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Behavior of a remolded silty-clay subjected to sequential loading

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Department:

BEHAVIOR OF A REMOLDED SILTY-CLAY SUBJECTED TO SEQUENTIAL LOADING

BY

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THESIS

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ABSTRACT

The investigation was undertaken to determine shear strength and pore pressure parameters of a soil when subjected to sequential loading. This method is intended to similate the in-situ condition of stress application under certain field conditions.

The testing procedure, a sequential triaxial shear test, is described. This test consists of successive stages of partial consolidation followed by undrained shear. Triaxial consolidation and shear strength characteristics of a sedimented remolded silty-clay are compared with the results from conventional tests on isotropically consolidated samples.

The undrained strength was found to be a function of moisture content, irrespective of the method of test and initial stress system for the normally consolidated soil.

A unique relationship was established between a pore pressure parameter, the ratio of the change in pore pressure to the effective vertical consolidation pressure, and undrained strength.

It is concluded that this sequential procedure of testing does change the strength characteristics of the soil. These cannot be predicted from the conventional test.

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NOTATIONS AND DEFINITIONS

A	=	Skempton's A parameter, $A = \frac{\Delta_u}{\Delta (\sigma_1 - \sigma_3)}$
A_f	=	parameter A at failure
a_{rp}	=	area of piston of the triaxial cell
a_v	=	coefficient of compressibility
B	=	Skempton's B parameter, $B = \Delta u/\Delta\sigma_3$
$\bar{c}$	=	cohesion, effective stresses
c_v	=	coefficient of consolidation
E	=	modulus of elasticity
e	=	void ratio
Δe	=	change in void ratio
K	=	ratio of effective major principal stress to effective minor principal stress, $\bar{\sigma}_1/\bar{\sigma}_3$
K_c	=	ratio of effective major principal consolidation stress to effective minor principal consolidation stress $\bar{\sigma}_{1c}/\bar{\sigma}_{3c}$
K_f	=	K at failure
K_o	=	ratio of major principal stresses during consolidation under a zero lateral strain condition
k	=	coefficient of permeability
ln	=	natural logarithm, $\log_e$
log	=	common logarithm, $\log_{10}$

OCR	=	overconsolidation ratio, $\bar{\sigma}_{cm}/\bar{\sigma}_c$
$\bar{p}_f$	=	$\frac{1}{2}(\bar{\sigma}_1 + \bar{\sigma}_3)_f$ at failure
$\bar{p}_c$	=	effective consolidation pressure
q_f	=	undrained strength at failure
q_o	=	undrained strength
S_{ff}	=	shear stress on the failure plane at failure
$S_{\beta f}$	=	shear stress on the plane of maximum obliquity at failure
S_{tf}	=	shear stress on the plane of tangency of the Mohr envelope at the time of failure
S_{mf}	=	maximum shear stress at failure
U	=	degree of consolidation
u	=	pore water pressure at any time
u_i	=	initial pore water pressure
V_T	=	total volume of the sample
w_f	=	water content at failure
w_o	=	water content corresponding to q_o
α	=	angle between plane of failure and major principal stress
Δ	=	a change
ε_f	=	compressive strain at failure
σ	=	total normal stress
$\bar{\sigma}$	=	effective normal stress
$\bar{\sigma}_c$	=	horizontal consolidation pressure
$\bar{\sigma}_{cm}$	=	maximum past consolidation stress
$\bar{\sigma}_{1c}$	=	vertical effective consolidation pressure
σ_{cell}	=	cell pressure

σ_{vc} = vertical consolidation pressure

$\overline{\phi}$ = effective stress friction angle

I. INTRODUCTION

A. PURPOSE OF INVESTIGATION

The continuing growth of cities combined with persisting demands for more housing, highway embankments, earth dams and a wide variety of other structures has made imperative the utilization of poor subsoil and improvement of poor subsoil conditions. This is due to the increasing scarcity of advantageously located available land which offers good foundations conditions.

The use of subsoil and the improvement of poor subsoil conditions, generally weak and highly compressible soils, requires the construction of a fill so as to raise the grade above any possible high water level. The addition of the fill creates problems of foundation stability and large settlements in poor subsoils.

The problem of foundation stability becomes a great concern to the engineer during construction of the fill. It is often necessary to predict and control the rate of loading - i.e., the rate of fill construction - to prevent a foundation failure and a partial loss of the fill. This knowledge becomes necessary on two accounts; one, to determine the feasibility of the project within the time and cost alloted to construction and two, to prevent a foundation failure with the involved losses of time and money. The economic and time constraints make it necessary to construct the fill with a factor of safety against a foundation failure

as close to unity as possible. All of these constraints require that the engineer be able to predict the total amount of deformation and the increase in undrained strength with partial consolidation of the foundation soil under successive stages of fill loading.

This general field problem constituted the basis for the development of this investigation. Its purpose was to determine the deformation and strength properties of a soil when subjected to prescribed loading paths which may, in general, simulate the above mentioned field problem.

A saturated, remolded silty clay was used for the investigation. The justification for its use was made in the following statements by Johnson and Whitman (1960):

> "The hope for a clearer understanding of shear strength behavior lies in isolating the many factors which determine strength and studying each of these factors in detail. To do so, the researcher must work with samples whose history of formation is known accurately and this requirement of necessity, means that remolded samples must be used. Basic research using saturated remolded samples has provided the main share of our present knowledge regarding the fundamentals of shear strength behavior, and will, in the future, provide the knowledge needed to piece together the desired comprehensive picture of strength behavior and to formulate practical rules and procedures."

The application of a selected loading path was made with the use of special triaxial shear tests, herein designated as sequential tests. These sequential tests consisted of successive stages of partial consolidation followed by

undrained shear.

The strength of the silty clay from the results of sequential shear tests were analyzed in terms of the failure condition, water contents, pore water pressures, effective stresses and consolidation stresses.

The deformation properties of the silty clay were analyzed in terms of the changes in such properties as permeability, coefficient of consolidation, and modulus of elasticity. These properties were also evaluated in terms of degree of consolidation and applied consolidation stresses.

B. REVIEW OF PAST INVESTIGATIONS

1. Consolidation

The basic theoretical consolidation theory was developed by Terzaghi (1925). The concept of the theory considers an ideal saturated soil subjected to vertical loading with zero lateral strain based on the following assumptions:

a. The soil is a homogeneous material.

b. The soil is completely saturated.

c. It is one-dimensional flow.

d. It is one-dimensional compression.

e. Darcy's law of flow is applicable.

f. The action of infinitesimal masses is no different from that of larger representative masses.

g. The compressibility of soil grains and water is negligible.

h. Constant values for certain soil properties which actually vary somewhat with pressure.

i. The coefficient of consolidation $c_v = \frac{k(1 + e)}{\gamma_w a_v}$ is constant during the consolidation process.

The consolidation process depends upon the thickness of the soil layer, the number of drainage surfaces of the clay layer and the coefficient of consolidation, c_v, of the soil. This coefficient is a function of the permeability of the soil, the unit weight of water and the rate of pore pressure dissipation.

Seed (1965) mentioned that triaxial cells are finding increasing use for investigation of the consolidation characteristics of soils. The effective stress path method suggested by Lambe (1964) is applicable to triaxial test. The analysis procedures are for situations where volume changes during consolidation are accompanied by changes in lateral strain.

2. Shear Strength

Failure criteria. The theory of shear failure of soil was first expressed by Coulomb (1776). The equation is:

$$s = c + \sigma \tan \phi \qquad (1)$$

where s is shear resistence

c is cohesive strength

σ is normal stress

ϕ is coefficient of friction.

Whitman (1960) defined shear strength in four possible ways as shown in Figure 1.

s_{tf} = shear stress on the plane of tangency of the Mohr envelope at the time of failure,

$s_{\beta f}$ = shear stress on the plane of maximum obliquity at failure,

s_{ff} = shear stress on the failure plane at failure,

s_{mf} = maximum shear stress at failure.

There are very small differences among s_{tf}, s_{ff}, and $s_{\beta f}$. For practical work Whitman suggested to choose s_{mf}, which is one-half of the deviator stress, to represent undrained shear strength of soil. For dealing with shear strength in terms of effective stress, s_{tf} (i.e. the Mohr envelope) should be used.

Which stage in the shear process represents failure? There is no general answer. The definition of strength must be suited to the problem which is being attacked. The undrained test is being run for the purpose of determining the s_{tf} vs $\bar{\sigma}_{tf}$ relationship, but gives the data which are useful in predicting the pore pressures which will be set up in the clay at various stages of construction. The end point as shown in Fig 2 by curve A appears to be the logical solution. Curve A is the line passing through the s_{ff}, the maximum obliquity at failure, whereas curve B is the line passing through s_{mf}, maximum shear stress at failure.

3. Effective Stress

The principle of effective stress originated in the work of Terzaghi (1923 and 1932). The concept was that the strength and deformation characteristics of soils

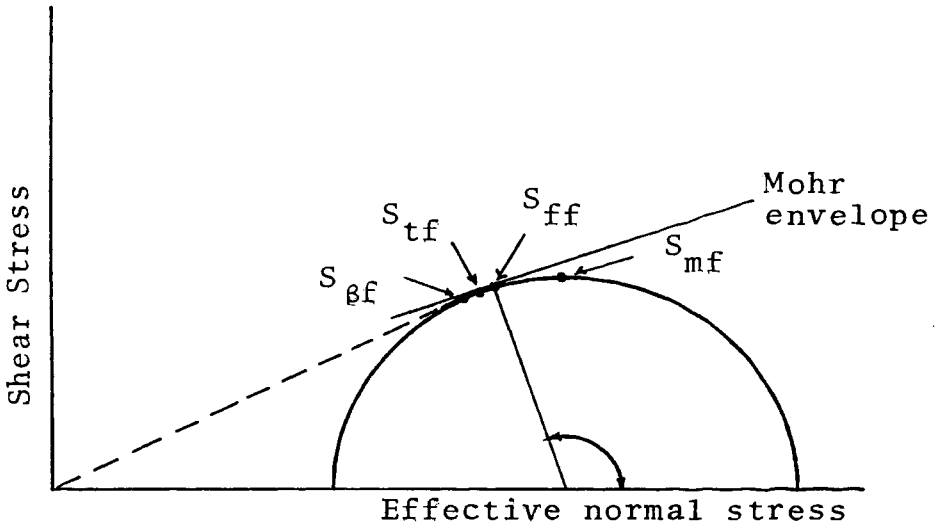

FIGURE 1. FOUR SHEAR STRESS REPRESENTING FAILURE

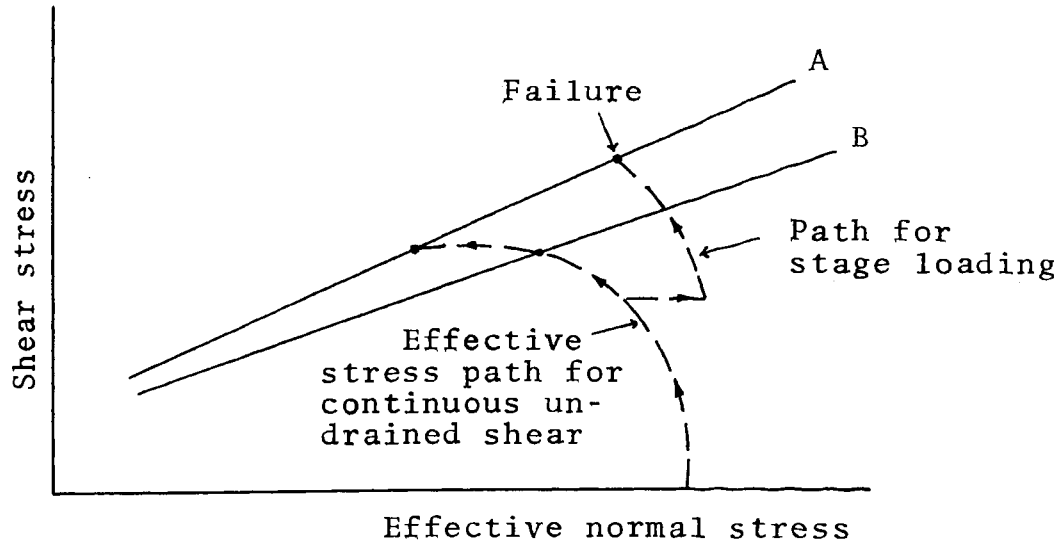

FIGURE 2. CHOICE OF FAILURE CRITERIA FOR STAGE LOADING
(After Whitman, 1960)

are governed by the effective stresses rather than the total stresses. For saturated soil, the effective normal stress is given by

$$\bar{\sigma} = \sigma - u \tag{2}$$

where σ is the total normal stress and u is the pore pressure in the pore space of the soil.

Then Coulomb's equation may be written as

$$s = \bar{c} + \bar{\sigma} \tan \bar{\phi}$$

$$= \bar{c} + (\sigma - u) \tan \bar{\phi} \tag{3}$$

The effective stress $\bar{\sigma}$, cannot be determined directly by experiment. Skempton (1960) justified the use of Equation 4 from a theoretical viewpoint. The determination of the effective stress depends on the pore pressure. Skempton (1954) proposed that the change in pore pressure Δu may be expressed by

$$\Delta u = B \left[\Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \right] \tag{4}$$

where $\Delta\sigma_1$ = change in major principal stress,

$\Delta\sigma_3$ = change in minor principal stress

A and B are the pore pressure parameters.

The parameter B is equal to unity for fully saturated soils.

The parameter A is not a constant for any soil and it depends on the following factors (Lo, K.Y., 1969):

a. stress history,

b. consolidation pressure,

c. direction of stress path (rate of strain or time),

d. sustained loading,

e. state of stress.

Lo (1969) proposed that the change in pore pressure with change in shear stresses is a phenomenon associated with strain rather than with stress and thus he rejects Skempton's parameter A. He used the ratio $\Delta u/\bar{p}c$ to show that it was a function only of axial strain and independent of stress history, consolidation pressure, stress state and stress system.

Taylor (1948) suggested that pore pressure does not affect the undrained shear strength directly. In other words, there is no effect on the undrained strength when the values of the intergranular stresses are expressed in terms of effective stresses. Lowe and Karafiath (1960) justified Taylor's concept by finding that the undrained strength is not identical for both isotropic and anisotropic consolidated compacted clays.

For previous work, it was found that the angle of effective friction is independent of stress system. Simons (1960) performed tests on Oslo clay and concluded that the same shear strength parameters with respect to effective stress were obtained for both isotropic and anisotropic consolidated samples. Henkel and Sowa (1963) found similar results.

4. Undrained Strength and Moisture Content Relationship

Rutledge (1947) proposed that for a given soil, the effective major principal stress and void ratio are independent of the principal stress ratio and the method of test. He also proposed that the undrained strength of a soil is determined by the major principal consolidation stress $\overline{\sigma}_{1c}$. Henkel (1956, 1959 and 1960) extended the idea by performing series of tests on normally consolidated Weald clay. Three conclusions were drawn:

a. Strength is uniquely related to the moisture content at failure.

b. Strength is uniquely related to effective stress at failure.

c. Effective stress is uniquely related to the moisture content at failure.

Ladd (1966) showed that there were differences among the values of strength obtained from the common test methods because of the effects of sample disturbance and/or differences in the state of stress at failure. He concluded that the in-situ strength is a function of the mode of failure and cannot be treated as a unique relationship of the water content.

5. Stress-Strain Relations

Immediate settlement is computed by elastic theory but soil properties measured in laboratory tests tend to over-estimate in-situ settlement. This discrepancy is attributed largely to the difficulty in prediction of field deformation. Ladd (1964) presented an analysis of the difficulties and

showed that the factors influencing the modulus determined by laboratory tests on remolded clay are:

a. the level of application of shear stress,

b. the type of shear test,

c. the stress system,

d. the direction of major principal stress.

6. Shear Testing

The cylindrical compression test is the common type of triaxial test used in research work and routine testing. Consolidated undrained tests with pore pressure measurement on saturated soil is traditionally not only utilized to determine the values of $\bar{c}$ and $\bar{\phi}$, but also to evaluate the values of parameter A and to study the effect of stress history.

Bishop and Bjerrum (1960) discussed about the uses of triaxial tests. The properties of cohesive soil as measured in the triaxial tests have been applied to the solution of stability problems such as bearing capacity of a clay foundation, cuts and excavations in clay, natural slopes, earth pressures on earth retaining structures, the stability of earth dams and many other special cases.

C. GENERAL CONSIDERATIONS ON METHOD OF INVESTIGATION

Professor Tinoco (1970) suggested that this investigation becomes the starting point for the development of suitable procedure for laboratory testing which may allow the soil engineer to predict the behavior of a soil layer under the

the stress sequences imposed during and after construction loading.

This prediction involves quantitative evaluations of the total amount, the rate and the time required of settlement, the rate and time required of pore pressure dissipation, the rate of increase of undrained strength with increasing or decreasing consolidation stresses and its relationship to the stability of the soil mass.

In his opinion, the stress history, the stress path and the applied stress system may significantly change the properties and behavior of the clay layer and thus, standard shear and consolidation tests may predict the behavior of the layer if the field loading path is relatively similar or if the factor of safety against a stability failure is larger than 2. However, prediction for field problems with factor of safety close to unity against stability failure or with an expected large amount of settlement, the standard methods may not reliably evaluate the behavior of the soil layer.

Partial fulfillments of the requirements imposed by the complex field problems may be obtained by triaxial shear tests which simulate the field loading path. This type of test is herein referred to as a sequential triaxial shear test. It should be pointed out that this type of test should not be confused with the multi-stage triaxial test (Taylor, D. W., (1950)) because the objectives of each of

them are different. The procedure and the name of the tests were also suggested by Professor Tinoco.

II. LABORATORY TESTS AND PROCEDURES

A. DESCRIPTION, PREPARATION AND PROPERTIES OF A SILTY-CLAY SOIL

The soil chosen for this research was a mixture of a commercial grundite clay and a laboratory prepared coarse grained silt, here-after referred to as Baxter silt. This mixture was chosen so as to speed up the making of samples in a short period of time.

The grundite clay is a dark grey soil, procured from the Illinois Clay Products Company. X-ray diffraction analysis was used to identify the clay minerals. It indicated that grundite clay consisted mainly of illite. The physical properties of the soil are listed as follows: (Jackson, A. T., 1968)

Liquid Limit = 56.0%

Plastic Limit = 27.0%

Specific Gravity = 2.75

X-ray diffraction analysis was also used to identify the mineral composition of the Baxter silt as primarily quartz.

Twenty percent of silt was mixed with eighty percent of grundite clay. The soils were mixed in a dry condition in a sealed paper cylinder. The cyliner was rotated and shaken until the clay and silt became a homogeneous mixture. The physical properties of the silty-clay mixtures are summaried as follows:

Liquid Limit = 45.0%

Plastic Limit = 23.0%

Specific Gravity = 2.74

The grain size distribution of the mixture as determined by hydrometer analysis is shown in Figure 3.

B. SAMPLE PREPARATION

Remolded samples were prepared by method of sedimentation. The sedimentation unit shown in Figure 4 is designed to perform one-dimensional consolidation. The unit consists of a plexiglass cylinder with 1.4 cm. inside diameter. The cylinder is fit to the base with a drainage line.

Before the soil slurry was poured into the cylinder, the base was drained with distilled water to remove air trapped in the drainage line. A porous stone and a moistened filter paper cut to exact size were placed in the base. The drainage line was closed by a pinch clamp.

A thin coating of inert silicone oil was applied to the inside wall of the plexiglass cylinder. The cylinder was then connected to the base and top by means of three threaded brass rods and wing nuts.

