CREEP RUPTURE OF SATURATED UNDISTURBED CLAYS

by

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ABSTRACT

The stress/strain relationship for most engineering materials is known to be time dependent. This is most evident during a creep test in which continual deformations are observed under constant stress conditions. In the laboratory, a specimen of cohesive soil subjected to a constant shear stress may fail after having deformed at relatively slow rates for a considerable time. This type of failure, termed creep rupture, is also known to occur in the field.

Results of drained and undrained triaxial creep rupture tests are presented in this thesis. These tests were performed on a sensitive marine clay from western Canada which was consolidated to various stress histories. Pore pressure measurements were taken during undrained tests using an electrical transducer. In addition to the creep rupture tests, incremental load and constant strain rate triaxial tests were performed for comparative purposes.

The strain rate during a creep rupture test was observed to initially decrease as the specimen strained, reach a transient minimum strain rate, and then increase until rupture. Failure was found to be inevitable whenever

the strain rate started to increase after having reached a minimum value. Pore pressures measured during the undrained tests did not reflect the onset of creep rupture at the transient minimum strain rate, and therefore, the onset of creep rupture cannot be explained in terms of effective stresses.

A relationship was found to exist between the deviator stress, strain and current strain rate during undrained triaxial tests having the same consolidation history. This relationship permitted the prediction of the results of constant strain rate tests based on the results of creep rupture tests. This resulted in an understanding of the interrelation between the transient minimum strain rate of a creep rupture test and the maximum deviator stress of a constant strain rate test.

Once the transient minimum strain rate had been reached, the results of creep rupture tests showed that the strain rate was inversely proportional to the time remaining before rupture. This relationship is independent of stress level, consolidation history and drainage conditions. As a result, it is suggested that measurement of deformations in the field can be used to predict the time until a sudden failure would be anticipated.

The upper yield strength, defined as the maximum compressive stress which will not cause a creep rupture failure, was evaluated from both creep rupture and constant strain rate tests. It was found that the compressive strength increased as a linear function of the cube root of the strain rate.

TABLE OF CONTENTS

Page

LIST OF TABLES

LIST OF FIGURES

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CHAPTER 1

INTRODUCTION

1.1 - Introduction

At present, 1970, Limit Design is used for most soil mechanics problems which involve the strength and stability of soil masses. This method compares the total available shear strength with the average strength required to maintain equilibrium, thereby providing a factor of safety against failure. The total available shear strength is based upon the soil's strength parameters--friction and cohesion--the slope and intercept, respectively, of the Mohr-Coulomb strength envelope. Although providing a means for design against total collapse, Limit Design does not estimate the magnitude of anticipated deformations.

In Soil Mechanics and Foundation Engineering, design is often governed by deformation criteria, rather than by ultimate strength and, therefore, it is necessary to predict deformations in addition to ultimate strength. Until the recent development of the finite element method of analysis, it has been necessary, for most practical cases, to estimate deformations based upon experience and semiempirical equations, since more exact computational methods were not available.

The finite element representation of the continuum has allowed evaluation of deformations based upon a multi-linear elastic representation of the stress/strain relation for the soil (Finn 1967, Clough and Woodward 1967). For some soils, this stress/strain relation is time dependent, an additional variable which, presently, has not been incorporated directly into the finite element method of analysis. However, with further development of finite element techniques, it should be possible to include this variable and predict deformations based upon stress/strain/time relationships. These analyses will require a more comprehensive understanding of the effect of time on stress/strain relations than presently exists and, therefore, further research is required in this field of study.

The influence of time on stress/strain relations can best be demonstrated with a creep test. In this test, the sample continues to strain, although the stress remains constant. This situation has been observed under field conditions by many, including Haefeli (1953a), Goldstein and Ter-Stepanian (1957), Saito and Uezawa (1961), and Suklje (1961). They report continual movements taking place in natural slopes in which the total stresses are remaining nearly constant. In certain cases, it has been reported that, after many years of this continual movement, deformation rates increased rapidly, leading to a catastrophic failure (Suklje 1961).

A loss of strength with time has also been observed in laboratory studies by many, including Bishop (1966), Casagrande and Wilson (1951) and Saito and Uezawa (1961). Therefore, a better understanding of the effect of time on strength parameters should be helpful for both the prevention and the prediction of long-term creep failures.

1.2 - Purpose and Scope

This thesis, based upon laboratory results, is an attempt to further clarify the effect of time on the strength properties of undisturbed saturated clays. The laboratory investigation, carried out on a locally-available sensitive clay, involved constant stress triaxial tests, with pore pressure measurements, at stress levels high enough to cause creep rupture. Constant strain rate and incremental stress triaxial tests were included in the investigation for comparative purposes. This thesis investigates the phenomenological behaviour of saturated clays in terms of both total and effective stresses, and a major portion of the thesis is devoted to investigating the occurrence and prediction of creep rupture.

CHAPTER 2

LITERATURE REVIEW

This literature review presents pertinent information in soil mechanics literature on the effect of time on both stress/strain relationships and strength. It also includes a brief mention of rheological studies carried out on metals and plastics and is presented in the following subsections:

2.1 - Field Observations of Creep

Haefeli (1953a) described many practical examples of situations where creep has been of importance in soils, snow and ice. In particular, Haefeli (1953b) described the effect of a creeping slope on an abutment of a railway bridge crossing the Landquart River in Switzerland. To stop translation of this abutment, a large horizontal beam was placed across the river between abutments. This beam was instrumented and the pressures exerted on the abutment due to creep were measured. Haefeli reported that between 1944 and 1950 the pressure tended to increase, although some seasonal fluctuations were noted.

Long-term failures in London clay slopes were investigated by Henkel (1957). At Northolt, one of the sites investigated, Henkel reported that substantial movements and a series of tension cracks along the top of the slope were observed at least six months before failure occurred.

Suklje (1961) reported a long-term creep landslide which occurred along the Gradot Ridge in Macedonia in 1956. Tension cracks up to 90 feet deep were observed 30 years before the failure. A year before the failure, the lower side of a crack was noted to have subsided 20 centimeters. Further subsidence was observed a few weeks before failure.

It is clear from these examples that failures do not always take place without previous warning. Therefore, a better understanding of the mechanism and time-dependent behaviour of long-term failures may result in methods for predicting impending failure.

2.2 - Studies of Time-dependent Behaviour of Metals and Plastics

The stress/strain response of most engineering materials is affected by variations in temperature and duration of test. Engineers have, therefore, carried out considerable research in an attempt to explain and predict this behaviour. Two basic approaches have evolved for these investigations. The first, usually referred to as the macroanalytical or phenomenological approach, describes the actual phenomena in terms of measurable physical parameters, such as stress, strain, time and temperature. The second, referred to as the micromechanistic approach, describes the phenomena in terms of the behaviour of the atomic or molecular structure of the material. Research carried out according to this approach includes the application of the rate process theory to plastics, rubber and asphalt (Ree and Eyring 1958, Herrin and Jones 1963). Results of these investigations have resulted in qualitative, but not quantitative, predictions of the response to applied shear stress. Therefore, although increasing emphasis is being placed upon this approach at present, it still appears to require considerable development before being useful for quantitative prediction of macroscopic behaviour.

On the other hand, a quantitative prediction can usually be made using a phenomenological approach. The disadvantage of this approach is that the prediction can normally be applied only to a limited number of situations, similar to that from which the relation was obtained, as the basis for the prediction may not be fundamental to the material's behaviour. In this thesis, the analysis of time-dependent behaviour is mainly restricted to a phenomenological investigation and, therefore, this literature review is likewise restricted.

A typical strain/time curve for a creep test is shown in Figure 1. The shape of this curve is similar for many engineering materials, such as metals, concrete and plastics. This curve has been subdivided by many researchers into four separate stages; these four stages are:

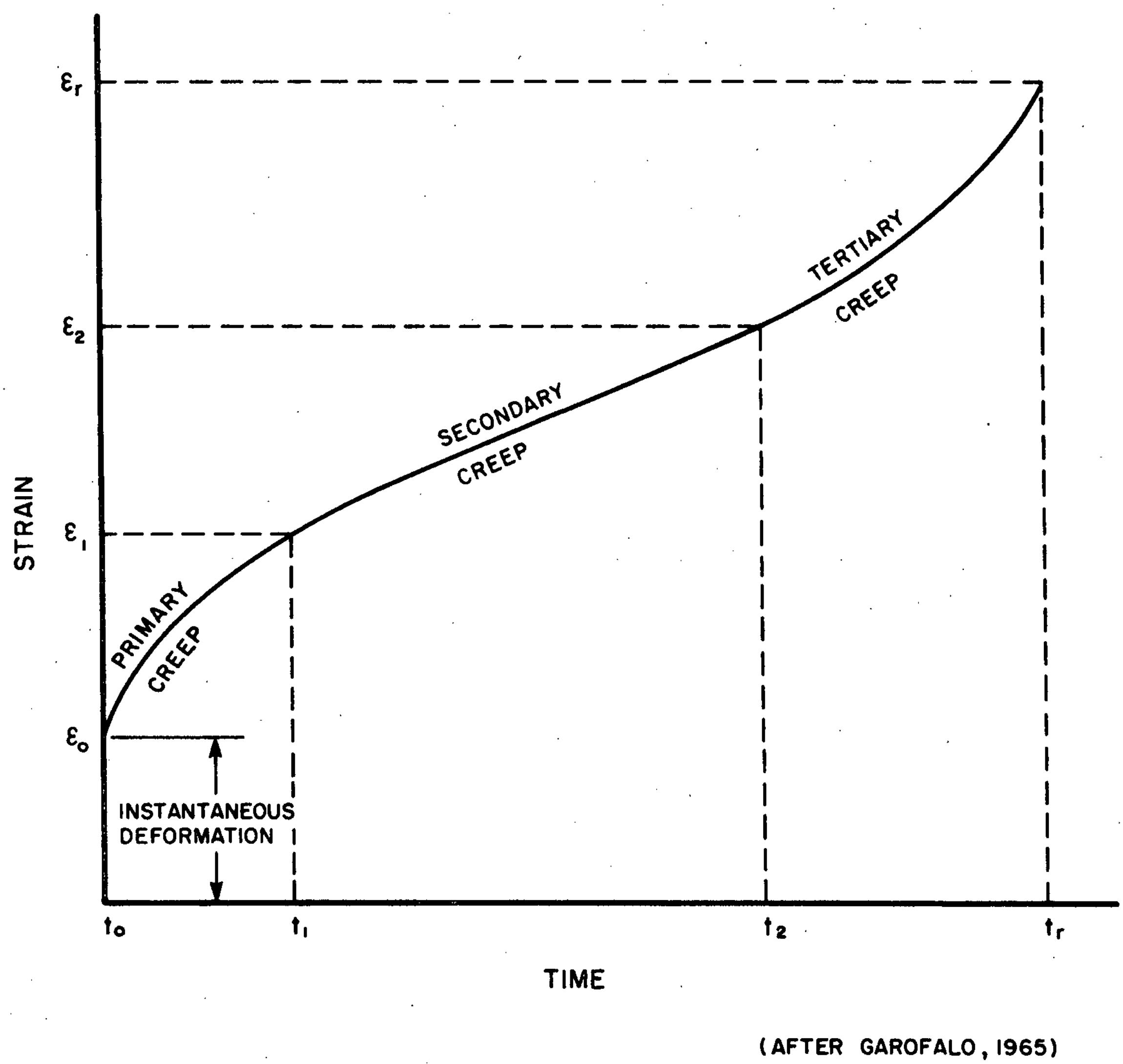


FIGURE 1 – TYPICAL CREEP RUPTURE CURVE

Stage 1 Instantaneous deformation (ε_0)

Stage 2 Primary creep (ε_0 to ε_1)

Stage 3 Secondary creep (ε_1 to ε_2)

Stage 4 Tertiary creep (ε_2 to ε_r)

The instantaneous deformation is often referred to as the initial elastic deformation, but in fact usually comprises both elastic and plastic deformations. Primary creep is defined as the stage during which the strain rate is continually decreasing, while secondary creep is defined as the stage during which the strain rate is nearly constant. Tertiary creep is the final stage resulting in creep rupture and, therefore, the strain rate increases during this stage. Most engineering materials exhibit both Stages 1 and 2, but secondary and tertiary creep may or may not be evident, depending upon the material and the applied stresses.

