UNDRAINED TIME DEPENDENT BEHAVIOR OF A LIGHTLY OVERCONSOLIDATED NATURAL CLAY

by

Mustapha Zergoun

B.Sc.A. Civil Engineering, Laval University, Quebec, 1976

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF

THE REQUIREMENTS FOR THE DEGREE OF

M.A.Sc.

in the Department

of

Civil Engineering

We accept this thesis as conforming to the

required standard

THE UNIVERSITY OF BRITISH COLUMBIA

August, 1982

In presenting this thesis in partial fulfilment of the requirements for an advanced degree at the University of British Columbia, I agree that the Library shall make it freely available for reference and study. I further agree that permission for extensive copying of this thesis for scholarly purposes may be granted by the head of my department or by his or her representatives. It is understood that copying or publication of this thesis for financial gain shall not be allowed without my written permission.

Department of CIVIL ENGINEERING

The University of British Columbia
2075 Wesbrook Place
Vancouver, Canada
V6T 1W5

Date AUGUST 11, 1982

ABSTRACT

Time dependence of stress-strain and strength characteristics of a sensitive, overconsolidated, intact clay have been investigated under undrained conditions. Various time loading histories in undrained triaxial compression, under a given consolidation history, showed that increase in speed of testing results in stiffer stress-strain response and higher strength, and that sustained stress or load causes a reduction of strength with time. In all tests, rupture was triggered at about the same critical level of strain. Correlation among the various tests supported the validity of the equation-of-state uniquely relating stress, strain and strain-rate. Comparison with the same clay destructured on normal consolidation showed that destructuration of the clay does not cause any significant change in its time dependent behavior.

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (cont'd)

LIST OF FIGURES (cont'd)

LIST OF SYMBOLS

A	Skempton's pore pressure parameter = $\dfrac{\text{Excess pore pressure}}{\text{Deviator stress}}$
A_1	intercept of $\ln \dot{\varepsilon}$ versus D, at $t = t_1$
D	shear stress level, $(\sigma_1-\sigma_3)/(\sigma_1-\sigma_3)_f$
m	slope of $\ln \dot{\varepsilon}$ versus $\ln t$, at a fixed D
t	time elapsed since the initiation of creep
t_f	rupture life
α	slope of $\ln \dot{\varepsilon}$ versus D, at any fixed time t
ε	axial strain
$\dot{\varepsilon}$	axial strain rate
$\dot{\varepsilon}_{min}$	minimum axial strain rate
$\bar{\sigma}_1, \bar{\sigma}_3$	principal effective stresses
$\sigma_1-\sigma_3$	deviator stress
$\bar{\sigma}_1/\bar{\sigma}_3$	effective stress ratio

ACKNOWLEDGEMENTS

Grateful acknowledgement is expressed to the National Research Council of Canada for financial support of the study. The author's deeply felt gratitude goes to his research adviser, Dr. Y.P. Vaid, for his constant guidance in the difficult task of obtaining reliable and accurate experimental data. He also wishes to thank the staff of the Civil Engineering Workshop of the University of British Columbia for their highly professional technical assistance.

CHAPTER 1

INTRODUCTION

Time is one of the various variables that affect both shear strength and deformation behavior of all geological materials. In most soils the effect of time is either difficult to isolate from other factors or has a relatively minor importance. However, for clays, time effects may have a considerable influence.

Following the traditional separation between deformation analysis and stability analysis in soil mechanics, time effects on clays have been considered, on one hand, in the application of oedometer tests to settlement problems, and on the other hand, in the application of shear strength tests to slope stability problems. The first phenomenon is concerned with volumetric deformations of clays. The second aspect deals with the time dependence of strength, and with the general relationship between stress, strain and time. During the shearing of a clay, two types of time effects are encountered depending on whether the volume of the material is varying (drained conditions) or is considered to be constant (undrained conditions). The knowledge of undrained time dependent behavior of clays has a considerable practical importance in the prediction of end of construction deformations and stability of earth structures involving such materials.

Undrained time dependence of stress-strain and strength characteristics of clays have been investigated by several researchers (1,2,3,8,10,14,24,32). It has been found that the undrained strength measured in the laboratory using the conventional shear tests depends on the speed of testing; the increase in strength with speed of testing being more pronounced in clays with a higher plasticity index. Similar time effects are observed during creep loading, where deformation increases under constant applied stresses, and the rupture life decreases as the level of creep stress increases (9,10,15,21,25,28,32,31). In general, the stress-strain and strength response of a clay is a function of the time history of loading or deformation. Prior to the study of time dependent behavior of an undisturbed clay by Vaid and Campanella in 1977 (32), no attempt was made by the previous investigators to correlate stress-strain-strength behavior from test results with different time deformation histories. Furthermore, most of the investigations of time dependent clay behavior have been restricted to normally consolidated clays. Similar studies on overconsolidated clays are very limited (15,31,33), even though such clays have a wide spread occurrence. Natural clays may be overconsolidated during their geological history by the weight of soil strata that were later eroded, by the weight of ice that later melted, by dessication due to temporary exposure to weather, or by aging (6,7). The time dependent behavior of these soils is important in predicting the end of construction deformations or assessing the stability of cuttings, foundations, and tunnel openings.

In the experimental study of sensitive clays, it is important to define clearly the overconsolidation state of a clay. Since sensitive clays undergo a radical change in their natural structure at consolidation stresses higher than the apparent preconsolidation pressure, it is necessary that this limiting pressure is never exceeded prior to shear testing if the structure of the clay is to be maintained intact. In this investigation, the overconsolidation of the clay is defined with respect to the in-situ apparent preconsolidation pressure as estimated from the incremental one-dimensional consolidation test. This definition is to be distinguished from the one used in most studies on overconsolidated clays where the samples are first subjected to laboratory normal consolidation, then rebounded to a lower stress. The overconsolidation is then defined with respect to the maximum consolidation stress applied in the laboratory.

In the present study, the influence of rate of deformation and loading history on the undrained stress-strain and strength characteristics of an undisturbed, saturated, sensitive, overconsolidated, marine clay is studied. A variety of time histories of loading are used to shear identical triaxial test samples. These include conventional constant rate of strain shear, constant stress creep, constant rate of loading, and constant load creep among others. The deformation behavior of the clay is correlated among various types of tests using the concept of the equation of state used in metal creep and also found applicable to the time dependent deformation of clays (32,33). Comparisons with the results of the similar study on the same clay in the normally consolidated state are also attempted.

CHAPTER 2

REVIEW OF SOME CONTRIBUTIONS TO THE KNOWLEDGE OF TIME EFFECTS ON CLAYS AT CONSTANT VOLUME

In the last three decades, there have been several contributions to the knowledge of time effects on the undrained behavior of clays. These studies may be classified in two groups. The first group concerns the time dependence of undrained strength of clays as evidenced by the effect of variation in the speed of testing in the laboratory and also in the field. The second, deals with the general relationship between stress, strain and time including the influence of rate of testing as well as creep effects on the deformation and strength behavior of clays. Some of these contributions will be reported hereafter following this distinction.

2.1. Effect of Rate of Testing on the Undrained Strength of Clays

The influence of rate of testing has been recognized in the measurement of the undrained strength of clays in the laboratory as well as in the field. In the laboratory, the effect of rate of strain on clay samples tested in undrained compression has received considerable attention. One of the earliest contributions to that subject was made by Taylor in 1943 (30). The results of his investigation showed that the undrained strength of remoulded Boston Blue Clay increases linearly with the logarithm of the axial strain rate. Numerous studies followed then on various types of clays in order to determine the effect of rate of strain on the value of the undrained strength, also termed apparent cohesion (3,8,14,23,24). It is now well established that the undrained strength of saturated clays increases by 5 to 10% for a 10-fold increase in rate of strain. However, there is a minimum value of rate of strain below which the undrained strength remains essentially constant.

In 1950, Casagrande and Wilson (10) reported an experimental study partly dealing with the effect of rate of loading on the strength of fully saturated brittle clays and clay shales at constant water content. Two types of tests were used: creep-strength tests in which loads were built up quickly and maintained constant until the specimen failed; and long-time unconfined compression tests in which the specimens were subjected to incremental axial loading, the elapsed time between increments of load varying for different tests. As the pore pressures were not measured, the effect of the test duration was related to total stresses. A strength ratio was defined as the ratio of the compressive strength corresponding to a normal rate of loading. In creep-strength tests a normal time to failure of 1 minute was used, and for long-time tests a normal time of loading of 10 minutes. In creep-strength tests at constant water content the strength was reduced to values of between 80 and 40 percent of its normal value in about 30 days. For long-time unconfined compression tests, the strength of fully saturated Mexico City clay showed a decrease of about 40% from its normal value in 30 days of total time of loading. These results clearly indicate that the undrained strength of saturated clays increases with the rate of loading and decreases under the effect of sustained loads.

In 1972, Bjerrum (5) reported several field studies demonstrating that the procedures used then for computing the end of construction stability of an embankment built on soft clay were unsatisfactory mainly because of incorrect values of shear strength introduced in the analysis. The dominant factor causing such errors in the estimation of the shear strength being the rate of testing. He observed that the discrepancy between the undrained strength measured in the field vane tests and the shear strength back-calculated from the reported failure cases was essentially dependent on the plasticity of the clay. He pointed out that the more plastic the clay, the larger the discrepancy between the vane and the field mobilized undrained strength. He then established a correlation between the field shear strength and the values measured in a vane test and proposed a correction factor with which a vane shear strength should be multiplied before it is introduced in a stability analysis. This correction factor varied with the plasticity index of the clay. He further carefully stated that the validity of the proposed correction of undrained strength is limited to such conditions where the factor of safety increases after the construction due to a gain in strength by consolidation.

The contributions discussed above were essentially concerned with the variation of the undrained strength of clays with the duration of stressing. They do not give any indication on the deformation behavior of such clays with time. Contributions on that aspect of time effects on undrained clays will now be presented.

2.2 General Undrained Stress-Strain-Time Behavior of Clays

The study of the relationship between stress, strain and time in the behavior of clays at constant volume was essentially conducted through the experimental observation of creep tests on clay samples. The accumulation of strain with time in constant stress creep tests constitutes a clear manifestation of the time dependence of stress-strain behavior of clays. The study of creep of clays was largely based on the experience accumulated in the analysis of creep of metals and other engineering materials (16,18,34). Mathematical and/or rheological models have been proposed by various researchers (17,21,25,29) for the description of creep deformation of clays. Generally, the creep strain is expressed as a function of the stress, temperature and time. It is customary to assume that the three effects are separable. Two fundamental assumptions leading to two different approaches have been used to describe time dependent response under arbitrary stress changes. It may be assumed that either:

a) the response of the material depends on the present state explicitly,

b) or, the material remembers its past explicitly and responds to the present in a manner that reflects its past history.

The first assumption leads to what is known as the equation of state formulation, while the second results in the so-called memory theory. The equation of state approach was adopted by most researchers for practical reasons of past successful experience with this theory, and simplicity in its application and understanding. Various expressions of the equation of state were proposed to relate creep strain, creep stress, time, and temperature. In all these expressions the effect of temperature is incorporated as a constant. One of the most known expressions is the so-called Bailey-Norton law:

$$\varepsilon = A\sigma^{m}t^{n} \tag{1}$$

where: ε = creep strain

σ = creep stress

t = time

A,m,n = constants function of temperature.

Other more complex expressions have also found use. Generally, the strain rate is of interest and the analysis gives rise to two possibilities. If the creep strain is differentiated with respect to time, the so-called time hardening formulation is obtained. Another formulation can be derived by eliminating time between the equation of state and the time hardening expression. This procedure leads to the so-called strain hardening formulation in which the current creep strain rate depends on the current stress, strain and, through the constants, temperature. Based on these fundamental considerations, phenomenological relationships were developed using empirical curve-fitting techniques for the characterization of creep of soils.

In 1968, Singh and Mitchell (28) used the time hardening approach for the case of constant stress creep, and proposed an apparently general function relating creep rate, creep stress and time in the form:

$$\dot{\varepsilon} = A_1 \exp(\alpha D)\left(\frac{t_1}{t}\right)^m \tag{2}$$

where:

$\dot{\varepsilon}$ = axial rate of strain

t = elapsed time under shear stress $\sigma_1 - \sigma_3$

t_1 = elapsed time at which the constant A_1 is defined

D = shear stress level $\dfrac{\sigma_1 - \sigma_3}{(\sigma_1 - \sigma_3)_f}$

$(\sigma_1 - \sigma_3)_f$ is obtained from a constant rate of deformation undrained triaxial compression test

m = slope of ln $\dot{\varepsilon}$ vs lnt at any fixed value of D

α = slope of ln $\dot{\varepsilon}$ vs D at any fixed time t.

This phenomenological relationship suffers from some serious limitations. The creep stress is assumed to remain constant while creep deformation is taking place, and the expression proposed is not valid either at the beginning or end of stressing. It is therefore difficult to apply this function to a variable stress situation, and the form of the relationship rules out the possibility of creep rupture in the laboratory tests.

In general, rheological models and empirical curve-fitting techniques do not necessarily take into account the mechanisms governing the creep deformation process. Comparisons of the prediction from these models with observed experimental data indicated that the models developed are only approximate and fail to duplicate the real stress-strain-time response of clays.

A greater success has been achieved by the application of the equation of state theory using the strain hardening approach to the correlation between constant strain rate and constant stress creep test data. In 1952, Pao and Marin (22) obtained a good agreement in correlating the results of the two types of tests on Plexiglass. In 1963, Coates, Burn and McRostie (12) suggested the validity of such a correlation for clays but did not have any data to support it. In an extensive study of the time dependent behavior of an undisturbed clay, Vaid and Campanella (32) presented in 1977 a clear experimental evidence supporting the equation of state theory. The results of triaxial compression tests on normally consolidated undisturbed Haney clay showed that increase in strain rate, rate of loading, length of aging, and thixotropic hardening all result in stiffer undrained stress-strain response and higher undrained strength. Based on the assumption that at a given level of strain (or structure), the shear stress is a function only of the instantaneous rate of strain, and is independent of the past strain rate history, successful correlations tied together the test results from various time loading histories. It was also shown that the effective stress failure envelope for the clay tested was unaffected by any kind of time effects. The equation of state was similarly found applicable to the study of the strain rate behavior of a heavily overconsolidated quick Leda clay (33). A satisfactory correlation was established for this clay between constant strain rate shear and constant stress creep loadings.

The similarity in the magnitude of the axial strain at failure - defined at minimum strain rate in creep tests, and at peak deviator stress in constant strain rate shear - supported the existence of a critical shear strain level at which rupture occurs as suggested by Coates and McRostie in 1963 (12). Such a suggestion, supported by experimental evidence for the existence of a critical level of strain responsible for triggering failure, has also been made by Leonards (19), Campanella and Vaid (9), and Vaid et al. (33).

In the present investigation, the time dependent behavior of a lightly overconsolidated, sensitive clay will be studied in order to obtain basic information on such behavior as the data available in the literature is still deficient on that subject. The applicability of the equation of state theory in its strain hardening formulation will be considered, and comparisons of the general form of behavior with the same clay in the normally consolidated state will be presented.

CHAPTER 3

EXPERIMENTATION

3.1 Material Tested

A local undisturbed medium stiff clay (called Haney clay) was used for the study. Haney clay is believed to have been deposited in a marine environment and later subjected to partial leaching due to surface infiltration. The clay was block sampled from an open pit and all test samples were trimmed to cylinders of ϕ 3.5 cm × 7.5 cm from blocks obtained from the same horizon. This ensured the least variation among individual samples. Some physical properties of the clay tested are outlined in Table 1. It is a grey silty clay with uniform horizontal layers of about 0.5 cm in thickness of dark grey organic material. Figure 1 shows a typical distribution of initial water content along a vertical sample. The higher values of water content are due to the organic layers. These layers were regularly encountered in all samples and did not seem to have any noticeable effect on the test results.

It may be of interest to report that during the unconfined compression test the samples exhibited a brittle rupture at very low axial strain. The failure planes formed

TABLE 1 PROPERTIES OF HANEY CLAY

Natural water conent	63 to 73%
Degree of saturation	100%
Liquid limit	89%
Plastic limit	35%
Plasticity index	54%
Liquidity index	0.52 to 0.70
Specific gravity of solids	2.80
Clay fraction (d < 0.002 mm)	85%
Silt fraction (0.002 mm < d < 0.06 mm)	13%
Activity	0.64
Maximum past pressure	3.5 kg/cm^2
Unconfined compression strength ($\dot{\varepsilon}$ = 1.1%/min, ε_f = 1.4%)	1.4 kg/cm^2
Sensitivity	6 to 10

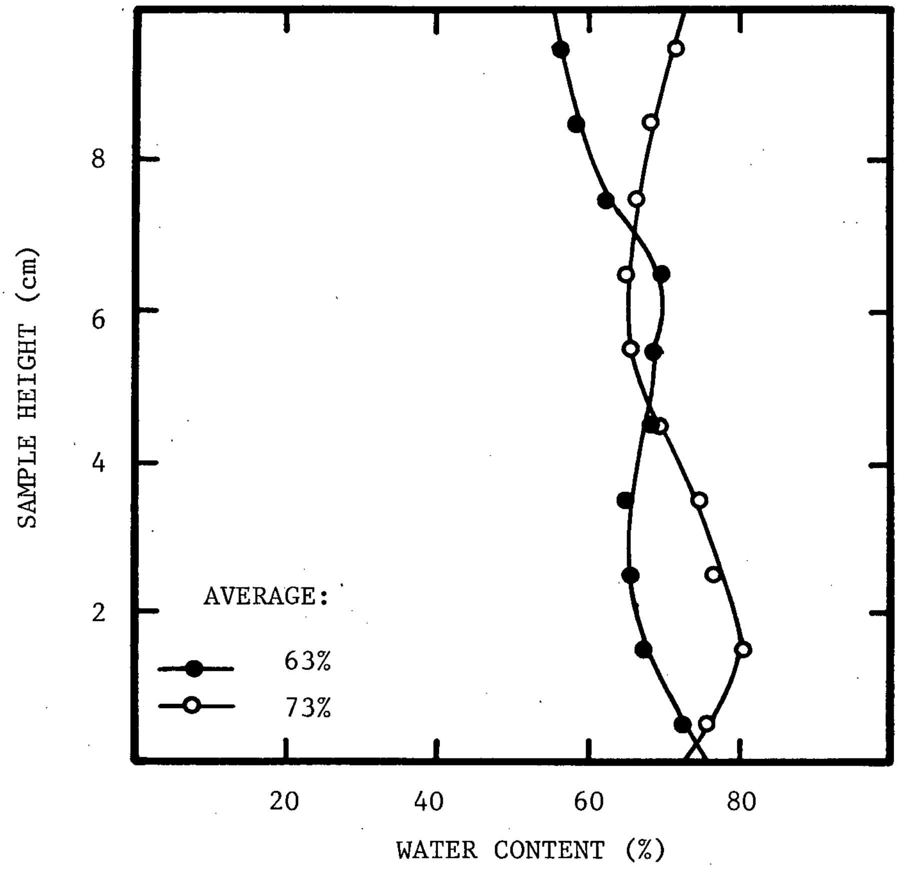

FIG. 1 DISTRIBUTION OF NATURAL WATER CONTENT ALONG VERTICAL SAMPLES OF HANEY CLAY.

a wedge cone close to the top of the specimens and extended vertically to the bottom. This type of failure occurred regardless of the direction of loading with respect to the bedding layers.