The prepared dry silty-clay mixture was mixed with distilled water. After several trials, it was found that 150 gm. of soil mixed with 135 ml. of distilled water would make a workable mixture. The soil slurry was mixed thoroughly in a soil dispersion mixer for 10 minutes. After the soil slurry was well mixed, it was poured into

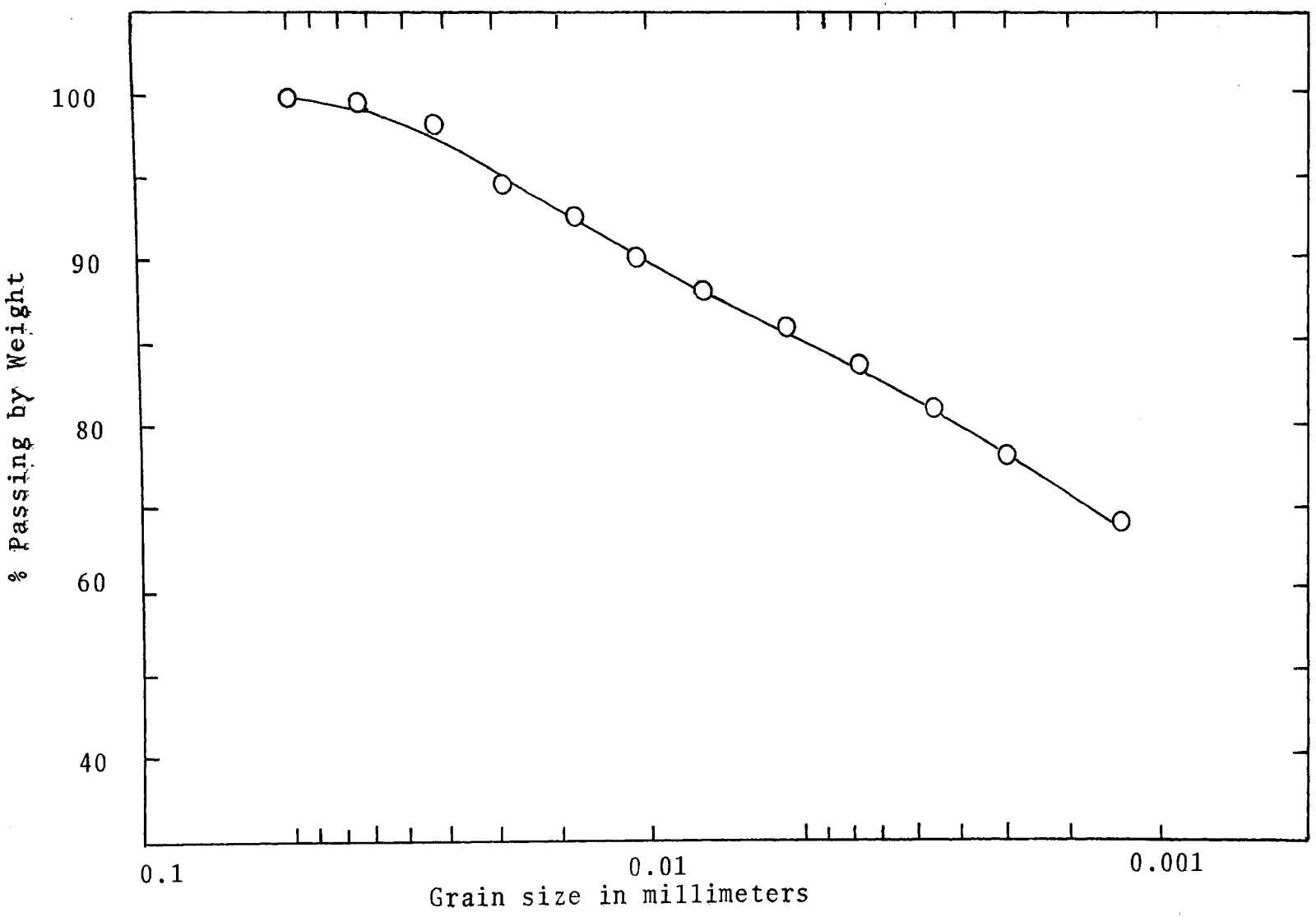

FIGURE 3. GRAIN SIZE DISTRIBUTION CURVES OF GRUNDITE CLAY AND BAXTER SILT MIXTURE

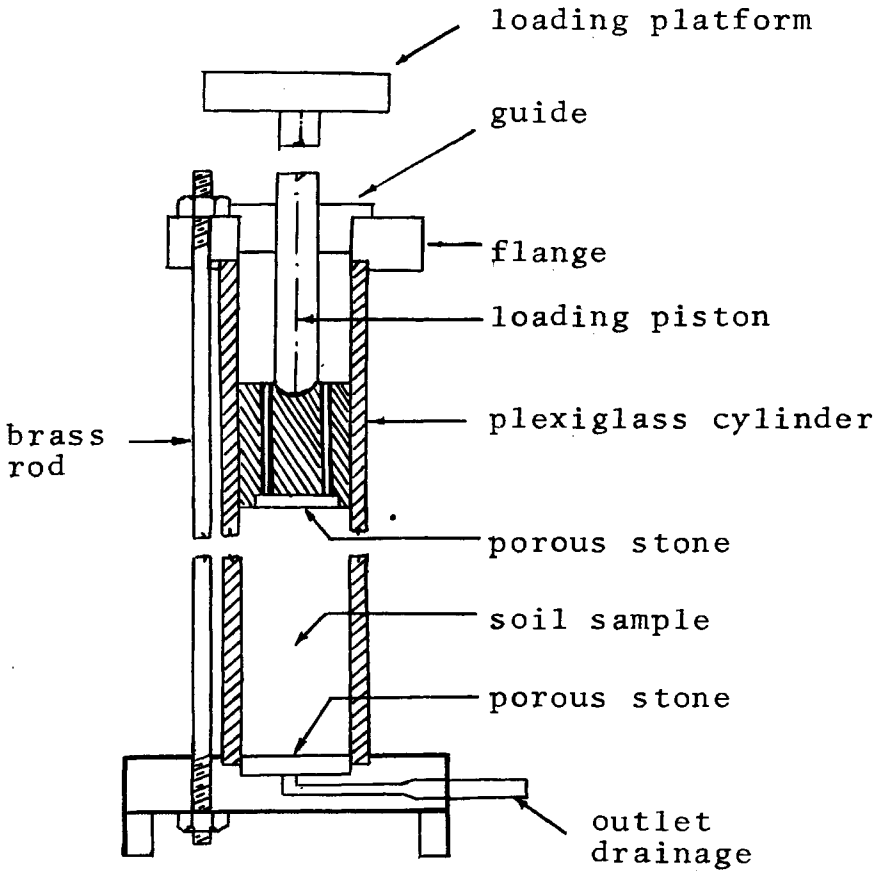

FIGURE 4. SEDIMENTATION UNIT

the sedimentation unit. The slurry was deaired by means of a vacuum system.

After the soil slurry had been deaired, an O-ring and a top piston were placed carefully into the cylinder. Then a steel rod with a platform was inserted on top of the piston. The piston was lowered slowly down the cylinder until it made contact with the top of the soil slurry. Then an axial load of 8 kg. was put on the loading system. The sample was allowed to consolidate for four days before it was suitable for test.

When the sample was ready for test, it was extruded out of the cylinder with the steel rod. The whole length of the sample was about 10 cm. It was then trimmed at both ends by a wire saw to a length of 8 cm. The weight, height and diameter of the sample were recorded.

The sample was subsequently placed in the triaxial cell.

The sample was then reconsolidated under all around pressure of 1 kg./sq. cm. in order to insure uniformity of moisture content.

A hydrometer analysis was performed on top and bottom half of one sample to check for segregation. It was found that the amount of segregation was negligible. The variation of moisture content of the sample before consolidation is given in Table IIin Appendix A.

C. STANDARD TRIAXIAL COMPRESSION TESTS

1. Apparatus

Triaxial cell. All the test samples were first consolidated in a cylindrical triaxial cell built by the Norwegian Geotechnical Institute, Oslo (Anderson, A. and Simons, N. E. (1960)). A rotating bushing in each triaxial cell was used to reduce the piston friction.

Bourdon gauges and pore pressure unit. The cell pressure was measured by Bourdon gauges. A BLH strain-indicator and C. E. C. Transducer type of 4-312-0001 were utilized to measure the pore pressure. Before the test, the Bourdon gauge was calibrated against the pore pressure measuring unit.

Loading device. An axial compression load was applied to the ends of the test sample by a constant strain-controlled type Mossco loading press. A calibrated proving ring with two dial gauges was used to measure the axial load and the vertical deformation of the sample.

2. Procedure

All the triaxial compression tests performed in the course of this investigation were of the consolidated-undrained type with measurement of pore pressure during both the consolidation and the shearing phases. Some samples were consolidated isotropically and others anisotropically in the triaxial cell. Single drainage from the top of the sample was provided by using porous

stones under the top cap with connection outside the cell. Slotted Whatman No. 54 filter paper drains were used for radial drainage. The bottom porous stones were used to measure pore pressures.

The initial length to diameter ratio of the sample is about 2.3 which is satisfactory for end effects and column action (Bishop, A. W. and Henkel, D. J., 1962).

Before mounting the sample, the vertical surfaces of top and bottom caps were polished and greased with silicone to reduce the leakage between the membrane and the cap (Poulos, S. J., 1964). Double Trojan rubber membranes of 0.002 in. each in thickness were used to case each sample. The rubber membranes were sealed against the loading cap and the base pedestal by two rubber o-rings. The cell was then filled with deaired water.

Since pore pressures were measured at the bottom porous stone during isotropic consolidation, drainage from the top cap was necessary. A connection from the loading cap led from the cell to a 50 c.c. burette. Pore pressures were measured at the base of the sample by using the transducer and the BLH strain indicator which was read to micro-divisions. It was found that 1 Kg./sq.cm. of pressure was equal to 710 micro-divisions. The parameter, B (Skempton, A. W., 1954) was determined for each increment of cell pressure to determine the degree of saturation of the sample. Samples were considered not sufficiently

saturated and were rejected for values of B less than 0.95.

In order not to prestress the sample, cell pressures were applied in increments of every 0.5 kg./sq.cm. and when the pore pressure achieved equilibrium, drainage was then permitted and the sample was allowed to consolidate. During consolidation, time, volume change and decrease of pore pressure at the base of sample were recorded.

When there was no further noticeable change in the volume of water in the burette, the consolidation of a sample was considered to be complete. All the drainage lines were then closed and the cell was mounted in the loading press. The cell pressure was increased to 3.0 kg./sq.cm. before shear. This was required that further consolidation at higher stress could be performed during sequential loading.

When the sample was ready for shear, a calibrated proving ring was brought into contact with the top of the piston.

In all cases during shearing, the cell pressure was kept constant and the axial load increased at a constant rate of strain of 0.0022 in. per minute. The reason for choosing this rate of strain was to balance the time available and the amount of strain at failure, so that the number of sequential loading stages would be constant when 20 to 25 percent strain was reached, the shearing was

stopped and the sample was removed from the triaxial cell. The final height of the sample was measured. Then the sample was cut into five sections for moisture content determinations. Results of moisture content of samples at the end of tests are summarized in Table IIIin Appendix A.

Results of the test were not corrected for the filter paper and rubber membranes because the stress corrections were considered to be negligible.

D. SEQUENTIAL LOADING IN TRIAXIAL COMPRESSION TESTS

In order to obtain a better understanding of the influence of the degree of consolidation on the undrained strength of a fine-grained material, a special test, the sequential triaxial test was developed. Samples were sheared sequentially in three or four stages. After each stage, the sample was consolidated at a higher effective pressure before undrained shear was restarted.

1. Apparatus

The apparatus utilized for this type of tests were the same as previsouly described in Section 1 of Part C. However, for the cases of anisotropic and K_o consolidation, a special Geonor loading device was used to apply the axial stress.

2. Notation and Definitions

The various types of triaxial tests which were carried out are summarized below. The dash above the notation of the test indicates that pore water pressure measurements

were made during the tests.

$\overline{\text{CIU}}$ test

One series of standard undrained tests were performed on isotropically consolidated samples with measurements of pore pressure.

$\overline{\text{CIU}}$-$\overline{\text{SCAU}}$ test

Another series of samples were consolidated isotropically and tested undrained in shear ($\overline{\text{CIU}}$). After a strain of 4 percent to 5 percent was reached, sequential loading was started consisting of sequences of anisotropic consolidation and tested undrained ($\overline{\text{CAU}}$) until a strain of 20 to 25 percent was reached.

$\overline{\text{CAU}}$-$\overline{\text{SCAU}}$ test

Samples were consolidated anisotropically and tested undrained in shear ($\overline{\text{CAU}}$). Then the samples were tested sequentially similar to that of the $\overline{\text{CIU}}$-$\overline{\text{SCAU}}$.

CA-$\overline{\text{UU}}$-$\overline{\text{SCAU}}$ test

A series of samples were consolidated anisotropically and then the consolidation shear stresses were released at constant volume followed by undrained shear for sequential stages.

$\overline{\text{OCIU}}$-$\overline{\text{SCAU}}$ test

The overconsolidated samples were first consolidated isotropically and then were allowed to swell to a reduced value of effective stress. The overconsolidation ratio (OCR), equals $\overline{\sigma}_{cm}/\overline{\sigma}_{c}$ where $\overline{\sigma}_{cm}$ is the maximum previous

consolidation pressure and $\bar{\sigma}_c$ is the consolidation pressure.

3. Procedure

CAU-SCAU test

Anisotropic consolidation was carried out by applying a major principal stress to the sample during consolidation. Axial loads were applied in small increments of about 0.1 kg./sq.cm. to obtain a predetermined ratio of horizontal to vertical stress for consolidation of the sample before the next increment of loading was applied (Lowe, J. and Karafiath, L., 1960).

When the consolidation of the sample was completed, the triaxial cell was mounted in the loading press, and the vertical loading device was removed. The proving ring was brought into contact with the piston, and a vertical stress equivalent to the previous stress was applied by moving the press to contact the sample until the proving ring dial read the amount computed by the following equation:

$$LD = \frac{W_A + W_H + W_{DL} + a_{rp}\,\sigma_3}{A_s} \qquad (5)$$

Where L = proving ring factor expressed in load per division,

D = proving ring deflection in divisions from zero load,

W_A = weight of the arm lever of the anisotropic loading device,

W_H = weight of hanger of the anisotropic loading device,

W_{DL} = weight of dead load on the hanger,

a_{rp} = area of the piston,

A_s = area of the sample at any time.

For the sequential tests, the samples were sheared until a maximum deviator stress was reached which generally occurred at about 4 percent to 5 percent of axial strain. The motor was then stopped and consolidation of the sample under the applied stress was started by opening the drainage line. The change in volume and pore pressure dissipation were recorded. During the consolidation phase, the vertical stress was kept constant. This was done by utilizing a hand wheel to make the contact between the piston and sample all the time. When a pore pressure decrease of 0.5 kg./sq.cm. was reached, the drainage lines were closed again and the second undrained shearing stage was immediately started. Two or three sequences of the above described procedure were performed until an amount of 20 to 25 percent of axial strain was reached, and the test was stopped at this point. The sample was removed from the triaxial cell and its height was measured. Then the sample was cut into five sections for determination of water content.

CAU-UU-SCAU test

The procedure for the consolidation of the sample was the same as that for CAU-SCAU test, but before shearing, the axial load was released without drainage until it became equal to the cell pressure. The sample was sheared

by the previously described method of sequential loading.

$\overline{\text{CIU-SCAU}}$ test

The procedure was the same as $\overline{\text{CAU-SCAU}}$ but with $\overline{\text{CIU}}$ consolidated isotropically and undrained in shear as previously described.

$\overline{\text{OCIU-SCAU}}$ test

The overconsolidated samples were prepared by first isotropically consolidating the sample to a cell pressure of either 3.0 kg./sq. cm. or 4.0 kg./ sq. cm. Once the consolidation of the sample was completed, all drainage lines were closed. The cell pressure was reduced to 1 kg./ sq. cm. The drainage lines were reopened and the swelling process initiated. For the swelling stage, 24 hours was allowed for equilibrium. After swelling was completed, the sample was sheared by the same procedure as described in $\overline{\text{CAU-SCAU}}$.

All the sample preparation and testing were performed in a temperature controlled room at 70° F.

E. CALCULATIONS

The calculations of the test data were reduced and summarized by computer, IBM 360/50. A sample program is given in Appendix A.

The computer program for calculation of strength parameters for the sequential triaxial test is similar to that for the standard triaxial compression test, however, the data of every sequential loading is regarded

as a new triaxial test by itself. New heights, diameters and volumes were calculated for each stage.

F. SUMMARY OF LABORATORY TESTS

Five series of tests were scheduled for this research. Table I summarized all types of tests and the details of each test.

III. PRESENTATION OF TEST RESULTS

A. CONSOLIDATION PHASE

1. Preshear

Prior to undrained shear, all the samples were initially consolidated either isotropically or anisotropically. Only those samples isotropically loaded are considered in this section. A typical test result of compression and degree of consolidation in percent versus time for $\overline{CIU}$-$\overline{SCAU}$ test is plotted in Fig. 5 . The compression is computed by $\Delta V/V_T$, where ΔV is the change in volume and V_T is the total volume of the sample at the beginning of the test. The degree of consolidation U is defined by the equation, (Taylor, D. W., 1948)

$$U = 1 - u/u_i, \tag{6}$$

where u is the pore water pressure at any time and u_i is the initial pore water pressure.

Both the compression and pore water dissipation curves are similar and they follow Terzaghi's theory of consolidation. However, by extra-polation of the curves, the pore water pressure dissipated completely before the volume change became negligible. A larger increment of total stress produces a greater time lag between the compression and the degree of consolidation as shown in Fig. 6 for values of 25 and 50 percent consolidation.

The coefficient of consolidation, c_v, was determined by the logarithm of time fitting method. The c_v for $\overline{CIU}$,

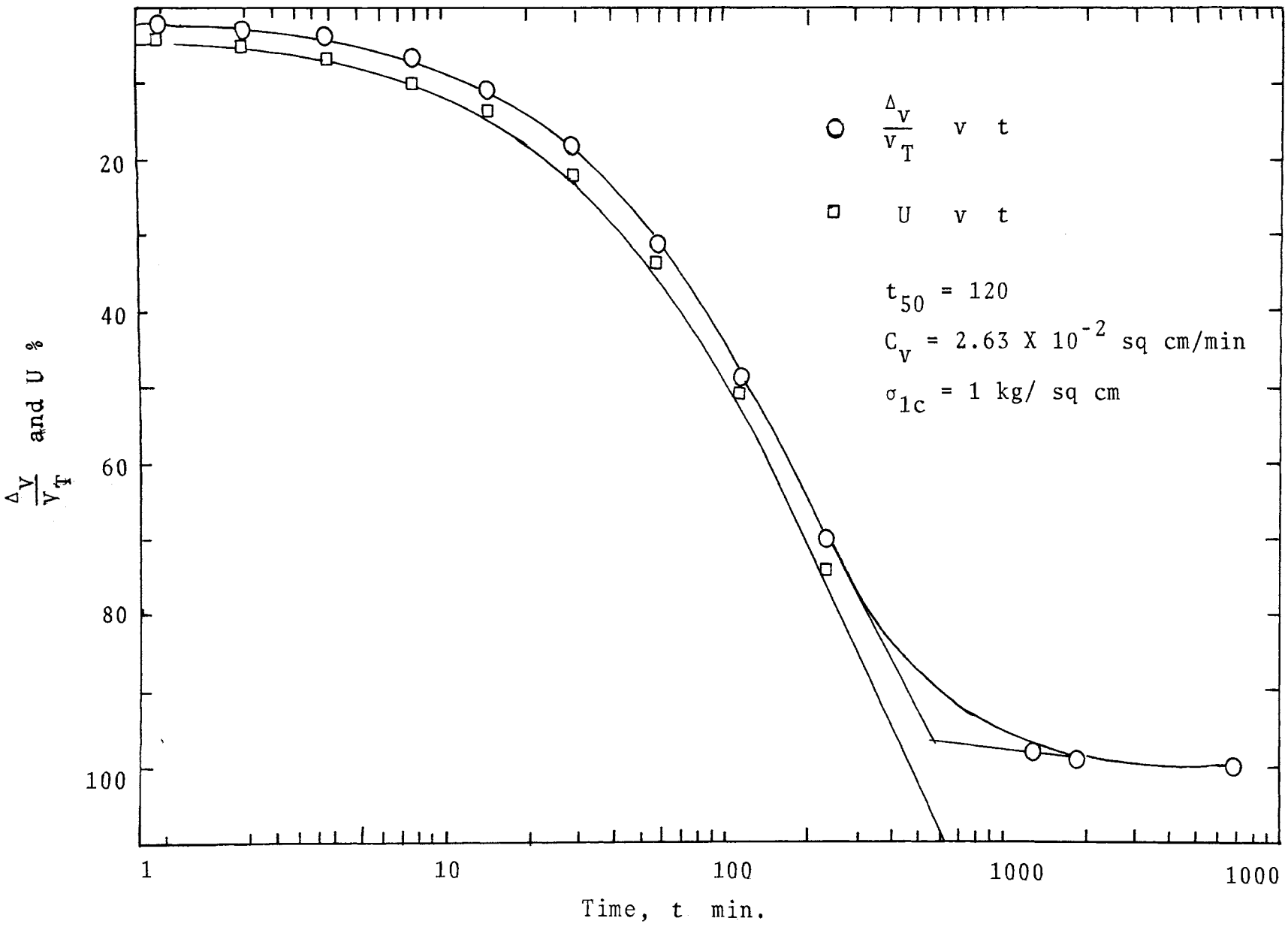

FIGURE 5. TIME COMPRESSION AND DEGREE OF CONSOLIDATION *CURVES FOR CIU-SCAU TEST*

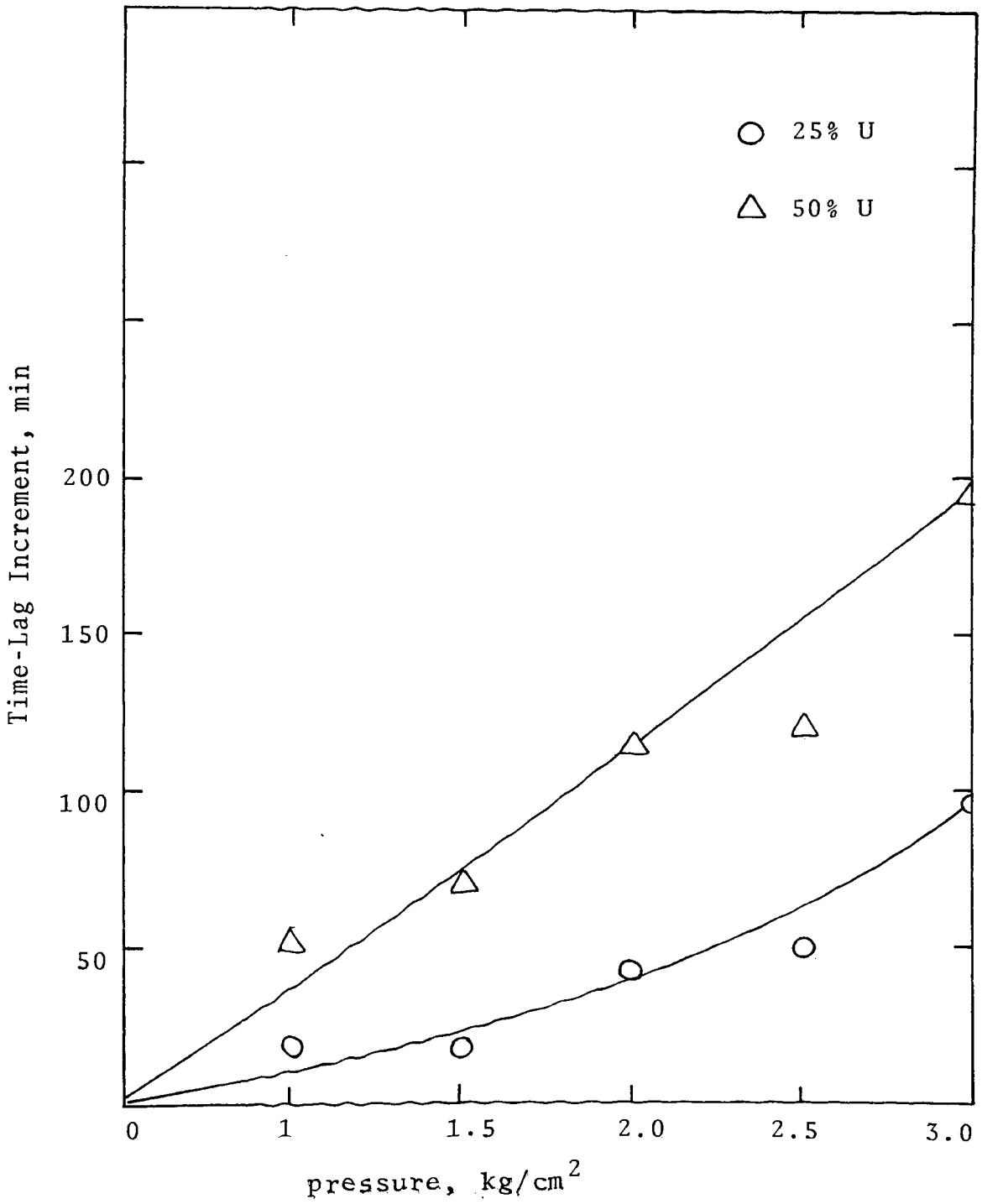

FIGURE 6. TIME-LAG-INCREMENT RELATION TO CONSOLIDATION PRESSURE

$\overline{CIU}$-$\overline{SCAU}$ and $\overline{OCIU}$-$\overline{SCAU}$ tests were found to vary from 2.63×10^{-2} cm^2/min. to 1.80×10^{-2} cm^2/min., and decreased with increasing consolidation pressure and t_{50}. This trend was also found by Taylor (1948) on Chicago and Boston clays for consolidation pressures less than 3.0 kg./sq. cm.

