OCR on Undrained Strength.

as an approximate engineering approach. There are both theoretical and practical reasons why such an approach is not precise. Of greatest consequence, it is probably unwarranted to assume with the present state of knowledge that all of the detrimental effects of sampling can be expressed simply as a reduction in effective stress. Further, accurate methods for determining $\bar{\sigma}_{ps}$ are not currently known. Had the specimen existed at a possibly varying K_o effective stress system for a geological time rather than for the time used in the laboratory for a $CA\text{-}\overline{UU}$ test, the value of effective stress following perfect sampling might be different. There is also the added problem of the variability in $\bar{\sigma}_r$ measurements. However, to the authors' knowledge, no other method exists for the quantitative evaluation of the very significant effects of disturbance on *UU* strengths.

EXAMINATION OF THE CONSOLIDATED-UNDRAINED TEST

One widely used technique to estimate the undrained shear strength s_u of an element of clay in the ground is to isotropically consolidate the specimen in the laboratory to a pressure equal to the effective vertical pressure in the ground, and then test the soil in undrained shear. This process involves first a reduction of effective stress and then a reapplication prior to undrained shear. For a normally consolidated specimen, for example, the stresses are reduced from the K_o effective stresses to the isotropic effective stress $\bar{\sigma}_r$ and then reconsolidated isotropically to a stress equal to $\bar{\sigma}_{vo}$. During this large reduction and subsequent increase in effective stresses, and also because of the change from a K_o to an isotropic stress system, a significant volume reduction normally occurs. Many (but not all, such as Taylor (23), p. 397) soil engineers feel that this volume decrease results in an undrained shear strength which is too large. For example, see Rutledge (7), p. 1216, Hansen and Gibson (8), p. 204, Osterberg (24), p. 70, and Bishop and Bjerrum (25), p. 449.

Published Methods of Correcting CIU Test Results:

Several methods have been suggested for evaluating the effects of volume change on undrained strength. These methods generally extrapolate, in various ways, a plot of log s_u from *CIU* triaxial tests versus void ratio e to a value of s_u corresponding to the *in situ* void ratio. For example, Schmertmann (26) draws a line through the point of intersection of lines of log s_u versus e for *CIU* tests on undisturbed and remolded specimens and parallel to the estimated slope of the *in situ* consolidation curve. The intersection of this line with the *in situ* void ratio yields the field undisturbed strength. Calhoon (27) shifts the plot of log s_u versus e from *CIU* triaxial tests upward in proportion to the per cent disturbance of the triaxial specimens. The per cent disturbance is deduced

TABLE 3—PREDICTION OF UNDRAINED STRENGTH AT IN SITU VOID RATIO FROM *CIU* TRIAXIAL TESTS BY PREVIOUS METHODS FOR KAWASAKI CLAY 1.

Method	s_u(kg/cm^2) for $\Delta e/(1 + e_o) = 0$
Schmertmann (**26**)	0.3 to 0.45
Calhoon (**27**)	~0.85
Casagrande and Rutledge (**28**)	~0.5
Average of unconfined[a] compression tests	0.4[b]
Average of $\overline{UU}$ and top one third of unconfined compression tests	0.5[b]

[a] Many of the specimens contained lenses of sand, silt, or shells which caused very low unconfined strengths.
[b] Corrected to correspond to $t_f = 5$ hr.