3.2 Apparatus

Undrained strength of overconsolidated Haney clay at small confining pressure is relatively low. The maximum deviator stress for an effective confining pressure of 0.5 kg/cm^2 and overconsolidation ratio of 7 is about 1.0 kg/cm^2 in conventional constant rate of strain shear. The accurate determination of the variation of undrained strength with time alone thus requires a great confidence in the measurement of the axial load applied to shear the samples. In a standard triaxial apparatus, friction is developed around the loading ram by the O-ring seal. The amount of friction generated during loading depends on the value of the cell-pressure and the rate of deformation or the rate of loading. It also depends on occasional bending of the loading ram due to any tilting of the top cap of the sample. The accurate measurement of the amount of friction generated is cumbersome and in any case questionable. Among the several methods available for the reduction of error due to friction effects on the measured axial load outside the triaxial cell, the hydrostatic seal system with a continuous air-leakage around the loading ram has proven to be quite satisfactory. Figure 2 shows the components of the continuously air-leaking system originally proposed by Chan (11).

A slightly modified design of this hydrostatic seal was incorporated in the triaxial cell used in this testing program. In using such a seal, the cell water pressure is counteracted by an equal air pressure applied through the top of the cell. This system forms a film of air continuously leaking upward around the loading ram. The tolerance between the fixed ring and the loading ram is such that friction may develop only at the contact of the loading ram and the stainless steel ball bushing. An attempt was made to measure the amount of friction generated then, and it was found to represent approximately 10 gram force. It is believed that no additional friction will develop since with the small magnitude of axial loads involved bending of the loading ram can be disregarded. The magnitude of cell pressure used (2.5 kg/cm^2) is not likely to cause any increase in ram friction.

The only difficulty that may be encountered during the application of the confining pressure is a slight delay in the generation of the same pressure in the chamber and through the seal system. The sealing air pressure and the cell water pressure must be applied simultaneously so that the level of the contact between air and water will be kept

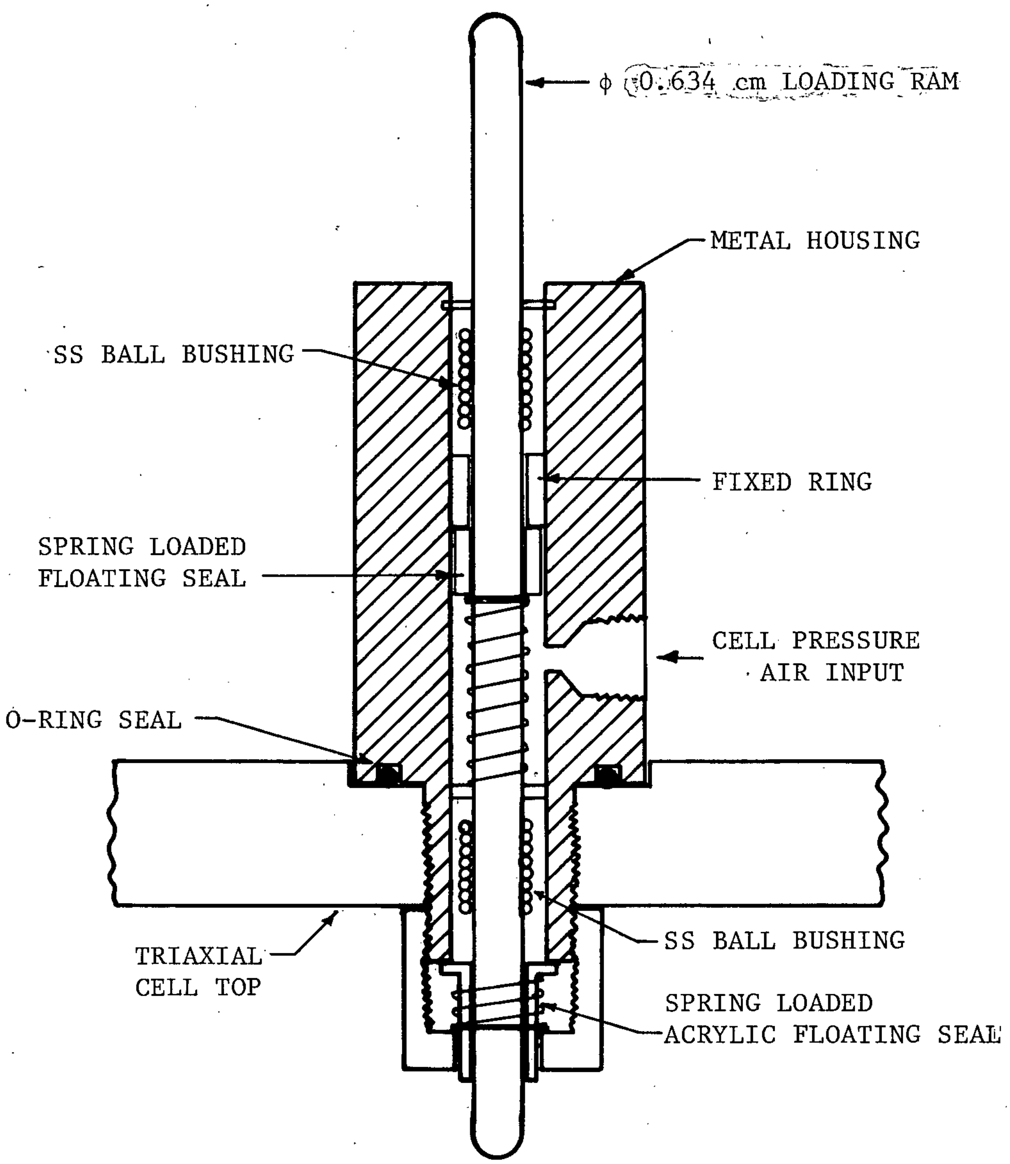

FIG. 2 COMPONENTS OF THE HYDROSTATIC SEAL SYSTEM FOR LOW FRICTION TRIAXIAL CELL.

above the top of the cell and below the air leak input. This may be done by coupling the air supply for the cell water pressure and for the air leaking seal system to the same air pressure regulator and by providing a large bore tube connection between a temporary plexiglass water reservoir and the triaxial chamber for the initial application of the confining pressure. When the level of the air-water contact is ensured to be above the top of the chamber, the cell water pressure connection may be switched to the small 0.3 cm outside diameter saran tube diffusion spiral system.

In undrained triaxial tests on clay, the measurement of the pore pressure generated during shearing is critical. For overconsolidated Haney clay tested in conventional constant rate of strain compression, the excess pore pressure generated at peak deviator stress is almost equal to the effective confining pressure and this has a tremendous effect on the ratio of major effective stress over minor effective stress. During isotropic reconsolidation, samples of Haney clay showed a volume change of about 0.5% for an overconsolidation ratio of 7, after 24 hours under double drainage conditions. Volume changes during reconsolidation are therefore very small. Both pore pressure and volume change measurements must be accurate and reliable. An electronic differential pressure transducer was used for volume change measurements. Pore water pressures were also monitored with an electronic pressure transducer having a capacity of 7 kg/cm^2, a sensitivity of 0.005 kg/cm^2, and a rated compliance of 0.0044 cm^3 for 7 kg/cm^2 change in pressure. The reliability was ensured by full saturation of the measuring system with thoroughly deaired water and spiral diffusion loops of at least 1 metre long of 0.3 cm outside diameter saran tube between the transducers, the back-pressure glass pipette, and the cell-pressure plexiglass reservoir. The spiral diffusion loops allowed the visual detection of any air bubbles and protected the pressure transducers and the chamber from the presence of diffused air from the air pressure supply. The 0.3 cm outside diameter saran tubing drainage lines were kept as short as possible between the triaxial apparatus and the electrical transducers to minimize the compliance of the system.

The axial deformation of the triaxial sample was measured by a Linear Variable Displacement Transducer DCDT having an accuracy of 3.0×10^{-4} cm for a maximum displacement of 1.5 cm. The L.V.D.T. was placed on an extension of one of the three rods retaining the top of the triaxial chamber. The vertical sample deformation was measured by the displacement of a rigid bar clamped on the ϕ 0.635 cm stainless steel loading ram. Since the cell pressure was constant throughout the test program, no displacement due to straining of the pressurized chamber will occur to alter the reference on which the L.V.D.T. was mounted.

3.3 Experimental Procedure

All test samples were subjected in one increment to an all-round effective pressure of approximately 0.5 kg/cm^2 and a back-pressure of 2.0 kg/cm^2. The overconsolidation ratio was equal to 7 assuming a maximum in-situ past pressure of 3.5 kg/cm^2. Double drainage was allowed for a period of 24 hours for all tests.

In a preliminary test, a sample was confined under an effective pressure of 0.5 kg/cm^2 for 38 hours, and the pore pressure generated due to the arrest of secondary consolidation at the end of the drainage period, was observed. Figure 3 shows the amount of pore pressure developed during an undrained period of about 2000 minutes. The small fluctuation in the measurement may be due partly to electronic noise effects and partly to small temperature variations. Nevertheless, the changes in these pore pressures are small. A stable maximum of 0.04 kg/cm^2 was anticipated for the longest time of reliability of undrained conditions (10,000 minutes), and it was considered not sufficient to influence the measurement of pore pressure during the shear process. Hence, for the remaining tests the

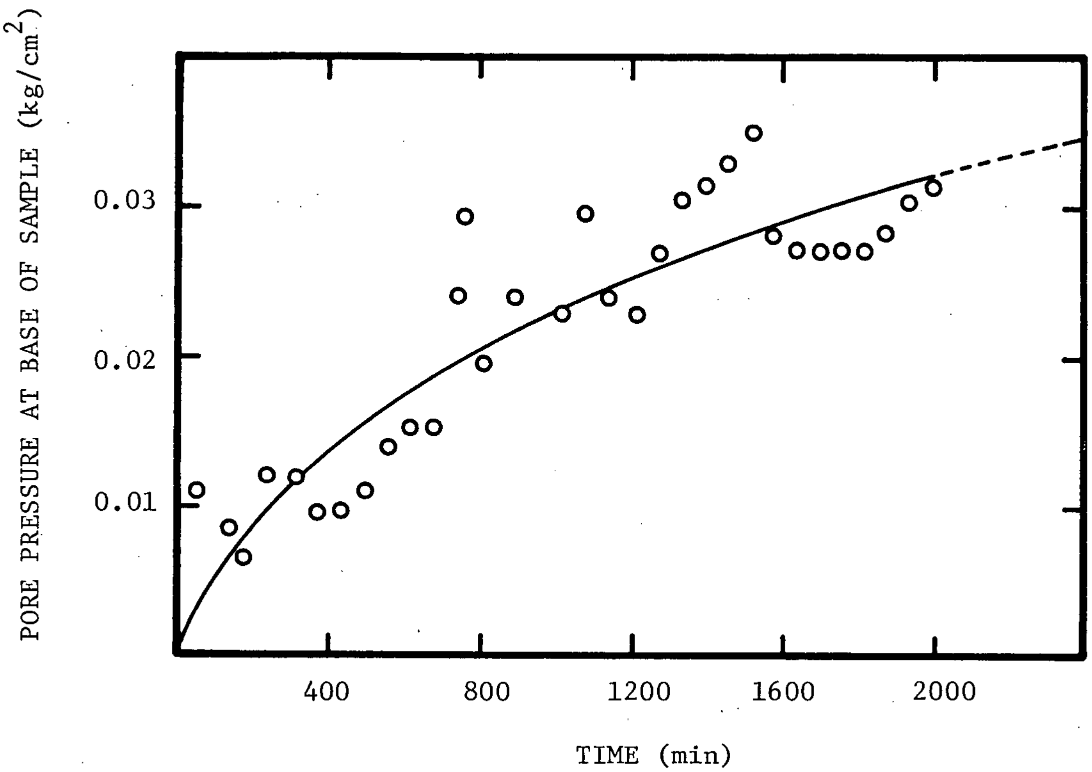

FIG. 3 PORE PRESSURE GENERATED DUE TO THE ARREST OF SECONDARY COMPRESSION AFTER 38 HRS CONSOLIDATION AT $\bar{\sigma}_c = 0.5$ kg/cm^2.

shear loading followed immediately after the consolidation period. This preliminary observation also ensured that no leakage will take place between the cell and the sample through the two thin sealing rubber membranes separated by a coat of high vacuum grease.

In all tests, vertical compression loading with constant confining pressure was imposed in undrained conditions with pore pressure measurement at the base of the specimen. The conventional constant rate of strain compression tests were performed using a 1 tonne capacity Wykeham Farrance loading machine. The results of these tests were used to choose the range of stress levels for the creep tests and the rates of loading in the constant rate of loading compression tests. In the constant stress creep tests, the axial force was applied instantaneously by a ϕ 3.81 cm frictionless Bellofram air piston. The constant air pressure supply for the loading piston was controlled by a precision Fairchild regulator. The load from the piston was transferred to the sample through a rigid metallic frame, allowing the addition of discrete small lead shot as dead weights to keep the vertical stress constant while the sample was compressing and the corresponding area was increasing.

In the constant rate of loading tests, the samples were loaded in compression under a continuously increasing axial force. A ϕ 5.08 cm frictionless Bellofram air piston was used to apply the load on the sample. The air supply for the loading piston was controlled by a variable speed servo-motor coupled to a precision Fairchild air pressure regulator. This system ensured a linear increase of air pressure supply with time resulting in a constant rate of vertical loading of the sample.

The constant load creep tests were performed similarly to the constant stress creep tests. The axial load was instantaneously applied and kept constant by a controlled air pressure supply acting in a ϕ 5.08 cm frictionless Bellofram piston. As compression was taking place, the horizontal area of the sample was increasing, resulting in a continuously decreasing creep stress.

All tests were performed in a constant temperature environment (maximum temperature variation of $\pm$ 0.25°C) in order to eliminate the influence of temperature on deformation rates and pore pressure measurement.

All measurements were carried out electronically and the test data were automatically recorded on a digital cassette tape using a high speed Vidar Digital Data Acquisition System.

CHAPTER 4

TEST RESULTS

4.1. Constant Rate of Strain Shear

Table 2 summarizes the results of conventional constant rate of strain shear tests. Since the stresses at the end of consolidation are similar, the only varying parameter from test to test is the difference in the constant rate of axial strain during shear loading.

The influence of variation in the constant rate of strain on the resulting stress-strain response of the clay is shown in Figure 4. It can be seen that the stress-strain relation for the overconsolidated Haney clay is dependent on the rate of strain. The stress-strain response prior to failure - defined as the peak deviator stress - becomes stiffer with increasing constant rate of strain. Differences in the peak deviator stress as high as 30% may be noted for a strain rate variation of about 3 orders of magnitude. It is interesting to note that the axial strain at peak deviator stress was essentially independent of the rate of strain and was in the range of 1.2 to 1.5%. The sharp decrease in strength past the peak deviator stress is typical of a sensitive and overconsolidated clay. Straining in the post

TABLE 2 RESULTS OF UNDRAINED CONSTANT RATE OF STRAIN SHEAR FOR OVERCONSOLIDATED HANEY CLAY

Test Number	6-80	2-80	29-80	4-80	8-80
Axial Strain Rate (%/min)	1.03×10^{0}	1.37×10^{-1}	1.40×10^{-2}	2.59×10^{-3}	8.72×10^{-4}
Void Ratio: Before consolidation After consolidation	2.025 2.015	2.037 2.020	1.967 1.962	1.953 1.948	1.967 1.967
Consolidaton Stresses: $\bar{\sigma}_1$(kg/cm^2) $\bar{\sigma}_3$(kg/cm^2) $\bar{\sigma}_1/\bar{\sigma}_3$	0.54 0.50 1.07	0.57 0.49 1.17	0.50 0.41 1.20	0.47 0.46 1.04	0.62 0.50 1.23
Deviatoric Stress $(\sigma_1-\sigma_3)$ (kg/cm^2): at ε = 0.8% at ε = 1.0% at ε = 1.2% $(\sigma_1-\sigma_3)_{max}$	 1.24 1.28 1.31 1.32	 1.08 1.18 1.21 1.21	 0.93 1.00 1.06 1.18	 0.92 1.01 1.06 1.07	 0.88 0.94 0.95 0.95
Maximum Stress Ratio $\bar{\sigma}_1/\bar{\sigma}_3$	195.9	∞	7.9	15.4	9.0
Axial Strain ε (%): at $(\sigma_1-\sigma_3)_{max}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	 1.41 1.41	 1.15 1.15	 1.54 1.54	 1.15 1.15	 1.50 0.93
P.P. Parameter A : at $(\sigma_1-\sigma_3)_{max}$ at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	 0.38 0.38	 0.40 0.40	 0.35 0.35	 0.36 0.36	 0.39 0.42

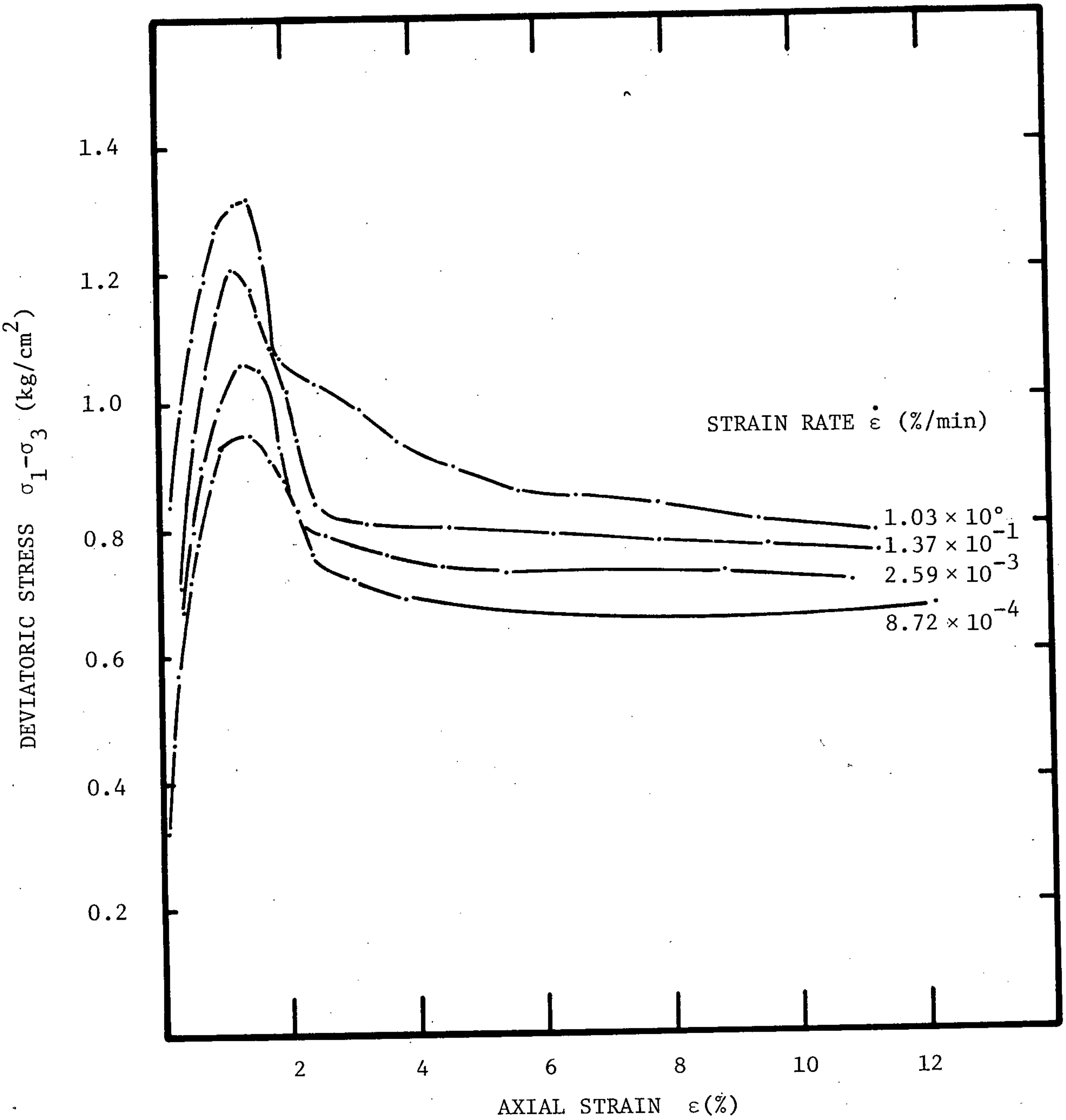

FIG. 4. INFLUENCE OF RATE OF STRAIN ON UNDRAINED STRESS-STRAIN BEHAVIOR OF OVERCONSOLIDATED HANEY CLAY IN CONSTANT RATE OF STRAIN SHEAR.

peak region of a sensitive clay is associated with the collapse of the structure of the material. As a consequence, strain softening is initiated, and for the clay tested this appears to trigger at a critical level of strain. Similar observations regarding a critical strain level have been made for other clays by Coates et al. (12), Vaid and Campanella (32), and Vaid et al (33).