2. After Undrained Shear

Relationship of Compression and Time. The results of partial consolidation with a maximum of 25 percent pore pressure dissipation in the series of sequential loadings are presented in Figures 7, 8 and 9. Linear relationships between Δe and Δu are shown in Figure 8 for a $\overline{CIU}$-$\overline{SCAU}$ test and in Figure 9 for $\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$. The coefficient of compressibility a_v, the slope of the Δe versus Δu plot, decreases as the intergranular stress ratio $K_c = \frac{\bar{\sigma}_{1c}}{\bar{\sigma}_{3c}}$ is increased. The linear relationship of Δe versus Δu is maintained during the sequential stages, and thus it is not affected by changes in permeability and compressibility. This verified the assumption of a linearity between Δe and Δu made in the Terzaghi's theory of consolidation (Terzaghi, K., (1943)). The equation is given by:

$$\Delta e = a_v \, \Delta u \tag{7}$$

The coefficient of permeability at each stage were evaluated by using the equation,

$$k = \frac{a_v \, c_v}{(1 + e) \, \gamma_w} , \tag{8}$$

from Terzaghi's theory of consolidation. The permeability of the sample decreased with the sequential increments in

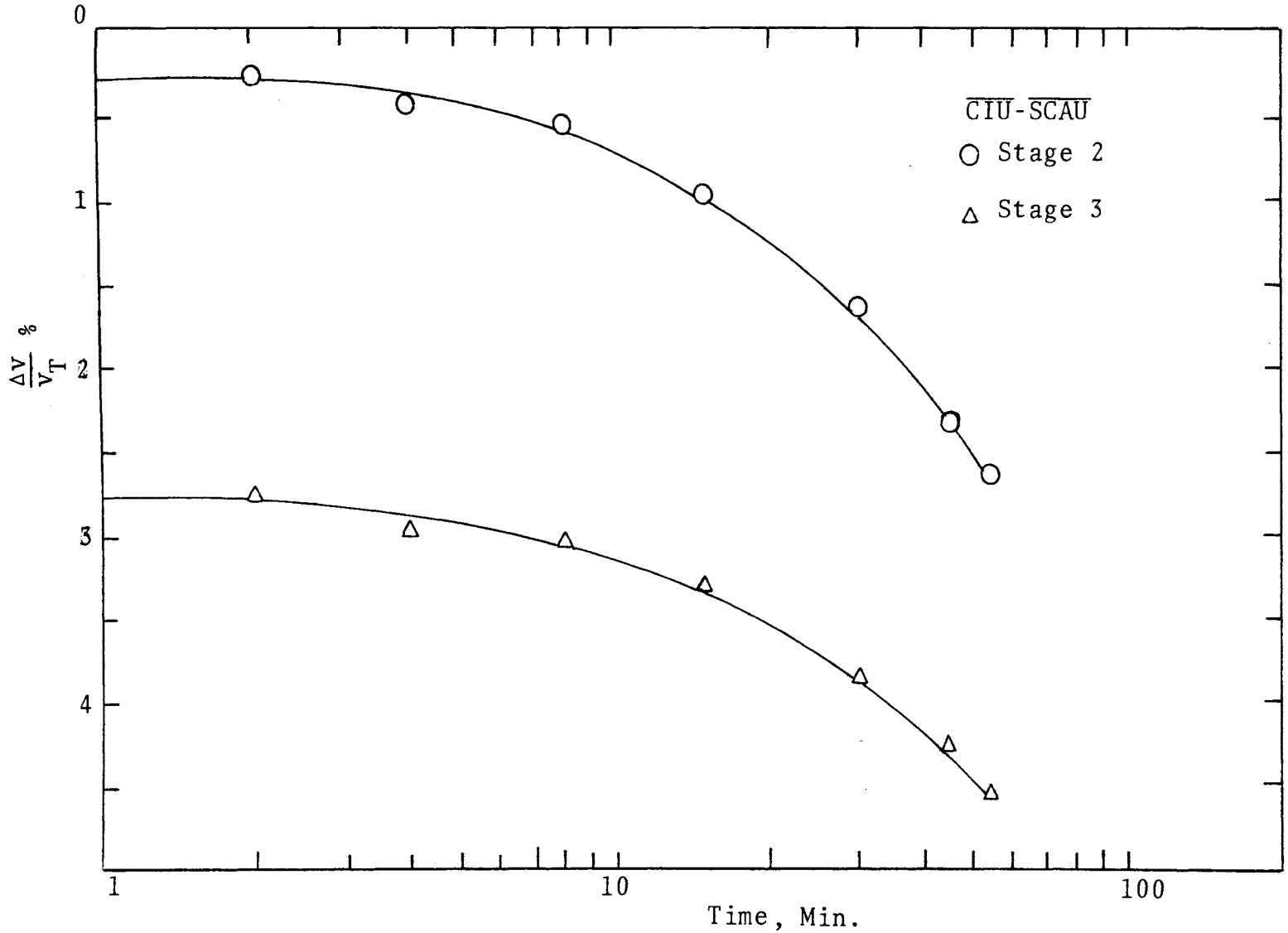

FIGURE 7. TIME-SETTLEMENT DURING CONSOLIDATION

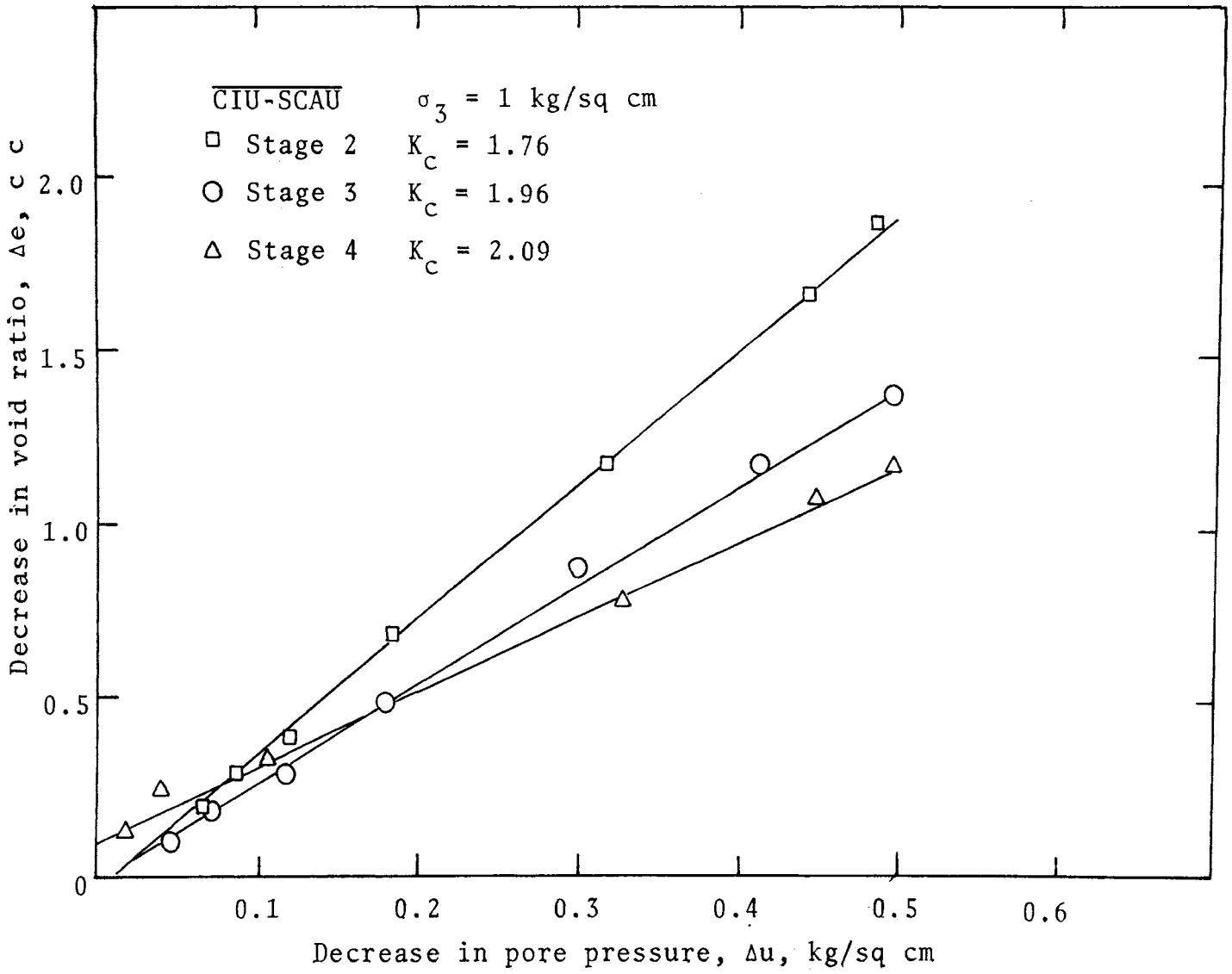

FIGURE 8. RELATIONSHIP BETWEEN DECREASE IN PORE PRESSURE AND DECREASE IN VOID RATIO

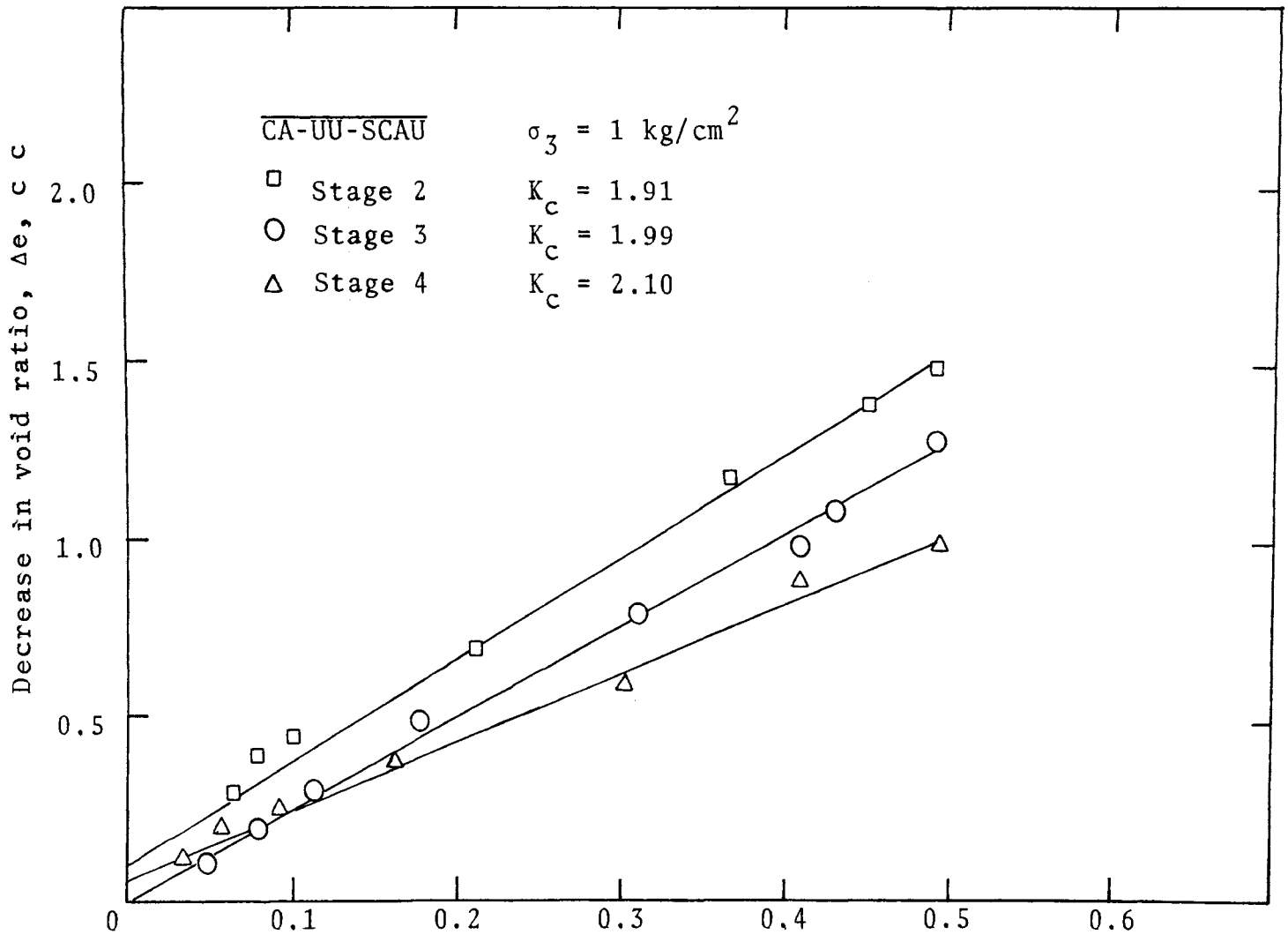

FIGURE 9. RELATIONSHIP BETWEEN DECREASE IN PORE PRESSURE AND DECREASE OF VOID RATIO

the K_c ratio. Curves of time-compression shown in Figure 7 decrease in gradient as the number of consolidation stages increase. This is due to the decrease of compressibility and permeability of the soil sample.

Effect of Pore Pressure Dissipation. The shape of the pore pressure dissipation profiles are nearly identical for all the sequential stages (Figure 10). The time taken for the pore pressure to dissipate 0.5 kg./sq. cm. were approximately the same. This allowed 25 percent of consolidation before shear again.

The decrease in compressibility and permeability of the sample did not affect the shape of the pore pressure dissipation curve. Figures 10 and 11 also show that these curves seem to be independent of the applied consolidation ratio.

The percent of volumetric compression during the sequential stages is very small, i.e. not larger than 2 percent of the total volume change for each stage. At a given time, the percent of volume change does not correspond with the percent of pore pressure dissipation. No more than 25 percent of the pore pressure was permitted to dissipate. This means that the time-compression and time-degree of pore pressure dissipation curves are not identical.

B. SHEAR

1. Stress-Strain Relationship

Axial stress differences for sequential stages are plotted against percent strain in Figures 12 and 13. It is