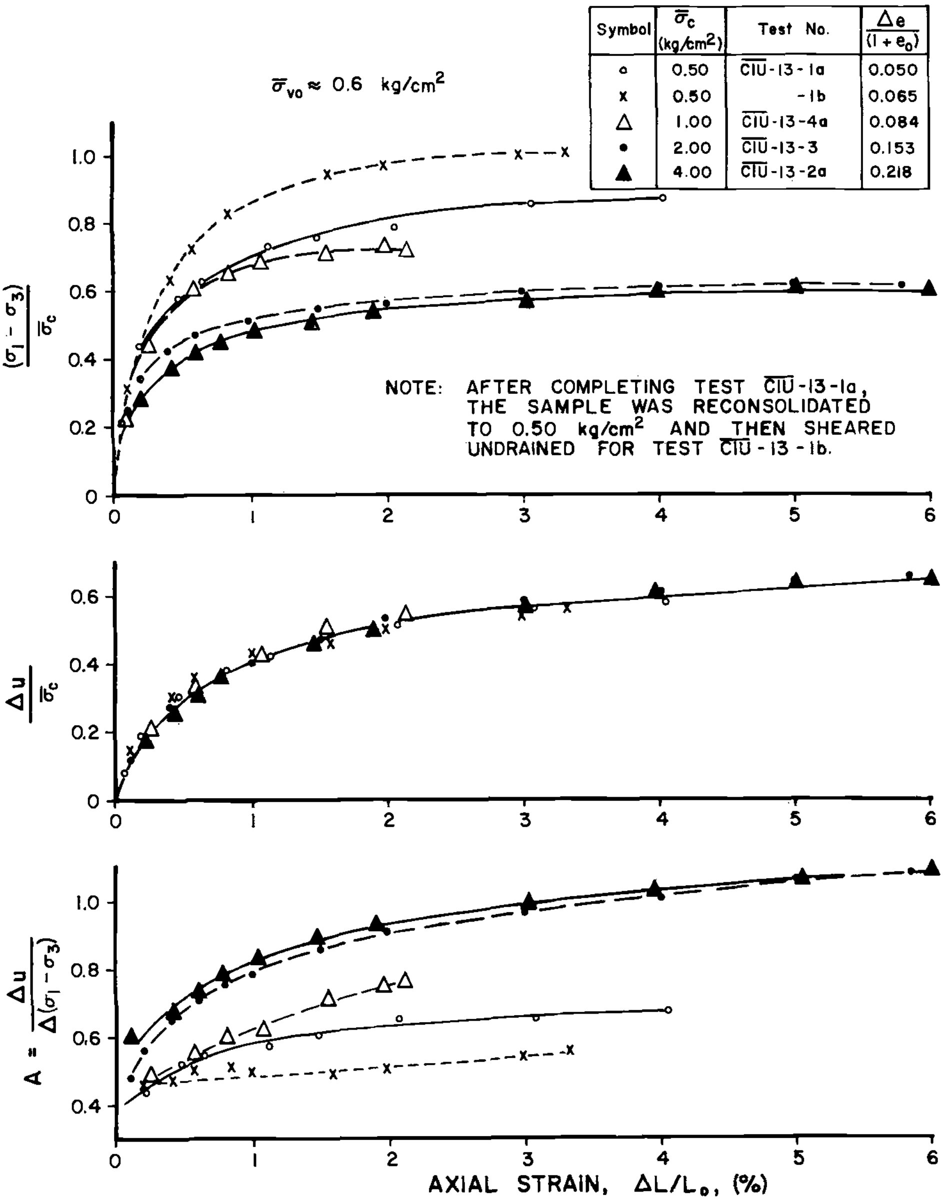


FIG 13—Effect of Consolidation Pressure and Reconsolidation on the Stress-Strain Behavior of Lagunillas Clay.