Figure 5 shows the variation of pore water pressure measured at the base of the sample. There is a sharp increase in pore pressure reaching to a maximum value close to the initial effective confining pressure, followed by a rapid decrease to a more stable value. The maximum pore water pressure occurs at the same time as the peak deviator stress and at a constant strain level of about 1.0 to 1.5%. The variation of pore water pressure with axial strain is clearly strain rate dependent prior to the peak value. However, it appears from Table 2 that Skempton's pore pressure parameter at failure A_f does not show any clear strain rate dependence. This would imply that differences in pore water pressure among various tests (Figure 5) are associated solely with the influence of strain rate on deviator stresses.

The results on Table 2 also indicate that the maximum effective stress ratio is reached at about the same strain as the peak deviator stress for all constant rate of strain

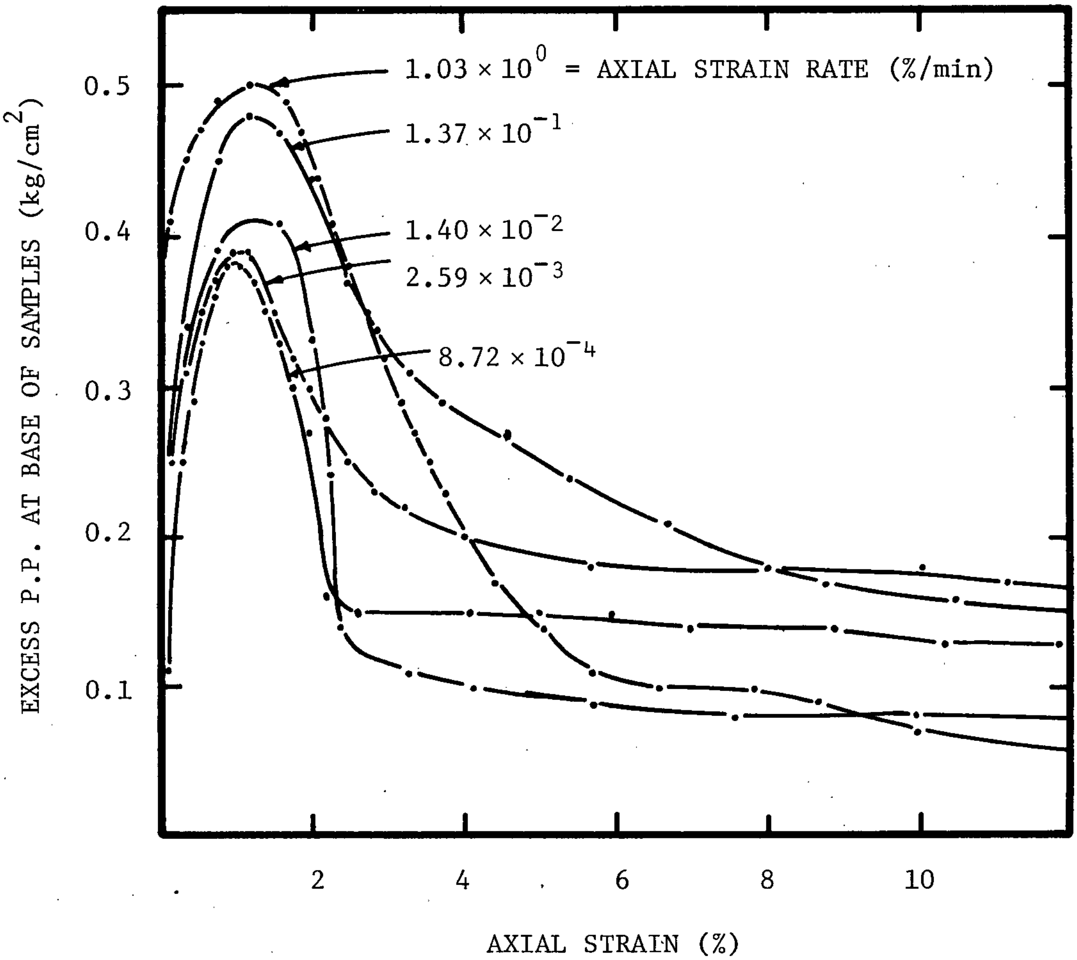

FIG. 5 VARIATION OF EXCESS PORE PRESSURE WITH AXIAL STRAIN FOR UNDRAINED O.C. HANEY CLAY IN CONSTANT RATE OF STRAIN SHEAR.

shear tests. The extremely high values of this ratio are a consequence of very low values of minor effective stress which, in turn, are related to the high pore pressures generated at that time. Other investigators (27) have reported that for lightly overconsolidated sensitive Norwegian clays the point of maximum effective stress ratio occurred fairly early in an undrained triaxial compression test, about 2% axial strain, and the point of maximum deviator stress was reached later in the test at an axial strain depending on the preconsolidation ratio. For that study the overconsolidated state of the clay was obtained by consolidating all the samples, under the conditions of no lateral strain, to a same stress largely in excess of the apparent preconsolidation pressure, then rebounding them to lower consolidation stresses. As noted previously this procedure results in a collapse of the natural sensitive structure of the clay. Therefore, a possible explanation for the contrasting behavior of Haney clay for which both maximum deviator stress and maximum effective stress ratio occur at an equal small level of strain may be the unaltered sensitive nature of the clay structure.

The strain rate dependence of the undrained strength of the clay tested is better shown in the semi-logarithmic plot of Figure 6. The results of these tests indicate a linear increase in undrained strength with the logarithm of axial

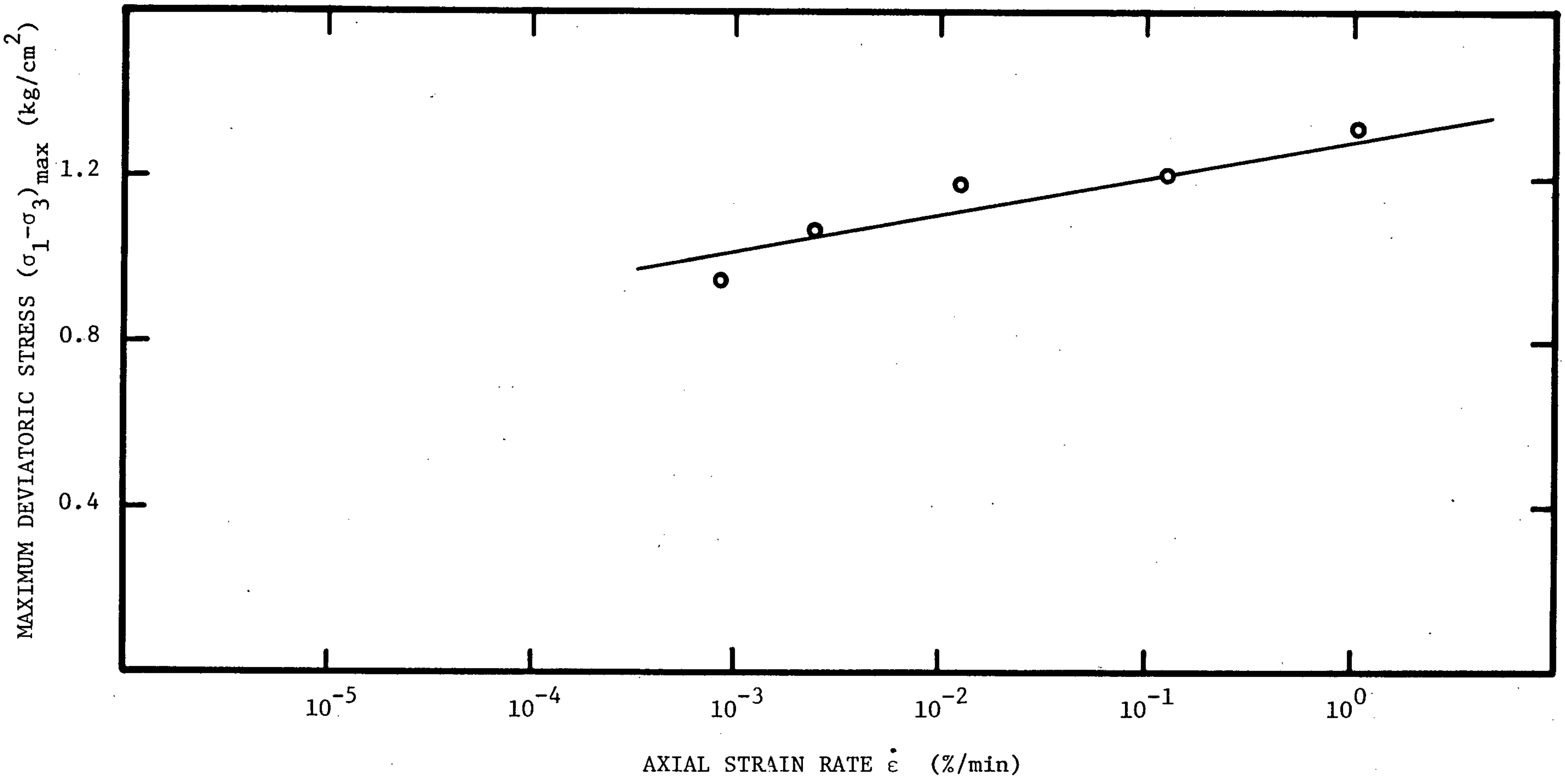

FIG. 6. VARIATION OF UNDRAINED STRENGTH WITH RATE OF STRAIN IN CONSTANT RATE OF STRAIN SHEAR.

strain rate of the order of 8% per log cycle. Similar increases ranging between 5 and 10% per log cycle of strain rate were reported by several other researchers for other clays, both normally consolidated and overconsolidated (1,2,3,5,8,12,14,23,24,30,32,33).

4.2. Constant Stress Creep

The results of this series of tests are summarized in Table 3. Since the end of consolidation conditions are essentially identical, the only variable from test to test is the constant deviator stress during creep loading.

The development of axial strain with time at various levels of creep stress is shown in Figure 7. For a stress level of $0.83\ kg/cm^2$ a continuously decreasing deformation rate was observed until the maximum time of reliability of undrained conditions when the test was terminated (10,000 minutes). It is believed that this stress level approaches the upper yield strength value and that no failure will develop with time under stresses smaller than this value. For the higher stress levels, the samples progressively strained with time until eventual rupture. It is clear from Figure 7 that the time of failure (total collapse of the clay sample) under a sustained stress decreases with increasing creep stress.

For the test at a stress level of $0.70\ kg/cm^2$, a step change in creep stress was suddenly applied after an elapsed

TABLE 3 RESULTS OF UNDRAINED CONSTANT STRESS CREEP FOR OVERCONSOLIDATED HANEY CLAY

Test Number	19-80	21-80	51-81	45-81	27-80	24-80	28-80	18-80	31-81
Deviatoric Stress Level (kg/cm^2)	1.15	1.13	1.06	1.01	0.98	0.95	0.93	0.83	0.70 1.08
Void Ratio:									
Before consolidation	1.878	1.822	1.845	1.774	1.883	1.768	1.952	1.755	1.891
After consolidation	1.876	1.822	1.852	1.771	1.888	1.811	1.996	1.774	1.927
Consolidation stresses:									
$\bar{\sigma}_1$ (kg/cm^2)	0.52	0.50	0.57	0.51	0.51	0.52	0.53	0.49	0.51
$\bar{\sigma}_3$ (kg/cm^2)	0.51	0.48	0.51	0.50	0.49	0.49	0.50	0.49	0.50
$\bar{\sigma}_1/\bar{\sigma}_3$	1.02	1.05	1.11	1.02	1.04	1.05	1.06	1.00	1.02
Axial Strain Rate $\dot{\varepsilon}$ (%/min):									
at $\varepsilon = 0.8\%$	–	1.4×10^{-1}	6.5×10^{-2}	1.7×10^{-2}	6.4×10^{-3}	3.5×10^{-3}	6.7×10^{-4}	2.7×10^{-5}	
at $\varepsilon = 1.0\%$	2.2×10^{-2}	2.5×10^{-2}	9.4×10^{-3}	3.1×10^{-3}	1.1×10^{-3}	2.4×10^{-4}	1.2×10^{-4}	9.0×10^{-6}	
at $\varepsilon = 1.2\%$	2.1×10^{-2}	1.1×10^{-2}	2.7×10^{-3}	1.0×10^{-3}	5.0×10^{-4}	1.4×10^{-4}	–	–	
$\dot{\varepsilon}_{min}$	1.5×10^{-2}	9.6×10^{-3}	2.0×10^{-3}	4.8×10^{-4}	2.5×10^{-4}	1.2×10^{-4}	8.3×10^{-5}	8.5×10^{-6}	
Time at $\dot{\varepsilon}_{min}$ (minutes)	19	46	160	800	1000	1300	2100	8000	
Axial Strain at $\dot{\varepsilon}_{min}$ (%)	1.42	1.30	1.45	1.55	1.55	1.60	1.10	1.00	
P.P. Parameter A at $\dot{\varepsilon}_{min}$	0.39	0.35	0.36	0.29	0.28	0.45	0.38	0.29 no rupture	

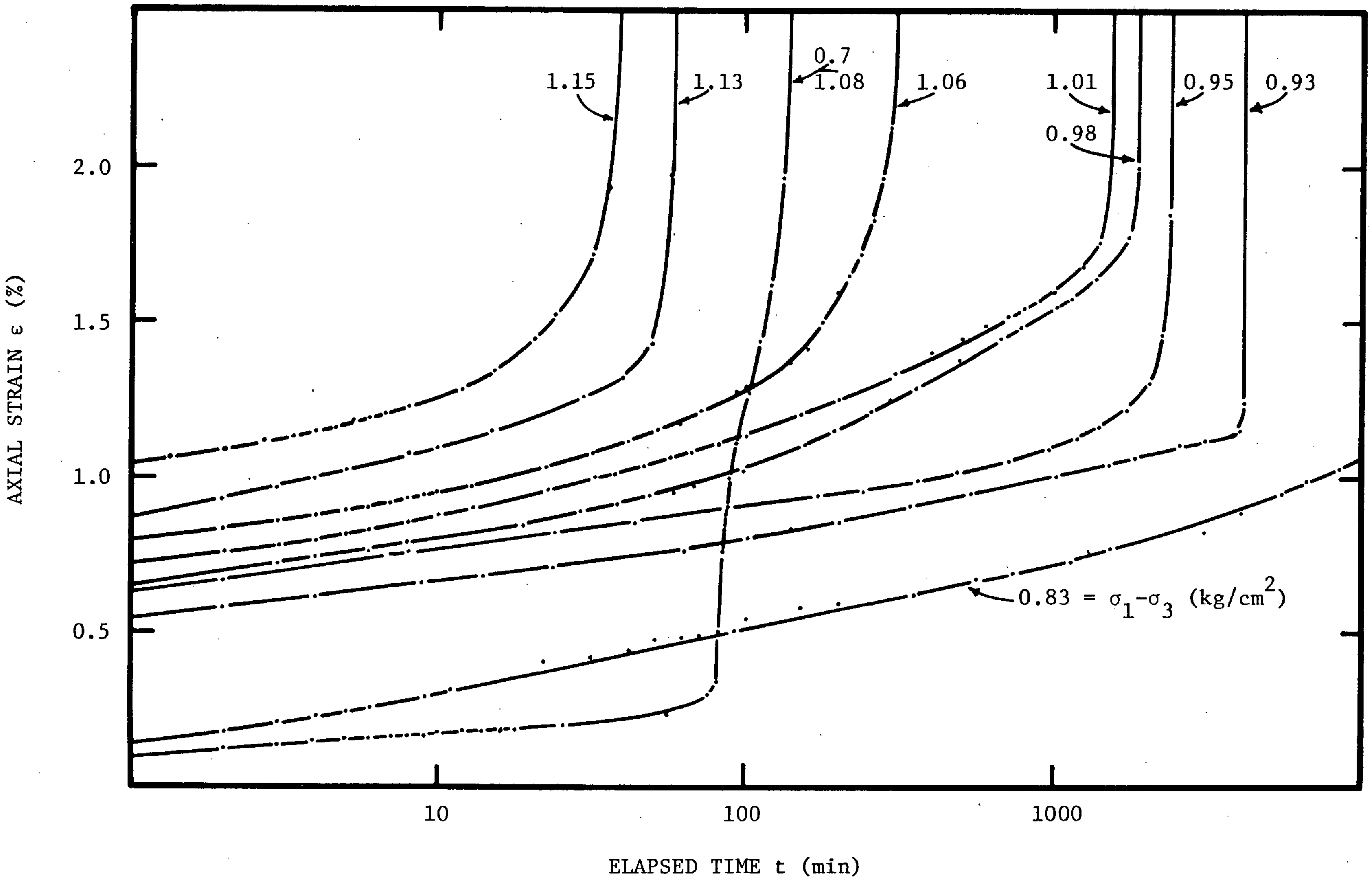

FIG. 7. AXIAL STRAIN VERSUS TIME RELATIONSHIP IN CONSTANT STRESS CREEP.

time of about 80 minutes, bringing the stress level to 1.08 kg/cm^2. As a result, the failure occurred at a time intermediate between the time of rupture in tests at stress levels of 1.06 and 1.13 kg/cm^2, which straddle the stress level 1.08 kg/cm^2. Such a result would be anticipated if the hypothesis that the relationship between current stress, strain, and strain rate is assumed unique and independent of the time loading history used to bring the clay to rupture. The existence of such a relationship for the clay tested is shown in Chapter 5.