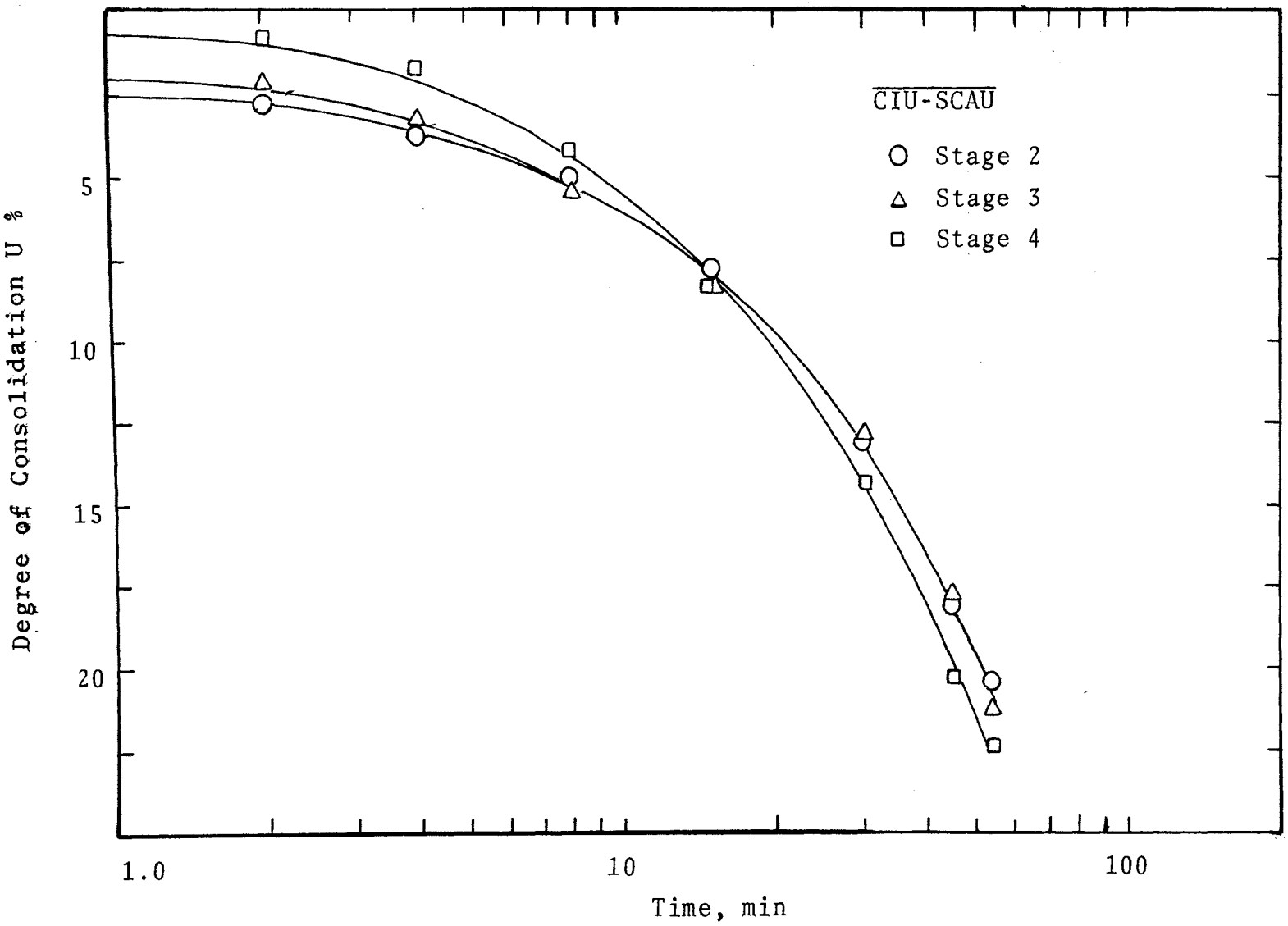

FIGURE 10. TIME-DEGREE OF CONSOLIDATION DURING SEQUENTIAL LOADING

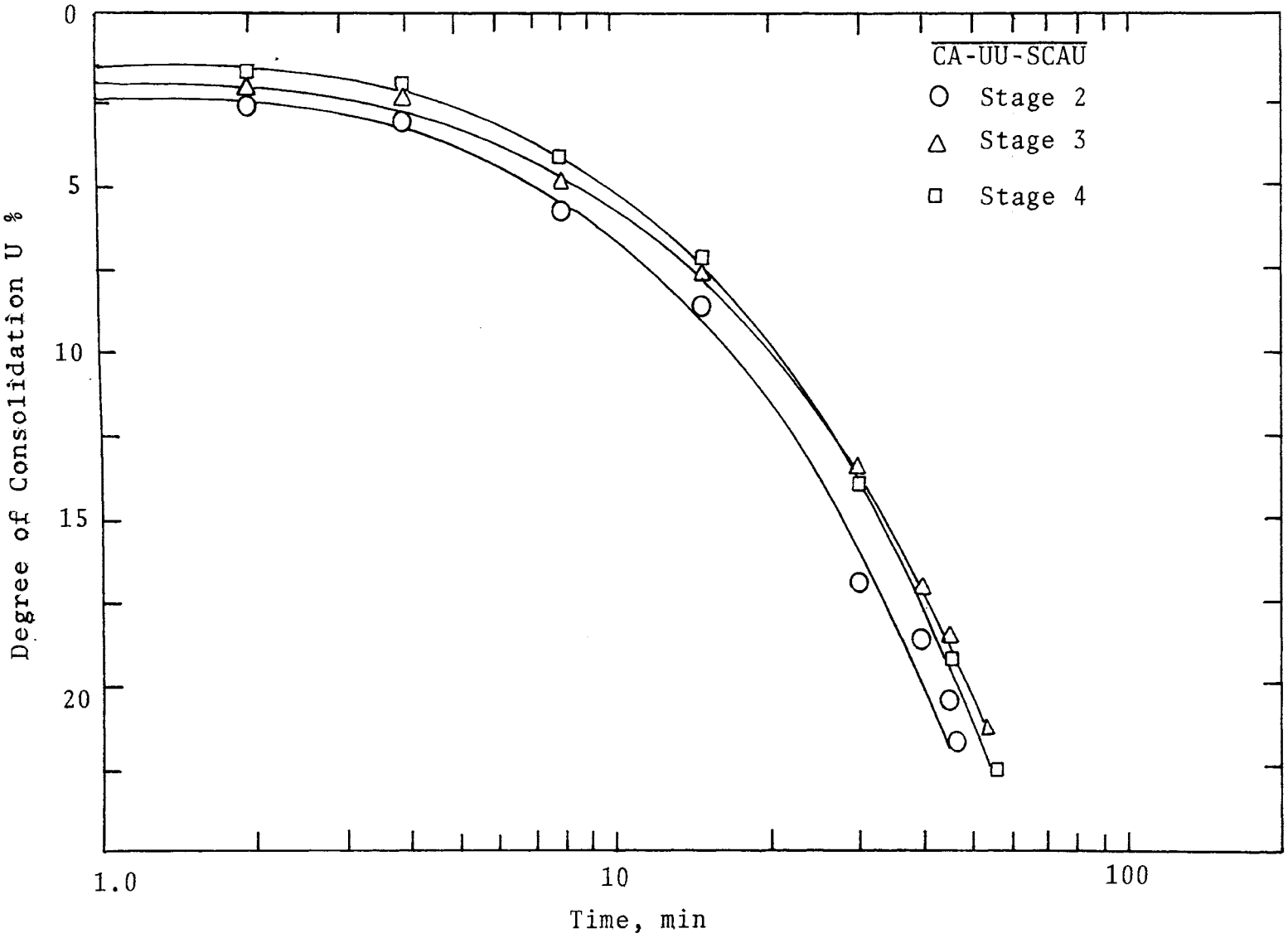

FIGURE 11. TIME-DEGREE OF CONSOLIDATION DURING SEQUENTIAL LOADING

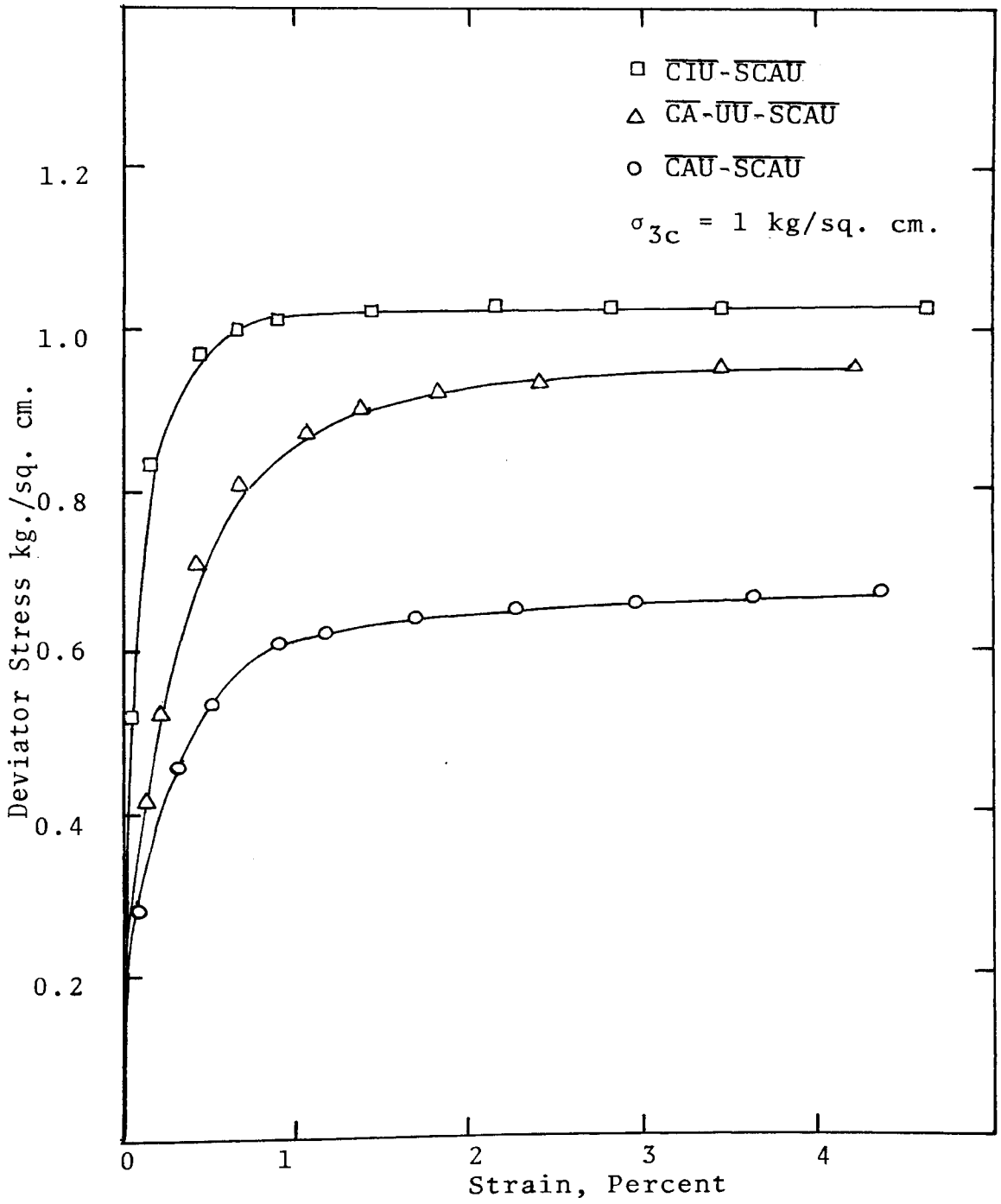

FIGURE 12. STRESS-STRAIN RELATIONSHIP FOR FIRST STAGE

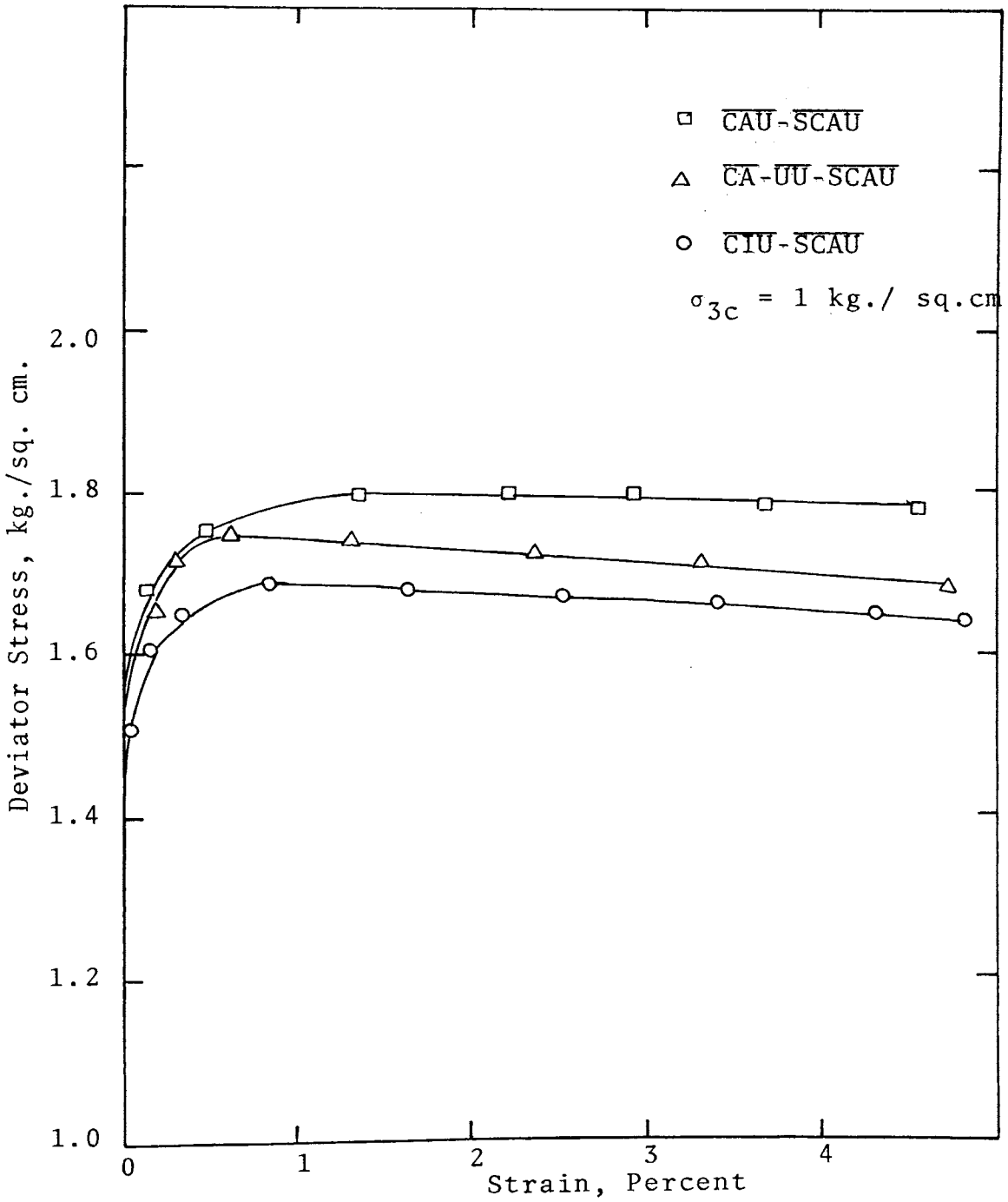

FIGURE 13. STRESS-STRAIN RELATIONSHIP FOR LAST SEQUENTIAL STAGE

interesting to note that the last stages of sequential tests reached maximum stress difference at a relatively low percent of strain. Figure 13 shows that these stress-strain curves are similar to those of slightly overconsolidated soils since the deviator stress decreases with larger strains. The mode of failure in normally consolidated soil is changed as further shear loading is imposed upon the sample. Then the soil seems to reach a maximum deviator stress with little amount of strain and then a decrease in load carrying capacity.

The variation in E, the secant modulus of elasticity, with stress level is shown in Figures 14 and 15 in which $E/\overline{\sigma_{1c}}$ is plotted versus the ratio of $(\sigma_1 - \sigma_3)_f/(\sigma_1 - \sigma_3)$. This is the ratio of deviator stress at failure to the applied stress, hereafter called factor of safety. For sequential loading, the ratio $E/\overline{\sigma_{1c}}$ tends to decrease with increasing consolidation pressure at a F.S. of 4. The values of $E/\overline{\sigma_{1c}}$ remain constant at F.S. of 3 or 4 and it is also constant between stages 3 or 4. The decrease of $E/\overline{\sigma_{1c}}$ with consolidation pressure was also found to be true in the standard $\overline{CIU}$ test.

2. Pore Pressure Parameters

The value of the parameter A_f is calculated by using the equation (Skempton, A. W., 1954),

$$A_f = \frac{\Delta u_f}{\Delta(\sigma_1 - \sigma_3)_f} \qquad (9)$$

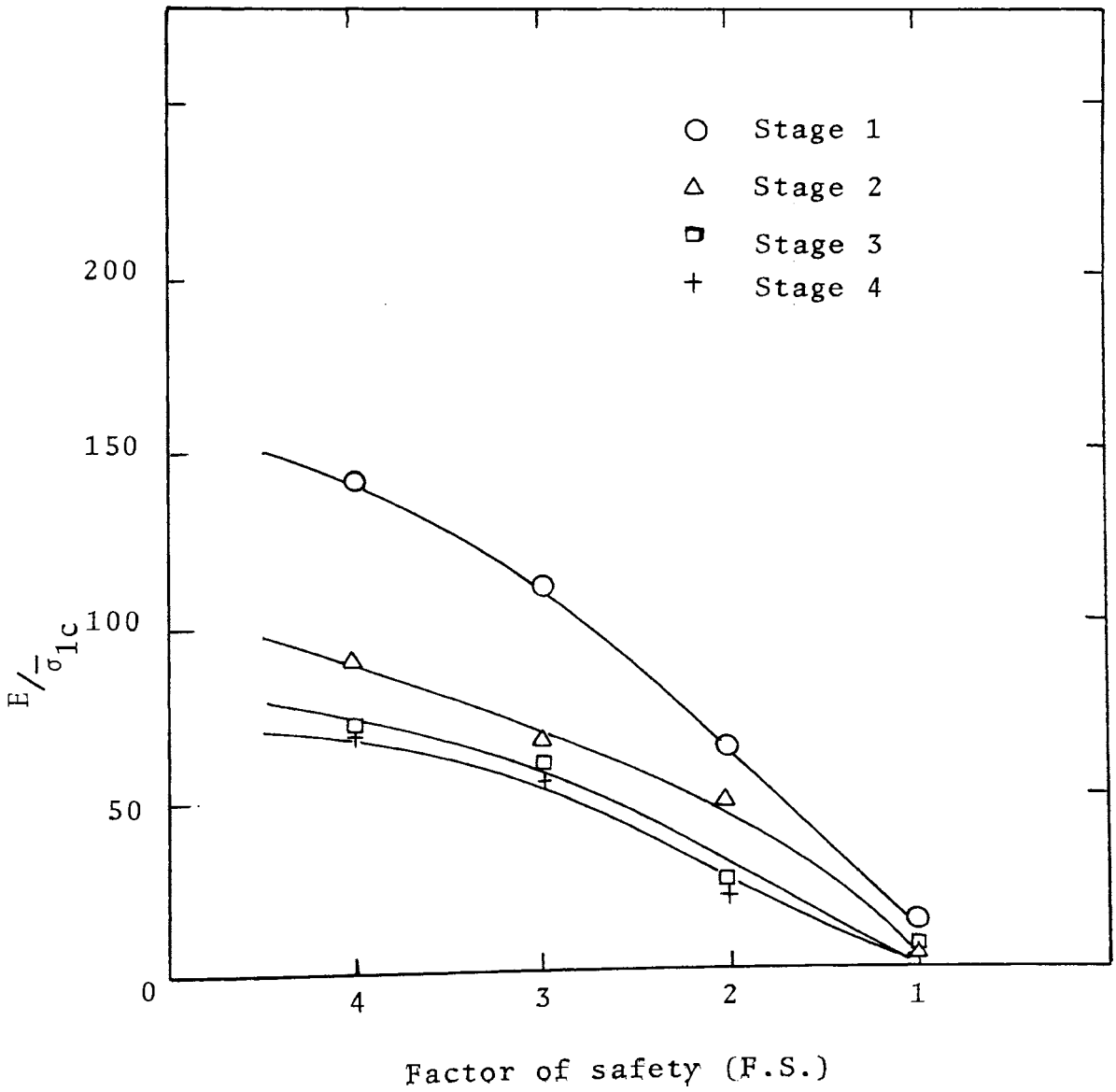

FIGURE 14. RELATION OF $E/\bar{\sigma}_{1c}$ - FACTOR OF SAFETY FOR $\overline{\text{CIU-SCAU}}$ TEST

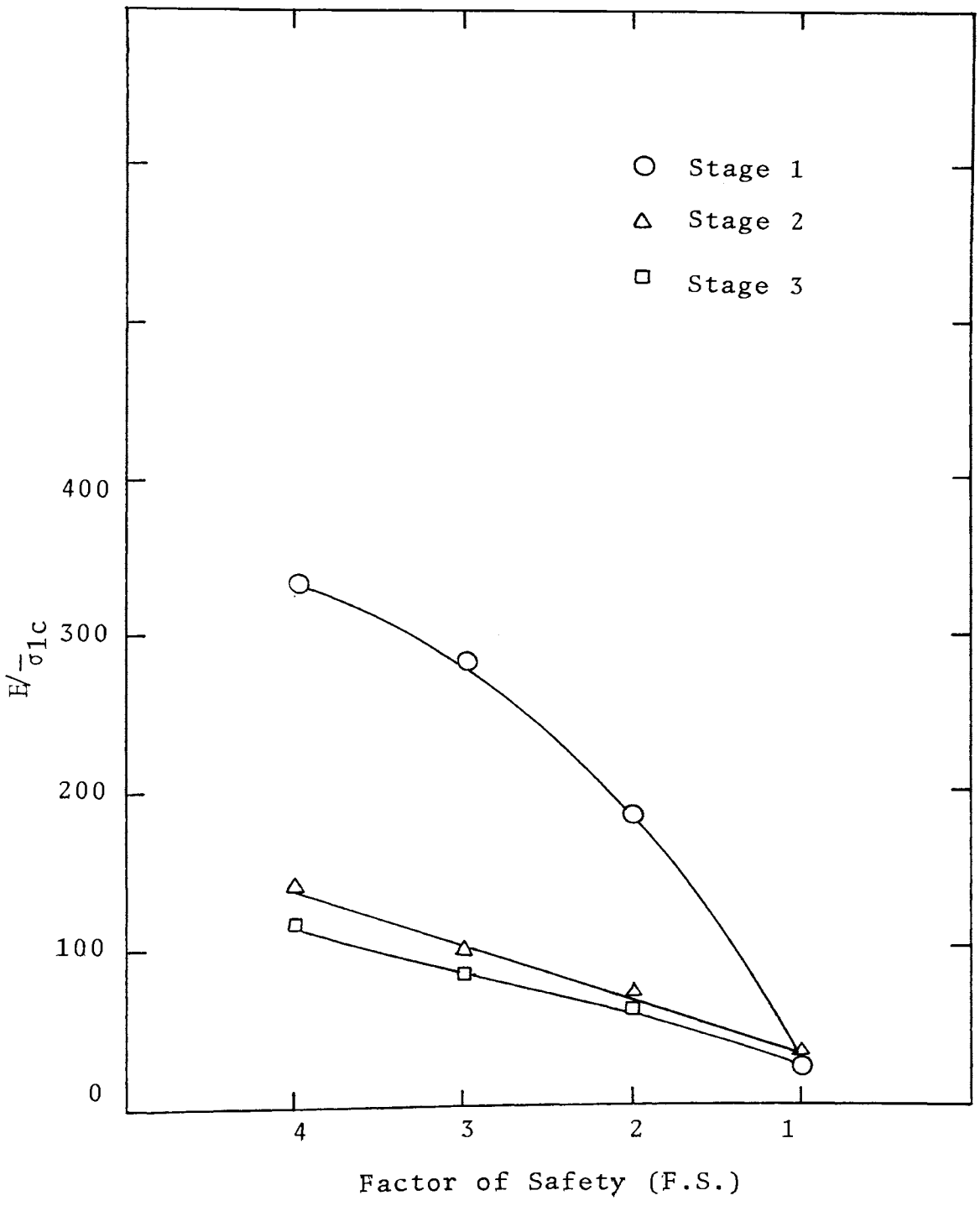

FIGURE 15. RELATION OF $E/\bar{\sigma}_{1c}$ - FACTOR OF SAFETY FOR $\overline{\text{OCIU}}$-$\overline{\text{SCAU}}$ TEST

Here the A_f values are not the highest values. They are taken at the maximum stress ratio. In order to understand the relationship between A_f and undrained strength, Figure 16 for the sequential stages was plotted. For normally consolidated soil, the sequential loading gives a maximum value of A_f of 1.5 for $\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$ and about 1.0 for $\overline{CAU}$-$\overline{SCAU}$. The $\overline{OCIU}$-$\overline{SCAU}$ seems to result in increasing A_f as sequential stages proceed with the A_f increasing to approximately 1.0. It is apparent that the sequential loadings do not affect A_f for the normally consolidated soil. However, the values of A_f vary with the stress history.

Figures 17 and 18 show the linear relationship of A_f versus q_f and $\Delta u_f/\bar{\sigma}_{1c}$ versus q_f for the first shearing stage of different sequential tests. Two trends can be obtained. For the consolidation pressure of 1 kg./sq. cm. (x - lines) $\overline{CIU}$-$\overline{SCAU}$ test gives a higher A_f and $\Delta u/\bar{\sigma}_{1c}$ than those of other tests. The line Y shows the linear relationship for those samples consolidated to 2.0 kg./sq. cm.