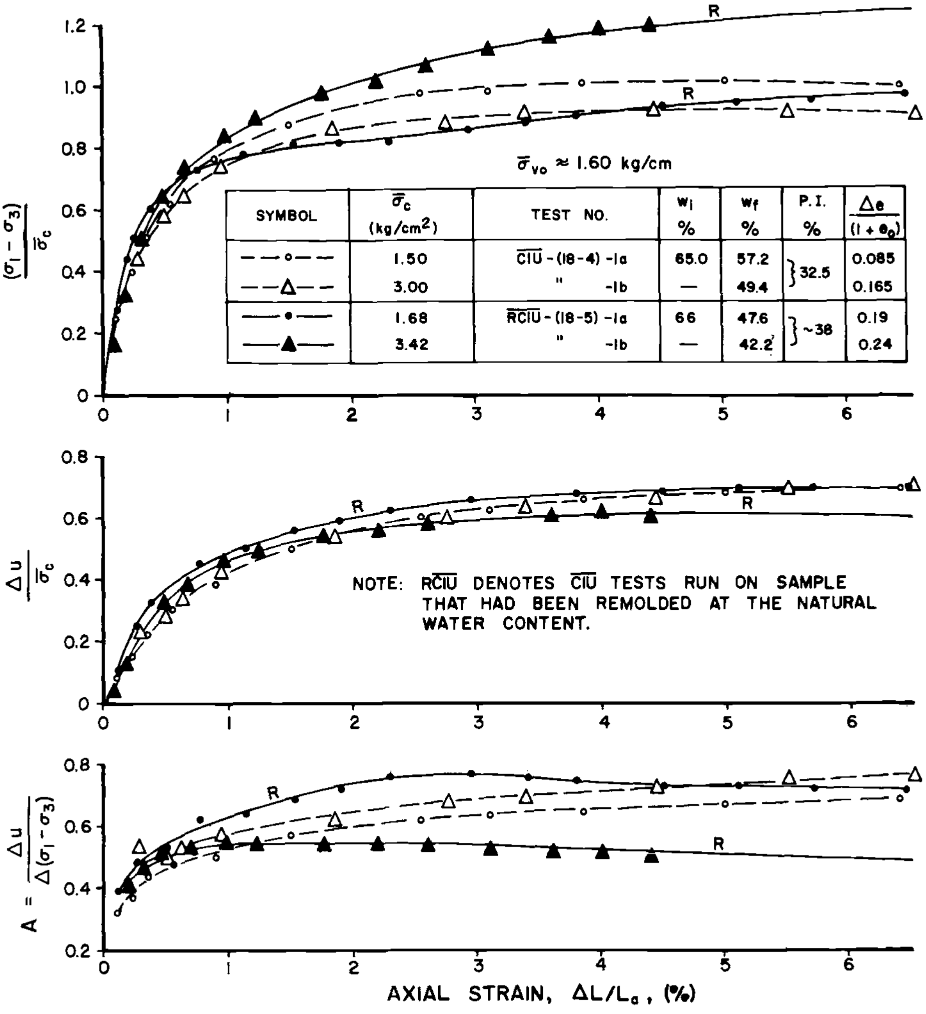


FIG. 14—Effect of Consolidation Pressure and Remolding on the Stress-Strain Behavior of Kawasaki Clay I.

from the location of the log $\bar{\sigma}_c$ versus e curve from the triaxial tests in relationship to log $\bar{\sigma}_c$ versus e plots from oedometer tests on varying size specimens and on a remolded specimen.[10] One of the simplest methods, that of Casagrande and Rutledge (28), as quoted by Hvorslev (9), suggests an extrapolation of the log s_u versus e line from CIU tests back to the *in situ* void ratio as an approximate approach for obtaining s_u corresponding to no volume change.

The application of these methods to the strength data on Kawasaki Clay I, Fig. 10, yielded the values of s_u cor-

[10] This method assumes that only trimming produces sample disturbance, which can be very incorrect, and it neglects the effect of K on the location of $\bar{\sigma}_c$ curves.

rected for volume change,[11] shown in Table 3. There is a wide divergence in the resulting strengths. Two of the methods yielded strengths which are too low since they are equal to, or smaller than, the 0.5 kg/cm^2 from the *UU* tests. On the other hand, the s_u from the Calhoon method is too high since *CIU* tests with $\bar{\sigma}_c = \bar{\sigma}_{vo}$ yielded an s_u of only about 0.8 kg/cm^2 (Fig. 8). Even if the Kawasaki Clay I represents an unusual case, these methods would appear to be questionable.

Proposed Methods of Correcting CIU Test Results:

One way to account for the effects on s_u of the volume change upon reconsolidation is to use the Hvorslev parameters (**29**), p. 210 or (**18**), p. 166 which can be expressed as follows: [$\frac{1}{2}(\sigma_1 - \sigma_3)_f$ is replaced by s_u since only undrained shear strength is considered]

$$s_u = \mathrm{H}\bar{\sigma}_e + \bar{\sigma}_{3f} \tan \bar{\theta}_e \quad (2)$$

where:

$\mathrm{H} = \mathrm{K}(\cos \bar{\phi}_e)/(1 - \sin \bar{\phi}_e)$,
$\tan \bar{\theta}_e = (\sin \bar{\phi}_e)/(1 - \sin \bar{\phi}_e)$,
$\mathrm{K} = \bar{c}_e/\bar{\sigma}_e$,
$\bar{c}_e$ = Hvorslev cohesion,
$\bar{\phi}_e$ = Hvorslev friction angle, and
$\bar{\sigma}_e$ = Hvorslev equivalent consolidation pressure.