Zener and Holloman (1946) proposed that a unique relation exists between stress, strain, strain rate and temperature for metals subjected to tensile loading, which was independent of the strain rate and temperature history of the metal; thus, the tensile stress could be expressed as:

$$S = f(\varepsilon, \dot{\varepsilon}, T) \tag{1}$$

where: S = current tensile stress

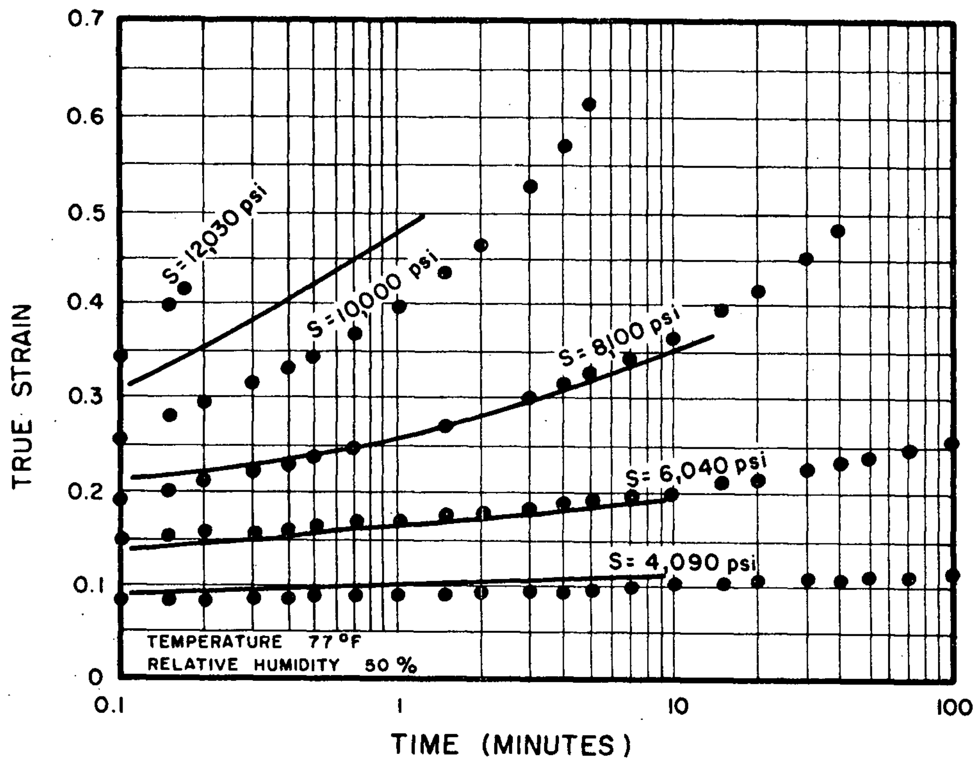
ε = current strain

$\dot{\varepsilon}$ = current strain rate

T = current temperature

Lubahn and Felgar (1961) stated that, upon investigation of available research on metals, overwhelming evidence is found to show that the temperature of prior straining does affect current behaviour. Lubahn and Felgar do suggest, however, that a relation exists between stress, strain and strain rate for metals at constant temperature, which is independent of the strain rate history.

Pao and Marin (1952) used this concept to predict the tensile creep behaviour of Plexiglas based upon the results of a series of constant strain rate tests. Figure 2 shows the predicted and actual tensile creep behaviour obtained by Pao and Marin. It is interesting to note that two of the tests shown in Figure 2 were at stress levels large enough to result in creep rupture. (This is not readily apparent, since the elapsed time is plotted on a



(AFTER PAO & MARIN, 1952)

LEGEND

——— PREDICTED FROM RESULTS OF CONSTANT STRAIN RATE TESTS

● ● ● TENSILE CREEP DATA

FIGURE 2 - PREDICTED AND ACTUAL TENSILE CREEP OF PLEXIGLAS

logarithmic scale.) Pao and Marin obtained predicted creep curves by graphical and numerical methods, rather than by evaluating a mathematical relationship between stress, strain and strain rate. The authors state that the exact form of an equation relating stress, strain and strain rate is relatively unimportant compared with the basic idea that such a general relationship exists.

This short section on time-dependent studies on metals and plastics has been included to show that similar results are obtained for most engineering materials, be they metal, plastic, rubber or, as will be shown, saturated clay. The mechanism of creep must be considered as a very complex phenomenon related to intermolecular forces and the "making and breaking" of bonds. When considering molecular behaviour saturated clay is basically the same as other materials, although having the complicating aspect of being a two-phase material. It is therefore possible that the results of research on metals and plastics may, in fact, provide guidelines for investigation of the time-dependent behaviour of saturated clay.

2.3 - Creep and Creep Rupture

Casagrande and Wilson (1951) investigated the effect of creep loadings on the shear strength of nine clays. Most of the tests reported were unconfined compression tests in which the load was built up quickly and maintained constant until the specimen failed. The application of load was achieved by applying four to eight increments of load at approximately one-minute intervals. For Mexico City clay, the time to failure was found to increase with decreased load, while the logarithm of the time to failure was found to be linearly related to the applied stress. Casagrande and Wilson defined failure as occurring when the deformation rate started to increase, since they often observed a failure plane developing shortly after the inflection point in the strain/time curve.

Goldstein and Ter-Stepanian (1957) confirmed a decrease in strength with increased duration of loading. They defined the "long-term" strength as that stress required to cause a failure after a "long" period of time. Based upon a proposed model for soil comprising viscous and brittle bonds, Goldstein suggested that creep rupture occurred at a specific value of strain, independent of time of loading and stress level. Subsequently, results by Sherif (1965) indicated that creep rupture strain is not independent of stress level for all soils.

Investigations by Vialov and Skibitsky (1957) on creep in frozen soils showed deformations initially occurring at a decreasing rate, followed by a stage of creep at a constant deformation rate and finally resulting in the tertiary failure stage. These observations were based upon the results of tests which involved shear along rods, initially frozen in soil. Vialov and Skibitsky suggested the existence of an ultimate continuous strength or upper yield strength based upon the result of a test which did not fail while under load for over four years. Direct shear tests performed on a dense tertiary clay also indicated the existence of an upper yield strength. Vialov and Skibitsky also noted that the horizontal deformation during a direct shear test could be satisfactorily described by the equation:

$$\gamma = a + b \log_{10} t \qquad (2)$$

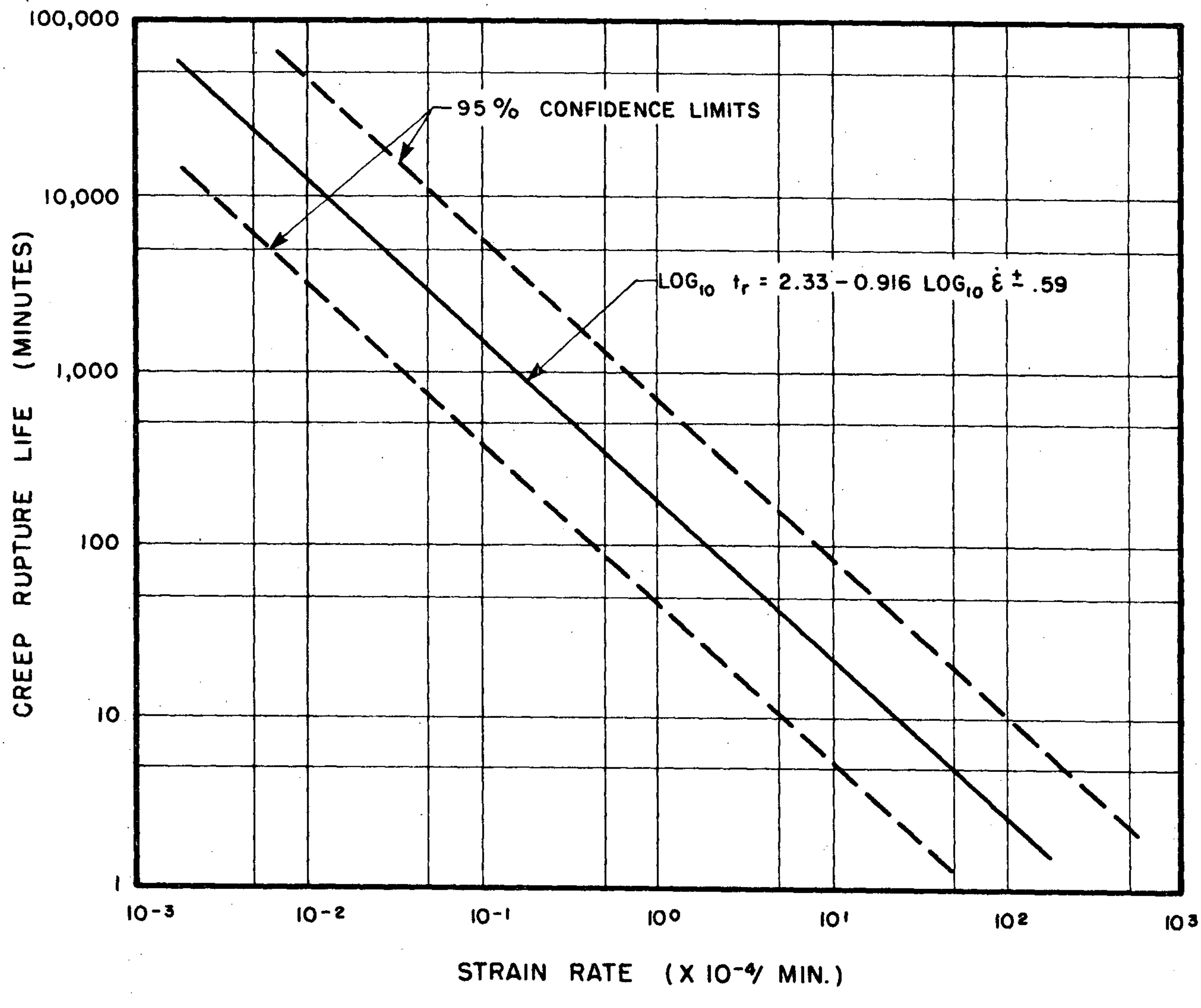
where: γ = horizontal deformation
t = elapsed time
a, b = constants

This relationship is not true for all soils, as the results of Campanella (1965) indicate a definite curvature to the strain/logarithm time curve for triaxial compression of remoulded illite.