It is interesting to note that a linear relation between $\Delta u/\bar{\sigma}_{1c}$ and q_f is obtained (Figure 19) for all tests in this soil. There is some scatter but the following was derived:

$$\frac{\Delta u}{\bar{\sigma}_{1c}} = M - N \log q_f/q_o \qquad (10)$$

where M is a constant evaluated at q_o, kg/cm^2, and N is the value of the slope of the line relating $(\frac{\Delta u}{\bar{\sigma}_{1c}})_f$ to the logarithm of q_f. Its value is 0.302. The constant M can

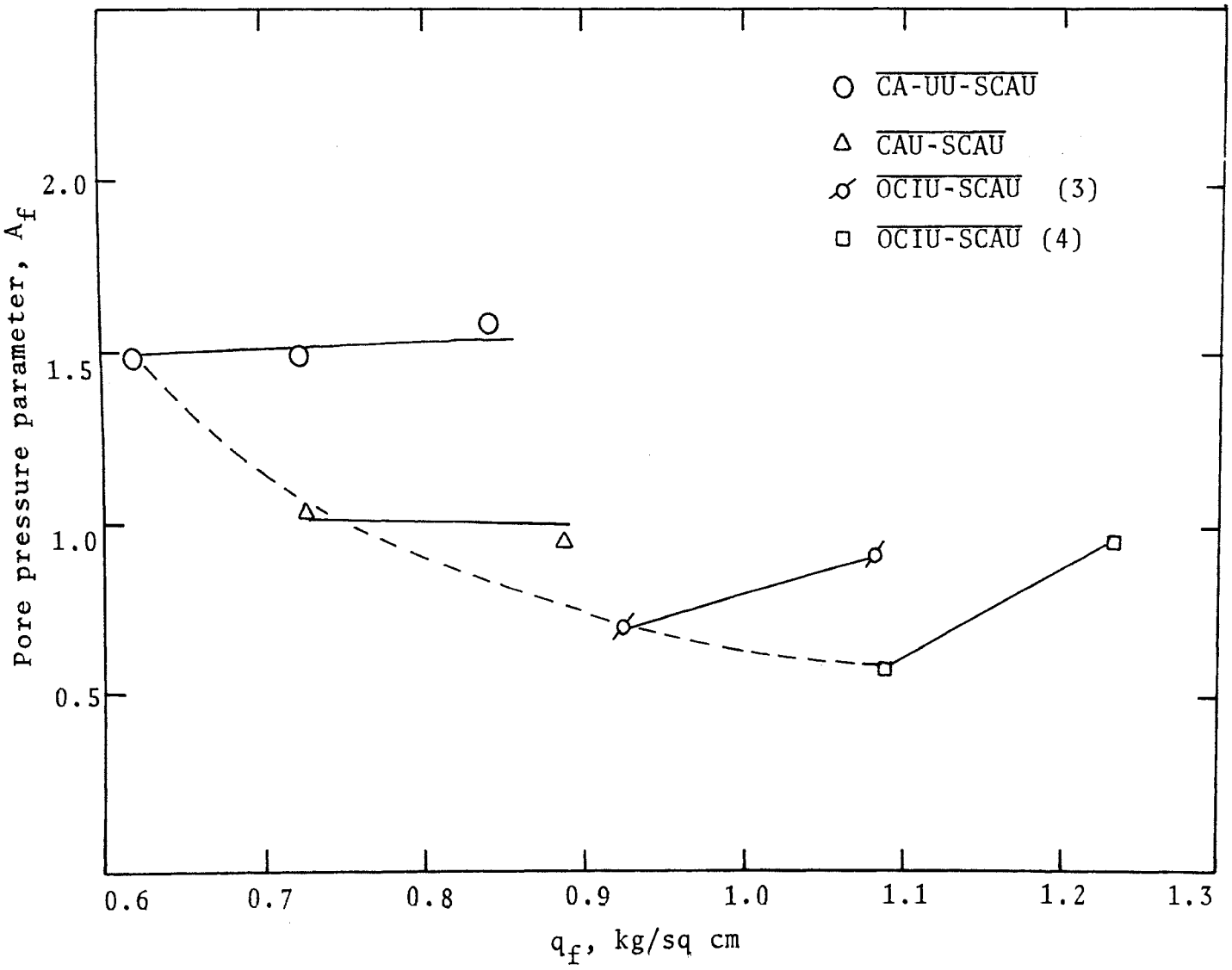

FIGURE 16. RELATION BETWEEN PORE PRESSURE PARAMETER, A_f AND UNDRAINED STRENGTH FOR SEQUENTIAL STAGES

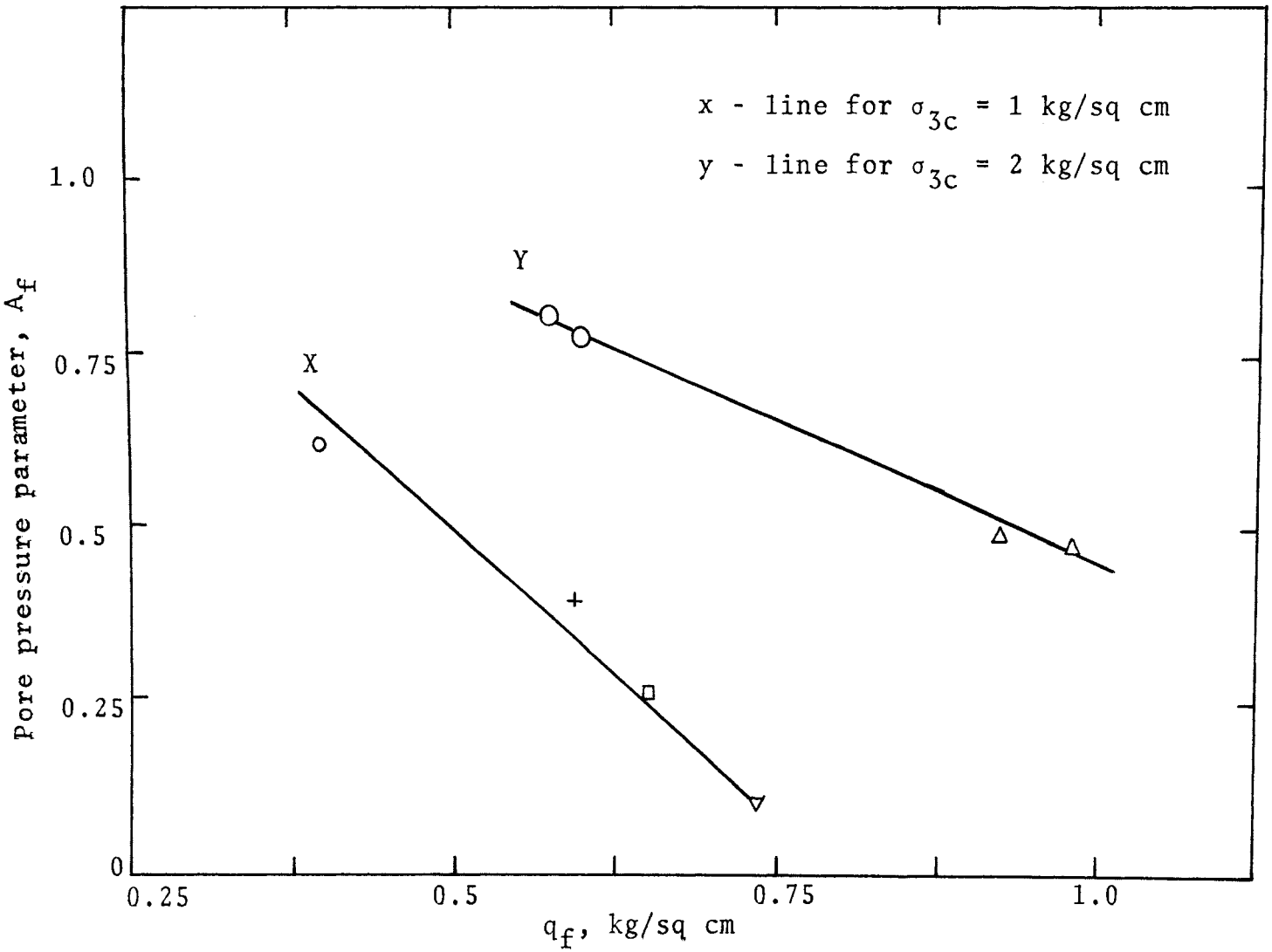

FIGURE 17. RELATIONSHIP BETWEEN PORE PRESSURE PARAMETER, A_f AND UNDRAINED STRENGTH FOR FIRST STAGE

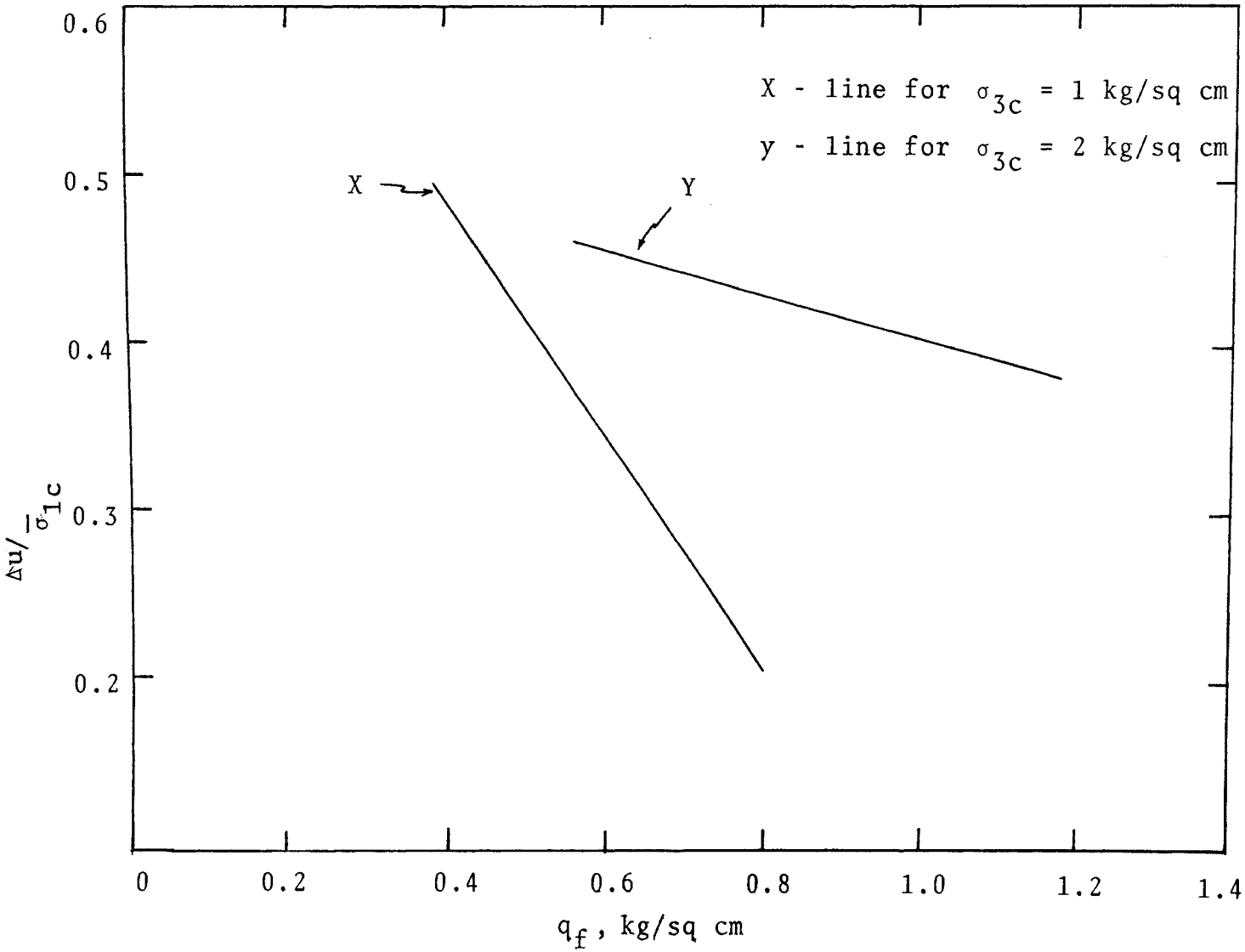

FIGURE 18. RELATIONSHIPS BETWEEN $\Delta u/\bar{\sigma}_{1c}$ AND UNDRAINED STRENGTH FOR FIRST STAGE

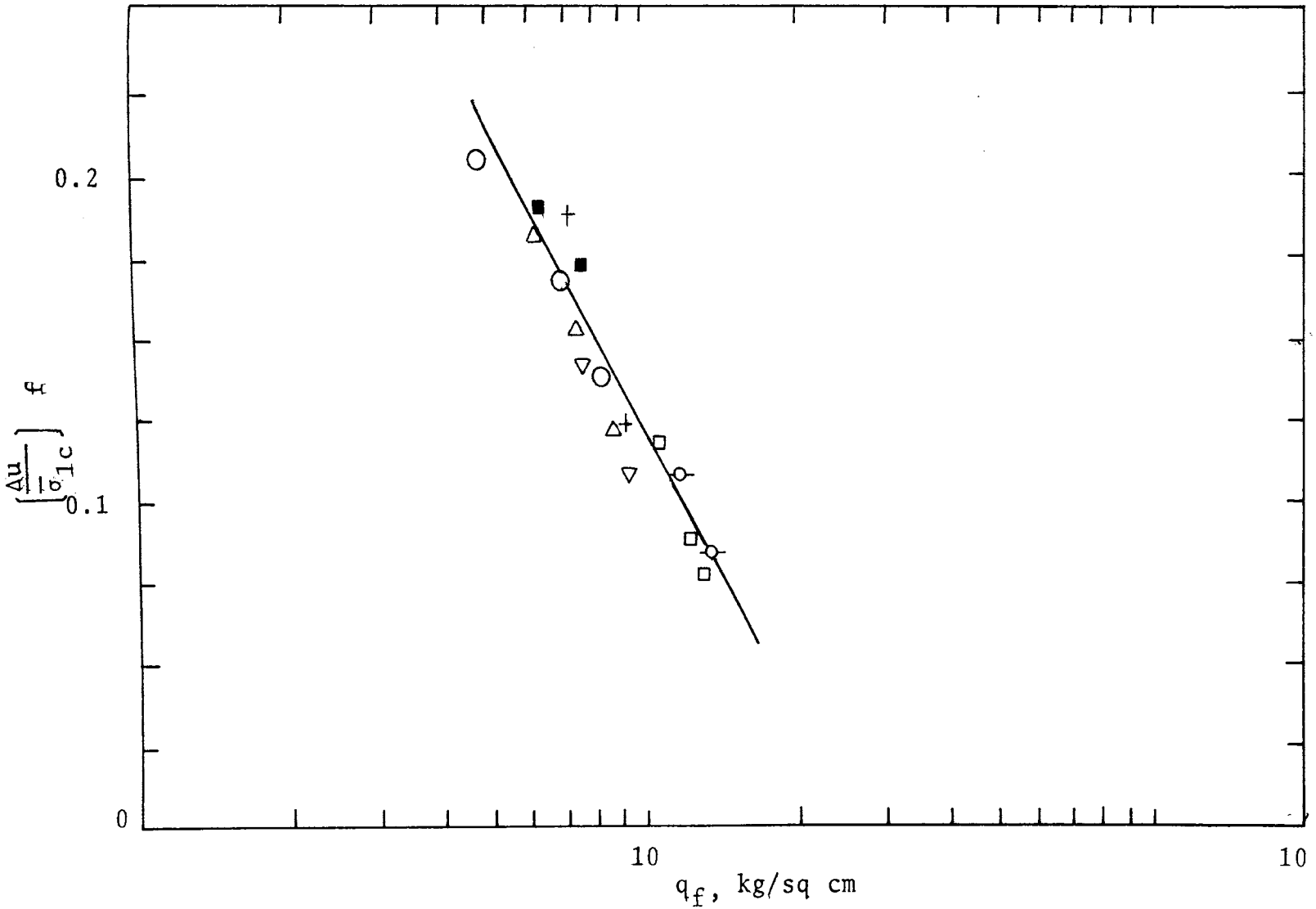

FIGURE 19. RELATION OF $\Delta u/\bar{\sigma}_{1c}$ AND UNDRAINED STRENGTH FOR ALL TEST DURING SEQUENTIAL STAGES

be defined as the intercept on the $\Delta u/\overline{\sigma_{1c}}$ axis for any given value of log q_f.

3. Shear Strength in Terms of Effective Stresses, $\bar{c}$ and ϕ.

It is more convenient to use a modified envelope plotted in terms of the maximum point of the Mohr failure circle (Lambe T. W., 1964) by

$$q_f = \frac{(\sigma_1 - \sigma_3)}{2}_f \quad \text{and} \quad \bar{p}_f = \frac{(\bar{\sigma}_1 + \bar{\sigma}_3)}{2}_f$$

The corresponding parameters in the Mohr-Coulomb diagram can be determined by

$$\sin \bar{\phi} = \tan \bar{\alpha} \tag{11}$$

$$\bar{c} = \frac{a}{\cos \bar{\phi}} \tag{12}$$

Figures 20 to 24 are plots with q_f versus $\bar{p}_f$ at maximum stress ratio. It is observed that the $\bar{\phi}$ values of effective friction angle for $\overline{\text{CIU}}$, $\overline{\text{CIU-SCAU}}$ and $\overline{\text{CA-UU-SCAU}}$ tests are approximately the same. For $\overline{\text{CAU-SCAU}}$ test, the effective friction angle seems to be 5 degrees higher.

Effective Stress Path. Figures 20 to 24 also present effective stress paths for the various types of tests. In Figure 20 are shown stress paths for the standard $\overline{\text{CIU}}$ test. The curves on the four tests with different consolidation pressures are approximately geometrically similar. The lines drawn through points of equal axial strain are approximately straight lines.