If two specimens were consolidated to the same pressure, but had different water contents and hence different values of $\bar{\sigma}_e$, then the difference in s_u for undrained shear could be reflected in changes in both $\mathrm{H}\bar{\sigma}_e$ and $\bar{\sigma}_{3f} \tan \bar{\theta}_e$. However, if $\bar{\sigma}_{3f}$ were independent of the water content change, the difference in s_u could be calculated from the change in $\mathrm{H}\bar{\sigma}_e$, assuming of course, that H and $\bar{\theta}_e$ were constants for the soil.[12]

Since $\bar{\sigma}_{3f}$ is directly related to the excess pore pressure at failure, one can look at the effects of volume change on the pore pressure behavior of $\overline{CIU}$ tests. Such data for the Lagunillas clay and the Kawasaki Clay I are presented in Figs. 13 and 14. The data show that $\Delta u/\bar{\sigma}_c$ versus strain is practically independent of (1) consolidation pressure for $\bar{\sigma}_c$ values greater than $\bar{\sigma}_{ps}$, even though the value of volumetric strain compared to the *in situ* value varies considerably with consolidation pressure (Figs. 9 and 10), (2) reconsolidation, with resultant volume decrease, after undrained shear (Test $\overline{CIU}$-13-1(*b*) in Fig. 13), and (3) remolding at the natural water content with subsequent consolidation and very large volume changes (Tests $R\overline{CIU}$-(18-5)-1(*a*) and -1(*b*) in Fig. 14). As a reasonable approximation it is therefore assumed that the volume change caused by the consolidation of $\overline{CIU}$ specimens to pressures between $\bar{\sigma}_{ps}$ and $\bar{\sigma}_{vo}$ has little effect on $\Delta u/\bar{\sigma}_c$ during undrained shear, and that the strain at failure is also unaltered. The decrease in s_u is then calculated from the change in Hvorslev cohesion, $\mathrm{H}\Delta\bar{\sigma}_e$, commensurate with the decrease in volume from the *in situ* condition, $\Delta e/(1 + e_o)$. An example is given in the next section.

Another possible approach to the problem of assessing the effects of volume change would be to use the results of *CU* tests at consolidation pressures much larger than $\bar{\sigma}_{vo}$, where the effects of specimen disturbance might be minimized (**13**), p. 194, (**30**), p. 33. This method, as with previous ones, requires negligible changes in soil structure with consolidation pressure. If the value of s_u corresponding to perfect sampling is desired, *CA-UU* tests should be used with consolidation pressures at least two to four times $\bar{\sigma}_{vo}$.

[11] There are insufficient data on the other cases in Table 2 to allow similar analyses.

[12] Bjerrum and Wu (**12**) show that H varies with consolidation pressure for undisturbed specimens of Lilla Edet clay, which is believed to have a significant amount of natural cementation.