Coates, Burn and McRostie (1963) presented creep rupture data for a very sensitive clay from Eastern Canada, which indicated that an increase in applied shear stress resulted in decreased time to failure. The authors questioned whether the time to failure should be defined as the total time to rupture, or the time until the strain rate started to increase. Most researchers use the first definition, although, as already mentioned, Casagrande and Wilson (1951) used the latter. Coates, Burn and McRostie suggested that constant strain rate tests may provide equivalent information to the constant stress test, but did not suggest any method for correlation of the results. As discussed, Pao and Marin (1952) successfully related results from these two types of tests on Plexiglas. Therefore, it appears logical to attempt such a correlation in this study, the results of which are discussed in Chapter 6.

A relatively complete summary of undrained creep and strength behaviour for cohesive soils is to be found in Mitchell, Seed and Paduana (1965). In this report, the results of many investigations have been subdivided into two groups--those in which a strength loss was noted and those in which either no change or a strength gain was observed. Strength gain was observed only for compacted samples with a high percentage of bentonite, known to be highly thixotropic. Strength loss was observed for most saturated clays independent of whether disturbed, undisturbed, consolidated or compacted.

Based upon the results of a series of unconfined creep tests, Saito and Uezawa (1961) proposed a linear relationship between the logarithm of the secondary strain rate and the logarithm of the total elapsed time to rupture (creep rupture life), as shown in Figure 3. Data used for obtaining this relationship included the results of creep tests on four natural Japanese soils and the results of Murayama and Shibata (1956), Casagrande and Wilson (1951) and Goldstein and Ter-Stepanian (1957). Shown in Figure 3



(AFTER SAITO & UEZAWA, 1961)

FIGURE 3 - RELATIONSHIP BETWEEN CREEP RUPTURE LIFE AND SECONDARY STRAIN RATE

is the error band which encloses 95 per cent of all the tests (95 per cent confidence limits). The equation of the linear relationship shown in Figure 3 is of the form:

$$\log t_r = C - m \log \dot{\varepsilon}_s \tag{3}$$

where: t_r = total elapsed time until creep rupture (creep rupture life)

$\dot{\varepsilon}_s$ = secondary strain rate

m, C = constants

When t_r is expressed in minutes and $\dot{\varepsilon}_s$ is expressed in 10^{-4} per minute, the equation, as evaluated by Saito and Uezawa, becomes:

$$\log t_r = 2.33 - .916 \log \dot{\varepsilon}_s \tag{4}$$

According to Saito and Uezawa, this equation is valid for different types of clay, consolidation history and drainage conditions.

If the constant m is assumed to be equal to one, then Equation 4 can be written as:

$$t_r \, \dot{\varepsilon}_s = \text{constant} \tag{5}$$

This gives a very simple relationship which states that the creep rupture life is inversely proportional to the secondary strain rate.

A similar correlation of creep rupture life with strain rate was presented by Monkman and Grant (1956) for various metals. The difference between the Saito and Uezawa and the Monkman and Grant relationships centres around the definition of the governing strain rate. Whereas Saito and Uezawa used the secondary or constant strain rate, Monkman and Grant suggested that a constant strain rate does not exist and used the minimum strain rate which occurred between primary and tertiary creep. Therefore, Monkman and Grant proposed a linear relation between the logarithm of the transient minimum strain rate and the logarithm of creep rupture life. For creep rupture tests, the value of the governing strain rate obtained, assuming the existence of a constant strain rate or a transient minimum strain rate, is nearly the same and, therefore, the results of Monkman and Grant are comparable with those of Saito and Uezawa. It is interesting to note that the Monkman and Grant 95 per cent confidence limits for any one metal are $\pm.55$ logarithm cycles, as compared with the Saito and Uezawa limits of $\pm.59$ logarithm cycles for all soils. This is, perhaps, surprising since metals are usually considered to be more homogeneous than most natural soils. However, as most soils are more plastic than metals, non-homogeneous areas within a sample will cause lower stress concentrations in a soil than in a metal. It is possible that these factors tend to balance one another, giving the same order of magnitude of error band for both metals and soils for the creep rupture criterion presented.

Having obtained a laboratory hypothesis for predicting creep rupture, Saito and Uezawa proceeded to apply it to field conditions. Saito (1965) presented additional case histories to confirm the application of the hypothesis to field conditions. These included the investigation of both natural slopes and full-scale experiments in which failure was induced by sprinkling water on the slope. The method used to predict the time of failure, as proposed by Saito, requires the continual measurement of displacement across a tension crack at the top of the slope. By dividing the rate of displacement by the total length of the slope, a strain rate is obtained. Using this strain rate, Equation 4 is solved for t_r (the creep rupture life), which is then claimed to represent the remaining time until failure of the slope. Although this method appears to be satisfactory on the basis of the reported field studies, it can be criticized on several grounds, the most severe being that the laboratory t_r was defined as the creep rupture life (time from initial loading to rupture) and this has been used to predict the remaining time until rupture after the measurement of the field strain rate.

2.4 - Use of the Rate Process Theory to Predict Creep Behaviour

The rate process theory (Glasstone, Laidler and Eyring 1941) postulates that a statistical rate of bond rupture occurs which is dependent upon the thermal and activation energy of the bonds. As long as no shear force is applied to a bond, the probability of rupture is equal in all directions. However, application of a shear stress to a cohesive soil applies a net directional component to the bond ruptures and therefore a deformation is observed.

The rate process theory has been applied to the creep deformation of clays by Murayama and Shibata (1961, 1964), Mitchell (1964), Christensen and Wu (1964), Campanella (1965) and Mitchell, Campanella and Singh (1968). Murayama and Shibata (1961) used the rate process theory to mathematically describe the nonlinear viscosity of the dash-pot in their rheological model. The equation for their model, which was considered to be applicable for stress levels below the upper yield strength, follows:

$$\varepsilon = \frac{\sigma}{E_1} + \frac{\sigma - \sigma_0}{E_2} + \frac{\sigma - \sigma_0}{B_2 E_2} \log At \tag{6}$$

where: ε = strain

σ = deviator stress

σ_0 = lower yield strength of soil (for stress below the lower yield strength there is no time-dependent deformation)

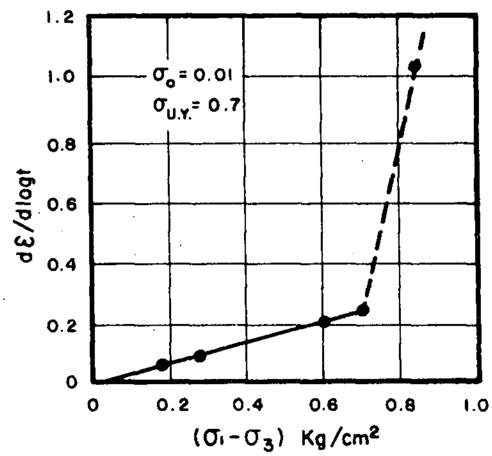
E_1, E_2 = elastic constants for spring elements in rheological model

A, B_2 = constants

t = time

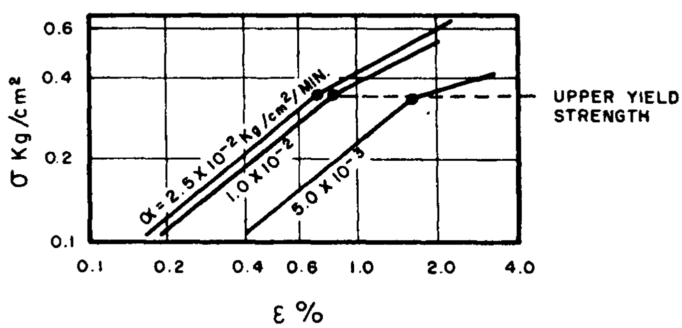
This equation predicts a linear relation between $\frac{d\varepsilon}{d \log t}$ and σ. Murayama and Shibata (1961) proposed that the first inflection point of the $\frac{d\varepsilon}{d \log t} - \sigma$ curve occurs at the upper yield strength as shown in Figure 4. Murayama and Shibata (1964) suggested another method for the evaluation of the upper yield strength of clays. Instead of requiring a series of creep tests, this method uses the results of only one test to predict the upper yield strength. The test used for this purpose is a stress-controlled compression test in which the compressive stress is applied in equal increments at uniform time intervals. Results of such tests by Murayama and Shibata show a linear relation between the logarithm of deviator stress and the logarithm of strain. The stress at which the first inflection point occurs is proposed to be equal to the upper yield strength as shown in Figure 5. No justification, based upon the rate process theory or rheological model, was presented by Murayama and Shibata; but, the upper yield strength obtained by this method was coincident with that obtained using the results of a series of creep tests on the same clay.

Christensen and Wu (1964) used the same approach as Murayama and Shibata and applied the rate process theory to describe the nonlinear viscosity of the dashpot in their



(AFTER MURAYAMA & SHIBATA, 1961)

FIGURE 4 - RELATIONSHIP BETWEEN $d\varepsilon/d\log t$ AND σ OBTAINED FROM CREEP TESTS



(AFTER MURAYAMA & SHIBATA, 1964)

FIGURE 5 - INFLUENCE OF RATE OF STRESS APPLICATION ON UPPER YIELD STRENGTH OBTAINED FROM INCREMENTAL LOADING TESTS

rheological model. Christensen and Wu assumed that each individual contact has a lower yield strength (the stress below which flow does not occur) which is not equal for each contact. Therefore, since very weak contacts can exist, Christensen and Wu state that the macrobehaviour of the soil may not exhibit a lower yield strength, which is contrary to the hypothesis of Murayama and Shibata (1961).

Mitchell (1964), Campanella (1965), Mitchell and Campanella (1963), Mitchell, Campanella and Singh (1968) have used the rate process theory to develop an equation which predicts the instantaneous strain rate during a creep test. This relationship is a function of the stress, temperature, soil structure and energy of activation of the bonds at the soil contacts. Tests devised by Campanella (1965) permit the evaluation of parameters in this equation by applying rapid changes in temperature and stress level during a creep test. The main purpose of their work has been to prove the validity of the rate process theory as applied to the shear behaviour of soils, rather than using it to describe the nonlinear behaviour of a rheological model. At present, insufficient information is available to make practical use of their equation.

Based upon a large amount of experimental data, Singh and Mitchell (1968) have proposed the following phenomenological equation for creep of cohesive soils:

$$\dot{\varepsilon} = A\,(t)^{m} \exp\,(\alpha\,\sigma_D) \qquad (7)$$

where: $\dot{\varepsilon}$ = strain rate
α, A, m = constants
t = elapsed time
σ_D = applied deviator stress

This equation is valid only for subfailure conditions and predicts a linear relation between $\log \dot{\varepsilon}$ and $\log t$ for undrained creep tests. Therefore, continually-decreasing strain rates are predicted by this equation and a constant or secondary stage of creep would not be anticipated.