Figures 21 and 22 show the effective stress paths for

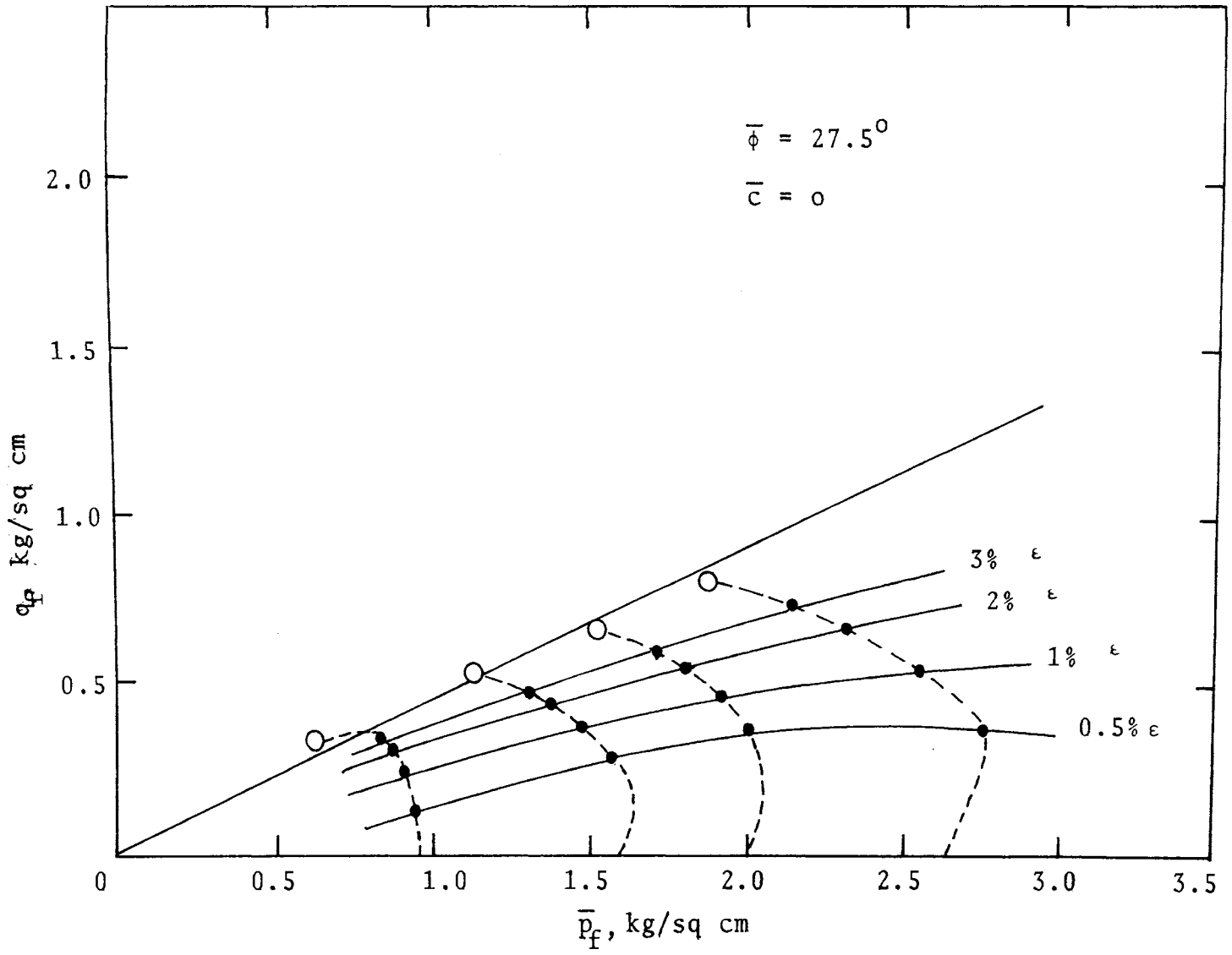

FIGURE 20. STRESS PATH AND K_f-LINE FOR $\overline{CIU}$ TESTS

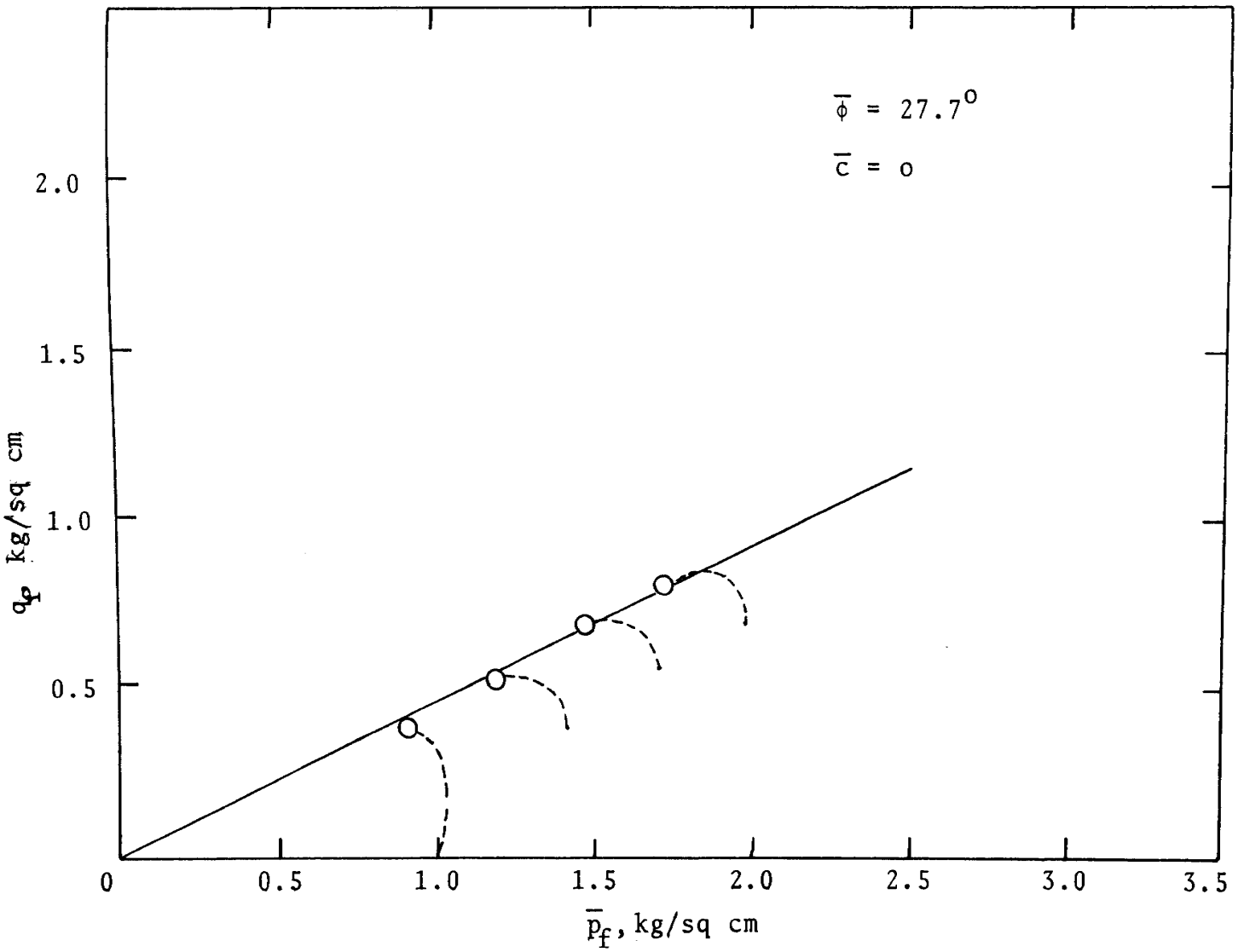

FIGURE 21. STRESS PATH AND K_f LINE FOR $\overline{\text{CIU-SCAU}}$ TEST

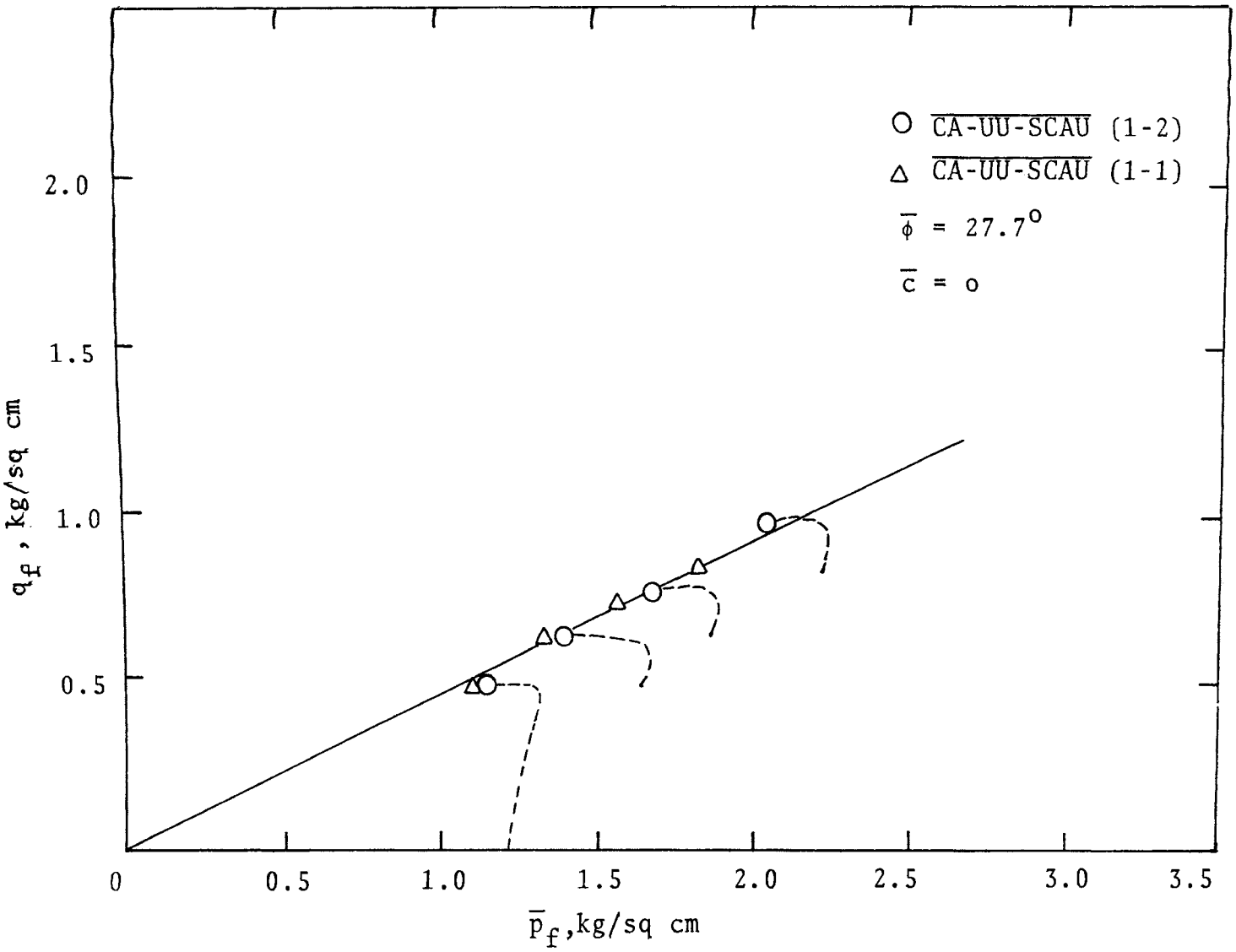

FIGURE 22. STRESS PATH AND K_f-LINE FOR $\overline{\text{CA-UU-SCAU}}$ TESTS

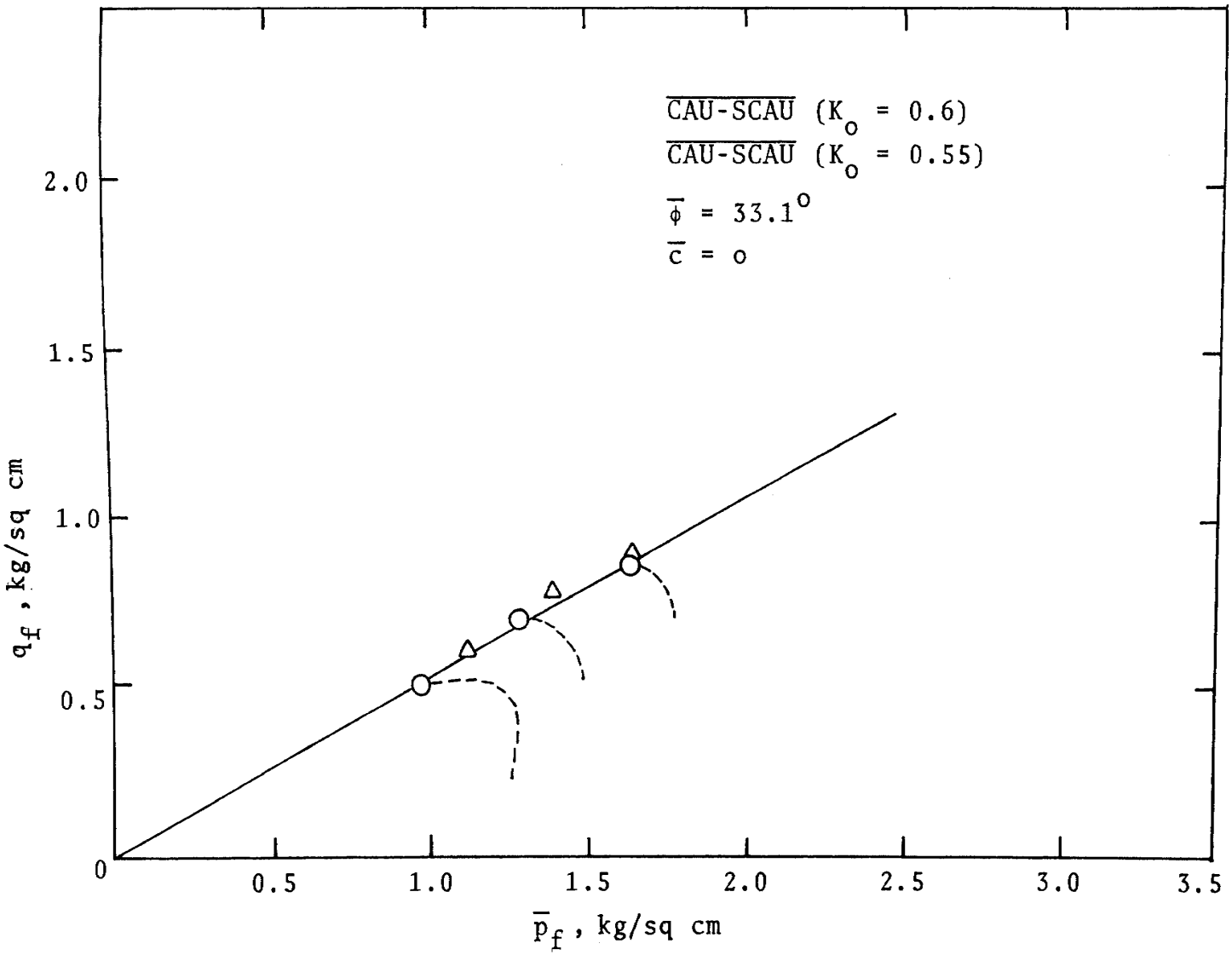

FIGURE 23. STRESS PATH AND K_f LINE FOR $\overline{\text{CAU-SCAU}}$ TESTS

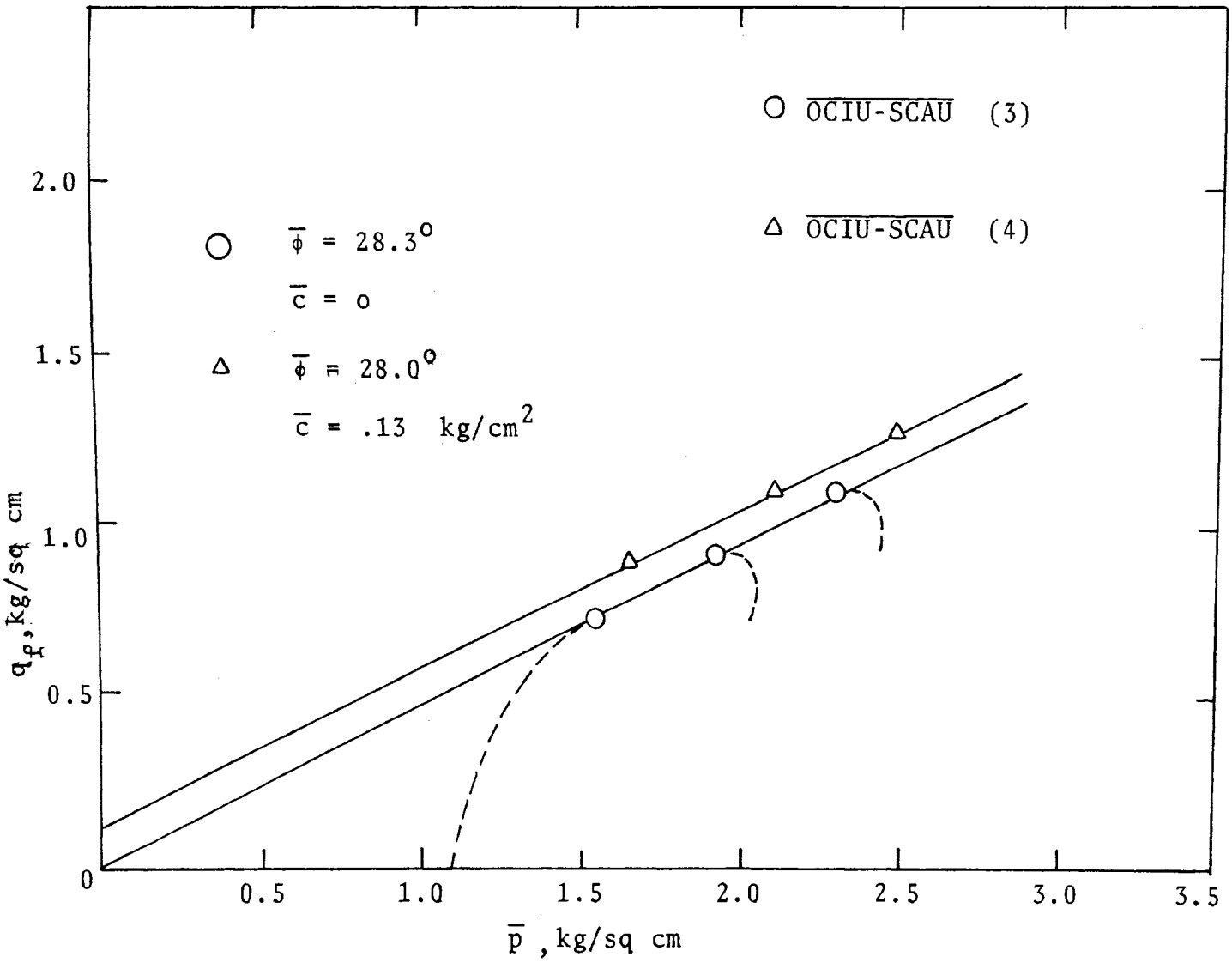

FIGURE 24. STRESS PATH AND K_f-LINE FOR $\overline{\text{OCIU-SCAU}}$ TESTS

$\overline{CIU}$-$\overline{SCAU}$ and $\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$ tests. As sequential stages proceed, the curves increase their radii of curvature. However, stress-strain similarity cannot be obtained from these types of paths.

Figure 23 shows stress paths for $\overline{CAU}$-$\overline{SCAU}$ test, starting with an initial anisotropic consolidation.

Figure 24 shows the effective stress path for $\overline{OCIU}$-$\overline{SCAU}$. In overconsolidated samples, decreasing pore pressure developed and the effective stress $\bar{p}$ increased during the first stage of shearing. However, for the second and third stages of shearing the excess pore pressure increased though not markedly. Lambe and Whitman (1969) suggested that the overconsolidated soil may have the same effect, i.e. friction angle, as normally consolidated soil if $\bar{p}_f$ is greater than one half of the maximum consolidation pressure.

4. Undrained Strength, Moisture Contents, Consolidation Pressures and Effective Stresses at Failure.

In Figure 25, moisture content is plotted versus undrained strength at maximum effective stress ratio for all tests. Samples subjected to different stress histories are plotted. For normally consolidated soil, there exists a unique relationship between undrained strength and moisture content. The moisture content and undrained strength, w_f - q_f, relationship is different for overconsolidated samples. The line diverges at the first and second stage of the $\overline{OCIU}$-$\overline{SCAU}$ test. However, this relation for the last stage of the test approaches the line plotted for the nor-

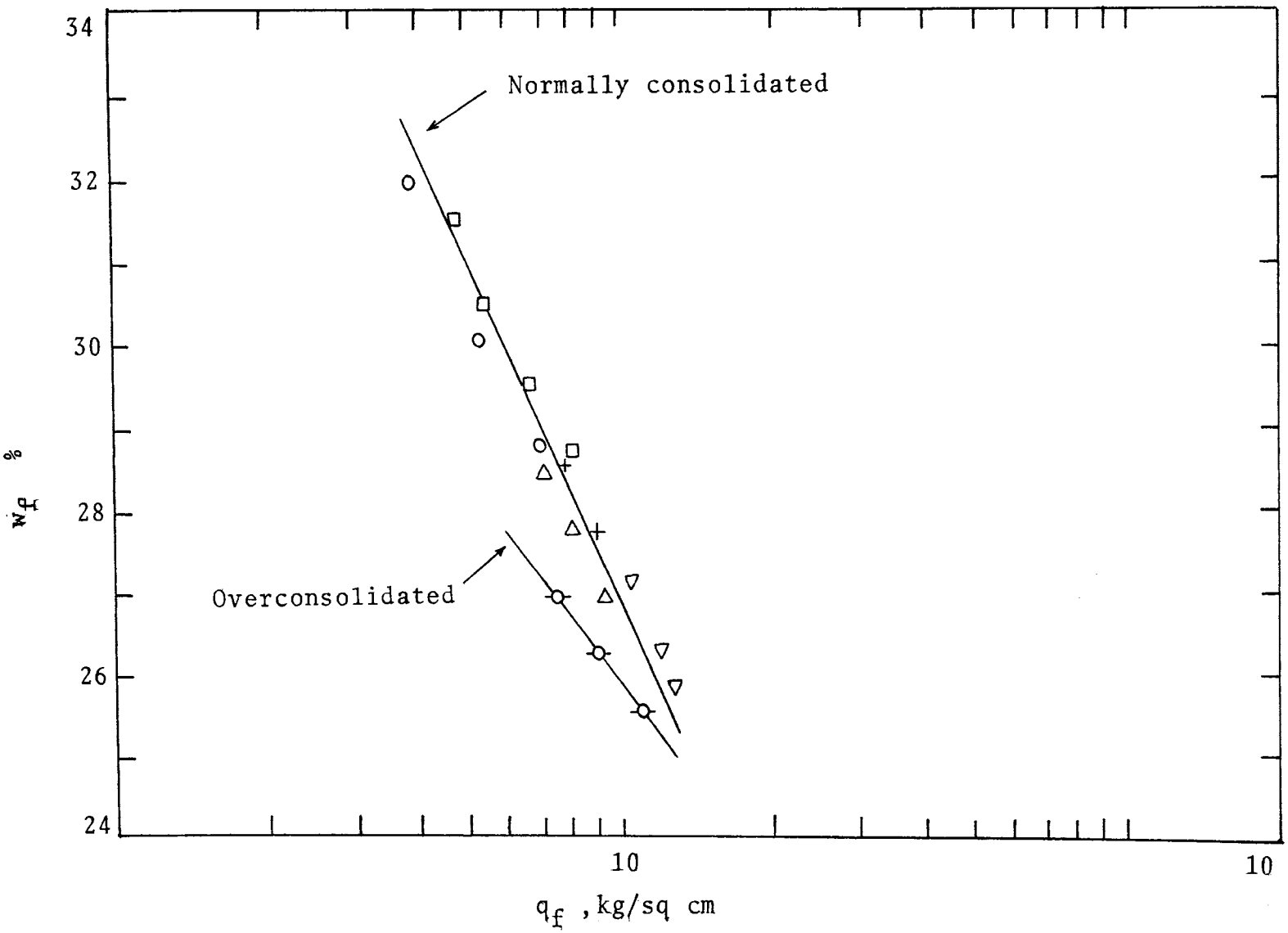

FIGURE 25. RELATIONSHIP BETWEEN WATER CONTENT AND UNDRAINED STRENGTH AT FAILURE

mally consolidated soil. Thus, sequential consolidation and shearing tend to decrease the effects of preconsolidation in terms of the w_f - q_f relation.

The relation of moisture content, undrained strength, consolidation pressures and effective stresses at failure for one typical result of $\overline{CIU}$-$\overline{SCAU}$ tests is shown in Figure 26. The linear relations of w_f - q_f, w_f - $\bar{p}_f$, and w_f - $\bar{p}_o$ are parallel to each other as previously shown by Rutledge (1947) and Henkel (1958, 1959 and 1960).

The undrained strength versus consolidation pressure, $\bar{p}_o$, in Figures 27 and 28 show a linear relationship. However, this relation is dependent on the stress history. This dependency is illustrated by a plot of moisture content at failure versus consolidation pressure, w_f - $\bar{p}_o$, as shown by Figure 29.

C. COMPARISON AND SUMMARY OF TEST RESULTS

Isotropic consolidation in the triaxial tests followed the Terzaghi theory of consolidation. For the silty-clay tested, the c_v decreased with consolidated pressure of 2.63×10^{-2} cm^2/min. for 1 kg./sq. cm. to 1.8×10^{-2} cm^2/min. for 3.0 kg./sq. cm.