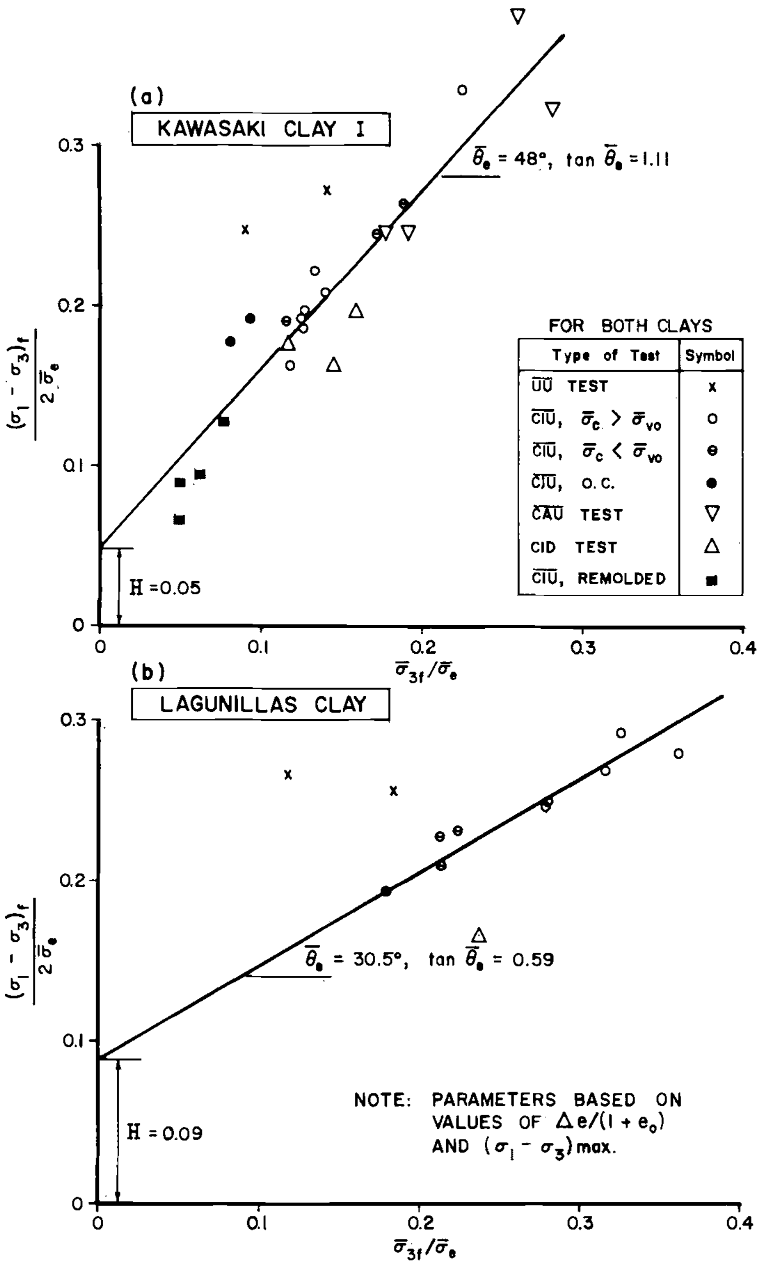


FIG. 15—Hvorslev Parameters.

TABLE 4—COMPARISON OF CORRECTED UNDRAINED STRENGTH FROM *UU* AND *CU* TESTS ON NORMALLY CONSOLIDATED KAWASAKI AND LAGUNILLAS CLAYS.

Location	Clay	Depth, m	$\bar{\sigma}_{vo}$, kg/cm²	$\bar{\sigma}_{ps}$, kg/cm²	$s_u/\bar{\sigma}_{vo}$ From *UU* Tests		$s_u/\bar{\sigma}_{vo}$ from *CIU* Tests				Measured[c] $\frac{s_u(UU)}{s_u(CIU)}$		Corrected[d] $\frac{s_u(UU)}{s_u(CIU)}$		$\frac{s_u(UU)^e}{s_u(CU)}$ $s_u(\overline{CU})$ from $CA\text{-}\overline{UU}$ Tests
							Measured[a]		Corrected[b]						
					Measured	Cor-rected[f]	$\bar{\sigma}_c = \bar{\sigma}_{vo}$	$\bar{\sigma}_c = \bar{\sigma}_{ps}$	$\bar{\sigma}_c = \bar{\sigma}_{vo}$	$\bar{\sigma}_c = \bar{\sigma}_{ps}$	$\bar{\sigma}_c = \bar{\sigma}_{vo}$	$\bar{\sigma}_c = \bar{\sigma}_{ps}$	$\bar{\sigma}_c = \bar{\sigma}_{vo}$	$\bar{\sigma}_c = \bar{\sigma}_{ps}$	
					(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Kawasaki, Japan.	Clay I,. *PI.* = 31%	20.5	1.60	0.92	0.31[g]	0.38	0.485	0.42	0.43	0.40	0.64	0.74	0.88	0.95	1.12
	Clay I, *P.I.* = 36%	25	1.88	1.08	0.26[g]	0.32	0.45	0.37	0.395	0.35	0.58	0.70	0.81	0.91	0.89
	Clay II, *P.I.* = 43%	36	2.62	1.64	0.27[g]	0.33	0.475	0.36	0.42	0.335	0.57	0.75	0.78	0.98	1.00
		Average.									0.59	0.73	0.82	0.95	1.00
Lagunillas, Venezuela.	plastic clay, *P.I.* = 37%	6.4	0.62	0.40 (Estimated)	0.255[h]	0.345	0.40	0.335	0.38	0.325	0.64	0.77	0.91	1.05	. . .

[a] From Figs. 7 and 8 or similar data. $\bar{\sigma}_c = \bar{\sigma}_{vo}$ means that s_u is taken at $\bar{\sigma}_c = \bar{\sigma}_{vo}$; for $\bar{\sigma}_c = \bar{\sigma}_{ps}$, s_u is taken at $\bar{\sigma}_c = \bar{\sigma}_{ps}$.

[b] Based on Hvorslev parameters. That is, $\Delta s_u = H\Delta\bar{\sigma}_e$.

[c] Columns (1) over (3) and (1) over (4) respectively.

[d] Columns (2) over (5) and (2) over (6) respectively.

[e] $s_u(CU)$ from $s_u/\bar{\sigma}_{1c}$ ratio from $CA\text{-}\overline{UU}$ tests at $\bar{\sigma}_{1c}$ values 2–4 times $\bar{\sigma}_{vo}$. $s_u(UU)$ from Column 2.