2.5 - Effect of Time on Strength Tests

Before presenting the effect of time on the shear strength of clays, it is necessary to briefly discuss the strength parameters which are used for effective stress and total stress analyses. The shear strength of soils is considered to primarily result from frictional forces at the contacts between soil particles. Thus, the shear strength of soils is normally considered to be a function of the effective normal stresses, rather than the total normal stresses. Conventional soil mechanics defines the shear strength parameters--cohesion (c') and friction (ϕ')--as the intercept and slope, respectively, of the effective stress Mohr-Coulomb failure envelope. Two failure criteria are commonly used in conjunction with triaxial testing; these being the maximum deviator stress $(\sigma_D)_{MAX.}$ and the maximum principal effective stress ratio $(\sigma_1'/\sigma_3')_{MAX.}$ failure criteria. For drained tests, these criteria are coincident, but for undrained tests these criteria often result in different values for the shear strength parameters, c' and ϕ', depending upon the type of soil, consolidation history and duration of the test.

The use of the principle of effective stress involves two steps; namely, the determination of the shear strength parameters-- c' and ϕ' --and the prediction of the pore pressures. To avoid having to predict pore pressures, shear strength is sometimes expressed in terms of total stresses. The most common application of this approach is the $\phi = 0$ analysis. This analysis is based upon the results of unconsolidated undrained triaxial tests on undisturbed saturated clays. Since the effective stresses at the start of the test are independent of the confining pressure for most clays, the maximum deviator stress is not dependent upon the initial confining pressure. The undrained shear strength is, therefore, equal to one-half of the maximum deviator stress, although the strength often quoted is the compressive strength or maximum deviator stress. The research reviewed in Section 2.3 is based on compressive strength and, in fact, all available research on creep rupture of clays is based upon total stress measurements.

Time effects on compressive strength were reported by Taylor (1948). Taylor presented results for the compressive strength of remoulded Boston blue clay, which showed an increase in strength with an increase in the speed of axial compression. Taylor attributed the difference in strength to a viscous or plastic resistance which varied with the speed at which the shear strain occurred. Casagrande and Wilson (1951) reported increased compressive strength with increased rates of loading on Mexico City clay. As the rate of loading was increased, the rupture strain was noted to decrease. These tests were performed by adding predetermined increments of stress at set intervals of time. On the other hand, Casagrande and Wilson also reported that, for a compacted clayey sand, the compressive strength increased with decreased rates of loading, accompanied with an increase in rupture strain. However, this soil was only partially saturated and, therefore, this increase in strength with time may be due to consolidation effects.

Perloff and Osterberg (1963) investigated the effect of various strain rates on remoulded illite. Undrained triaxial tests were performed for various consolidation histories, ranging between normally consolidated and an overconsolidation ratio of 20. For remoulded illite, Perloff and Osterberg reported that the greater the overconsolidation ratio the greater the effect of strain rate on the maximum deviator stress. The effect of strain rate on the maximum deviator stress was the same as previously reported by others. For normally consolidated illite, the variation in the maximum deviator stress was small for the range of strain rates tested. This may indicate the importance of structure on the effect of strain rate on the maximum deviator stress, since for normally consolidated remoulded illite, which would have a dispersed structure, the effect of strain rate is small. Whereas for the overconsolidated illite, the effect of strain rate is larger, possibly due to the structural bonds created by overconsolidation.

Bjerrum, Simons and Torblaa (1958) reported decreased compressive strength with increasing time to failure for a soft marine clay. The testing program consisted of a series of normally consolidated undrained triaxial tests at widely different strain rates. It was found that ϕ' , the effective angle of friction, also decreased with increasing time to failure, with failure defined at the maximum deviator stress. The validity of this result is somewhat questionable, since Bjerrum, et al, admitted the possibility of leakage of water through the membrane into the sample.

Results of normally consolidated drained tests at various strain rates resulted in no change in shear strength with variations in time to failure. It was suggested that any decrease in strength due to increased time to failure was compensated for by an increase in strength due to an additional volume decrease from secondary compression.

Crawford (1959) investigated the influence of strain rate on the effective stresses in a sensitive clay from Eastern Canada. The maximum deviator stress of strain rate controlled undrained triaxial tests was found to decrease linearly with the logarithm of the time to failure. The effective angle of friction, ϕ', at maximum deviator stress, increased with time to failure. This is the exact opposite of the results of Bjerrum, et al.

The results of Bjerrum, et al, and Crawford indicate that the effective stress strength parameters obtained from undrained triaxial tests are affected by the testing rate if failure is defined as occurring at the maximum deviator stress. Unfortunately, the triaxial tests in both investigations were discontinued before the maximum principal effective stress ratio was reached and, therefore, the effect of time on the shear strength parameters measured at the maximum principal effective stress ratio is unknown.

As mentioned previously, research on creep rupture of clays has been investigated in terms of total stresses. Therefore, in an attempt to extend the basic understanding of undrained creep, pore pressures were measured throughout all tests in the research to be described.

2.6 - Summary

Creep failures have been observed in the field. A better understanding of the mechanism and time-dependent behaviour of soil deformation might enable prediction of these failures.

Similar stress/strain/time behaviour has been observed for many engineering materials. Therefore, it may be possible to successfully apply theories proposed for metals and plastics to the creep of saturated clays.

There is considerable agreement within the literature that a decreased compressive strength is associated with increased time to failure for both constant stress and constant strain rate tests on saturated cohesive soils.

At present, no theory exists which permits the prediction of a creep test from the results of constant strain rate tests for shear of saturated clays.

Undrained creep rupture of clay has only been investigated in terms of total stresses.

CHAPTER 3

LABORATORY TESTING

3.1 - Description of Clay

The clay used in this study was obtained from an open pit which belonged to the Haney Brick and Tile Co. Ltd., Haney, B.C. This pit is located about 20 miles east of Vancouver on the north bank of the Fraser River. Block samples were obtained in August, 1965, by digging a pit around an undisturbed mass of clay about three feet square. Then, by carefully cutting the clay with piano wire, blocks of clay, approximately one cubic foot in size, were removed. These blocks were waxed in the field and transported to the University of British Columbia Soils Laboratory where additional coatings of wax were applied. The blocks were stored in a moist room until required. Although the testing program extended over a two-year period, no noticeable change in water content was observed and, therefore, the method of storage is assumed to have been adequate.

This clay, which will be referred to as Haney clay, has been used for research by others at U.B.C. (Byrne 1966, Hirst 1966, Gupta 1967, Lou 1967). Haney clay, according to

Armstrong (1957), is a glacio-marine deposit laid down in front of and beneath the ice of the Sumas Valley glaciation which occurred about 11,000 years ago. Due to rebound of the area with the retreat of the ice, the area is now above sea level and, as a result of leaching over the ensuing years, Haney clay now has a sensitive structure. Armstrong defines it as a plastic marine silty clay, with clay 46 per cent, silt 42.5 per cent and fine sand 11.5 per cent. The clay obtained from the open pit, being slightly less sandy, had clay 46 per cent, silt 51 per cent and fine sand 3 per cent, based on the M.I.T. grain size classification. Table I gives typical physical properties of Haney clay.

Standard one-dimensional consolidation tests were run to obtain the preconsolidation pressure. A value of approximately 38 psi for the preconsolidation pressure was obtained, which was in agreement with the results of Byrne (1966) and Hirst (1966). In the analysis of the shape of stress paths for drained tests, initially consolidated to 40 psi (Byrne 1966), a considerable effect apparently due to preconsolidation could still be observed. Since it was desired to eliminate the effect of preconsolidation, an

TABLE I

Physical Properties of
Haney Clay

Specific gravity	2.80
Liquid limit	50 per cent
Plastic limit	27 per cent
Plasticity index	23 per cent
Natural water content	41.5 per cent ±1 per cent
Per cent finer than two microns	46 per cent
Activity	0.5
Undisturbed unconfined compressive strength	1,650 psf
Remoulded unconfined compressive strength	330 psf
Sensitivity*	5

$$\text{*Sensitivity} = \frac{\text{Undisturbed unconfined compressive strength}}{\text{Remoulded unconfined compressive strength at 20 per cent axial strain}}$$

isotropic consolidation test was performed in triaxial equipment to investigate whether the type of consolidation affected the apparent preconsolidation pressure. The isotropic consolidation test estimated the preconsolidation pressure at 50 psi, about 30 per cent higher than that obtained from one-dimensional consolidation. To ensure elimination of the effect of preconsolidation in the testing program, it was decided to isotropically consolidate all samples to an effective stress of 75 psi.

3.2 - Development of Testing Program

The development of a testing program to investigate creep rupture requires an understanding of the effects of consolidation history and drainage conditions on the compressive strength of clay.

It is well known that, when normally consolidated clays are sheared with full drainage, there is a decrease in volume; while, in the undrained condition, the tendency for volume decrease is reflected by a pore water pressure increase. Thus, for a normally consolidated clay, a lower compressive strength is obtained from the undrained test, assuming the same total stress path to failure.

Throughout the remainder of this section, whenever the relative magnitudes of compressive strength obtained from drained and undrained triaxial tests are compared, it must be understood that these comparisons apply only to drained and undrained tests which have been subjected to the same total stress path and the same initial consolidation history.

Heavily overconsolidated clays, upon shearing in the drained condition, absorb water and an increase in volume results. If sheared in an undrained condition, the pore water pressure decreases and the effective stresses increase. Therefore, for a heavily overconsolidated clay, a lower compressive strength is obtained from the drained test. For most clays, with an overconsolidation ratio of between four and six, there is little tendency for volume change during shear and, therefore, drained and undrained compressive strengths are nearly the same (Roscoe, Schofield and Wroth 1958).

Therefore, depending upon the consolidation history of a clay, a lower compressive strength may be measured from either a drained or undrained test. As a result, should creep studies be performed for both drained and undrained conditions? Certainly, for the consideration of long-term behaviour, the assumption that a soil is completely drained does not seem unreasonable, but can undrained creep occur under field conditions? It is suggested that the slow movement of some extensive hillsides is the result of shearing of thick layers of clay which have such low permeabilities and long drainage paths that drainage is effectively zero, even over a considerable length of time. This condition would be equivalent to the undrained creep of clay and, therefore, undrained creep is considered to have practical significance.

If drainage is allowed during creep of a normally consolidated clay, the resulting increase in compressive strength due to the decrease in volume will reduce the possibility of a failure due to loss of strength with time. Therefore, the undrained condition is the controlling drainage condition under which a creep rupture failure will occur. A normally consolidated clay subjected to a creep loading will not fail in a drained or partially-drained condition, since it would have already failed in an undrained condition at an earlier stage of the test. Again, it must be remembered that this applies only if the total stress paths are the same.

For lightly overconsolidated clay (overconsolidation ratios of between four and six), the small difference in compressive strength between drained and undrained conditions allows investigation of creep rupture in either the drained or undrained state.

Although a heavily overconsolidated clay subjected to a creep loading may fail due to a decrease in undrained strength with time, additional softening with drainage (which may require considerable time) will certainly reduce the strength to a minimum. Therefore, to fully investigate the effect of time on the compressive strength of a heavily overconsolidated clay, drainage must be permitted. This should not be interpreted, however, as suggesting that a creep failure would take place in a completely drained condition, but rather that the softening with time from additional water contributes to the initiation of the failure. As failure is approached, deformation rates increase and, at failure, negative pore pressures will not be dissipated.