The $E/\overline{\sigma_{1c}}$ at a F.S. of 4 for $\overline{OCIU}$-$\overline{SCAU}$ test is twice that of the $\overline{CIU}$-$\overline{SCAU}$ test. The values of $E/\overline{\sigma_{1c}}$ at given ratio of 3 remain unchanged for third and fourth sequential stages of the $\overline{CIU}$-$\overline{SCAU}$ test. The value of $E/\overline{\sigma_{1c}}$ for anisotropically consolidated soil is greater than for

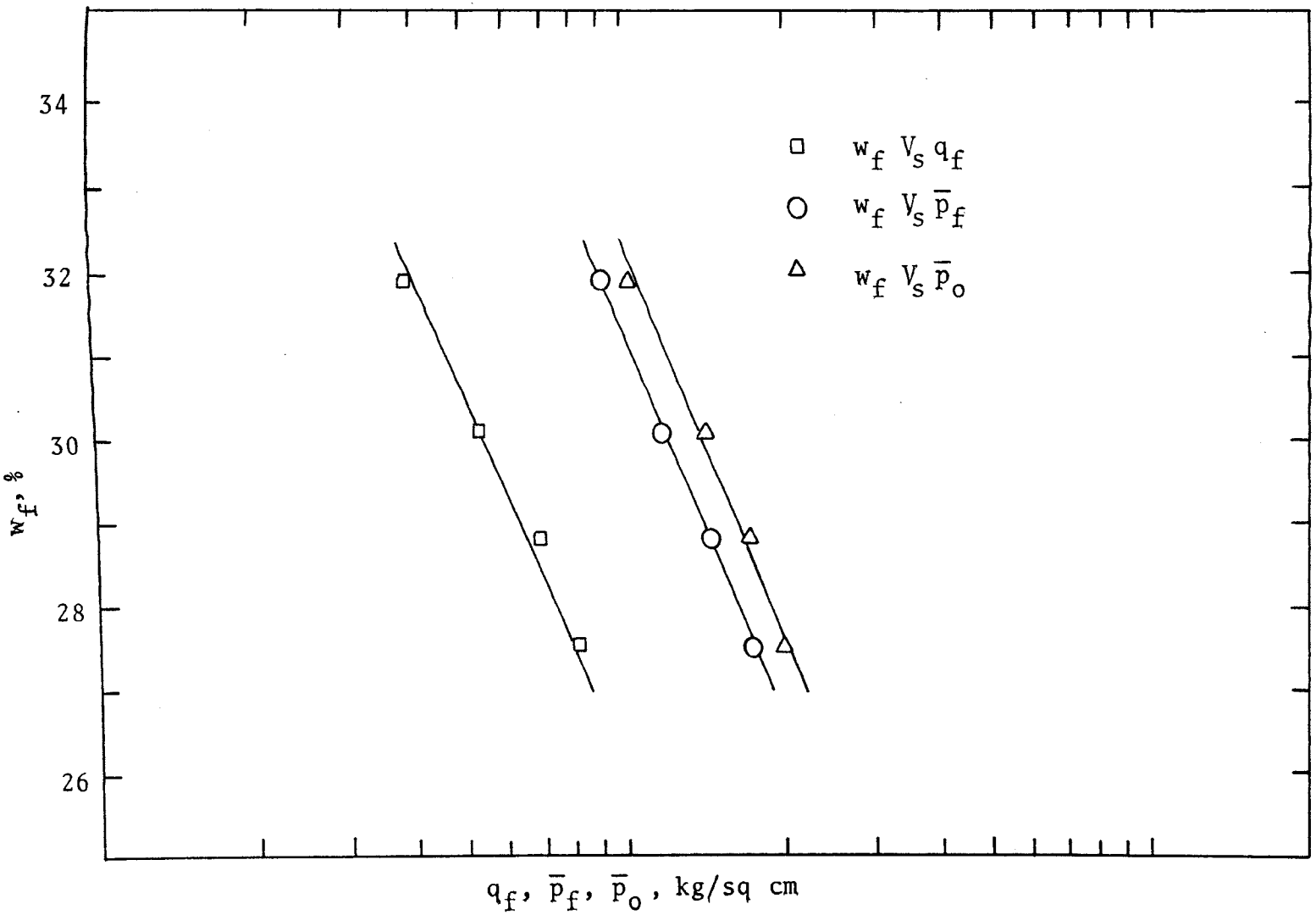

FIGURE 26. STRESS-VOLUME RELATIONSHIPS FOR $\overline{\text{CIU-SCAU}}$ TEST

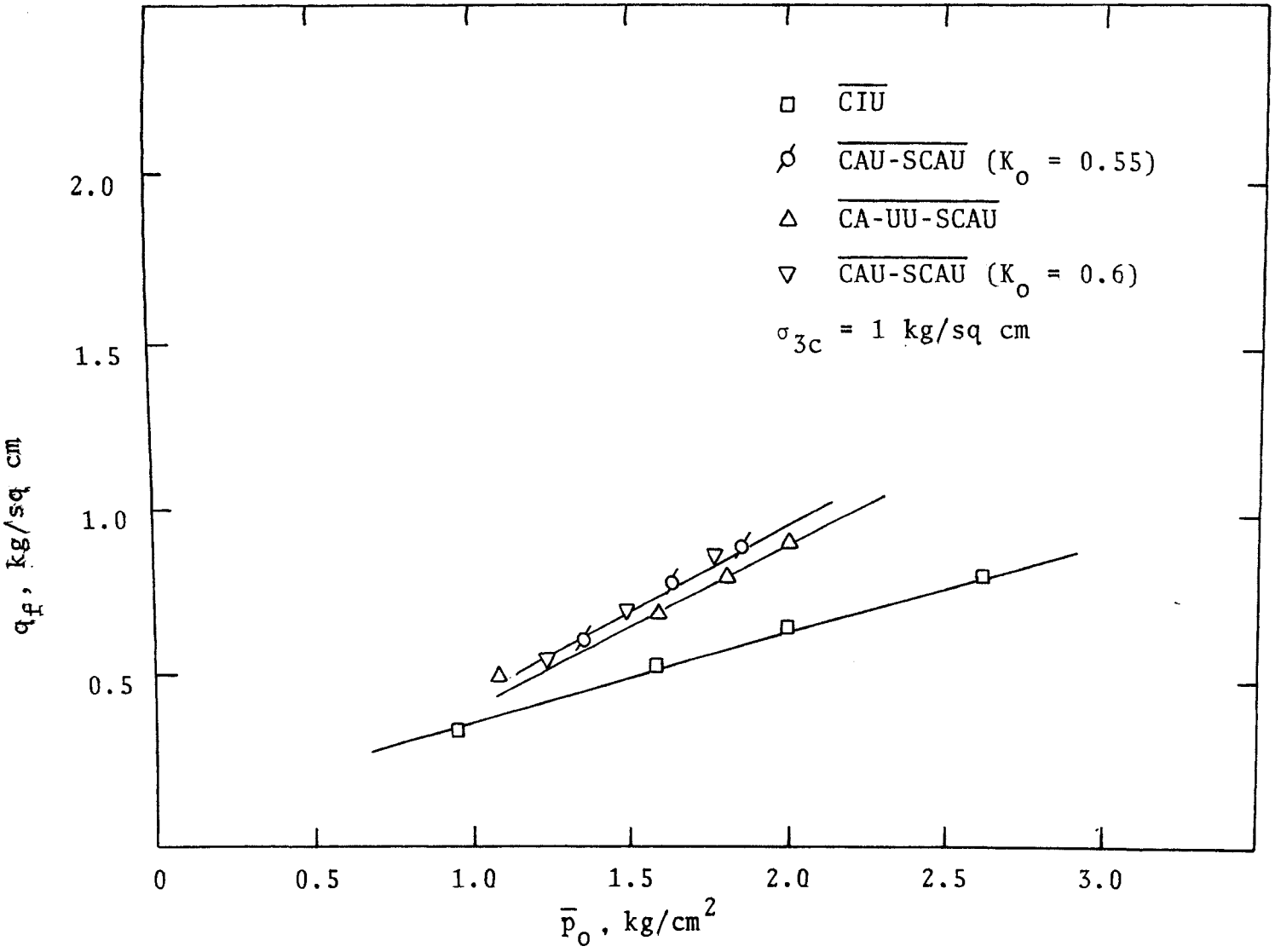

FIGURE 27. RELATIONSHIP BETWEEN UNDRAINED STRENGTH AND EFFECTIVE CONSOLIDATION PRESSURE AT FAILURE

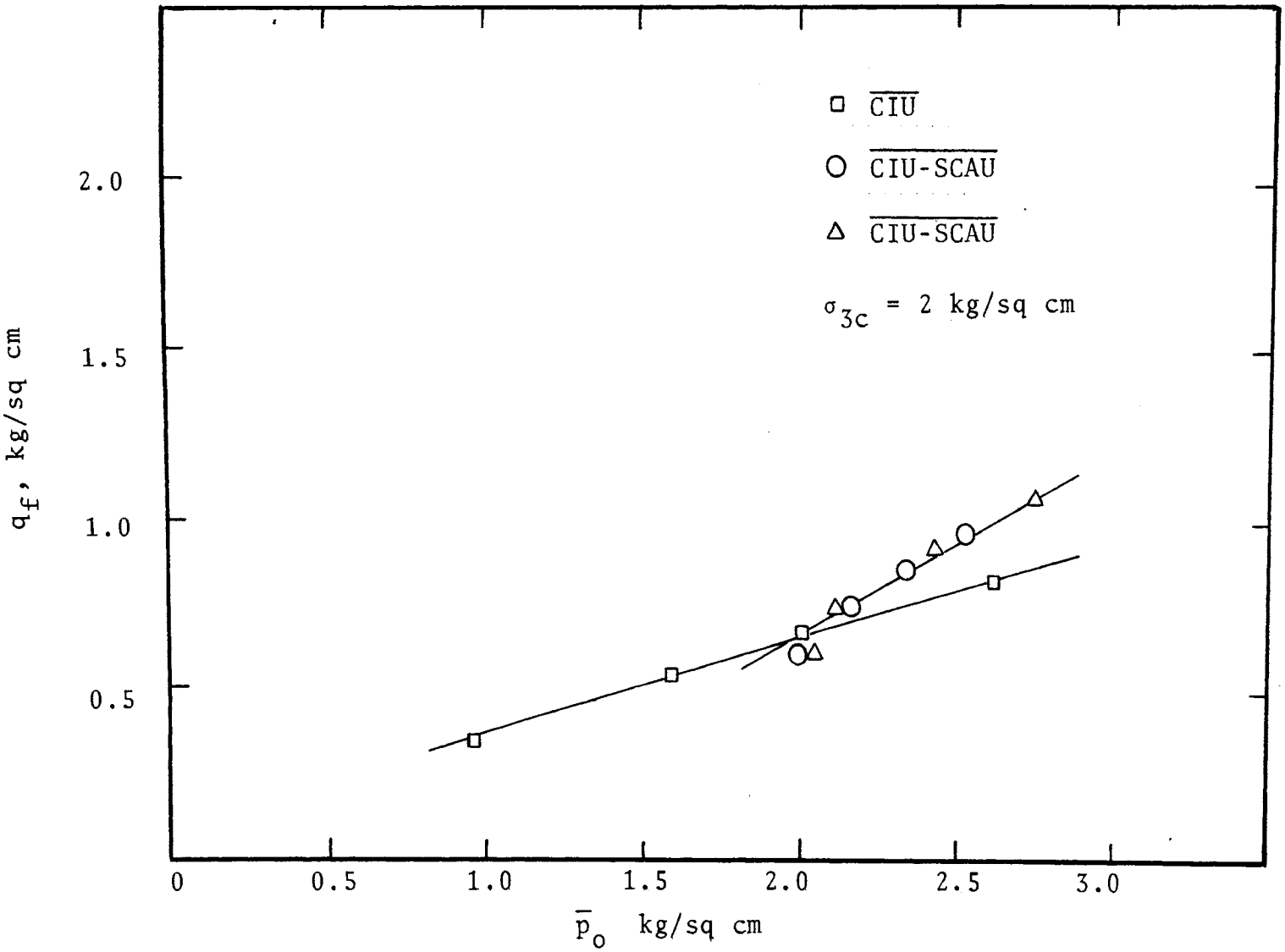

FIGURE 28. RELATION BETWEEN UNDRAINED STRENGTH AND EFFECTIVE CONSOLIDATION PRESSURE AT FAILURE

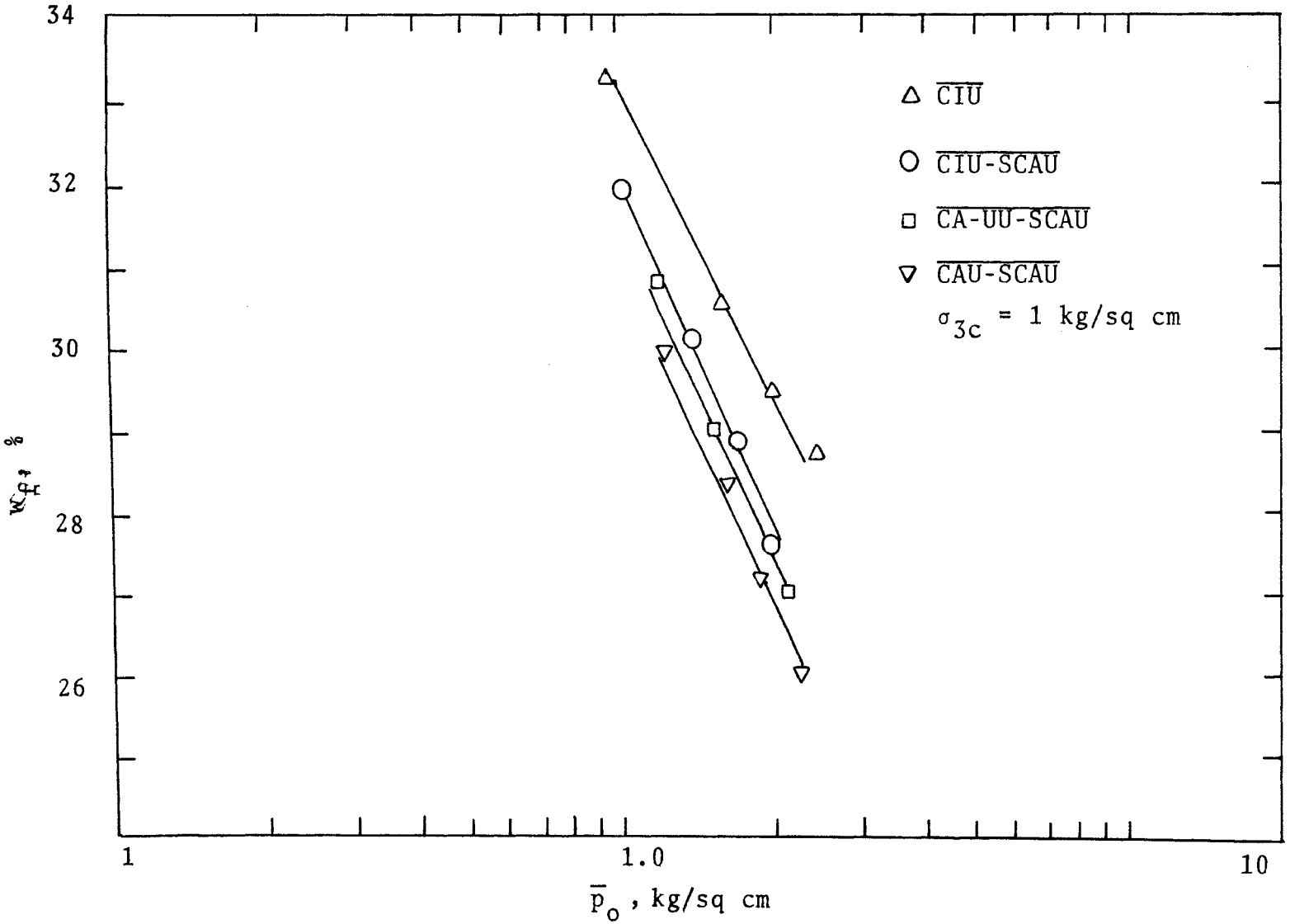

FIGURE 29. RELATIONSHIP BETWEEN MOISTURE CONTENT AND CONSOLIDATION PRESSURE

isotropically consolidated soil for F.S. of 3 and 4. However sequential loading decreases these differences by 50 percent.

Skempton (1954) and Bishop and Henkel (1957) found that values of pore pressure parameter A at failure for normally consolidated soils vary between 0.7 to 1.0. Figure 17 shows that A_f tends to have values 1.5 for $\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$ and 1.0 for $\overline{CAU}$-$\overline{SCAU}$. The $\overline{OCIU}$-$\overline{SCAU}$ gives a lowest value of A_f. This falls between the range of 0.25 to negative value found by Skempton (1954). The A_f for this kind of soil apparently varies with initial stress history. The $\overline{CIU}$-$\overline{SCAU}$ gives higher values of A_f and $\frac{\Delta u}{\overline{\sigma}_{1c}}$ than those obtained from $\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$ or $\overline{CAU}$-$\overline{SCAU}$ for the first stage of shearing.

As shown in Figure 23, the $\overline{\phi}$ for $\overline{CAU}$-$\overline{SCAU}$ tests is higher than those of the other sequential tests. The strength in terms of effective stresses is a function of stress path. If the stress path is changed, different values of $\overline{\phi}$ may be obtained.

There is a unique relationship between moisture content and undrained strength for this normally consolidated silty-clay. It is linear on a semi-log plot (Figure 25). This relationship is independent of test procedure. This means that both the standard triaxial and the sequential tests give the same unique relationship. Both undrained strength and moisture content are linearly related to consolidation pressures but these relations depend on whether the sample has been subjected to isotropic and anisotropic consolidation. Since the relation of moisture content and undrained

strength at failure is unique, this shows that the relation is independent of the amount of pore pressures induced during shear, which are different for each of the type of tests performed (Figures 17 and 18).

A linear relation between the parameter $\Delta u/\bar{\sigma}_{1c}$ and undrained strength at failure in a semi-logarithmic plot exists. This is also independent of the type of test performed for the sequential stages.

Results of all shear test data are summarized in Table IV in Appendix A.

IV. DISCUSSION OF RESULTS

A. RELATIONSHIP TO PAST INVESTIGATIONS AND BASIC CONCEPTS

1. Consolidation

During isotropic consolidation, the degree of pore pressure dissipation and of volume change against time closely reproduce the theoretical curve predicted for this soil by Terzaghi's theory of consolidation. It was shown in the previous chapter that the change in void ratio was linearly related to the decrease in pore pressure. According to Taylor (1948) and other (Barden, 1968), this is the critical assumption of the Terzaghi consolidation theory. These results may need to be checked further for other soils with different permeabilities in order to determine the limits of applicability of Terzaghi theory when applied to isotropic triaxial consolidation tests. Sequential consolidation tests did point out that the theory was obeyed for small degrees of consolidation in terms of pore pressure dissipation i.e. less than 25 percent. The permeability of the soil decreased as the number of sequential stages increased.

During the sequential consolidation stages, the amount of volume change did not correspond to the degree of pore pressure dissipation during the allowed time for consolidation. That is, the decrease in the coefficient of consolidation and in the permeability of the soil changed the expected amount of settlement. It did not, however, change the time-degree of consolidation relationship which was shown to

be independent of the applied K_c ratio during sequential loading.

Further research is needed at this point because the implication is very important. If undrained strength is a unique function of moisture content for normally consolidated soil, the fact that there has been a certain degree of pore pressure dissipation does not necessarily imply that a corresponding change in moisture content or volume has occurred as the dissipation of pore pressure may be predicted from purely theoretical reasoning. In other words, the change in moisture content or void ratio is linearly related to the change in pore pressure. But for a given time, the percent of total settlement is not the same as the percent of pore pressure dissipation, during sequential loading.

2. Deformation During Undrained Shear

The stress-strain relations were analyzed by means of the ratio $E/\bar{\sigma}_{1c}$ which was plotted versus the factor of safety. Test results (Figures 14 and 15) showed that the ratio $E/\bar{\sigma}_{1c}$ varies with the applied shear stress level, the stress system, the type of shear test, and the stress history. The effect of stress history decreased and was finally eliminated as the amount of deformation increased and as the number of undrained shear stages increased. Ladd (1964), using standard triaxial shear tests, found that the ratio $E/\bar{\sigma}_{1c}$ varies with the level at which shear stress is applied, the stress system and the type of shear test.

3. Pore Pressure Parameters

The parameter A at failure for the tested soil depends on the consolidation pressure, the stress system and the stress history. The parameter A_f was not uniquely related to the undrained strength, in either the standard or the sequential test, and the prediction of A_f from a knowledge of the undrained strength is not possible unless the previously mentioned factors are known.

These results are similar to those shown by Lo (1969) with undisturbed and remolded samples of normally consolidated clay soils.

Lo (1969) introduced the pore pressure parameter $\Delta u/\bar{p}_c$. The value of $\bar{p}_c$ refers to the consolidation stresses applied in the failure plane. An equivalent parameter, $\Delta u/\bar{\sigma}_{1c}$, was selected to present the results in the previous chapter. The most interesting relationship found relates the ratio $u/\bar{\sigma}_{1c}$ at failure defined by maximum effective stress ratio and the undrained strength.

The plot of the results for the standard tests is very similar to the relation between A_f and q_f. However, during sequential loading, a plot of the ratio $\Delta u/\bar{\sigma}_{1c}$ versus the undrained strength in a logarithmic scale was found to be linear and this relation was expressed in:

$$\frac{\Delta u}{\bar{\sigma}_{1c}} = M - N \log \frac{q_f}{q_o} \qquad (13)$$

This relation was found to be independent of the consolidation stress, the ratio of the major to the minor

principal consolidation stresses (K_c), and stress path for normally consolidated soil.

Equation 13 is rewritten below in terms of natural logarithms:

$$\frac{\Delta u}{\bar{\sigma}_{1c}} = M - \frac{1}{N'} \ln \left(\frac{q_f}{q_o}\right) \tag{14}$$

Equation 14 will be used in the following developments since it simplifies the mathematical expressions.

In the previous chapter, a linear relation was found between the moisture content and the logarithm of the undrained strength. This relation as previously described, is independent of consolidation stress ratio, K_c, the type of shear test and the stress path. Such a relation may also be expressed in terms of natural logarithms by:

$$w_f = w_o - \frac{1}{E'} \ln \frac{q_f}{q_o} \tag{15}$$

where w_f is the average moisture content at failure, w_o is the moisture content at an arbitrary selected value of q_o (undrained shear strength) and q_f is the shear strength at failure of the sample.