[f] Correction for Kawasaki based on taking O.C.R. = 2.85 ($\bar{\sigma}_r/\bar{\sigma}_{ps}$ = 0.35 which corresponds to values for the better samples). Correction for Lagunillas based on taking O.C.R. = 2.8 ($\bar{\sigma}_r/\bar{\sigma}_{ps}$ = 0.36).

[g] Average from *UU* and top one-third of *U* tests corrected to t_f = 5 hr.

[h] Average from *UU* tests for which $\bar{\sigma}_r$ measurements available.

COMPARISON OF CORRECTED UU AND CU TEST RESULTS

Results for Kawasaki and Lagunillas Clays:

Specific examples illustrating the application of the authors' methods of correcting *UU* and *CU* test data for sample disturbance will be presented first. In reiteration, the primary objective of the correction is to arrive at an undrained shear strength for triaxial compression of a specimen subjected to perfect sampling, that is, the specimen had a preshear effective stress equal to $\bar{\sigma}_{ps}$ (Eq 1) and the *in situ* void ratio. Test data for Kawasaki Clay I will be used for the illustrative examples.

Correction of UU Data for Test $\overline{UU}$-(18-5)-1 (strain rate effects will be neglected).

1. *Measured data* (Fig. 6)—s_u = 0.46 kg/cm² and $\bar{\sigma}_r$ = 0.34 kg/cm².

2. *Equivalent OCR* (Fig. 4)—Since specimen depth was 21.2 m, $\bar{\sigma}_{ps}$ = 0.95 kg/cm². Therefore, the equivalent OCR = $\bar{\sigma}_{ps}/\bar{\sigma}_r$ = 0.95/0.34 = 2.80.

3. *Corrected strength*—For an OCR = 2.80, and using Fig. 12, (s_u at $\bar{\sigma}_c$)/(s_u at $\bar{\sigma}_{cm}$) = 0.82. Since this ratio is also assumed equal to (s_u at $\bar{\sigma}_r$)/(s_u at $\bar{\sigma}_{ps}$), the corrected s_u = 0.46/0.82 = 0.56 kg/cm².

Correction of CU Data for Depth of 23 M:

1. *Hvorslev parameters*—Values of the Hvorslev parameters are shown in Fig. 15 for both the Lagunillas clay and Kawasaki Clay I where $\bar{\sigma}_e$ was based on values of $\Delta e/(1 + e_o)$ in Figs. 9 and 10 rather than water content, which varied erratically. The parameters were determined on the basis of $\overline{CU}$ and *CID* tests, although $\overline{UU}$ and $\overline{RCIU}$ test data have been added for interest.

2. *Measured data*—For a depth of 23 m Fig. 4 shows that $\bar{\sigma}_{vo}$ = 1.75 kg/cm² and $\bar{\sigma}_{ps}$ = 1.00 kg/cm². From Fig. 8 at $\bar{\sigma}_c = \bar{\sigma}_{ps}$ = 1.00 kg/cm², the measured s_u = 0.68 kg/cm². For $\bar{\sigma}_c$ = 1.00 kg/cm², a representative value of $\Delta e/(1 + e_o)$ = 0.04 from Fig. 10.

3. *Equivalent consolidation pressures* (Fig. 10)—For $\Delta e/(1 + e_o)$ = 0.04, $\bar{\sigma}_e$ = 2.60 kg/cm²; this represents isotropic consolidation of a lab specimen to $\bar{\sigma}_c = \bar{\sigma}_{ps}$ = 1.00 kg/cm². For perfect sampling, $\Delta e(1 + e_o)$ = 0 and $\bar{\sigma}_e = \bar{\sigma}_{vo}$ = 1.75 kg/cm².

4. *Corrected strength*—From Eq 2 if $\Delta\bar{\sigma}_{3f}$ = 0, $\Delta s_u = \mathrm{H}\Delta\bar{\sigma}_e$; H = 0.05 from Fig. 15 and $\Delta\bar{\sigma}_e$ = (1.75 − 2.60) = −0.85 kg/cm² from above. Therefore the correction to s_u for the volume change = 0.05 (−0.85) = −0.043 kg/cm², and the corrected s_u = 0.68 − 0.043 ≈ 0.64 kg/cm².