The time history of deformation rates in constant stress creep tests is illustrated in Figure 8. This plot was derived by differentiating strain-time curves of Figure 7. For samples which eventually failed, the deformation rate initially decreased until a minimum value was reached before its subsequent acceleration leading to rupture. For the sample at the upper yield stress level, a continuously decreasing deformation rate with time was observed. As pointed out by other researchers (4,9,15,17,21,25,28,29,32) a time region, termed secondary creep, over which the rate of strain is essentially constant does not seem to exist.

It is interesting to note on Table 3 that, irrespective of the creep stress level, the failure of the clay samples - defined at minimum strain rate - occurred at an axial strain level ranging between 1.0 and 1.6%. This strain level is identical to the axial strain at peak deviator stress in the

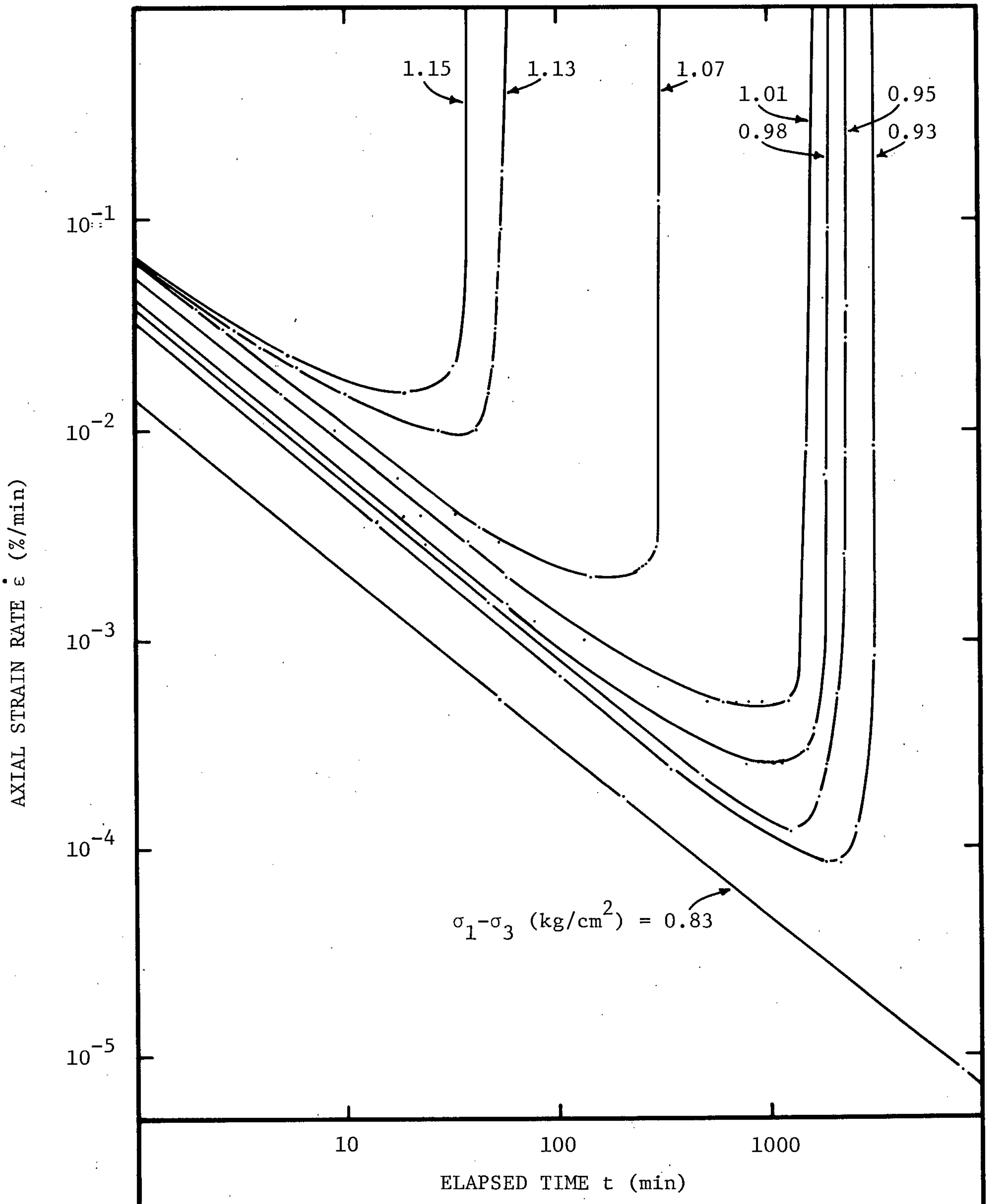

FIG. 8. AXIAL STRAIN RATE VERSUS TIME RELATIONSHIP IN CONSTANT STRESS CREEP.

constant rate of strain shear tests described in the previous section. This result is a further indication of the existence of a critical strain level beyond which the structure of this particular clay collapses and failure ensues.

The variation of the pore water pressure, measured at the base of the sample, with time is shown in Figure 9. Essentially, there was a sudden rise of pore pressure at the application of the axial stress, followed by a slow decrease until collapse of the sample occurred, when the pore pressure dropped very rapidly to a small stable value. The vertical arrows on the plot of the pore pressure versus time indicate when the minimum rate of axial strain was reached. In creep tests, the pore pressure behavior does not give any indication of impending failure of the sample. The pore pressure is still steadily decreasing slowly when rupture is about to take place. For the step-creep test starting with a constant stress of 0.70 kg/cm^2, the pore water pressure was reaching a stable value following the initial rise when it was 'instantaneously' brought up to a higher value due to the step change in creep stress. Consequently, the pore pressure started decreasing until failure occurred. The curve for this test shows clearly the sudden initial rise of pore pressure at the application of creep stress as this part was missing in the plot of the remaining constant stress creep

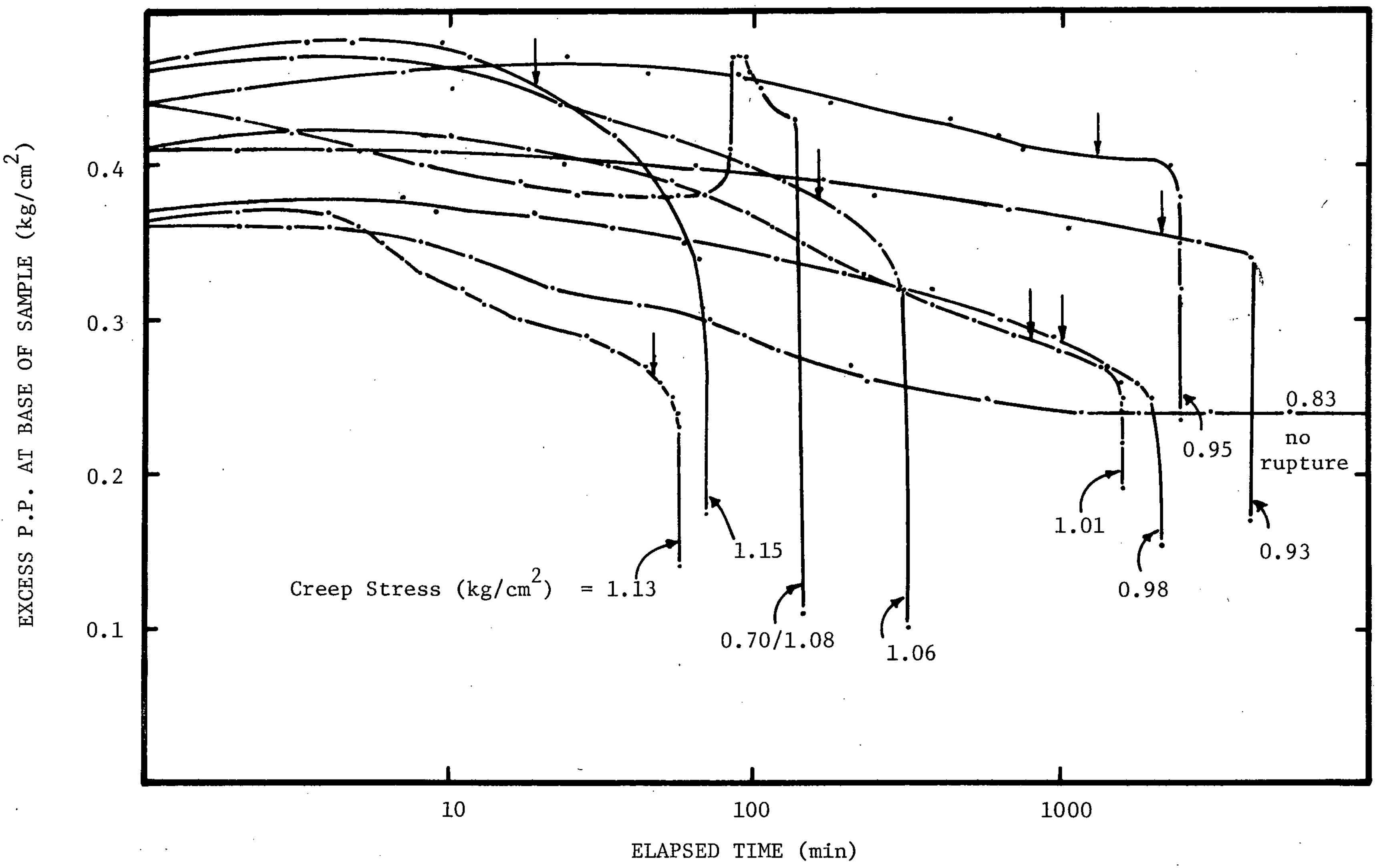

FIG. 9 PORE PRESSURE RESPONSE AT THE BASE OF THE SAMPLE WITH TIME IN CONSTANT STRESS CREEP ON O.C. HANEY CLAY.

tests because of the low frequency of recording at the start of loading.

It is also interesting to note that, in these tests, the effective stress ratio $\bar{\sigma}_1/\bar{\sigma}_3$ has a similar variation with time as the pore pressure since both major and minor total stresses are maintained constant during shear. Hence, the maximum effective stress ratio will occur at the same time as the maximum pore water pressure, that is at the very beginning of the application of the creep stress.

Figure 10 shows the variation of rupture life (time elapsed from the initiation of creep until final collapse) with creep stress. It may be seen that a given level of stress cannot be sustained by the clay for more than a fixed time without undergoing collapse. The plot of Figure 10 represents, in fact, the reduction of undrained strength with time for this particular clay.

In Figure 11, the minimum strain rates are plotted against the corresponding stress levels. The variation of the logarithm of minimum strain rate shows essentially a linear increase with stress level. Comparison of this relationship with the variation of undrained strength with rates of deformation from constant strain rate shear tests will be discussed later.

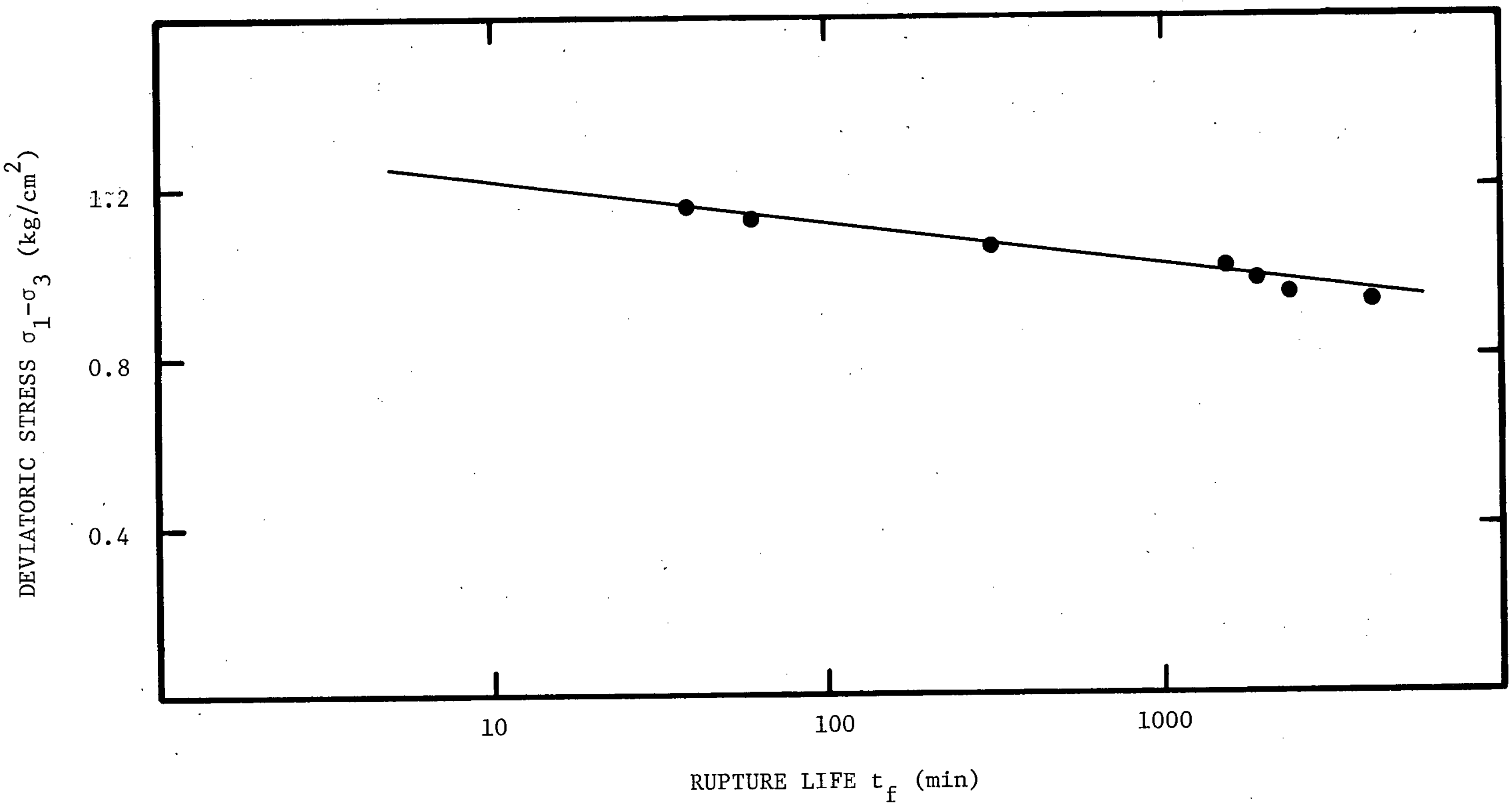

FIG. 10. TIME DEPENDENCE OF UNDRAINED STRENGTH IN CONSTANT STRESS CREEP.

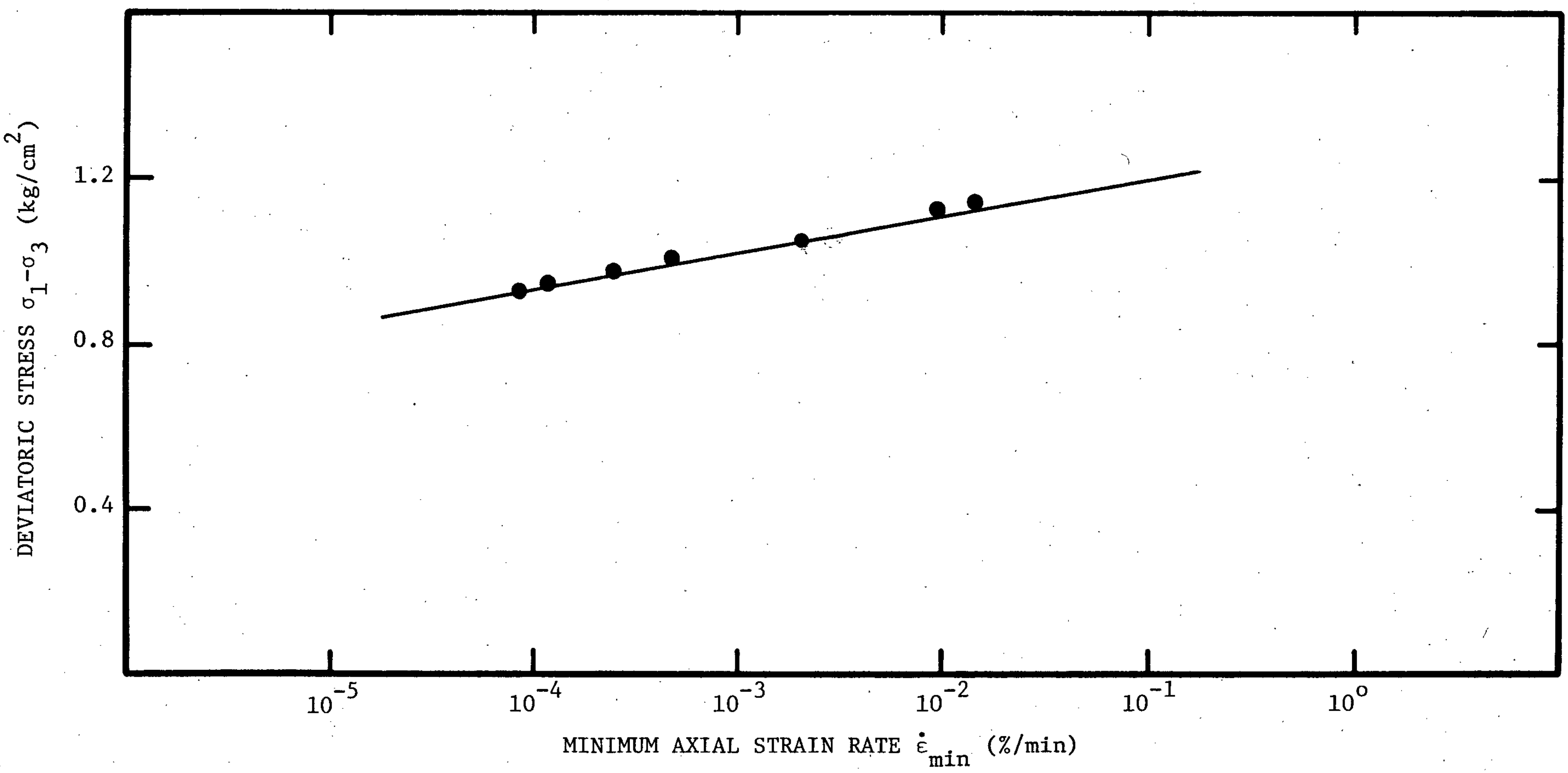

FIG. 11. VARIATION OF UNDRAINED STRENGTH WITH MINIMUM RATE OF STRAIN IN CONSTANT STRESS CREEP.

4.3. Constant Rate of Loading Shear

In this series of tests, the samples were loaded by linearly increasing the axial force with time. The results of these tests are summarized in Table 4. As before, the conditions of the samples are essentially identical at the end of the consolidation period, and the only varying parameter during shear is the rate of loading. The effect of rate of loading on the stress-strain response of the clay is shown in Figure 12. For this type of tests, the rupture of the samples occurred at a very large rate of deformation. Since only the maximum deviator stress and the stress-strain relation prior to failure were of particular interest in this study, the continuous measurement of the post-peak behavior was not recorded. In some of these tests, the variable speed servo-motor used to regulate the continuous increase of loading air-pressure was turned on at the start of shear. A slight delay was experienced in the system to reach a constant rate. This may account for some discrepancy in the results at various loading rates. In spite of the narrow range of variation in the rate of loading, a tendency to an increase in stiffness and strength with increasing speed of testing may be noted.