A change of temperature during an undrained triaxial test on saturated clay is known to affect the pore pressure. This change in pore water pressure, in turn, directly affects the effective stresses and thus the strength of a sample. Therefore, to eliminate random temperature effects, control should be maintained over the temperature while testing. Normally, this entails reducing temperature fluctuations to a practical minimum, but in certain cases controlled changes in temperature have been used in testing programs. Campanella (1965) used a temperature change during creep tests to evaluate parameters in the rate process equation. Under field conditions, temperature changes at depth are normally small and seasonal in fluctuation and, therefore, all tests in this program were performed at constant temperature. To do this, an enclosure was built around all the necessary testing equipment and a small air conditioner was used to maintain the temperature at 20 degrees C, ± 0.5 degrees C.

The testing program for this thesis was planned to include undrained tests with the following consolidation histories--normally consolidated and overconsolidated with overconsolidation ratios of two and six. To investigate the behaviour of heavily overconsolidated clay, drained and undrained tests with an overconsolidation ratio of 25 were performed. All samples were initially normally consolidated to an effective stress of 75 psi and rebounded to the desired final effective stress. The overconsolidation ratio, η, as used in this thesis, is the ratio of the normally consolidated effective stress (75 psi) to the overconsolidated or "rebounded" effective stress.

During all creep tests, small additional loads were continuously placed on the sample to maintain the deviator stress at a constant value. Large samples were used in order to get reasonable consistency between tests on natural undisturbed Haney clay. Thus, instead of using the more conventional 1.4-inch by 2.8-inch triaxial sample, 2.5-inch by 5.0-inch samples were trimmed from the block samples. As soon as the sample size is increased, consolidation times are considerably increased also. To reduce consolidation times by convenient means, double drainage and slit continuous filter paper (Whatman No. 54) side drains were used. This procedure reduced the theoretical time for 100 per cent consolidation to about 45 minutes.

Pore water pressures were measured at the base stone of the sample during all undrained tests using an electrical pressure transducer (manufactured by Data Sensors Inc.). When measuring pore water pressures, care must be taken to ensure that the pressure indicated by the measuring system is the pore pressure within the sample. Two errors which exist in the measurement of pore water pressures in the triaxial test are:

(1) - The time-lag in response of the measuring apparatus due to its compliance; and

(2) - The nonuniform pore water pressure within the sample due to nonuniform stress conditions.

The response time of the measuring apparatus is a function of the compliance of the measuring system, the permeability of the sample and the area over which the pore water pressure is measured. Based on a relationship for piezometers (Penman 1960), the time required to measure 95 per cent of an instantaneous pressure change within the sample would be under 30 seconds for the measuring system used in this testing program. Consideration was given to the use of hyperdermic needle pore pressure probes to measure the pressures at various levels within the sample. This idea was discarded when it was realized that it would take about 20 minutes to register 95 per cent of a pressure change. This difference in response times is due only to the difference in area over which the pressure is being measured; the larger the area, the quicker the water can be withdrawn from the sample to accommodate the compliance of the measuring system without greatly affecting the pore pressure in the sample. Even though a measuring system is very "stiff," the response time of the apparatus should be investigated when testing soils with low permeabilities.

Nonuniformity of stress within the triaxial specimen due to end restraints results in nonuniform pore water pressures throughout the sample (Blight 1963). Testing at a slow rate will ensure time for migration of pore water to balance the nonuniform pressures. This results in small changes to void ratio within the sample, although the average void ratio for the sample does not change. According to Blight, for Haney clay with a coefficient of consolidation, c_v, equal to 7×10^{-4} cm^2/sec, the test duration for 95 per cent equalization of pore pressure should be about one hour for 2.5-inch by 5.0-inch samples with side drains. Theoretically, this would be the length of time before any measured pore pressure could be expected to be 95 per cent accurate and, therefore, would give an indication of the length of time after creep loading before pore pressure measurements can be trusted.

The nonuniformity of stresses can be reduced by the use of frictionless end platens (Rowe and Barden 1964); but, frictionless end platens reduce the area over which base pore pressure measurements can be made. Therefore, although non-uniform pore pressures are reduced, the response time of the pore pressure measuring system is increased. This situation can be improved if a method is devised to connect the side drains to the pore pressure measuring apparatus without affecting the frictionless end platens. It should be realized that, for clay with low permeability, it is not always possible to satisfy both criteria for quick response of pore pressure measurements.

In order to estimate the compressive stress which, when applied to consolidated samples, would eventually cause a creep rupture failure, it was necessary to perform a standard strength test. Therefore, for each consolidation history a standard strength test was performed using the same testing apparatus as used in the creep studies. For the purpose of this thesis, the standard strength test was defined as an incremental loading test in which the rate of load application (i.e., one kilogram every three minutes) was maintained until failure occurred. In these tests, the rate of loading was chosen such that failure would take place after approximately six hours of loading.

During the incremental loading test, readings were taken at the end of each loading step. These loading steps were intensionally kept small, never exceeding about 4 per cent of the maximum deviator stress. However, near failure, these increments were further reduced and the frequency of loadings increased to maintain a constant rate of loading throughout the complete test. Strain rates during the test were calculated by the same method used for the creep tests (see Appendix B). It should be noted that these strain rates may be slightly in error since this method requires three strain and time readings, and during the incremental loadings tests, the deviator stress was not constant for these three readings. However, as already mentioned, the variation in deviator stress between adjacent readings was small.

In the latter stage of the testing program, constant strain rate triaxial tests were performed on samples with the same consolidation histories as those of the creep series. The results from these tests were used to correlate data between constant strain rate tests and creep tests. To ensure that correlation was possible, both series of tests were performed on samples obtained from the same group of block samples. Due to a shortage of clay, it was necessary to resort to performing some constant strain rate tests on 1.4-inch by 2.8-inch samples. This led to certain difficulties in correlating data due to differences in side drain and membrane effects.

Details of test equipment and test procedures are presented in Appendix A and an explanation of the parameters calculated from the tests, together with a few typical computer outputs of reduced data, is presented in Appendix B.

CHAPTER 4

RESULTS OF CREEP RUPTURE TESTS

4.1 - Undrained Creep Rupture Tests on Normally Consolidated Samples

All samples in this series were allowed to consolidate for 36 hours, at which time the average water content was 32.2 per cent with a standard deviation of the average equal to ± 0.2 per cent. At the end of the tests, water contents were found to be, on an average, 0.4 per cent above that determined by the sample side trimmings at the start of the test and volume change measurements during the test. For the 2.5-inch by 5.0-inch samples, this amounted to about 2.1 grams of excess water. It was believed that this excess water came from the filter paper side drains. To check this, a filter paper side drain was weighed dripping wet, as when installed on the sample, and again after being removed from a sample at the end of the test. The filter paper was noticeably drier upon removal, due to small pore water tensions drawing water from the side drains. In fact, the difference in weight was approximately 2.5 grams, which more than accounted for the excess of water already mentioned.

This confirmed the source of the 0.4 per cent excess water and, therefore, leakage of water through the membranes or around the O-ring seals was negligible.

After consolidation was completed, samples were left undrained for eight hours. During this time, a pore pressure build-up was observed which had an average value of ± 2.6 psi during the 8-hour period. Figure 6 shows the typical shape of the pore pressure build-up during the 8-hour period. At first, it might be thought that this pore pressure increase was due to leakage, but, if so, the pore pressure increase would likely be approximately linear with time. As discussed, water contents before and after tests checked quite satisfactorily. To explain this pore pressure increase, consider the sample just before the drainage was closed off. According to classical Terzaghi consolidation theory, primary consolidation had been completed for at least 34 hours, but secondary compression was still continuing. Thus, the pore pressure rise is considered to be a reflection of the tendency for continued secondary compression after the drainage lines were closed. This was investigated by a test which was consolidated for 24 hours instead of 36 hours. Since the

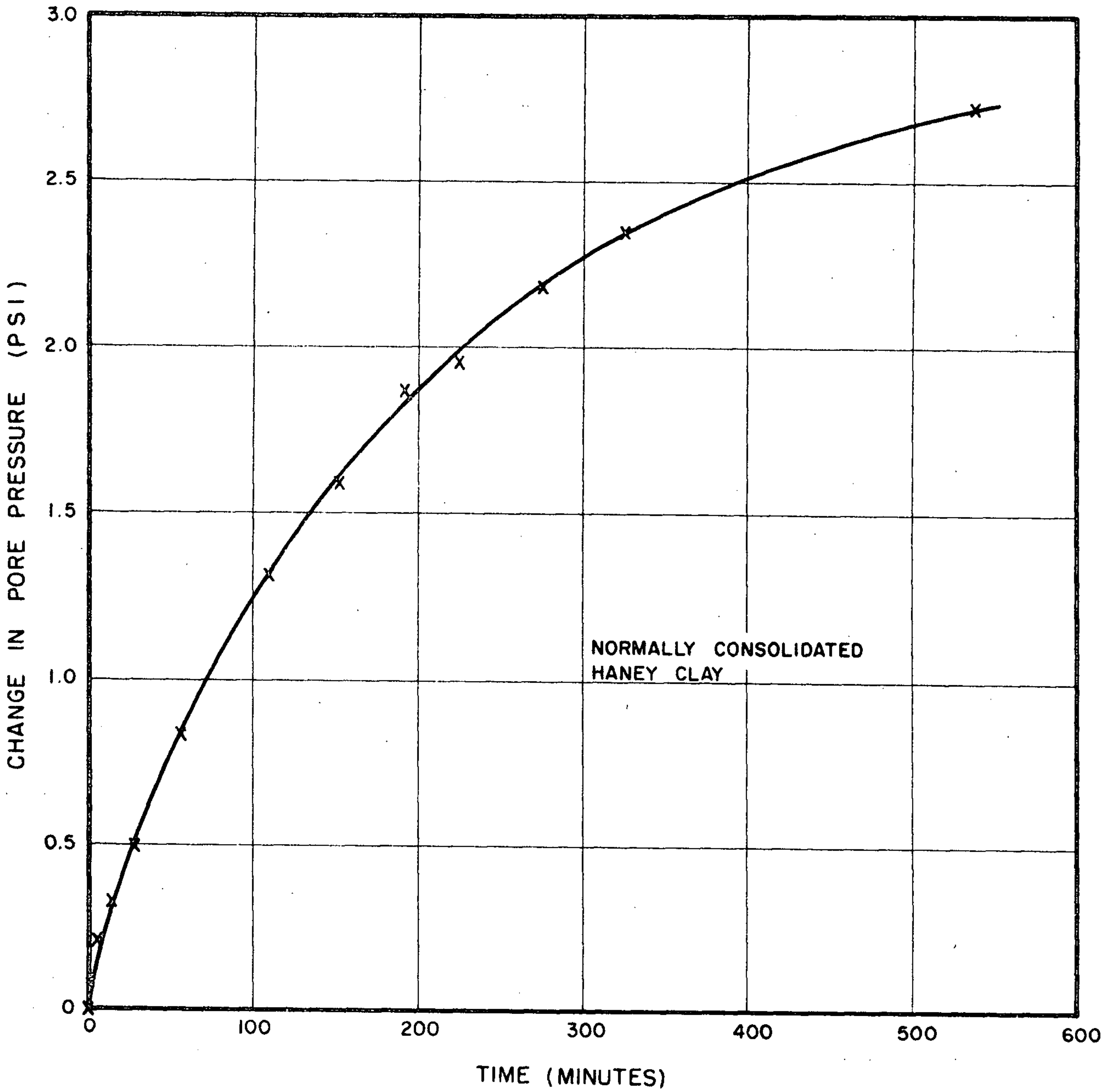


FIGURE 6 - TYPICAL PORE PRESSURE BUILD-UP AFTER 36 HOURS OF CONSOLIDATION

secondary compression remaining to take place after 24-hour consolidation is greater than that remaining after 36-hour consolidation, it would be expected that a larger pore pressure rise during an 8-hour period would occur. This was confirmed by the test consolidated for 24 hours, which had a pore pressure rise of 8.1 psi instead of the 2.6 psi mentioned earlier for 36-hour consolidation. These results are similar to those observed by Lou (1967) and discussed by Byrne (1966).