Table 4 presents the comparison of s_u from *UU* and *CU* tests as measured and after correction for the Kawasaki and Lagunillas clays. Columns 1 and 2 show the effect on s_u of treating $\bar{\sigma}_{ps}/\bar{\sigma}_r$ as an OCR for the *UU*-type tests; the strength was increased by 28 ± 6 per cent. Columns 3 through 6 present the effects of correcting for the volume decrease upon reconsolidation based on Hvorslev parameters at consolidation pressures of both $\bar{\sigma}_{ps}$ and $\bar{\sigma}_{vo}$; the strength was decreased by 5 to 12 per cent. Columns 7 through 10 present ratios of measured and corrected s_u from *UU* tests to that from *CIU* tests based on the data in Columns 1 through 6. The effect of performing the authors' second method of correcting *CU* data for volume decreases is shown in Column 11.

Columns 10 and 11 are of greatest interest. If the analyses proposed by the authors were entirely correct, the ratios in these columns should have equalled unity since (1) The *UU* strength data had been adjusted to make $\bar{\sigma}_r = \bar{\sigma}_{ps}$, and (2) The *CU* strength data had been adjusted to make $\bar{\sigma}_c = \bar{\sigma}_{ps}$ and $\Delta e/(1 + e_o)$ = 0 by two methods.

That is, all of the data had been adjusted, in theory, to conditions corresponding to perfect sampling. The actual values in these columns varied from unity by a maximum of only 12 per cent, which is considered good agreement in view of the large differences (Column 8 shows 23 to 30 per cent) in strengths before corrections. However, these methods require further investigation to establish their limitations.

As previously pointed out, there should not be agreement between s_u from *UU* tests and that from *CIU* tests with $\bar{\sigma}_c = \bar{\sigma}_{vo}$ even for perfect samples, since $\bar{\sigma}_{ps}$ will be less than $\bar{\sigma}_{vo}$ for normally consolidated clays. This is illustrated by the data in Column 9 where the ratio was only 0.85 ± 0.06.

SUMMARY AND CONCLUSIONS

This paper considers the undrained shear strength of saturated "undisturbed" clays as determined by laboratory tests. For most *normally* consolidated clays, the value of undrained shear strength, s_u, measured with *UU* tests, is only 40 to 80 per cent of that measured with *CIU* tests having a consolidation pressure equal to the *in situ* vertical effective stress. A portion of this difference is attributed to the fact that sampling necessarily involves the release of *in situ* shear stresses, since K_o is significantly less than unity. In addition, further disturbance due to tube sampling, extrusion, trimming, etc., generally reduces the actual effective stress in soil specimens to a value far below that existing in the ground.

An estimate of the isotropic effective stress in a specimen following perfect sampling, in which only the *in situ* shear stresses are released, can be obtained by reconsolidating a specimen to the K_o condition and then releasing the shear stresses at constant mass. A comparison of this theoretical stress, termed $\bar{\sigma}_{ps}$, with the effective stress $\bar{\sigma}_r$ as actually measured in laboratory specimens is used to indicate the degree of sample disturbance. Test data on tube samples of several moderately sensitive clays show average values of 2.8 to 5 for the ratio $\bar{\sigma}_{ps}/\bar{\sigma}_r$.

It is felt that *UU* strengths on tube samples are often significantly below those for truly undisturbed samples and that some of the reported agreements between such strengths and those back-figured from field observations may well have involved compensating errors. Furthermore, values of s_u, $\bar{\phi}$, and A_f measured from $\overline{CIU}$ tests on tube samples, which generally exhibit significant volume decreases during reconsolidation in the laboratory, are liable to errors on the unsafe side; s_u and $\bar{\phi}$ being too large and A_f being too low.

Methods are proposed for adjusting the values of s_u measured from *UU* and *CU* tests to that corresponding to an undrained triaxial compression test on a specimen after perfect sampling. An examination of *UU* test results suggests that such specimens behave as overconsolidated specimens. Based on this concept, the authors suggest that the ratio $\bar{\sigma}_{ps}/\bar{\sigma}_r$ be considered as an overconsolidation ratio and the *UU* strengths corrected accordingly.

The paper proposes that the volume decrease attendant with the decrease in effective stresses during sampling and their reapplication during reconsolidation in the laboratory prior to a *CIU* test can be accounted for, with many clays, by correcting the results of the *CIU* test through Hvorslev parameters. An alternate approach for obtaining an adjusted s_u from *CU* tests employs test data on specimens consolidated to pressures much greater than the *in situ* overburden pressure.