The effect of rate of loading on the clay behavior, as shown in Figure 13, indicates a definite increase in undrained strength with increasing logarithm of the loading

TABLE 4 RESULTS OF UNDRAINED CONSTANT RATE OF LOADING SHEAR FOR OVERCONSOLIDATED HANEY CLAY

Test Number	13-80	12-80	9-80	44-81	10-80	46-81	14-80	52-81
Loading Rate (kg/min)	2.54×10^{1}	2.74×10^{0}	2.6×10^{-1}	1.6×10^{-1}	1.4×10^{-1}	4.9×10^{-2}	1.6×10^{-2}	8.3×10^{-3}
Void Ratio:								
Before consolidation	1.963	1.997	1.990	1.804	1.998	1.763	1.986	2.054
After consolidation	1.966	1.984	1.996	1.939	2.002	1.878	1.975	1.987
Consolidation stresses:								
$\bar{\sigma}_1$ (kg/cm^2)	0.50	0.48	0.54	0.52	0.44	0.52	0.53	0.53
$\bar{\sigma}_3$ (kg/cm^2)	0.42	0.46	0.43	0.50	0.41	0.50	0.54	0.51
$\bar{\sigma}_1/\bar{\sigma}_3$	1.21	1.06	1.25	1.04	1.06	1.04	0.98	1.03
Deviatoric Stress $\sigma_1-\sigma_3$(kg/cm^2)								
at ε = 0.8%	1.45	1.37	1.10	0.95	0.99	1.02	1.08	0.90
at ε = 1.0%	-	1.40	1.16	1.04	1.09	1.12	-	1.01
at ε = 1.2%	-	-	1.19	1.10	1.16	1.14	-	1.07
$(\sigma_1-\sigma_3)_{max}$	1.52	1.40	1.24	1.14	1.22	1.15	1.12	1.02
Axial Strain Rate $\dot{\varepsilon}$ (%/min):								
at ε = 0.8%	-	6.4×10^{1}	6.5×10^{-2}	3.6×10^{-2}	2.9×10^{-2}	4.7×10^{-3}	6.0×10^{-3}	1.6×10^{-3}
at ε = 1.0%	-	1.0×10^{1}	9.0×10^{-2}	4.8×10^{-2}	3.8×10^{-2}	1.5×10^{-2}	-	2.2×10^{-3}
at ε = 1.2%	-	-	1.5×10^{-1}	6.5×10^{-2}	5.0×10^{-2}	5.0×10^{-2}	-	4.2×10^{-3}
Maximum Stress Ratio $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	21.06	10.37	13.73	50.00	11.23	33.22	10.87	23.66
Axial Strain (%): at $(\sigma_1-\sigma_3)_{max}$	0.96	1.06	1.60	1.52	1.44	1.33	0.89	1.02
at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	0.96	1.06	1.60	1.52	1.44	1.17	0.89	1.59
P.P. Parameter A: at $(\sigma_1-\sigma_3)_{max}$	0.23	0.29	0.28	0.41	0.31	0.40	0.37	0.45
at $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$	0.23	0.29	0.28	0.41	0.31	0.40	0.37	0.39

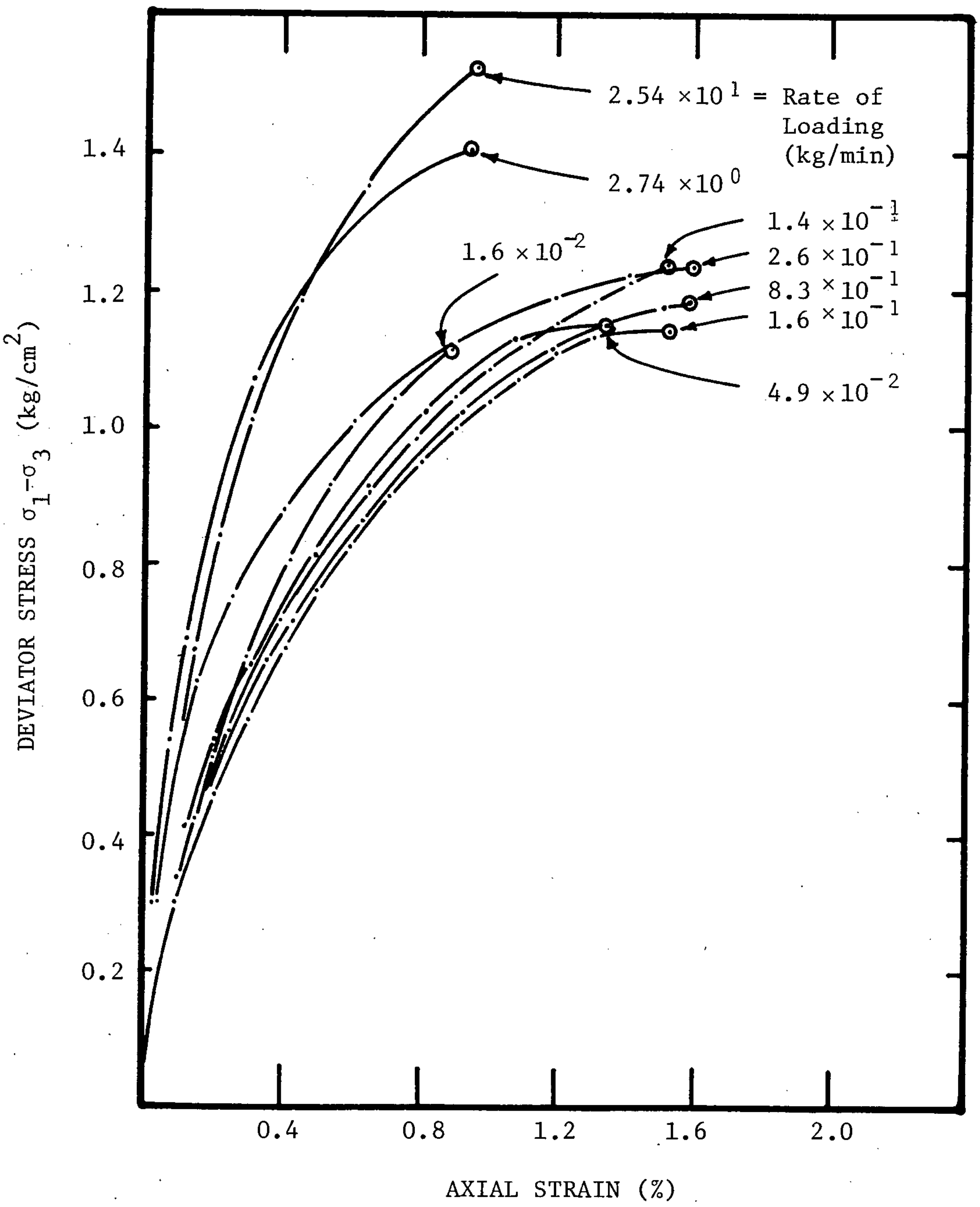

FIG. 12 EFFECT OF RATE OF LOADING ON THE STRESS-STRAIN RESPONSE OF O.C. HANEY CLAY IN CONSTANT RATE OF LOADING SHEAR.

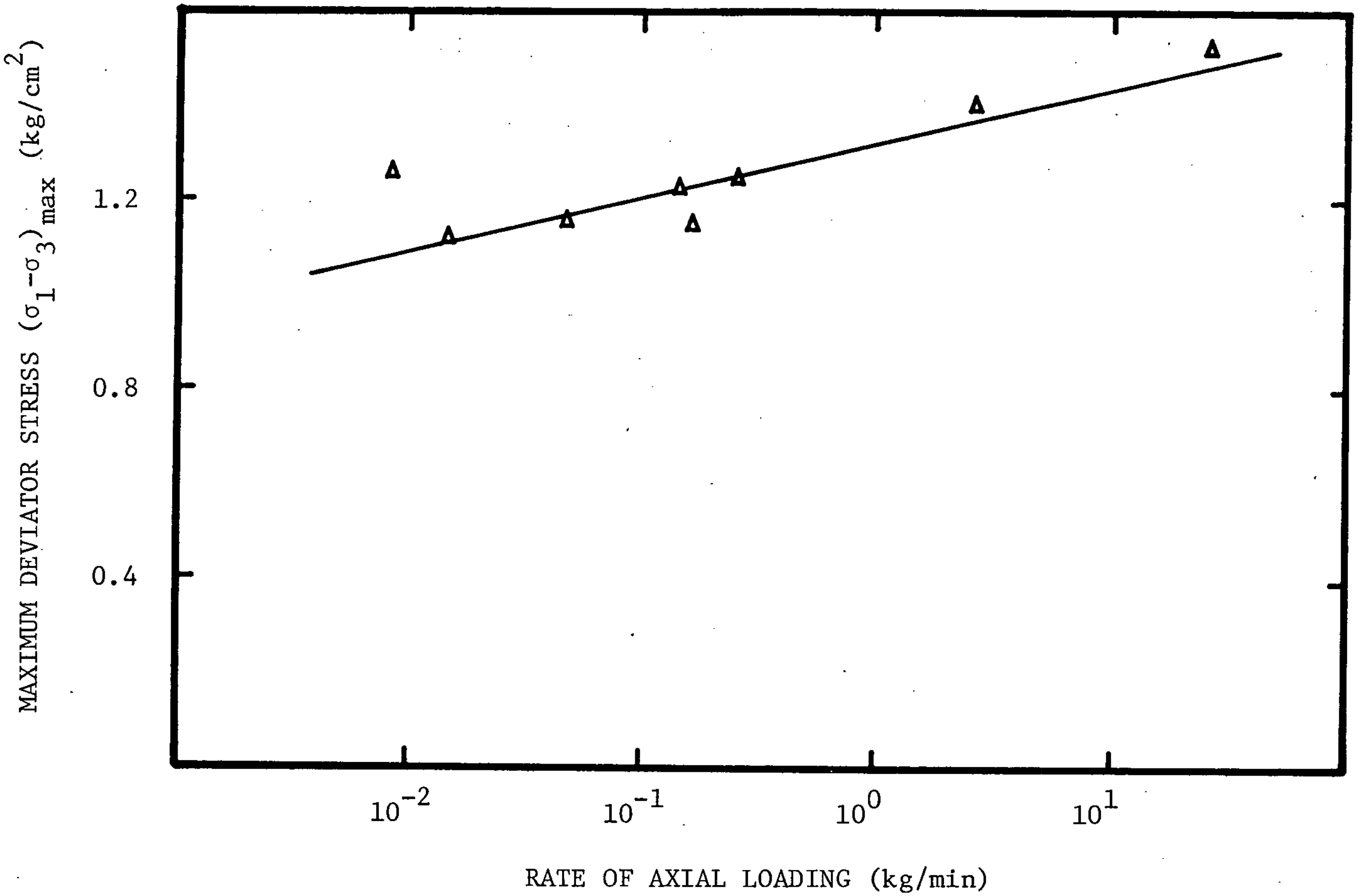

FIG. 13 VARIATION OF UNDRAINED STRENGTH OF O.C. HANEY CLAY WITH RATE OF LOADING IN CONSTANT RATE OF LOADING SHEAR.

rate. This increase in undrained strength is approximately 8% per log cycle in rate of loading. This behavior is similar to the observations reported by Casagrande and Wilson on saturated Mexico City clay (10) and by Vaid and Campanella on normally consolidated Haney clay (32).

4.4. Constant Load Creep

Samples in this series of tests were instantaneously loaded with predetermined loads which were held constant with time. As the deformation progressed, the sample area increased, thereby resulting in a continuous decrease in creep stress.

Table 5 summarizes the results of constant load creep tests. In Figure 14 it can be seen that the deformation behavior with time under constant load is similar to that under constant stress (Figure 7). Due to the low level of strain at failure, the stress decrease during creep was not significant. It is most likely that the test at the initial deviator stress level of 0.92 kg/cm^2 will result in a collapse of the sample had sufficient time for creep been allowed. This argument is based on the fact that the sample had already strained in excess of 1.0% axial strain when the test was terminated; and as shown earlier, a critical level of axial strain of about 1.2 to 1.5% would result in sample undergoing failure.

TABLE 5 RESULTS OF UNDRAINED CONSTANT LOAD CREEP FOR OVERCONSOLIDATED HANEY CLAY

Test Number	41-81	39-81	42-81	43-81	25-80	26-80	17-80
Initial Deviatoric Stress Level (kg/cm)	1.20	1.18	1.14	1.10	1.02	0.92	0.88 1.01
Void Ratio:							
Before consolidation	1.907	1.991	1.860	1.825	1.941	1.989	1.806
After consolidation	2.033	2.160	2.016	1.814	1.940	1.993	1.814
Consolidation stresses:							
$\bar{\sigma}_1$ (kg/cm^2)	0.49	0.51	0.49	0.54	0.52	0.49	0.51
$\bar{\sigma}_3$ (kg/cm^2)	0.48	0.49	0.48	0.49	0.50	0.50	0.50
$\bar{\sigma}_1/\bar{\sigma}_3$	1.02	1.02	1.03	1.11	1.04	0.97	1.02
Deviatoric Stress $\sigma_1-\sigma_3$(kg/cm^2)							
at ε = 0.8%	-	-	1.14	1.10	1.02	0.92	
at ε = 1.0%	1.20	1.17	1.14	1.10	1.02	0.92	
at ε = 1.2%	1.20	1.17	1.14	1.10	1.02	-	
Axial Strain Rate $\dot{\varepsilon}$ (%/min):							
at ε = 0.8%	-	-	3.8×10^{-1}	1.1×10^{-1}	3.3×10^{-2}	5.4×10^{-4}	
at ε = 1.0%	8.0×10^{-1}	2.8×10^{-1}	5.0×10^{-2}	9.0×10^{-3}	5.6×10^{-3}	9.0×10^{-5}	
at ε = 1.2%	2.1×10^{-1}	1.1×10^{-1}	1.0×10^{-2}	9.5×10^{-3}	1.5×10^{-3}	-	
$\dot{\varepsilon}_{min}$	1.1×10^{-1}	6.0×10^{-2}	1.0×10^{-2}	6.0×10^{-3}	1.3×10^{-3}	8.4×10^{-5}	
Time at $\dot{\varepsilon}_{min}$ (minutes)	3	6	20	70	215	2900	
Axial Strain at $\dot{\varepsilon}_{min}$ (%)	1.40	1.50	1.30	1.50	1.40	1.00	
P.P. Parameter A at $\dot{\varepsilon}_{min}$	0.35	0.33	0.33	0.33	0.35	0.38 no rupture	

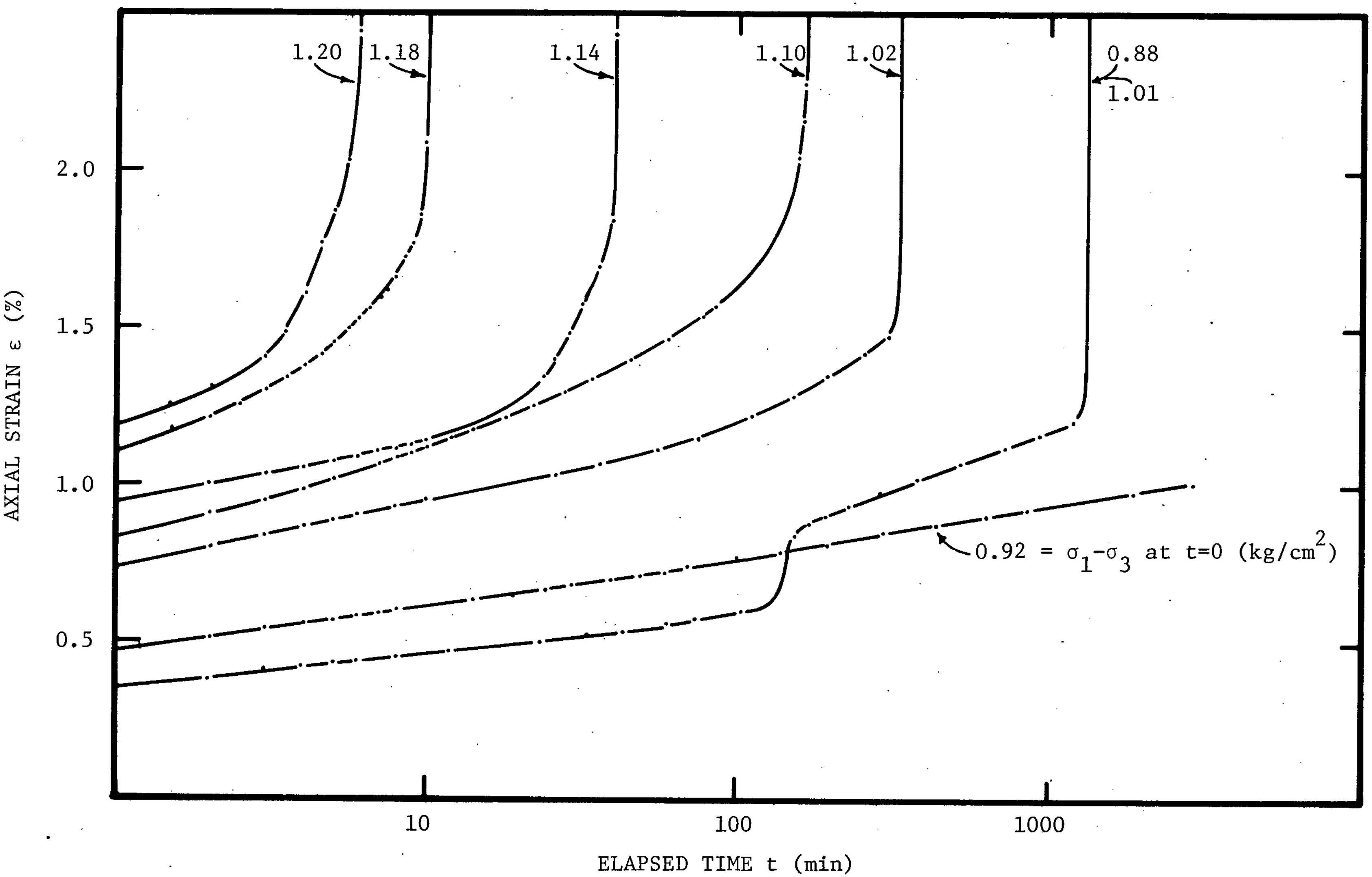

FIG. 14. AXIAL STRAIN VERSUS TIME RELATIONSHIP IN CONSTANT LOAD CREEP.

By differentiating strain time curves of Figure 14, the time history of creep rates was obtained in Figure 15. The deformation rates initially decreased until a minimum value before finally increasing, thus signaling the onset of failure. It may be noted that the shape of the axial strain rate versus logarithm of time relationship under the stress level of $0.92\ kg/cm^2$ does point to the likelihood of reaching the minimum creep rate, and then eventual failure. Figure 16 shows the variation of rupture life (time elapsed from the initiation of creep until final collapse) with initial creep stress. It can be seen that this relationship is also very similar to the results from constant stress creep in Figure 10.

In Figure 17, the minimum strain rates are plotted against the stress levels at the start of shear. The variation of the logarithm of minimum strain rates is essentially a linear increase with initial stress levels. Results shown in Figure 16 and Figure 17 would be expected to be similar to those for constant stress creep, since creep stresses did not vary excessively in constant load creep due to low levels of axial strain accumulation.