Once the magnitude of the pore pressure rise to be expected had been determined by a few tests, this became an extremely valuable way of determining whether there was any leakage. If the pore pressure rose six or seven psi during the 8-hour period, leakage into the sample was indicated and was later confirmed by a water content check at the end of the test. Some tests run in this experimental program had to be deleted due to leakage. The tests to which this applied were some of the tests in which the membranes were being used for a second time.

An incremental loading test gave a maximum deviator stress of 52.0 psi at an axial strain of about 4.3 per cent. Using this value as a reference compressive strength, creep rupture tests were run at deviator stresses of 51.3, 48.3, 46.3, 44.9, 43.7, 43.4 and 42.8 psi. Figure 7 shows the typical strain/time and pore pressure/time curves for a creep rupture test on normally consolidated Haney clay. The data are from Test C-6, which was loaded with a deviator stress of 43.7 psi. It should be noted that it would not be difficult to divide this strain/time curve into the standard sections; namely, primary creep, secondary creep and tertiary creep.

The pore water pressure rose continually during the test and there was no change in curvature or noticeable discontinuity in the pore water pressure curve during the period of secondary creep. Yet, the strain rate was decreasing in the early stages of the test and increasing at the end. It would appear difficult, then, to predict that the sample was going to rupture at a later stage in the test based upon the pore pressure measurements.

As already mentioned in Section 3.2, the theoretical equalization time for nonuniform pore pressures was of the order of one hour. Figure 8 shows the pore pressure/strain curve for Test C-6 and a dotted line which shows a

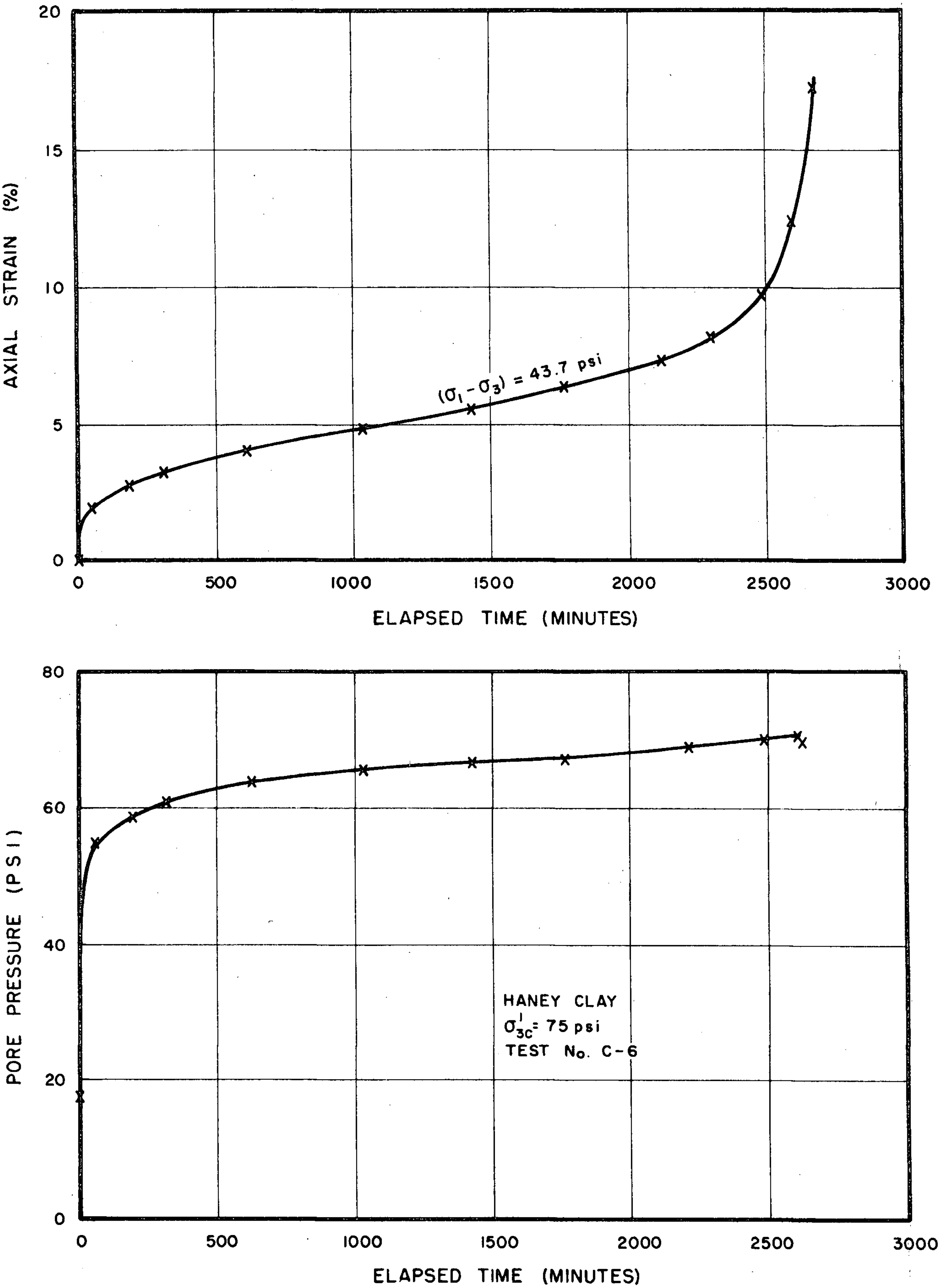


FIGURE 7 - UNDRAINED CREEP TEST
NORMALLY CONSOLIDATED

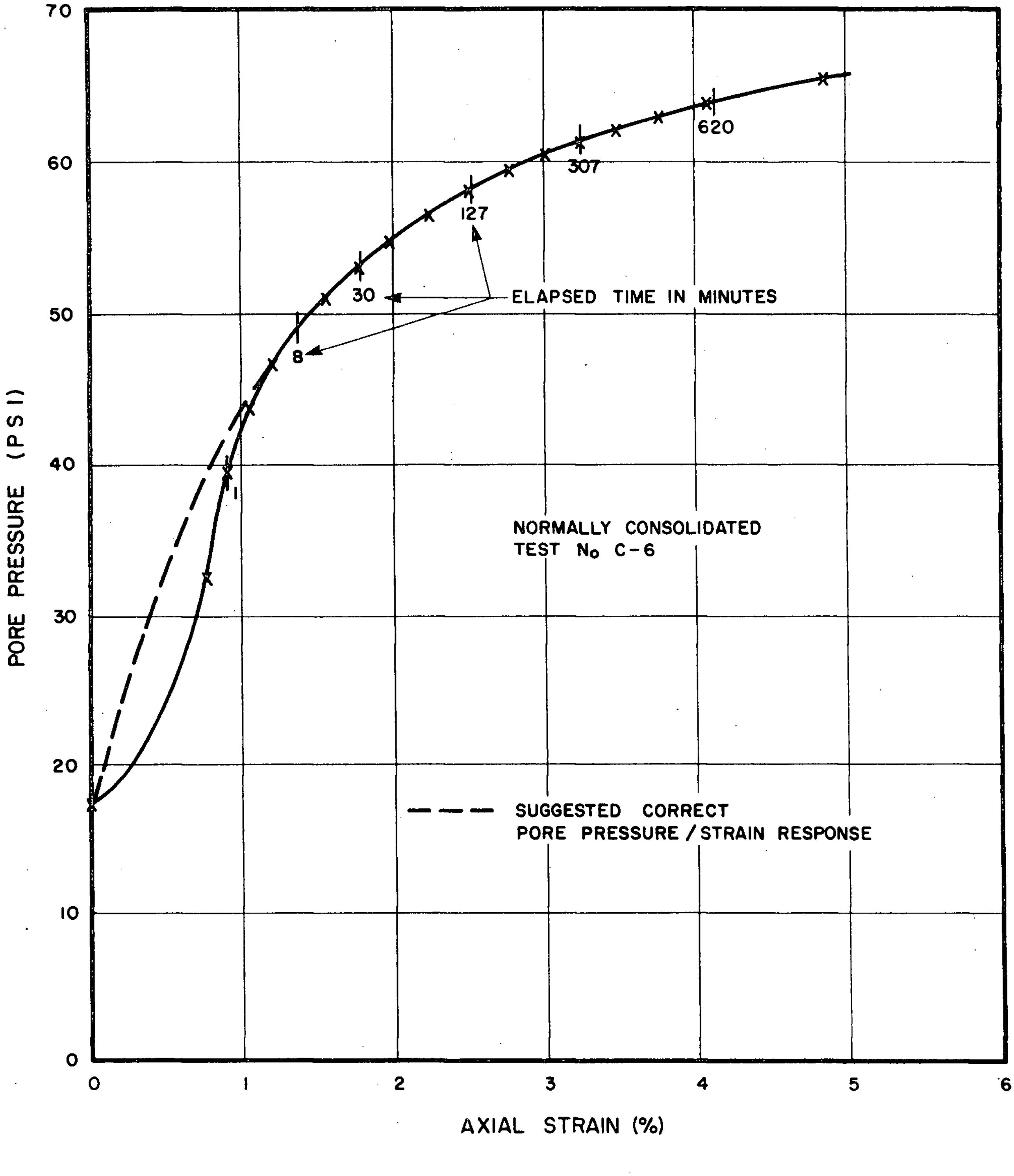


FIGURE 8 - PORE PRESSURE / STRAIN CURVE
NORMALLY CONSOLIDATED

suggested correct pore pressure response for strains less than 1.5 per cent. The dotted line predicts an increasing pore pressure whose rate of increase continually decreases with strain. This suggests that the pore pressure transducer is registering the correct pore pressure after about 1.5 per cent axial strain. This is equivalent to an elasped time in the test of about 16 minutes. This approach, when used for all normally consolidated samples, gave values of about 10 to 30 minutes to reach pore pressure equalization, which is less than the theoretical equalization time. However, for overconsolidated samples, a smooth curve through the initial pore pressure is not possible and, therefore, this method cannot be used to estimate equalization times. For example, in Figure 9 the pore pressure/strain curve is plotted for Test C-16 which was an overconsolidated sample. Upon loading, the pore pressure rose and after the first reading the pore pressure was observed to drop. It is impossible to hypothesize the true pore pressure behaviour between zero and two per cent strain and, therefore, an estimate of the time until pore pressure equalization is obtained is not possible. Thus, the pore pressure/strain curve can only be suggested as a probable