The similarity of Equations 14 and 15 leads to their comparison and by a simple manipulation, the following equation is obtained:

$$\frac{\Delta u}{\bar{\sigma}_{1c}} = Q - R\,(w_o - w_f) = Q - R\Delta w \tag{16}$$

where Q and R are constants and Δw is equal to the difference of $(w_o - w_f)$. In other words, the greater is the reduc-

tion in moisture content, the larger is the decrease in the ratio $\Delta u/\bar{\sigma}_{1c}$.

The form of Equation 16 resembles the form of Equation 8 where the decrease in void ratio or moisture content is accompanied by a linear decrease in pore water pressure during consolidation.

These results strongly suggest the need for more research in the direction of firmly establishing a law or principle relating the change in moisture content or void ratio to either the change in pore pressure or the change in the ratio $u/\bar{\sigma}_{1c}$, which may be independent of previous loading and boundary conditions imposed upon the sample.

4. Effective Stresses

Terzaghi developed the principle of effective stresses and proposed that the strength of a soil was governed by the effective stress at failure. Present test results show that the strength of the silty-clay is governed by the effective stresses at failure. However, in undrained loading the sample consolidated anisotropically gave a higher effective friction angle than that of the isotropically consolidated samples.

The difference in the effective friction angle is not due to the moisture content of the sample. Hvorslev (1960) suggested that the effective friction angle is independent of moisture content. The other factor that may explain the difference in friction angle is the value of pore water pressure at failure. The value of the pore pressure para-

meter A_f in Figure 16 for anisotropic consolidation soil is less than that for isotropically consolidated for both standard and sequential tests.

This result contradicts the results found by Whitman, Ladd and da Cruz (1960). They used the failure criterion of maximum deviator stress as opposed to the use of maximum effective stress ratio used in this study. Simons (1960) performed conventional drained and undrained triaxial shear tests on Oslo clay and also concluded that effective friction angles for the Oslo of clay were the same for isotropically and anisotropically consolidated samples. This particular point needs to be investigated further. Experimental error cannot account for a 5 degree difference in friction angle.

5. Undrained Strength Relations

For a given stress history the undrained strength of this soil is a function of moisture content at failure. This result is in agreement with Rutledge (1947). Henkel (1958, 1959 and 1960) found the same unique relationship by performing standard triaxial tests on remolded London and Weald Clays.

Test results showed that the relationship between undrained strength and consolidation pressure is dependent of stress history (Figure 27). Rutledge (1947) suggested the working hypothesis that undrained strength of a soil is determined by the effective consolidation stress. He also suggested that the relation between the effective consolida-

tion pressure and the void ratio for a given soil are independent of the method of test. However results shown in this investigation do not agree in that the moisture content at failure has been shown to be dependent on whether isotropic or anisotropic consolidation was employed.

B. FUTURE RESEARCH

The following recommendations for the future research are suggested:

1. Further investigation of the linearity of the relation between $\frac{\Delta u}{\bar{\sigma}_{1c}}$ and the logarithm of the undrained strength (q_f) using other soils with lower permeability and higher sensitivity should be made.

2. During initial anisotropic consolidation of a sample, measurement of volume change and pore water pressure dissipation for every increment of loading added may be used because it was not done in this study. This may help to understand the anisotropic consolidation characteristics of a soil.

3. Due to the limitation of using only two consolidation pressures, it is recommended that higher cell pressures may be used before starting shear test.

4. The cause of difference in effective friction angle between isotropically consolidated and anisotropically consolidated samples should be investigated.

5. The use of higher overconsolidation ratios may provide a better insight into the behavior of heavily over-

consolidated soils.

C. CONCLUSIONS

The following conclusions were drawn from this investigation.

1. During the consolidation phase of the sequential loading tests, the change in moisture content was linearly related to the change in pore pressure, but for a given time, the percent of settlement with respect to the total settlement is not the same as that of the percent pore pressure dissipation.

2. Sequential shear tests results showed that the ratio $\Delta u/\overline{\sigma}_{1c}$ is uniquely related to the logarithm of undrained strength and to the reduction of moisture content.

3. The undrained strength is a unique function of moisture content for normally consolidated soil, and this relation is independent of consolidation pressures, type of shear test and stress path.

4. The use of the sequential triaxial shear test may be justified by the significant departure in the value of certain properties, such as modulus of deformation, coefficient of consolidation, and permeability, as compared to the values obtained during conventional tests.

5. Neither the moisture content nor the undrained strength are uniquely related to the consolidation pressure. The relations were dependent on the type of triaxial shear tests, the stress history and the stress path.

V. APPENDIX A
TEST RESULTS

TABLE I

SUMMARY OF TESTS PROGRAMED

Types	$\bar{\sigma}_c$ kg/cm^2	σ_{vc} kg/cm^2	σ_{cell} kg/cm^2	K_c	OCR
$\overline{CIU}$-1	1.0	1.0	3.0	1.0	
$\overline{CIU}$-2	1.5	1.5	3.0	1.0	
$\overline{CIU}$-3	2.0	2.0	3.0	1.0	
$\overline{CIU}$-4	2.5	2.5	3.0	1.0	
$\overline{CIU}$-$\overline{SCAU}$-1	1.0	1.0	3.0	1.0	
$\overline{CIU}$-$\overline{SCAU}$-2	2.0	2.0	3.0	1.0	
$\overline{CAU}$-$\overline{SCAU}$-1	1.0	1.5	3.0	1.5	
$\overline{CAU}$-$\overline{SCAU}$-2	2.0	3.0	3.5	1.5	
$\overline{CAU}$-$\overline{SCAU}$-3	1.0	1.8	3.0	1.8	
$\overline{CAU}$-$\overline{SCAU}$-4	1.0	2.0	3.0	2.0	
$\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$-1	1.0	1.5	3.0	1.5	
$\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$-2	2.0	3.0	3.5	1.5	
$\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$-3	1.0	1.8	3.0	1.8	
$\overline{CA}$-$\overline{UU}$-$\overline{SCAU}$-4	2.0	3.6	3.5	1.8	
$\overline{OCIU}$-$\overline{SCAU}$-1	3.0	3.0	3.0	1.0	3
$\overline{OCIU}$-$\overline{SCAU}$-2	4.0	4.0	3.0	1.0	4

Note: $\bar{\sigma}_c$ = effective consolidation pressure

$\bar{\sigma}_{vc}$ = vertical consolidation

σ_{cell} = cell pressure

$K_c = \bar{\sigma}_{1c}/\bar{\sigma}_{3c}$

OCR = overconsolidation ratio $\bar{\sigma}_{cm}/\bar{\sigma}_c$

TABLE II

INITIAL MOISTURE CONTENT OF THE SAMPLE BEFORE CONSOLIDATION

Sample Section	Moisture Content %
Top 1	36.15
2	35.80
3	39.45
4	40.65
Bottom 15	41.80

TABLE III

MOISTURE CONTENT OF SAMPLE AT THE END OF TESTS

Sample Section	Moisture Content %					
	Type of Test					
	$\overline{CIU\text{-}SCAU}$		$\overline{CAU\text{-}SCAU}$		$\overline{OCIU\text{-}SCAU}$	
	1 kg/sq cm	2 kg/sq cm	1 kg/sq cm	2 kg/sq cm	3 kg/sq cm	4 kg/sq cm
1	26.46	27.03	28.38	26.84	26.40	26.84
2	26.96	26.61	28.30	26.40	26.20	26.50
3	27.27	26.55	28.48	26.30	26.38	26.60
4	28.70	26.72	28.20	26.23	26.60	26.70
5	27.99	27.35	28.60	27.00	26.78	28.23

TABLE IV

SUMMARY OF SHEAR TEST DATA

Type of Test	Stage No.	K_c	ϵ_f %	$\bar{\sigma}_1/\bar{\sigma}_3$ f	q_f kg/sq cm	$\bar{p}_f$ kg/sq cm	$\bar{p}_o$ kg/sq cm	A_f	$\Delta_u/\bar{\sigma}_{1c}$	w_f
$\overline{CIU}$	1	1.0	20.5	3.79	0.35	0.60	0.96	1.01	0.54	33.3
	1	1.0	15.0	2.85	0.55	1.13	1.60	0.93	0.64	30.5
	1	1.0	11.7	2.53	0.67	1.54	2.00	0.85	0.58	29.5
	1	1.0	17.2	2.51	0.81	1.89	2.64	0.96	0.59	28.7
$\overline{CIU\text{-}SCAU}$ (1-1)	1	1.0	4.2	2.49	0.39	0.92	1.01	0.62	0.48	31.9
	2	1.7	10.2	2.63	0.53	1.19	1.41	1.26	0.20	30.1
	3	1.9	15.9	2.80	0.70	1.47	1.70	1.27	0.17	28.8
	4	2.1	22.7	2.80	0.82	1.72	1.98	1.31	0.14	27.6
$\overline{CIU\text{-}SCAU}$ (2-1)	1	1.0	4.2	2.11	0.60	1.68	2.02	0.78	0.46	28.6
	2	1.7	9.5	2.39	0.76	1.85	2.17	1.49	0.17	27.6
	3	1.9	14.8	2.52	0.87	2.06	2.35	1.72	0.13	26.7
	4	2.1	20.6	2.53	0.97	2.25	2.54	1.74	0.11	25.9

-continued-

Table IV continued

$\overline{\text{CIU-SCAU}}$ (2-2)	1	1.0	4.3	2.05	0.57	1.67	2.03	0.81	0.46	27.5
	2	1.7	9.8	2.30	0.76	1.93	2.14	1.09	0.14	26.7
	3	1.9	15.1	2.42	0.94	2.26	2.44	1.09	0.10	26.5
	4	2.0	20.2	2.44	1.07	2.56	2.75	1.20	0.08	25.4
$\overline{\text{CA-UU-SCAU}}$ (1-1)	1	1.0	4.4	2.45	0.48	1.15	1.22	0.57	0.45	30.0
	2	1.9	10.6	2.64	0.63	1.40	1.65	1.36	0.18	28.4
	3	2.0	16.2	2.71	0.77	1.68	1.86	1.29	0.13	27.1
	4	2.2	20.3	2.84	0.98	2.05	2.24	1.24	0.11	26.0
$\overline{\text{CA-UU-SCAU}}$ (1-2)	1	1.0	4.4	2.57	0.48	1.13	1.17	0.59	0.48	31.4
	2	1.9	10.1	2.72	0.62	1.35	1.59	1.49	0.18	30.4
	3	2.0	16.3	2.73	0.73	1.57	1.82	1.48	0.15	29.2
	4	2.1	22.5	2.71	0.84	1.83	2.07	1.52	0.13	28.2

-continued-

Table IV continued

$\overline{CA\text{-}UU\text{-}SCAU}$ (2-1)	1	1.0	4.6	2.49	0.92	2.16	2.16	0.50	0.43	26.6
	2	2.1	9.5	2.70	1.10	2.41	2.70	1.68	0.12	25.7
	3	2.2	16.1	2.70	1.23	2.67	2.89	1.65	0.09	24.9
	4	2.5	18.9	2.68	1.38	3.01	3.17	1.74	0.08	24.7
$\overline{CA\text{-}UU\text{-}SCAU}$ (2-2)	1	1.0	4.6	2.47	0.98	2.33	2.28	0.47	0.41	27.1
	2	2.1	10.0	2.61	1.08	2.42	2.82	1.75	0.13	26.5
	3	2.2	15.3	2.64	1.17	2.59	2.92	1.70	0.10	25.9
	4	2.2	19.5	2.61	1.24	2.79	3.09	1.80	0.08	25.6
$\overline{CA\text{-}UU\text{-}SCAU}$ (1-1)	1	1.0	4.6	3.20	0.59	1.13	1.00	0.39	0.47	29.9
	2	2.1	10.5	3.22	0.70	1.32	1.62	1.60	0.18	28.4
	3	2.2	16.1	3.17	0.82	1.57	1.82	1.57	0.14	27.7
	4	2.3	20.6	3.07	0.94	1.84	2.07	1.70	0.12	26.9

-continued-

Table IV continued

$\overline{\text{CA-UU-SCAU}}$	1	1.0	5.0	2.78	1.07	2.26	2.00	0.38	0.40	26.2
(2-1)	2	2.3	11.0	2.89	1.18	2.42	2.75	1.80	0.11	25.5
	3	2.4	13.8	2.84	1.31	2.74	2.91	1.72	0.08	24.8
$\overline{\text{CAU-SCAU}}$	1	1.5	4.6	3.26	0.53	0.99	1.25	0.51	0.36	30.9
(1-1)	2	2.1	11.0	3.50	0.71	1.29	1.49	1.07	0.19	28.9
	3	2.4	16.7	3.38	0.89	1.64	1.79	0.97	0.13	27.1
$\overline{\text{CAU-SCAU}}$	1	1.5	5.0	2.81	0.92	1.93	2.57	0.57	0.34	26.6
(2-1)	2	2.2	11.3	3.09	1.08	2.12	2.43	1.10	0.14	25.7
	3	2.4	16.8	3.04	1.22	2.42	2.63	1.15	0.09	25.0
$\overline{\text{CAU-SCAU}}$	1	2.8	4.5	3.45	0.62	1.12	1.37	0.37	0.26	30.2
(1-1)	2	2.3	10.5	3.68	0.80	1.40	1.65	1.16	0.17	28.5
	3	2.4	16.0	3.45	0.91	1.64	1.86	1.12	0.13	27.7

-continued-

Table IV continued

$\overline{\text{CAU-SCAU}}$	1	2.0	4.5	3.39	0.67	1.24	1.45	0.30	0.21	29.6
(1-1)	2	2.3	10.5	3.42	0.77	1.42	1.73	1.12	0.17	28.4
	3	2.4	16.6	3.32	0.87	1.63	1.91	1.15	0.14	26.6
$\overline{\text{OCIU-SCAU}}$	1	1.0	4.7	2.80	0.74	1.56	1.11	0.20	0.26	26.9
(3)	2	2.1	10.1	2.83	0.93	1.94	2.05	0.72	0.15	26.0
	3	2.3	15.5	2.85	1.11	2.31	2.45	0.93	0.12	25.4
$\overline{\text{OCIU-SCAU}}$	1	1.0	5.1	3.35	0.90	1.67	1.19	0.21	0.36	26.0
(4)	2	2.4	11.3	3.20	1.12	2.13	2.17	0.60	0.09	25.2
	3	2.5	15.3	3.14	1.29	2.50	2.64	0.95	0.08	24.7

COMPUTER PROGRAM FOR SHEAR TEST

```
0001          DIMENSION SDIA(50),SLENG(50),DVOL(50),SLENGF(50),ALDR(50,50),
             1ADDR(50,50),ATDR(50,50),P1(50),P3(50),WW(50,3),DW(50,3),CW(50,3),
             2CM(50,3),CPER(50),NUMREA(50),EV(50,3),SG(50)
0002          DIMENSION XSTRES(50),XSTRAN(50),XUP(50),XSTRA(50)
0003          READ(1,13)NUMSAM
0004       13 FORMAT(I2)
0005          DO 100 N=1,NUMSAM
0006          READ(1,1) SDIA(N),SLENG(N),DVOL(N),SLENGF(N),P1(N),P3(N),CPER(N),S
             1G(N),NUMREA(N)
0007        1 FORMAT(2F3.2,F4.2,3F3.2,F3.0,F3.2,I3)
0008          NUMBER=NUMREA(N)
0009          READ(1,2) (ADDR(N,I),ALDR(N,I),ATDR(N,I),I=1,NUMBER )
0010        2 FORMAT(F4.0,F4.1,F4.0)
0011          READ(1,3) (WW(N,L),DW(N,L),CW(N,L),L=1,3)
0012        3 FORMAT(3F5.2)
0013          SAREA= (SDIA(N)**2.0)*3.1415/4.0
0014          VOLUME=(SAREA*SLENG(N))
0015          COVOL=VOLUME-DVOL(N)
0016          DEFORM=ADDR(N,NUMBER)*0.002540
0017          COLENG=SLENGF(N)+DEFORM
0018          COAREA=COVOL/COLENG
0019          WRITE(3,4)N
0020        4 FORMAT('1',14X,'SEQUENTIAL TRIAXIAL TEST PERFORMED ON 100% SATURAT
             1ED SAMPLE NUMBER'I3)
0021          ANISOR=  P1(N)/P3(N)
0022          WRITE(3,5) CPER(N), ANISOR
0023        5 FORMAT(//20X,'CONSOL PRESSURE  =',F4.0,6X,'ANISOTROPIC RATIO='
             1,F5.2)
0024          WRITE(3,19) P1(N),P3(N)
0025       19 FORMAT(/19X,'VERTICAL CONSOLIDATION PRESSURE=',F5.2,6X,'LATERAL CO
             1NSOLIDATION PRESSURE='F5.2)
0026          WRITE(3,6)
0027        6 FORMAT(/1X,'STRESS',2X,'STRAIN',3X,'AREA',2X,'PORE T',2X,'PORE D',
             12X,'SIGMA1',2X,'SIGMA3',2X,'SIG1EF',2X,'SIG3EF',2X,'DEVSTR',2X,'ST
             1RESS',2X,'ACOEFF',1X,'DEVST/2P3',1X,'SIG3EF/P3',1X,'V',6X,'H')
0028          WRITE(3,7)
0029        7 FORMAT( 1X,'KGSQCM',2X,'PERCEN',3X,'SQCM',2X,'KGSQCM',2X,'KGSQCM',
             12X,'KGSQCM',2X,'KGSQCM',2X,'KGSQCM',2X,'KGSQCM',2X,'KGSQCM',2X,'RA
             1TIO',3X,'VALUE',4X,'VALUE',5X,'VALUE')
0030          DO 200 I=1,NUMBER
0031          STRAIN=ADDR(N,I)*0.002540/COLENG
0032          XSTRAN(I)=STRAIN
```

```
0033                PERSTR=STRAIN*100.0
0034                TOTLD=ALDR(N,I)*0.2807
0035                AREA=COAREA/(1.0-STRAIN)
0036                SIGMA1=P1(N)+TOTLD/AREA
0037                SIGMA3=P3(N)
0038                UPP=ATDR(N,I)*0.0014
0039                XUP(I)=UPP
0040                DUPP=UPP-ATDR(N,1)*0.0014
0041                SIG1EF=SIGMA1-UPP
0042                SIG3EF=SIGMA3-UPP
0043                DEVS=SIGMA1-SIGMA3
0044                STRAT=SIG1EF/SIG3EF
0045                XSTRA(I)=STRAT
0046                IF(I.EQ.1) GO TO 33
0047                ACOEFF=DUPP/(TOTLD/AREA)
0048                GO TO 22
0049             33 ACOEFF=0.0
0050                GO TO 22
0051             22 STRESS=TOTLD/AREA
0052                XSTRES(I)=STRESS
0053                X=SIG3EF/P3(N)
0054                Y=DEVS/(2.0*P3(N))
0055                V=(SIG1EF-SIG3EF)/2.0
0056                H=(SIG1EF+SIG3EF)/2.0
0057                WRITE(3,8) STRESS,PERSTR,AREA,UPP,DUPP,SIGMA1,SIGMA3,SIG1EF,SIG3EF
                   1,DEVS,STRAT,ACOEFF,Y,X,V,H
0058             8 FORMAT(/1X,F6.3,2X,F6.3,2X,F5.2,1X,F6.3,2X,F6.3,2X,F6.3,2X,F6.3,2X
                   1,F6.3,2X,F6.3,2X,F6.3,2X,F6.3,2X,F6.3,3X,F6.3,4X,F6.3,2X,F6.3,4X,
                   1F6.3)
0059           200 CONTINUE
0060                CALL PPLT(XSTRAN,XSTRES,NUMBER)
0061                WRITE (3,30)
0062                CALL PPLT(XSTRAN,XUP,NUMBER)
0063                WRITE (3,31)
0064                CALL PPLT(XSTRAN,XSTRA,NUMBER)
0065                WRITE (3,32)
0066             30 FORMAT (//10X'STRAIN V STRESS')
0067             31 FORMAT (//10X'STRAIN V UPP')
0068             32 FORMAT (//10X'STRAIN V STRAT')
                C       MOISTURE CONTENTS AND VOID RATIOS
0069                WRITE (3,20)
0070             20 FORMAT('1')
```

```
0071              WRITE(3,9)N
0072            9 FORMAT(    6X,'MOISTURE CONTENTS OF SAMPLE NO.',I3)
0073              DO 300 L=1,3
0074              CM(N,L)=(WW(N,L)-DW(N,L))/(DW(N,L)-CW(N,L))
0075              CMPER=CM(N,L)*100.
0076              WRITE(3,10)L,CMPER
0077           10 FORMAT(//6X,'MOISTURE CONTENT SECTION :',I3,'=',F8.3,'%')
0078          300 CONTINUE
0079              CMAV=((WW(N,1)+WW(N,2)+WW(N,3))-(DW(N,1)+DW(N,2)+DW(N,3)))/((DW(N,
                 11)+DW(N,2)+DW(N,3))-(CW(N,1)+CW(N,2)+CW(N,3)))
0080              WATER=CMAV*100.
0081              WRITE(3,11) WATER
0082           11 FORMAT(////36X,'AVERAGE MOISTURE CONTENT =',F8.3,'%')
0083              WRITE(3,12)N
0084           12 FORMAT(//6X,'VOID RATIOS OF SAMPLE NO.',I3)
0085              WRITE(3,14)
0086           14 FORMAT('0')
0087              DO 400 L=1,3
0088              EV(N,L)=SG(N)  *CM(N,L)
0089              WRITE(3,15)L,EV(N,L)
0090           15 FORMAT(//6X,'VOID RATIO SECTION:',I3,'=',F8.3)
0091          400 CONTINUE
0092              EVAV=SG(N)*CMAV
0093              WRITE(3,16)EVAV
0094           16 FORMAT(////36X,'AVERAGE VOID RATIO =',F8.3)
0095          100 CONTINUE
0096              STOP
0097              END
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VI. BIBLIOGRAPHY

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VII. VITA

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