Two case studies are presented in which the results of *CIU* and *UU* tests, which had showed a 50 per cent discrepancy in strength, are analyzed in accordance with the procedures proposed in this paper. Following correction of the test data for sample disturbance, there was good agreement (within 5 to 10 per cent) between the strengths obtained from *UU* and *CIU* tests.

It is suggested that values of residual effective stress, $\bar{\sigma}_r$, be determined on representative "undisturbed" samples for all important jobs as a quantitative measure of the amount of disturbance caused by sampling. It is further hoped that the methods proposed here will be investigated for other clays and that the undrained shear strength corresponding to perfect sampling can eventually be related in a rational manner to actual field strengths.

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APPENDIX I

The notations employed in this paper are given below:

Stresses and Stress Ratios:

Suffix f = failure conditions,
Prefix Δ = a change,
σ = total normal stress,
$\bar{\sigma}$ = effective normal stress,
σ_1, $\bar{\sigma}_1$, σ_3, $\bar{\sigma}_3$ = total and effective, major and minor principal stresses,
σ_c = chamber pressure,
$\bar{\sigma}_c$ = consolidation pressure,
$\bar{\sigma}_e$ = Hvorslev's equivalent consolidation pressure,
$\bar{\sigma}_{1c}$, $\bar{\sigma}_{3c}$ = major and minor principal consolidation pressures,
σ_v, $\bar{\sigma}_v$, σ_h, $\bar{\sigma}_h$ = total and effective, vertical and horizontal stresses,
$\bar{\sigma}_{vc}$, $\bar{\sigma}_{hc}$ = vertical and horizontal consolidation pressures,
$\bar{\sigma}_{vo}$, $\bar{\sigma}_{ho}$ = *in situ* vertical and horizontal effective stresses,
$\bar{\sigma}_{cm}$ = maximum past pressure,
$\bar{\sigma}_{vm}$ = maximum past vertical pressure,
K = $\bar{\sigma}_h/\bar{\sigma}_v$
K_o = value of K for no lateral strain = coefficient of earth pressure at rest,
u = pore water pressure,
τ = shear stress,
q = $\frac{1}{2}(\sigma_1 - \sigma_3)$,
$\bar{p}$ = $\frac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)$,
s_u = $\frac{1}{2}(\sigma_1 - \sigma_3)$ maximum for undrained shear,
A = Skempton's A parameter,
B = Skempton's B parameter,
A_u = an A parameter for undrained release of shear stresses from a K_o condition,
$\bar{\sigma}_{ps}$ = effective stress after perfect sampling from a K_o condition,
$\bar{\sigma}_r$ = residual effective stress after actual sampling, and

u_r = residual pore pressure after actual sampling.

Types of Triaxial Compression Tests:

$\overline{\quad}$ = an elevated bar over letters denoting a type of shear test indicates that pore pressures were measured during shear

UU, $\overline{UU}$ = unconsolidated-undrained shear test ($\sigma_c > 0$),

U = unconfined compression test,

CU, $\overline{CU}$ = consolidated-undrained shear test,

CIU, $\overline{CIU}$ = isotropically consolidated-undrained shear test,

$\overline{RCIU}$ = isotropically consolidated-undrained shear test with pore pressure measurement on a specimen that had previously been remolded,

CAU, $\overline{CAU}$ = anisotropically consolidated-undrained shear test,

$CA\text{-}\overline{UU}$ = anisotropically consolidated specimen in which the consolidation shear stresses are released at constant volume (that is perfect sampling) followed by undrained shear with pore pressure measurement, and

CID = isotropically consolidated-drained shear test.

Miscellaneous Symbols:

$\bar{\phi}$ = slope of effective stress envelope tangent to Mohr circles at failure,

$\bar{\alpha}$ = slope of effective stress envelope on a q_f versus $\bar{p}_f$ plot,

$\bar{\phi}_e$ = Hvorslev friction angle,

$\bar{c}_e$ = Hvorslev cohesion,

K = $\bar{c}_e/\bar{\sigma}_e$,

$\bar{\theta}_e$ = modified version of Hvorslev friction angle,

H = modified version of Hvorslev K

w_i = initial water content,

w_f = water content at failure,

e = void ratio,

e_o = *in situ* void ratio,

w_L = liquid limit,

PI = plasticity index,

LI = liquidity index,

t_f = time to failure,

S_t = sensitivity,

OCR = overconsolidation ratio, and

OCR = $\bar{\sigma}_{cm}/\bar{\sigma}_c$ or $\bar{\sigma}_{vm}/\bar{\sigma}_{vc}$

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