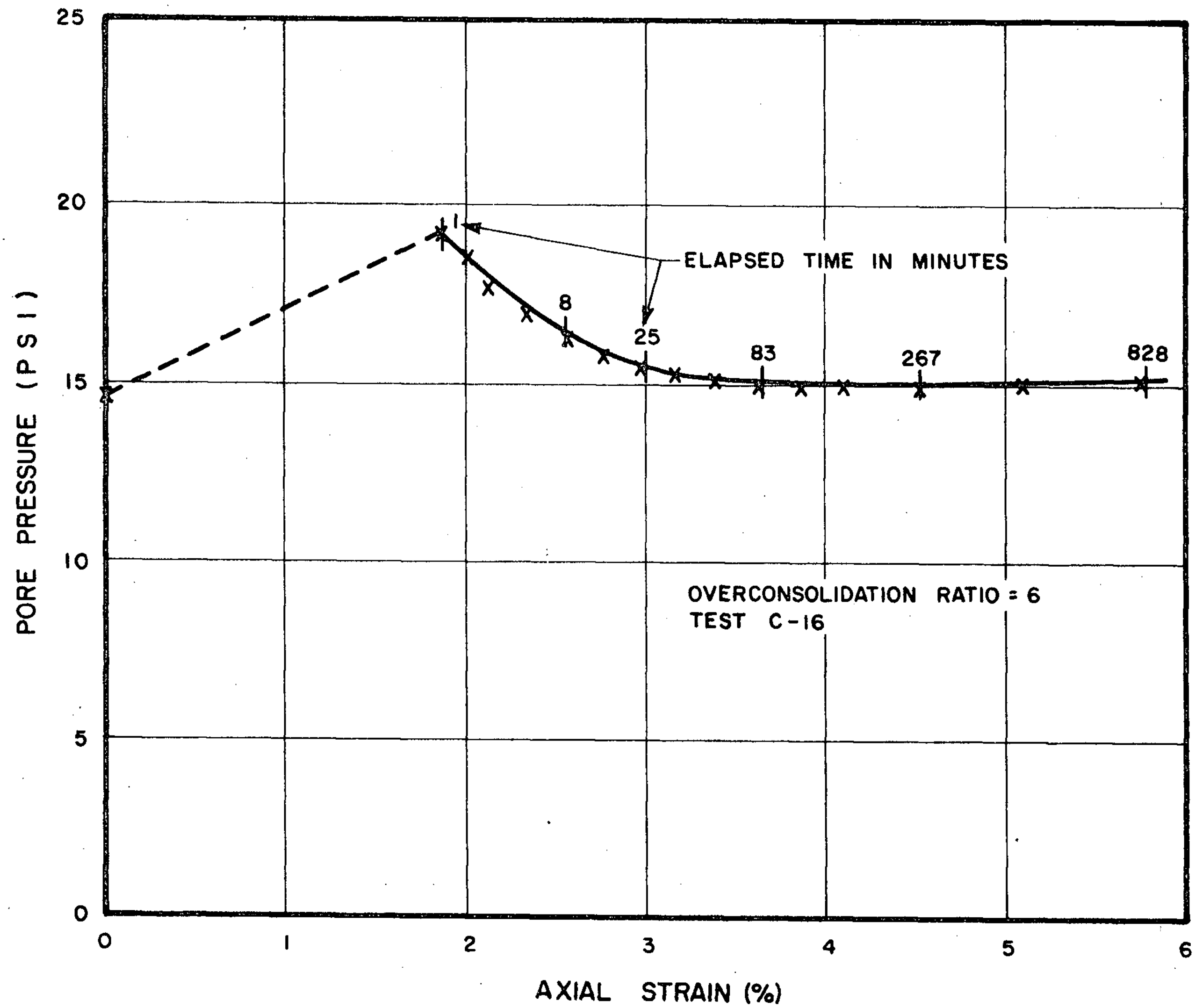


FIGURE 9 – PORE PRESSURE / STRAIN CURVE ($\eta = 6$)

check on the equalization times for nonuniform pore pressures in the normally consolidated test series. As a result, pore pressures measured after the first hour of creep are assumed to be correct and, in fact, those measured after the first 20 minutes of creep are likely close to the correct values.

Pore pressures measured during the last few minutes of a creep rupture test are also questionable, since at high strain rates pore pressure equalization cannot take place. Thus, failure criteria based on effective stress parameters at rupture are difficult to confirm. In addition, as rupture approaches, it is difficult to keep the deviator stress constant due to the changing cross-sectional area. Furthermore, the failure strain is difficult to estimate (see Figure 7). The only parameter which can be measured with reasonable accuracy at rupture is the elapsed time of creep loading. Thus, it is necessary to look for a failure criterion which is not dependent on parameters measured during the final part of the test. It was therefore decided to look more closely at the variation of strain rates during the test.

Figure 10 is a plot of calculated strain rates against elapsed time on a log-log plot for Test C-6. There

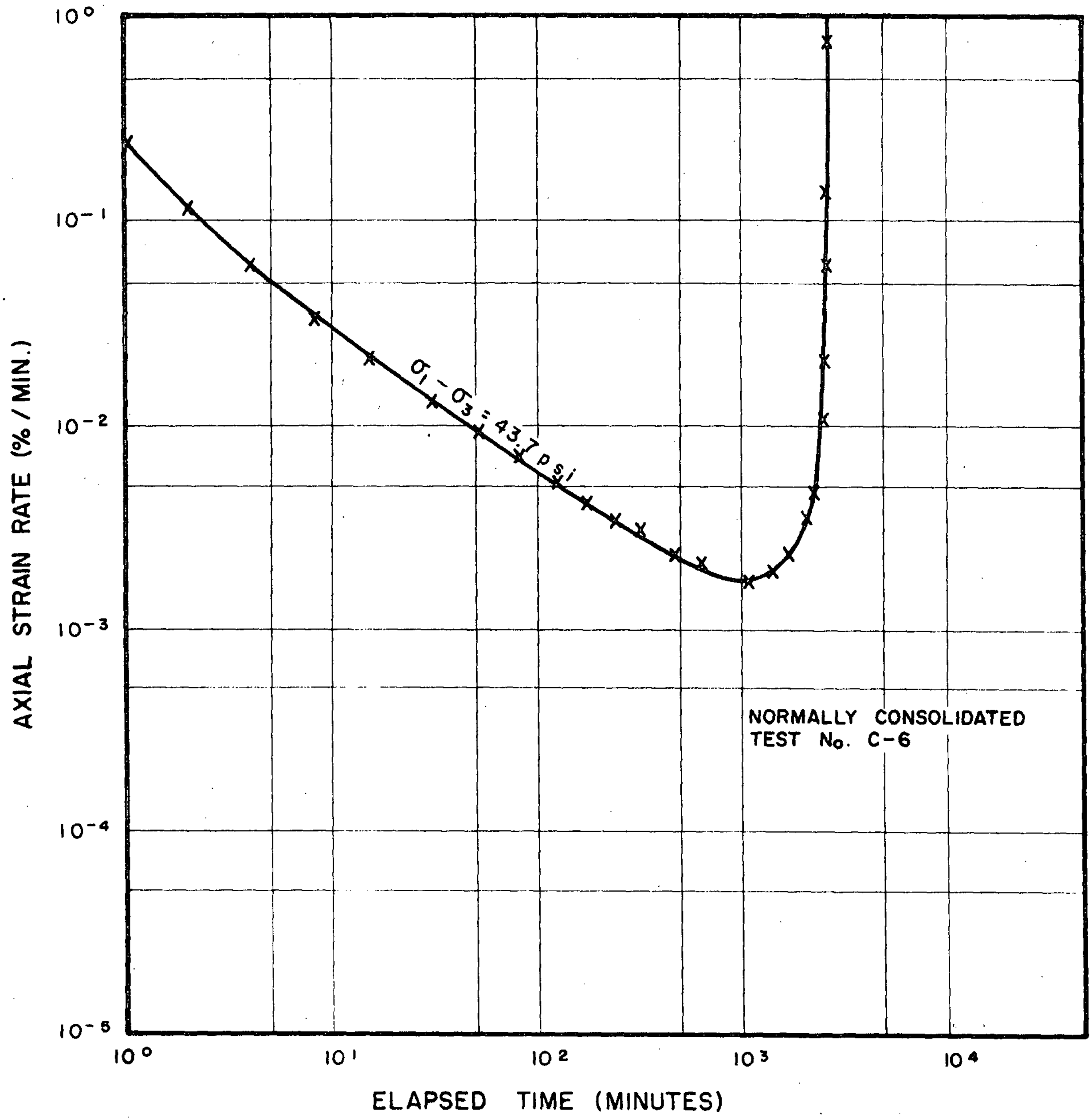


FIGURE 10 - LOG STRAIN RATE VERSUS
LOG ELAPSED TIME (NORMALLY CONSOLIDATED)

is a well-defined variation of strain rate during the complete test, the variation being a continuous function and having a definite transient minimum strain rate with no particular interval of time over which it could be said the strain rate was constant. It should be remembered that this is the same test shown in Figure 7. It had been suggested that the strain/time curve in Figure 7 could be divided into primary, secondary and tertiary creep. However, on the strain rate plot (Figure 10), it appears that there was no secondary creep in Test C-6. Therefore, it is suggested that secondary creep does not exist for Haney clay and is only recognized on the strain/time curve because of the scale of the strain ordinate. Once the strain rate becomes fairly small (i.e., approaching a horizontal line on a strain/time curve), a change of slope by a factor of two becomes difficult to distinguish. As a result, if secondary strain rates were evaluated for Haney clay from strain/time curves, the secondary strain rate obtained could be a function of the duration of the test. For example, Figures 11 and 12 present the strain/time curves for Test C-5 which did not reach a transient minimum strain rate.