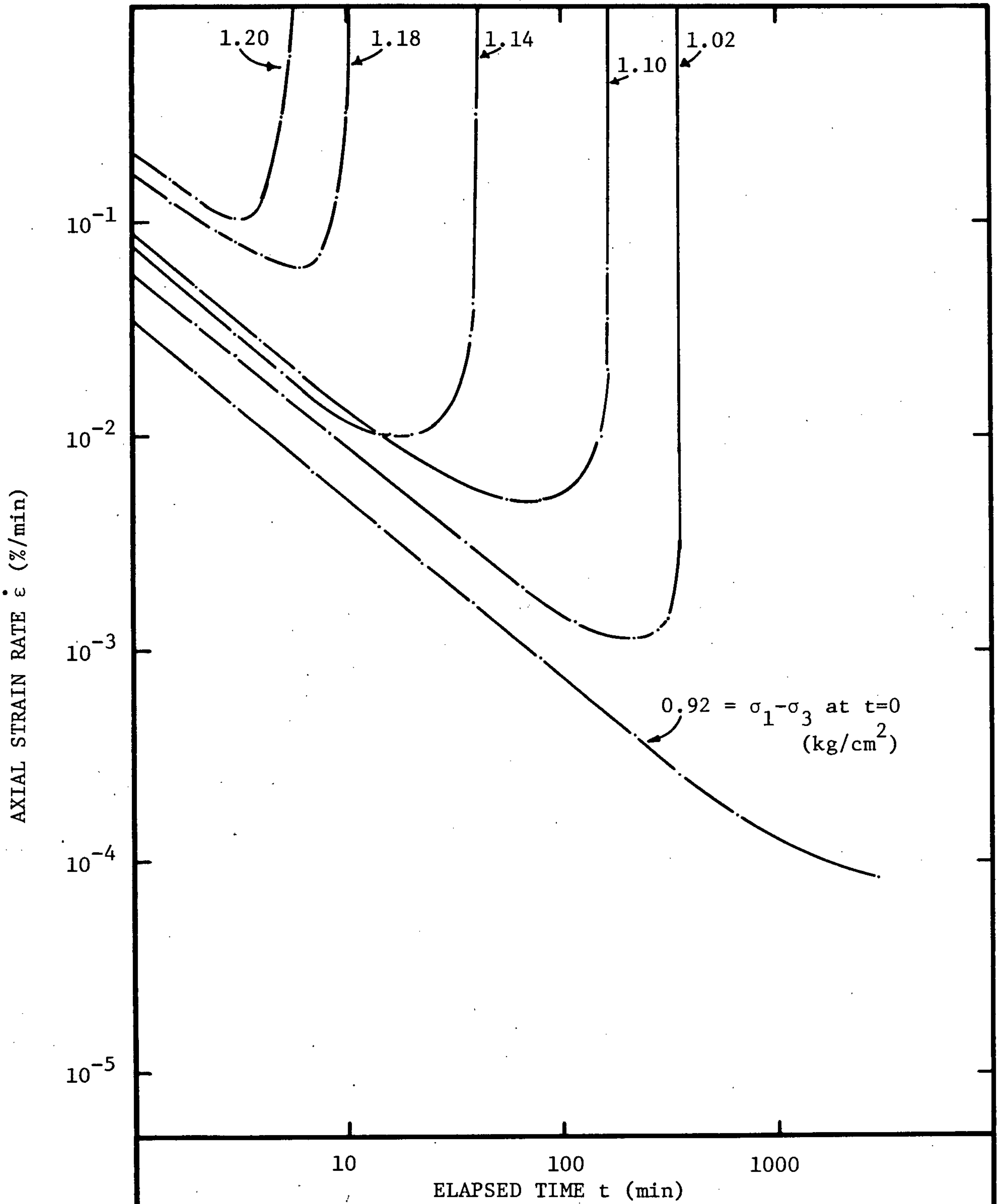

FIG. 15. AXIAL STRAIN RATE VERSUS TIME RELATIONSHIP IN CONSTANT LOAD CREEP.

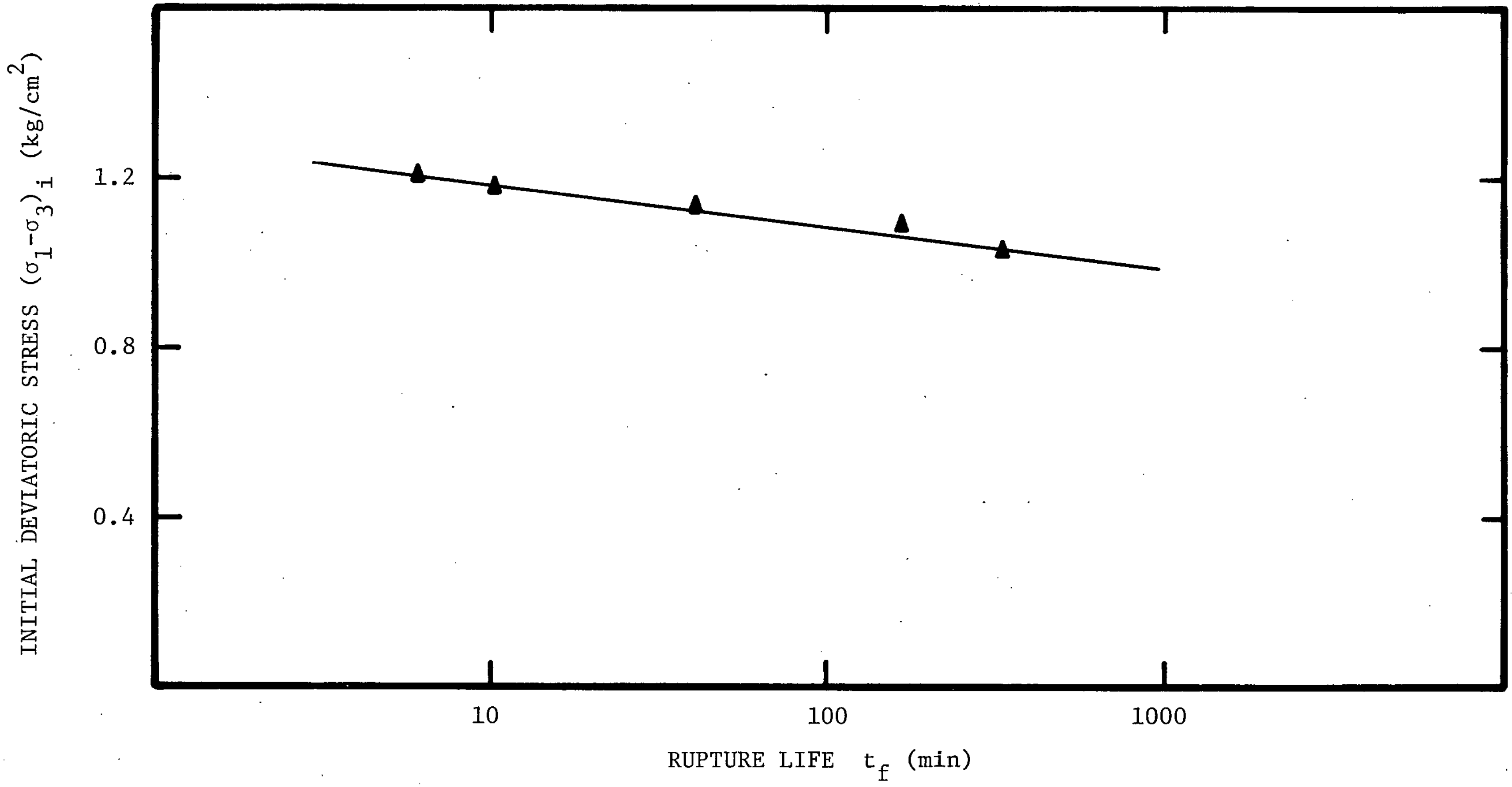

FIG. 16. INITIAL DEVIATOR STRESS VERSUS TIME TO FAILURE RELATIONSHIP IN CONSTANT LOAD CREEP.

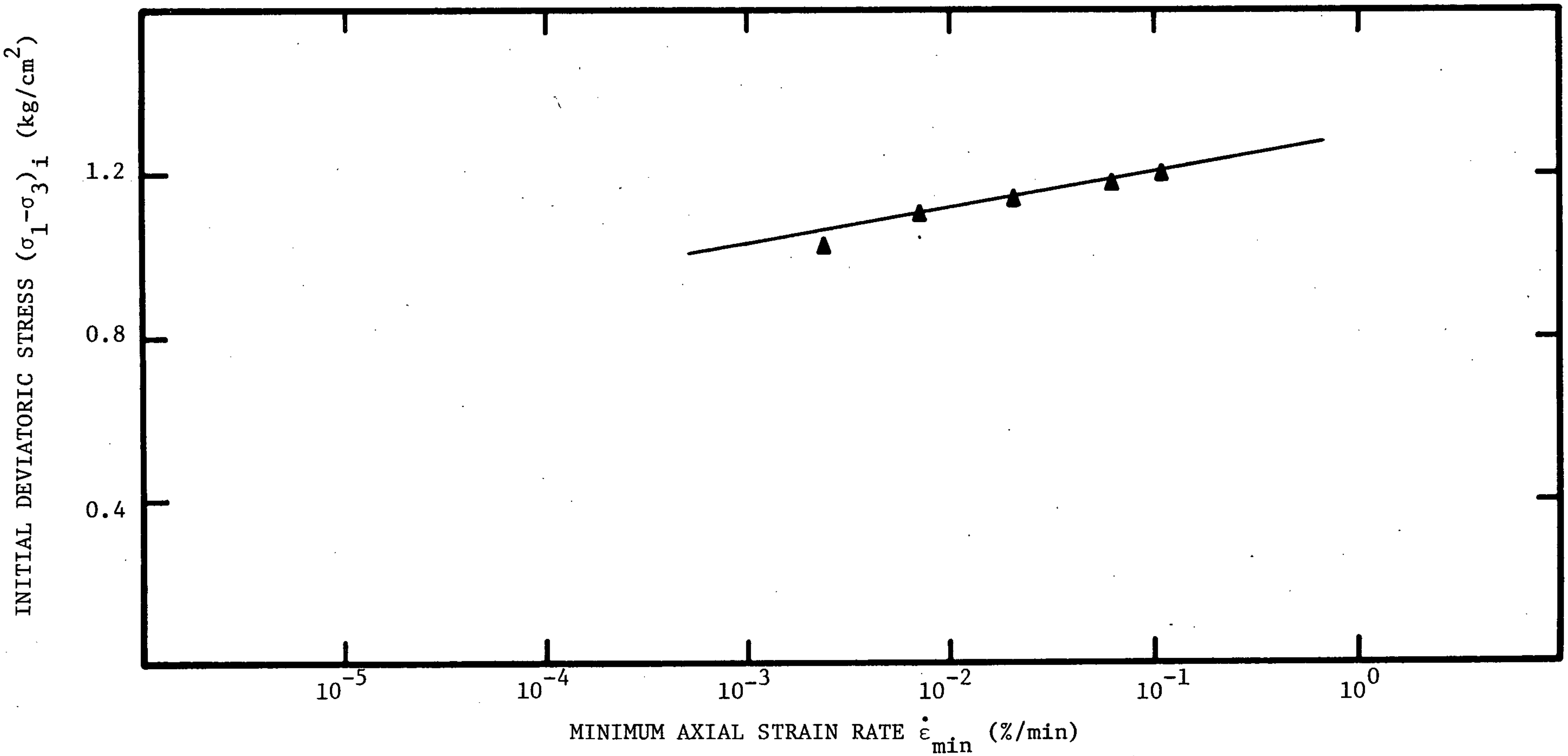

FIG. 17. INITIAL DEVIATOR STRESS VERSUS MINIMUM AXIAL STRAIN RATE RELATIONSHIP IN CONSTANT LOAD CREEP.

CHAPTER 5

CORRELATIONS OF RESULTS FROM TESTS WITH VARIOUS TIME LOADING HISTORIES

5.1. Stress-Strain-Strain Rate Relationship

In order to establish correlations of results from the tests with the different time loading histories considered in this investigation, the rate of strain is used as a unifying variable among the various tests. The uniqueness of the relationship between current stress and current strain rate at any given level of strain during shear is assumed. This is identical to assuming the validity of the equation of state in its strain hardening formulation:

$$\dot{\varepsilon} = f(\sigma,\varepsilon) \qquad (3)$$

where:

$\dot{\varepsilon}$ = creep rate or axial strain rate

ε = creep deformation or axial strain

σ = creep stress or deviator stress

In this expression, the current rate of strain is a function of both the current stress and strain and is independent of the past strain rate history. A direct relationship may be established between strain rate and stress provided that the strain level is maintained fixed to given values.

Since the strain level is essentially the same at maximum deviator stress in constant rate of strain shear and at minimum strain rate in constant stress creep, the first proof of the validity of the proposed equation of state is attempted between the results of constant strain rate shear and constant stress creep tests in terms of undrained strength versus strain rate. Figure 18 illustrates such a correlation. An excellent agreement may bè seen to exist between the results of these two types of tests. As mentioned previously, comparable results have been obtained by other investigators for both normally consolidated and overconsolidated clays (15,32,33) as well as for other engineering materials (16,22).

A similar correlation may be attempted among all the tests in this investigation in terms of deviator stress versus axial strain rate at fixed levels of strain prior to failure. These are shown in Figures 19a, 19b and 19c for the chosen strain levels of respectively 0.8, 1.0 and 1.2%. For each chosen strain level, it may be noted that the data point from tests with a variety of time loading histories fall essentially on the same curve, thus supporting the validity of the equation of state. All the correlations are

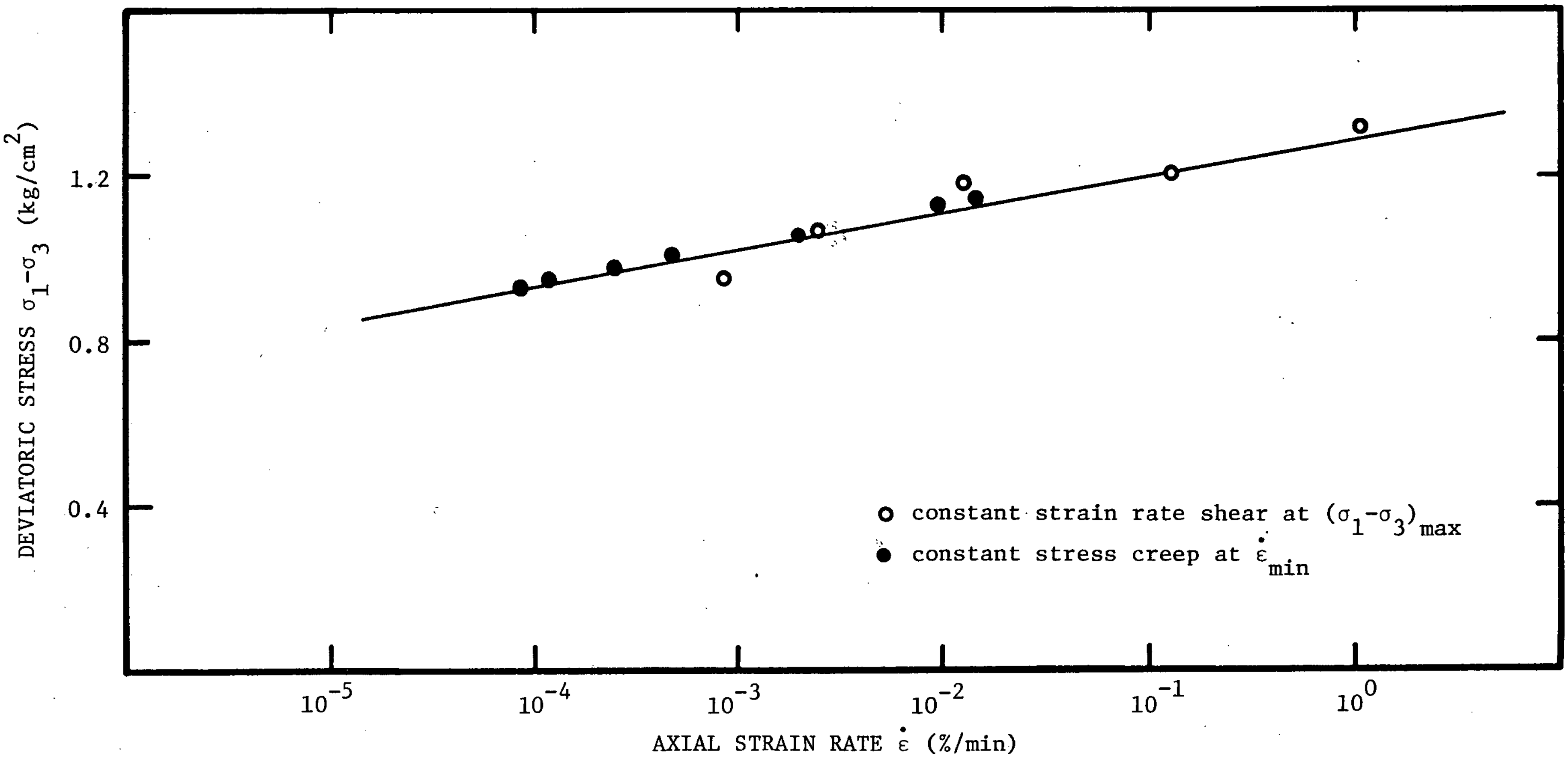

FIG. 18. COMPARISON OF STRAIN RATE DEPENDENCE OF UNDRAINED STRENGTH IN CONSTANT RATE OF STRAIN SHEAR AND CONSTANT STRESS CREEP.

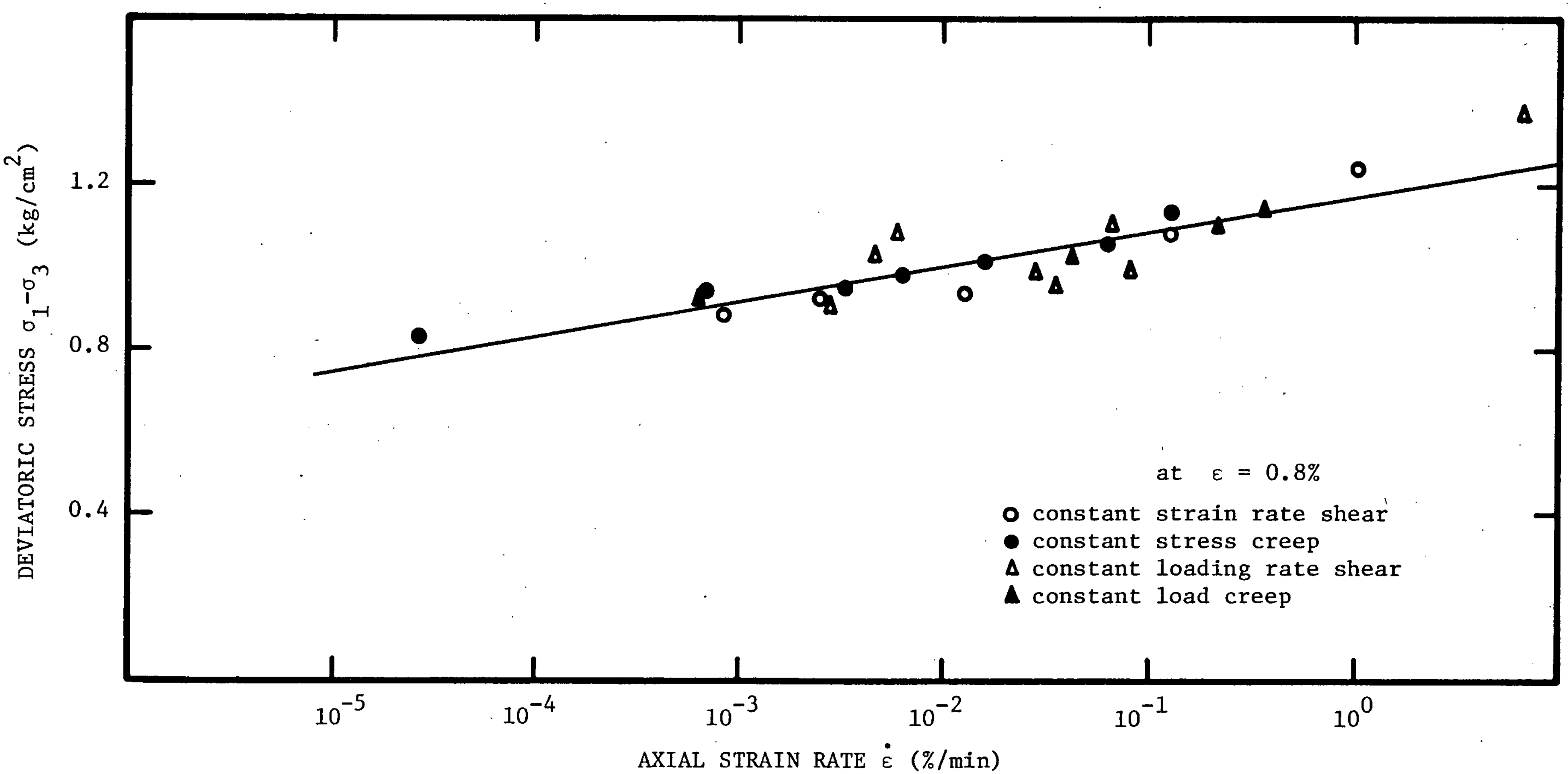

FIG. 19a. DEVIATOR STRESS VERSUS AXIAL STRAIN RATE AT EQUAL STRAIN LEVEL RELATIONSHIP IN CONSTANT RATE OF STRAIN SHEAR, CONSTANT STRESS CREEP, CONSTANT RATE OF LOADING SHEAR, AND CONSTANT LOAD CREEP.

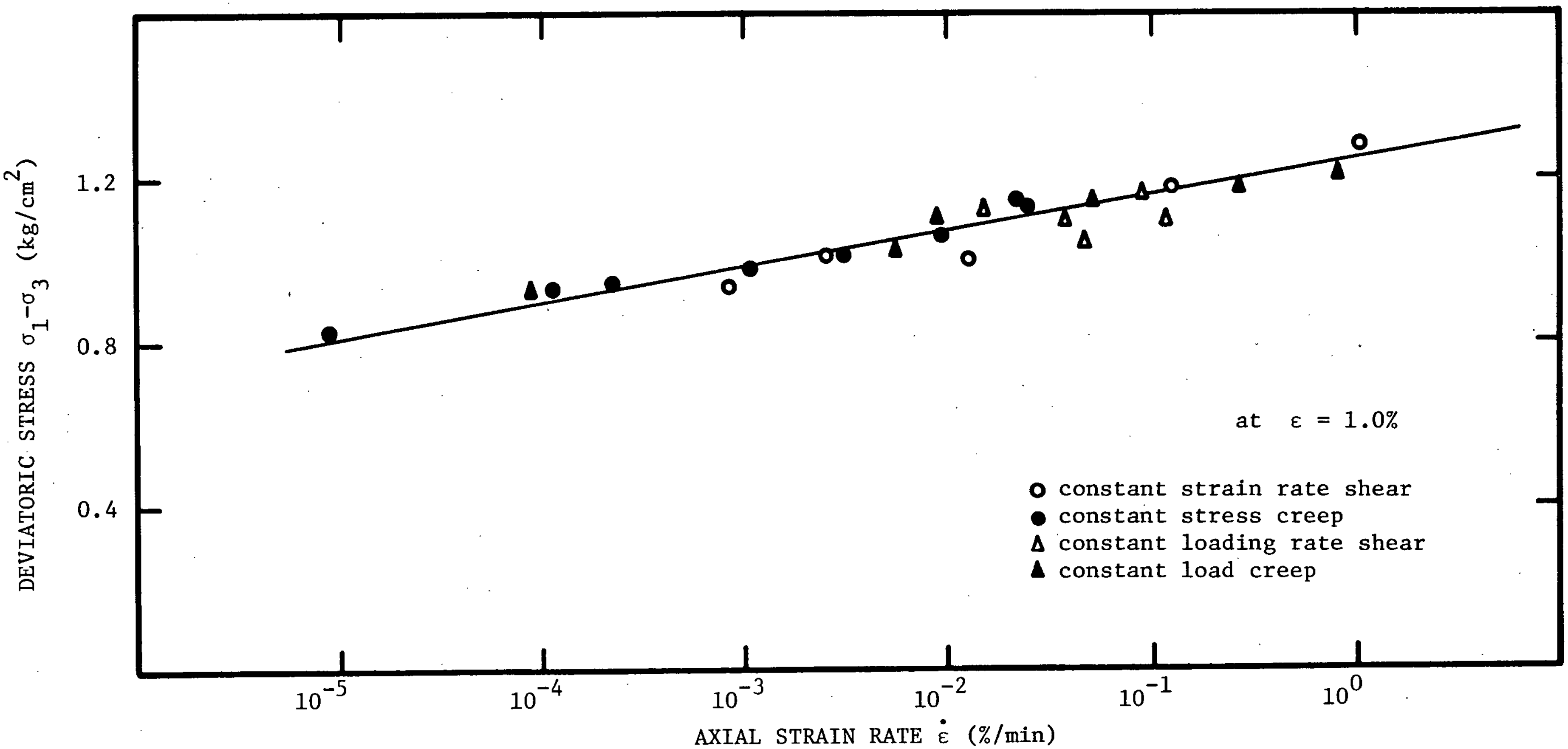

FIG. 19b. DEVIATOR STRESS VERSUS AXIAL STRAIN RATE AT EQUAL STRAIN LEVEL RELATIONSHIP IN CONSTANT RATE OF STRAIN SHEAR, CONSTANT STRESS CREEP, CONSTANT RATE OF LOADING SHEAR, AND CONSTANT LOAD CREEP.

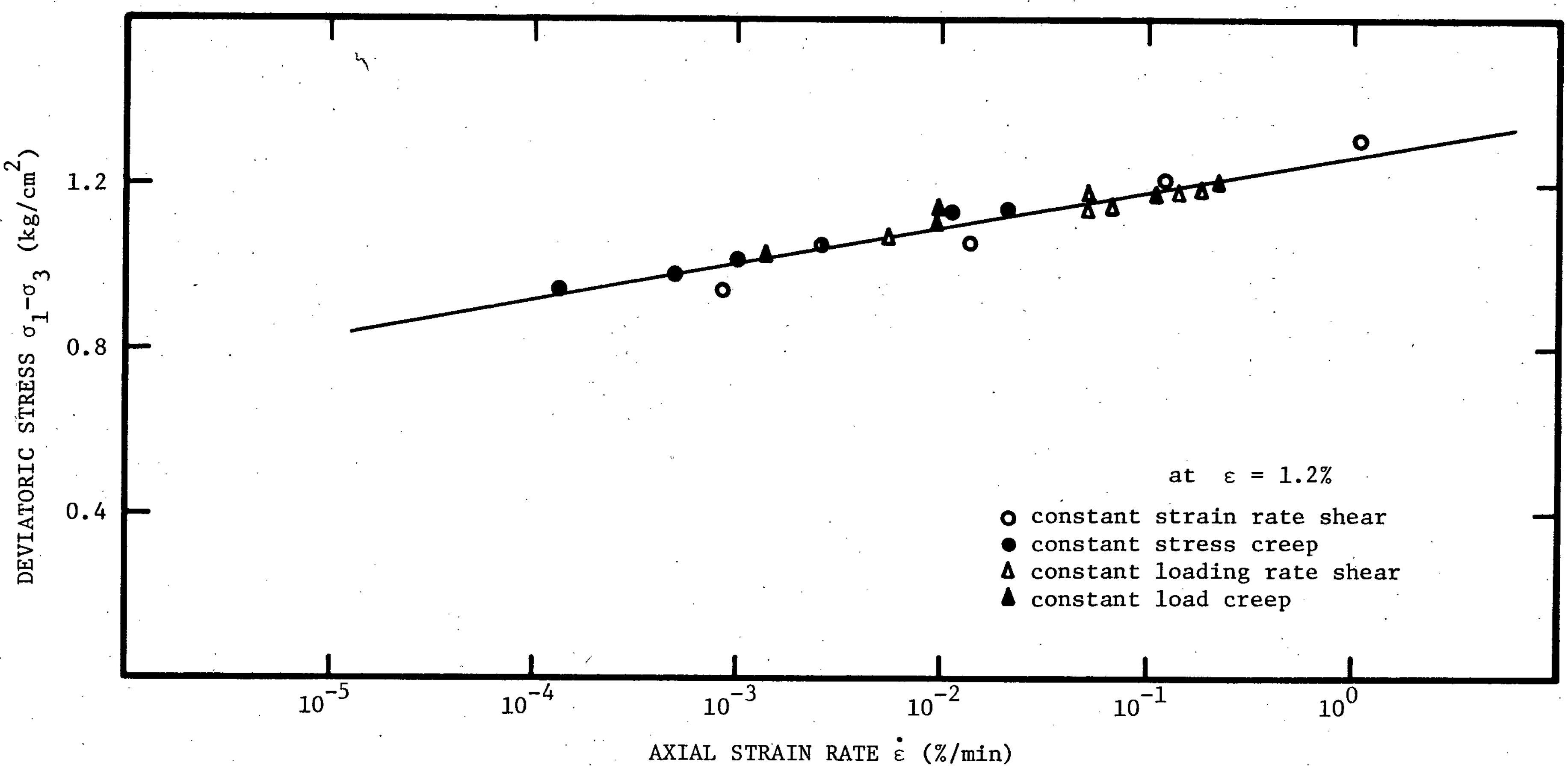

FIG. 19c. DEVIATOR STRESS VERSUS AXIAL STRAIN RATE AT EQUAL STRAIN LEVEL RELATIONSHIP IN CONSTANT RATE OF STRAIN SHEAR, CONSTANT STRESS CREEP, CONSTANT RATE OF LOADING SHEAR, AND CONSTANT LOAD CREEP.

summarized in Figure 20, and it may be seen that the essentially linear relationship obtained between deviator stress and logarithm of the axial strain rate, at each fixed level of axial strain, has approximately the same slope as the variation of undrained strength with log strain rate. A similar plot was presented by Berre and Bjerrum in a study of a plastic clay from Drammen (2). However, that study was restricted to triaxial compression tests using conventional constant rate of strain tests only. This diagram essentially shows that the rate of shear strain increases exponentially with an increase in the shear stress provided the change in strain is insignificant, and this independently of the time loading history used to shear the clay in undrained triaxial compression.

5.2. Comparison with the Behavior of Normally Consolidated Haney clay.

The stress-strain and strength behavior of Haney clay were previously investigated in the normally consolidated state by Vaid and Campanella (32). It would be interesting to compare the data obtained from that study with the present results on overconsolidated Haney clay. This is particularly important because of the sensitive nature of the clay used. Consolidation of a sensitive clay past the apparent preconsolidation pressure into the normally consolidated

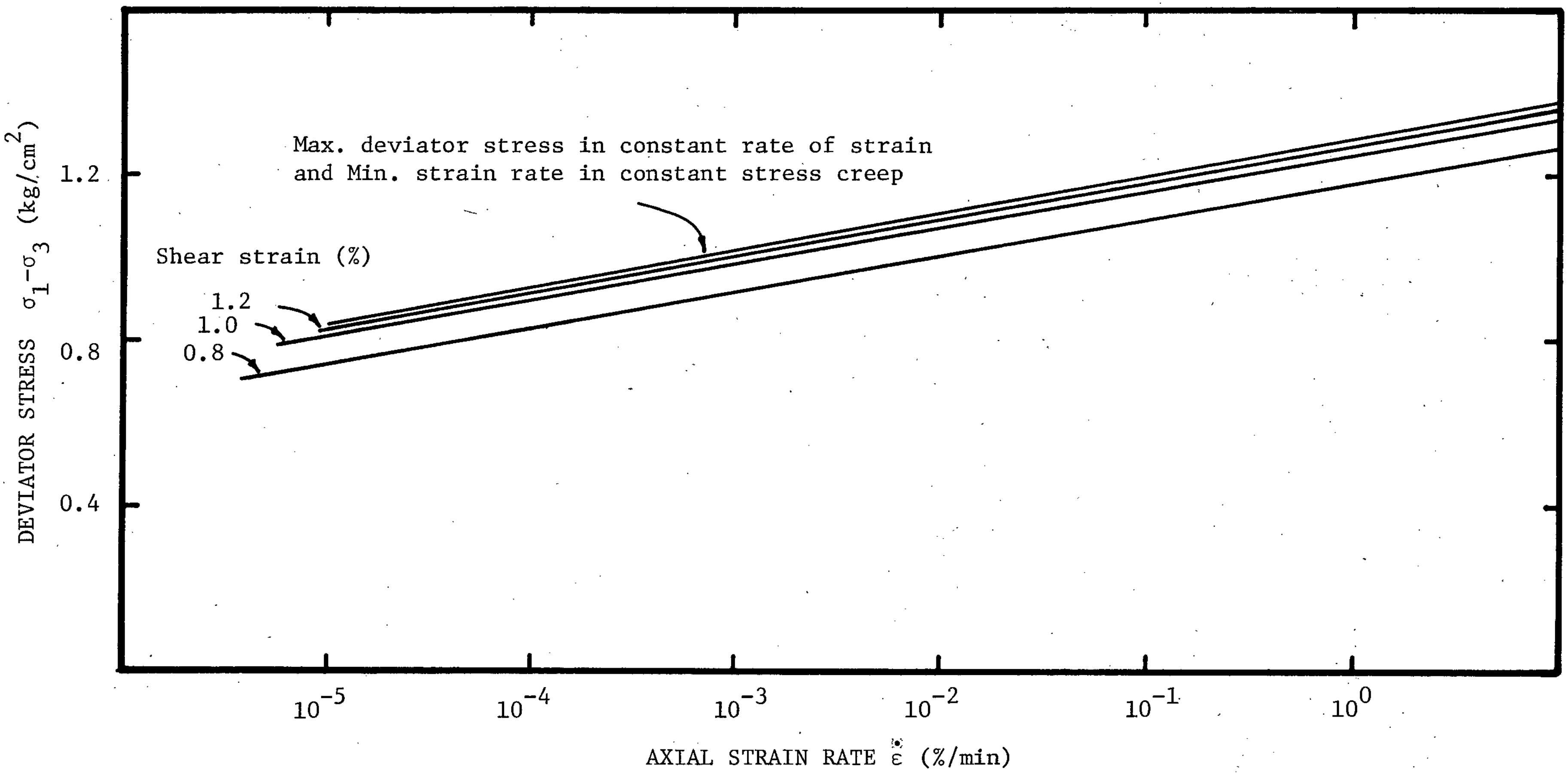

FIG. 20 STRESS-STRAIN-STRAIN RATE RELATIONSHIP FOR UNDRAINED TRIAXIAL COMPRESSION ON O.C. HANEY CLAY USING VARIOUS TIME LOADING HISTORIES.

region is associated with a radical change in its structure. Thus the behavior of the natural clay in its overconsolidated state may, because of the unaltered structure, reflect time dependent characteristics significantly different from those after it has been destructured upon normal consolidation in the laboratory.

It may be pointed out that such a comparison can only be qualitative because of the significant difference in the batches of the clay used in this study and that used in Vaid and Campanella study. The normally consolidated clay had a natural water content of about 40%, a liquid limit of 44%, a plastic limit of 26%, and a clay content (less than 0.002 mm) of approximately 50%. All these physical properties presented higher values for the clay tested in the present study, as shown previously on Table 1. The occurrence of uniform thin layers of organic materials in the present clay is a further indication that this clay was obtained from a different horizon than the one tested by Vaid and Campanella.

The stress-strain behavior of normally consolidated Haney clay in constant rate of strain shear did not exhibit the same sharp decrease in stress past the peak value as it was observed on the overconsolidated clay in Figure 4. The strain softening of the normally consolidated samples occurred more gradually past the maximum deviator stress. This difference in behavior is essentially due to the destructuration of the normally consolidated clay occurring prior to shear loading when the clay was subjected to an effective confining stress higher than the apparent preconsolidation pressure. However, some similarities were observed in the behavior of the clay in both consolidation states. The strain at failure was essentially independent of the rate of strain in both cases although the peak deviator stress was reached at an axial strain of about 2.5 to 3.0% in the normally consolidated state as compared to 1.2 to 1.5% for the overconsolidated clay.

The variation of undrained strength with logarithm of strain rate from constant rate of strain and constant stress creep is shown in Figure 21. While the overconsolidated clay presented a regular increase in undrained strength of about 8% per log cycle of strain rate, the normally consolidated clay exhibited no change in undrained strength at low strain rates (less than 5×10^{-3} %/min), then a more pronounced increase of about 9% per log cycle of strain rate for higher speed of deformation.

The comparison of the reduction of undrained strength with time from the constant stress creep tests is shown in Figure 22. It may be seen that the behavior of the clay has a similar trend in both consolidation states. However, larger reductions of strength were observed in the normally consolidated clay for short time failures (less than 100

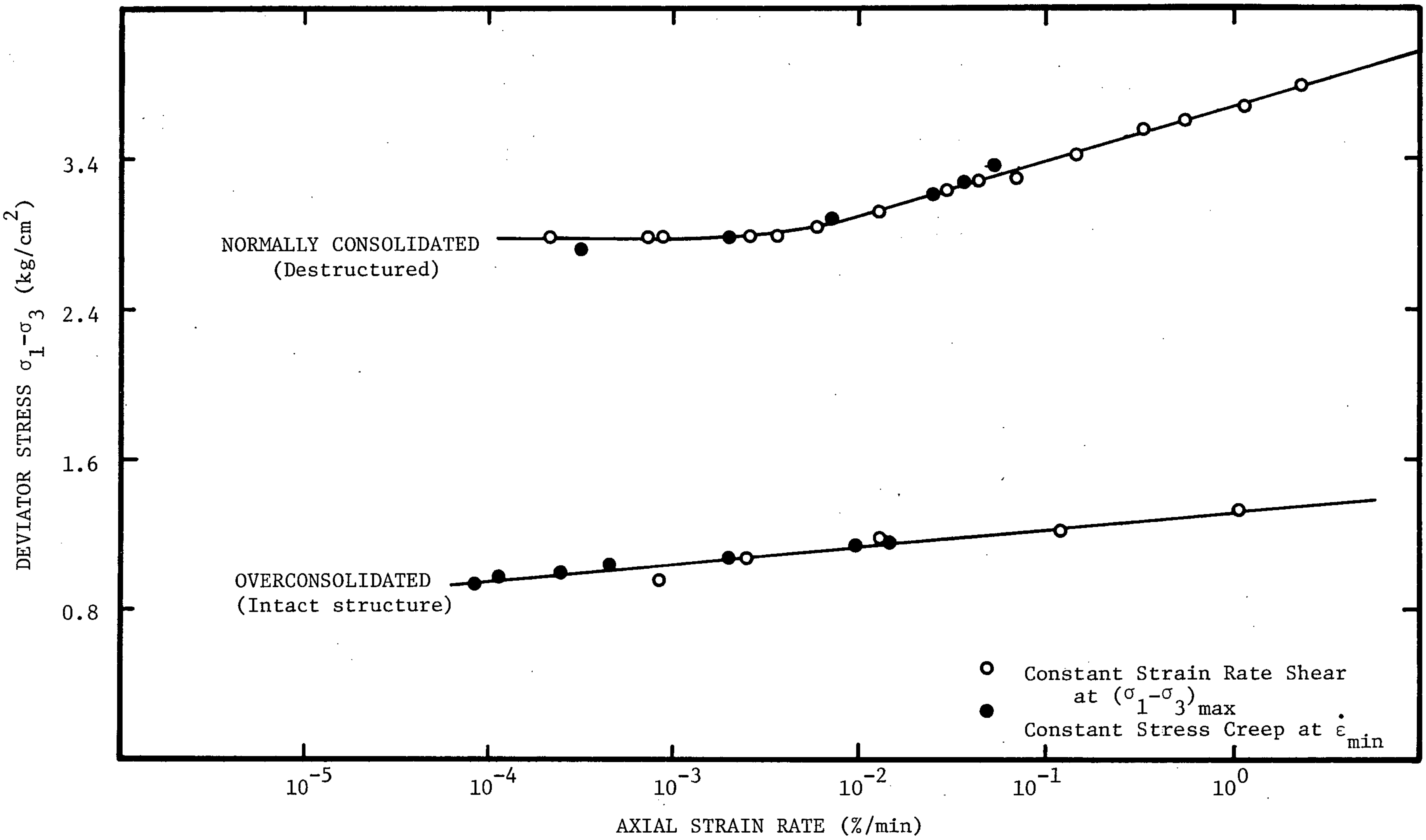

FIG. 21 VARIATION OF UNDRAINED STRENGTH WITH RATE OF STRAIN IN CONSTANT STRAIN RATE SHEAR AND CONSTANT STRESS CREEP FOR O.C. AND N.C. HANEY CLAY.

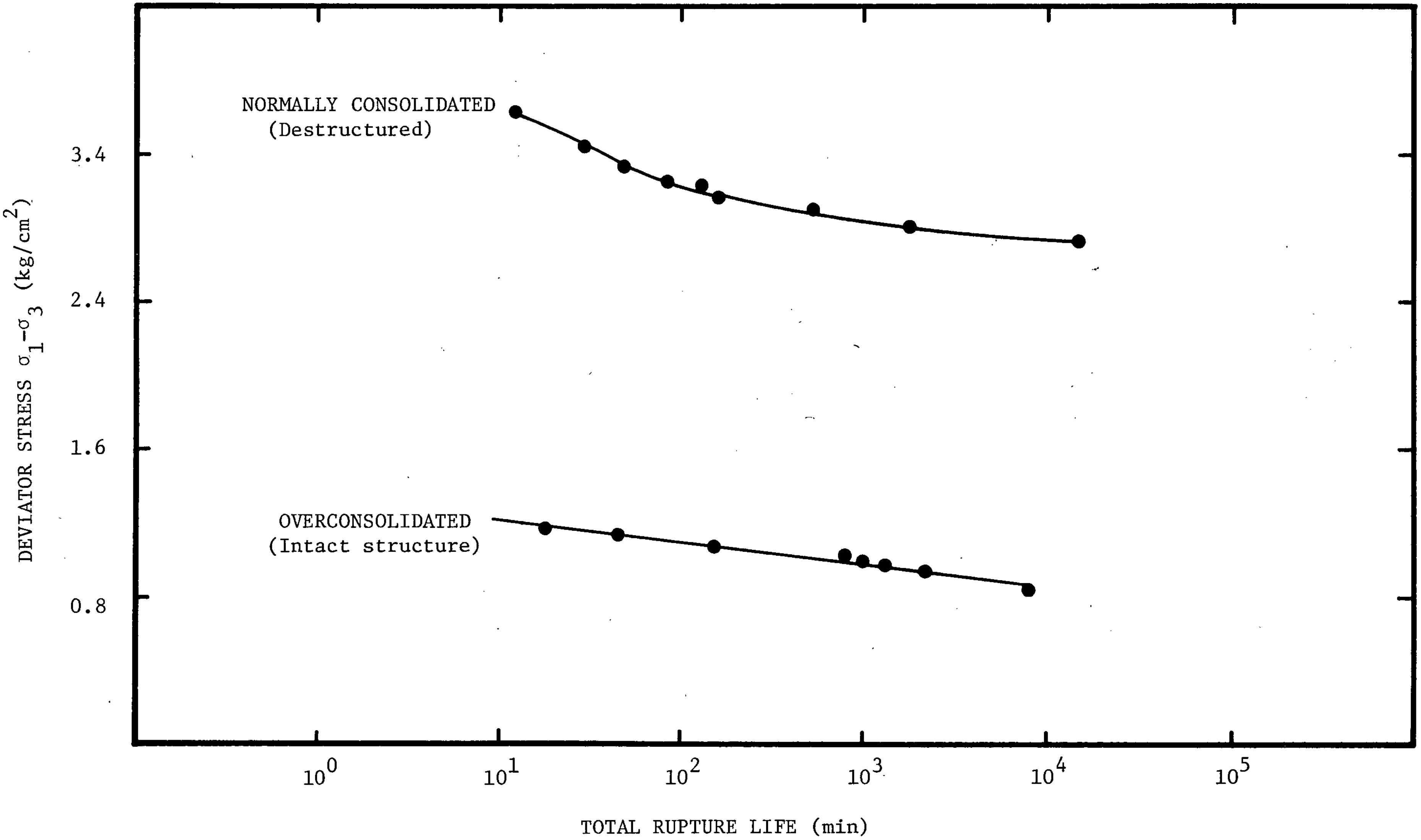

FIG. 22 TIME DEPENDENCE OF UNDRAINED STRENGTH OF O.C. AND N.C. HANEY CLAY IN CONSTANT STRESS CREEP.

minutes in total rupture life).

Since the validity of the equation of state has been proved for both clays, which means that the unique relationship between stress, strain, and strain rate is essentially independent of the time loading history used to shear the clay, the results of constant rate of loading shear and constant load creep tests for the two clays will certainly be as comparable as the constant rate of strain shear and the constant stress creep tests. Therefore, such a comparison will not be necessary.

From the comparison between a clay that has been destructured by normal consolidation in the laboratory and the same clay in its intact, structured and overconsolidated state with respect to the in-situ apparent preconsolidation pressure, it appears that the destructuration of the clay has not caused any significant change in the time dependent behavior of the material. A minor variation in the magnitude of the time effects on the undrained strength may be noted, but the characteristic time dependent behavior remains basically the same. Both clays showed essentially the same increase in undrained strength with increasing rate of strain or rate of loading, and the same reduction in undrained strength with time under the effect of sustained stress or load. In both clays, the current stress was uniquely related to the current strain rate at any given level of strain prior to failure, regardless of the time loading history used during shear.

5.3. Stress Conditions at Maximum Effective Stress Ratio

Figure 23 shows the stress conditions in the modified Mohr plot at maximum effective stress ratio for all tests on overconsolidated Haney clay. All the test data points are located in a narrow range between the failure envelope of the normally consolidated clay and the 45 degree line indicating the limit of the stress conditions in the triaxial tests ($\bar{\sigma}_3 = 0$). This diagram does not show any clear evidence of a time dependence of the results. It is however different from the results of the normally consolidated clay where the data points of all the various tests were lined up along a unique failure envelope.

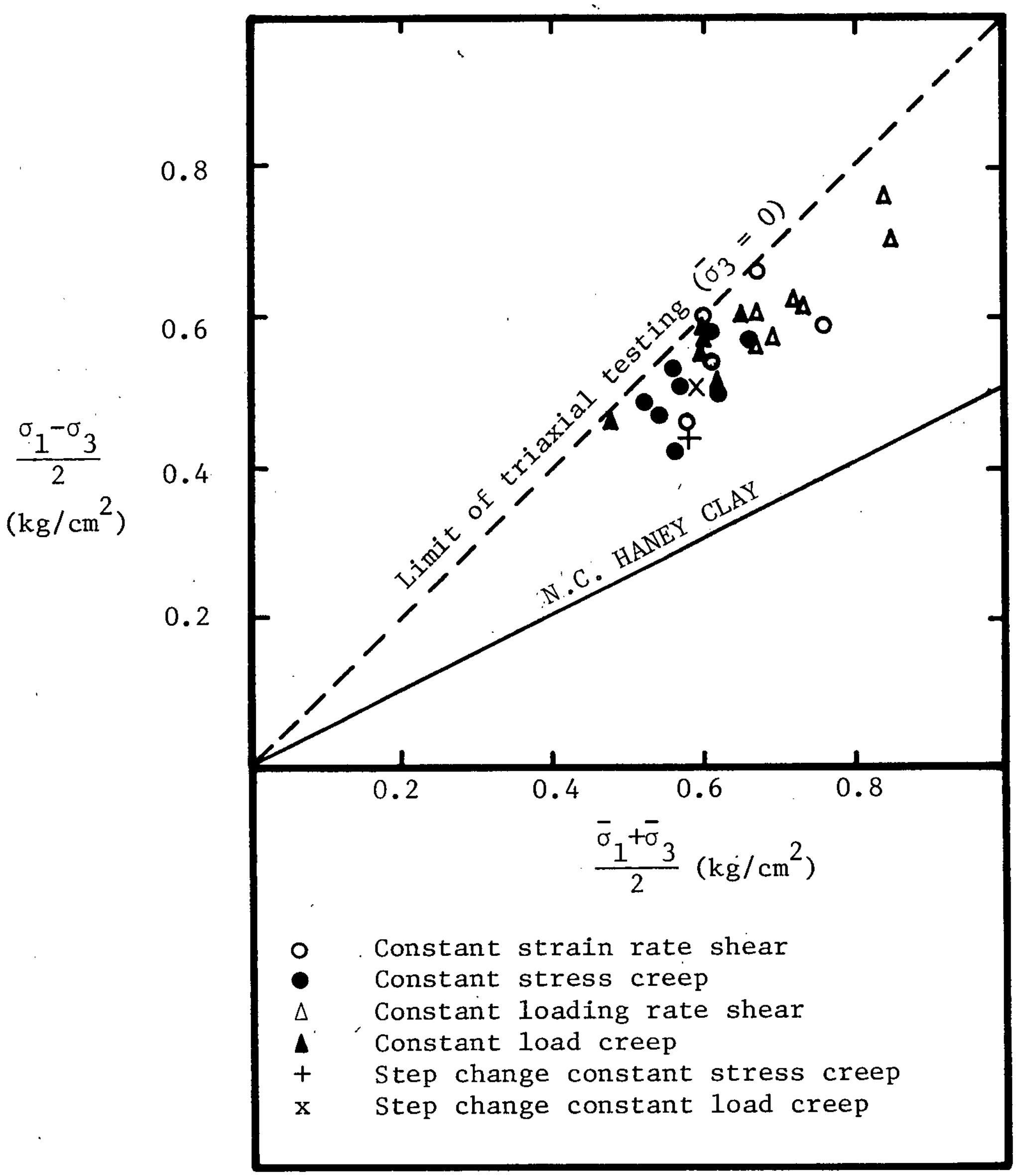

FIG. 23. STRESS CONDITIONS FOR ALL TESTS AT $(\bar{\sigma}_1/\bar{\sigma}_3)_{max}$ ON O.C. HANEY CLAY.

CHAPTER 6

CONCLUSION

The time dependence of undrained strength and deformation behavior of a saturated, sensitive, overconsolidated, natural clay have been examined in triaxial compression, for a given consolidation history, under a variety of time loading histories. The tests showed that increase in strain rate and rate of loading result in stiffer undrained stress-strain response and higher undrained strength for the clay investigated. A critical low level of strain appears to trigger rupture in all the tests performed in this study. A correlation among the various tests supported the validity of the equation of state in its strain hardening formulation, that is, the current strain rate is uniquely related to the current stress at any given level of strain prior to failure, regardless of the time loading history.

A qualitative comparison with the results of a similar study of the same sensitive clay, destructured upon laboratory normal consolidation, indicated that the destructuration of the clay has not caused any significant change in its time dependent behavior. The only differences observed and related to the natural structure of the overconsolidated clay were in the stress-strain response obtained from conventional constant strain rate tests, and in the stress conditions at maximum effective stress ratio of all tests plotted on a modified Mohr diagram. The overconsolidated clay exhibited a pronounced collapse of its structure at failure, and it did not show any well defined effective stress failure envelope at maximum effective stress ratio, regardless of the time loading history of shearing.

LIST OF REFERENCES

1. Alberro, J.A. and Santoyo, E.Y., "Long Term Behavior of Mexico City Clay", Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow 1973, Vol. 1.1, pp. 1-9.
2. Berre, T. and Bjerrum, L., "Shear Strength of Normally Consolidated Clays", Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, 1973, Vol. 1.1, pp. 39-49.
3. Bishop, A.W. and Henkel, D.J., "The Measurement of Soil Properties in the Triaxial Test", 2nd edition, Edward Arnold Ltd., London, England, 1962.
4. Bishop, A.W. and Lovenbury, H.T., "Creep Characteristics of Two Undisturbed Clays", Proceedings, 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969, Vol. 1, pp. 29-37.
5. Bjerrum, L., "Embankments on Soft Ground", Proceedings, ASCE Specialty Conference on Earth and Earth Supported Structures, 1972, Vol. 2, pp. 1-54.
6. Bjerrum, L., "Engineering Geology of Norwegian Normally Consolidated Marine Clays as Related to Settlements of Buildings", Seventh Rankine Lecture, Géotechnique, Vol. 17, 1967, pp. 81-118.

7. Bjerrum, L. and Lo, K.Y., "Effect of Aging on the Shear Strength Properties of a Normally Consolidated Clay", Géotechnique, Vol. 13, 1963, pp. 147-157.

8. Bjerrum, L., Simons, N. and Torblaa, I., "The Effect of Time on the Shear Strength of a Soft Marine Clay", Proceedings, Brussels Conference on Earth Pressure Problems, Vol. 1, pp. 148-158.

9. Campanella, R.G. and Vaid, Y.P., "Triaxial and Plane Strain Creep Rupture of an Undisturbed Clay", Canadian Geotechnical Journal, Vol. 2, 1974, pp. 1-10.

10. Casagrande, A. and Wilson, S., "Effect of Rate of Loading on Strength of Clays and Shales at Constant Water Content", Géotechnique, Vol. 2, No. 3, 1951, pp. 251-264.

11. Chan, C.K., "Low-Friction Seal System", Journal of the Geotechnical Engineering Division, ASCE, Vol. 101, No. GT9, 1975, pp. 991-995.

12. Coates, D.F., Burn, K.N., McRostie, G.C., "Strain-Time-Strength Relationships in a Marine Clay", Engineering Institute of Canada, Transactions, EJG-63, Geot. 11, October 1963.

13. Coulon, R.J., "Landslide on the Toulnustauc River, Quebec", Canadian Geotechnical Journal, Vol. 3, No. 3., 1966, pp. 113-144.

14. Crawford, C.B., "The Influence of Rate of Strain on Effective Stresses in Sensitive Clay", Papers on Soils, ASTM STP, No. 254, 1959, pp. 36-48.

15. Finn, W.D.L. and Snead, D.E., "Creep and Creep Rupture of an Undisturbed Sensitive Clay", Proceedings, 8th International Conference on Soil Mechanics and Foundation Engineering, Moscow, 1973, Vol. 3, pp. 135-142.

16. Finnie, J. and Heller, W.R., "Creep of Engineering Materials", McGraw-Hill Book Company, Inc., New York, 1959.

17. Goldstein, M. and Ter-Stepanian, G., "The Long-term Strength of Clays', Proceedings, 4th International Conference on Soil Mechanics and Foundation Engineering, London, 1957, Vol. 2, pp. 311-314.

18. Kraus, H., "Creep Analysis", John Wiley and Sons, Inc., New York, 1980.

19. Leonards, G.A., "Strain Rate Behaviour of Saint-Jean-Vianney Clay: Discussion", Canadian Geotechnical Journal, Vol. 17, 1980, pp. 461-462.

20. Mesri, G., Febres-Cordero, E., Shields, D.R. and Castro, A., "Shear Stress-Strain-Time Behaviour of Clays", Géotechnique, Vol. 31, No. 4, 1981, pp. 537-552.

21. Murayama, S. and Shibata, T., "Rheological Properties of Clays", Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, France, 1964, Vol. 1, pp. 269-273.

22. Pao, Y.H. and Marin, J., "Prediction of Creep Curves from Stress Strain Data", Proceedings ASTM, Vol. 52, 1952, pp. 51-57.

23. Perloff, W.H. and Osterberg, J.O., "The Effect of Strain Rate on the Undrained Shear Strength of Cohesive Soils", Proceedings, 2nd Pan American Conference on Soil Mechanics and Foundation Engineering, Vol. 2, pp. 103-128.

24. Richardson, A.M. and Whitman, R.V., "Effect of Strain Rate Upon Undrained Shear Resistance of a Saturated Remoulded Fat Clay", Géotechnique, Vol. 13, No. 4, 1963, pp. 310-324.

25. Saito, M. and Uezawa, H., "Forecasting the Time of Occurrence of a Slope Failure", Proceedings, 5th International Conference on Soil Mechanics and Foundation Engineering, Paris, France 1961, Vol. 1, pp. 315-318.

26. Scott, R. and Hon-Yim Ko, "Stress-Deformation and Strength Characteristics of Soils", State-of-the-Art Volume, 7th International Conference on Soil Mechanics and Foundation Engineering, Mexico, 1969, pp. 28-38.

27. Simons, N.F., "The Effect of Overconsolidation on the Shear Strength Characteristics of an Undisturbed Oslo Clay", ASCE Research Conference on Shear Strength of Cohesive Soils, Boulder, Colorado, June 1960, pp. 747-763.

28. Singh, A. and Mitchell, J.K., "General Stress-Strain-Time Function for Soils", ASCE, Journal of Soil Mechanics and Foundations Division, Vol. 94, SM1, 1968, pp. 21-46.

29. Šuklje, L., "Rheological Aspects of Soil Mechanics", John Wiley and Sons Ltd., London, England, 1969.

30. Taylor, D.W., "Fundamentals of Soil Mechanics", John Wiley and Sons Inc., New York, N.Y., 1948.

31. Tavenas, F., Leroueil, S. LaRochelle, P., Roy, M., "Creep Behavior of an Undisturbed Lightly Overconsolidated Clay", Canadian Geotechnical Journal, Vol. 15, 1978, pp. 402-423.

32. Vaid, Y.P. and Campanella, R.G., "Time Dependent Behaviour of Undisturbed Clay", ASCE, Journal of the Geotechnical Engineering Division, Vol. 103, GT7, 1977, pp. 693-709.

33. Vaid, Y.P., Robertson, P.K., Campanella, R.G., "Strain Rate Behaviour of Saint-Jean-Vianney Clay", Canadian Geotechnical Journal, Vol. 16, No. 1, 1979, pp. 34-42.

34. Symposium on Stress-Strain-Time-Temperature Relationships in Materials. ASTM Special Technical Publication No. 325, New York, N.Y., 1962.