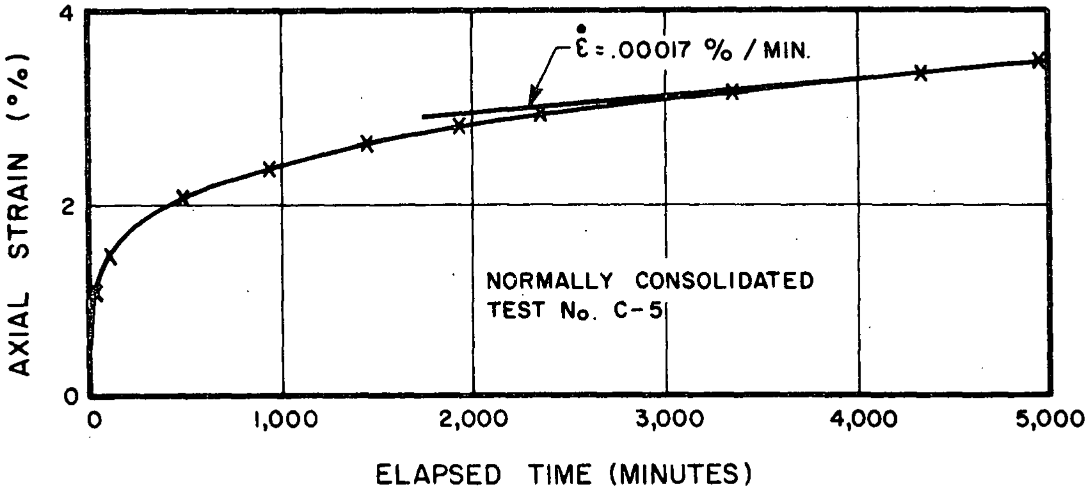


FIGURE 11 - STRAIN / TIME CURVE
0-5,000 MINUTES

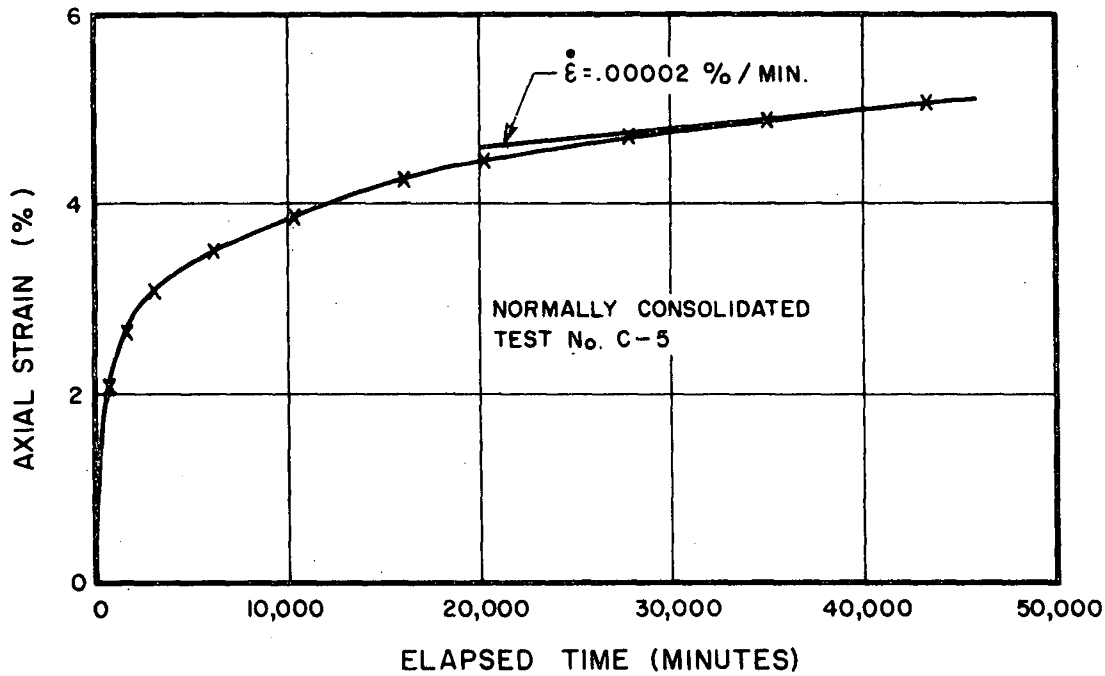
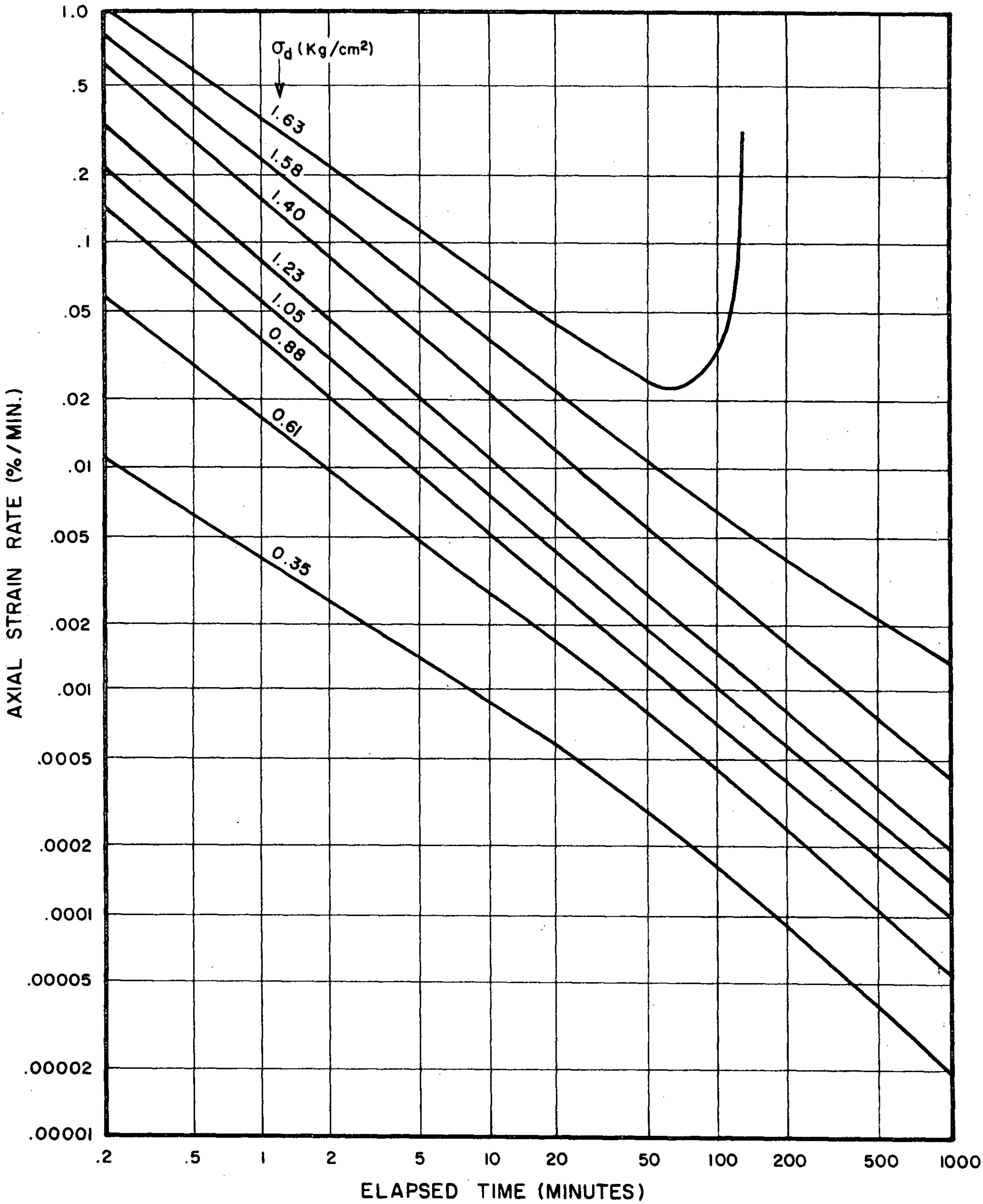


FIGURE 12 - STRAIN / TIME CURVE
0-50,000 MINUTES

Figure 11 shows the results plotted for the first 5,000 minutes and, if the test were to be stopped at this point, the "secondary strain rate" obtained would be .00017 per cent per minute. However, on Figure 12 the complete test is plotted (total duration greater than 40,000 minutes) and the "secondary strain rate" obtained is .00002 per cent per minute. Of course, both of these "secondary strain rates" are meaningless.

Campanella (1965) showed that, for stresses below the upper yield strength of consolidated illite, the strain rate was continually decreasing for the duration of the test. It is observed that Campanella's results of strain rate versus elapsed time are nearly parallel straight lines on a log-log plot for various subfailure stress levels (Figure 13), as predicted by the phenomenological equation of Singh and Mitchell (1968). Once the strain rate is observed to increase, all test results indicate that the strain rate will continue to increase and eventually the sample is bound to fail.

Therefore, the existence of a transient minimum strain rate can be used as an indication of a time dependent impending failure. This is similar to the failure criterion



(AFTER CAMPANELLA, 1965)

FIGURE 13 - LOG STRAIN RATE VERSUS LOG TIME FOR SATURATED ILLITE

suggested by Casagrande and Wilson (1951) and Coates, Burn and McRostie (1963). However, they did not use a strain rate plot to obtain the point of inflection in the strain/time curves. To avoid confusion throughout the remainder of this thesis, whenever failure of creep tests is discussed, it will be clearly stated whether failure is being defined at the transient minimum strain rate or at rupture.

Table II lists the values of measured parameters at the transient minimum strain rate. At this point in a test, since the strain rate is low, the deviator stress and pore water pressure can be measured more accurately than at rupture. The results in Table II indicate that the value of the transient minimum strain rate decreased with decreased deviator stress, while the elapsed time to the transient minimum strain rate increased. The principal effective stress ratio, the strain and Skempton "A" are all shown to increase at the transient minimum strain rate for decreasing deviator stress. Included in Table II are the results of the incremental loading test for the purpose of comparison. It is interesting to compare the maximum deviator stress and strain rate for the incremental loading test with the deviator stresses and transient minimum strain

TABLE II

Normally Consolidated Haney Clay
Undrained Creep Rupture Tests

Deviator Stress (psi)	Transient Minimum Strain Rate (Per cent per Minute)	At Transient Minimum Strain Rate σ_1'/σ_3'	At Transient Minimum Strain Rate Strain (Per cent)	At Transient Minimum Strain Rate Skempton "A"	Time to Transient Minimum Strain Rate (Minutes)
51.3	.165	2.45	2.4	.72	6
48.3	.040	2.61	2.7	.85	29
46.3	.0150	2.53	2.9	.92	80
44.9	.0037	2.75	4.2	1.05	510
43.7	.00190	2.82	5.2	1.15	1,200
43.4	.00038	2.88	5.4	1.17	5,600
42.8*	.000018	2.95	5.0	1.19	40,800

*Did not reach a transient minimum strain rate--parameters evaluated just before stopping test.

- -

Incremental Loading Test*

Maximum Deviator Stress (psi)	Strain Rate (Per cent per Minute)	σ_1'/σ_3'	Strain (Per cent)	Skempton "A"	Total Elapsed Time (Minutes)
52.0	.40	2.80	4.3	1.12	327

*Failure based on maximum deviator stress.

rates obtained from the creep tests. The maximum deviator stress obtained in the incremental loading test is almost the same as the deviator stress which caused failure very quickly in a creep test, and the magnitude of the strain rates are also similar.

Figure 14 shows the development of σ_1' / σ_3' with strain for Test C-6. This development is entirely due to the increasing pore water pressure as the sample strained. The transient minimum strain rate occured at 5.2 per cent strain and the strain rate continually increased from this stage in the test until rupture. Figure 15 compares the maximum recorded σ_1'/σ_3' for each deviator stress at which a creep test was performed. Although pore water pressures are known to be questionable near rupture in creep tests, it is interesting to note that the maximum principal effective stress ratio at or near creep rupture was approximately 3.20.

As mentioned previously, if effective stress strength parameters are used for design, it is necessary to estimate the anticipated pore pressures. Figure 16 shows the pore pressure/strain curves for three creep tests with widely different times to reach five per cent strain. The

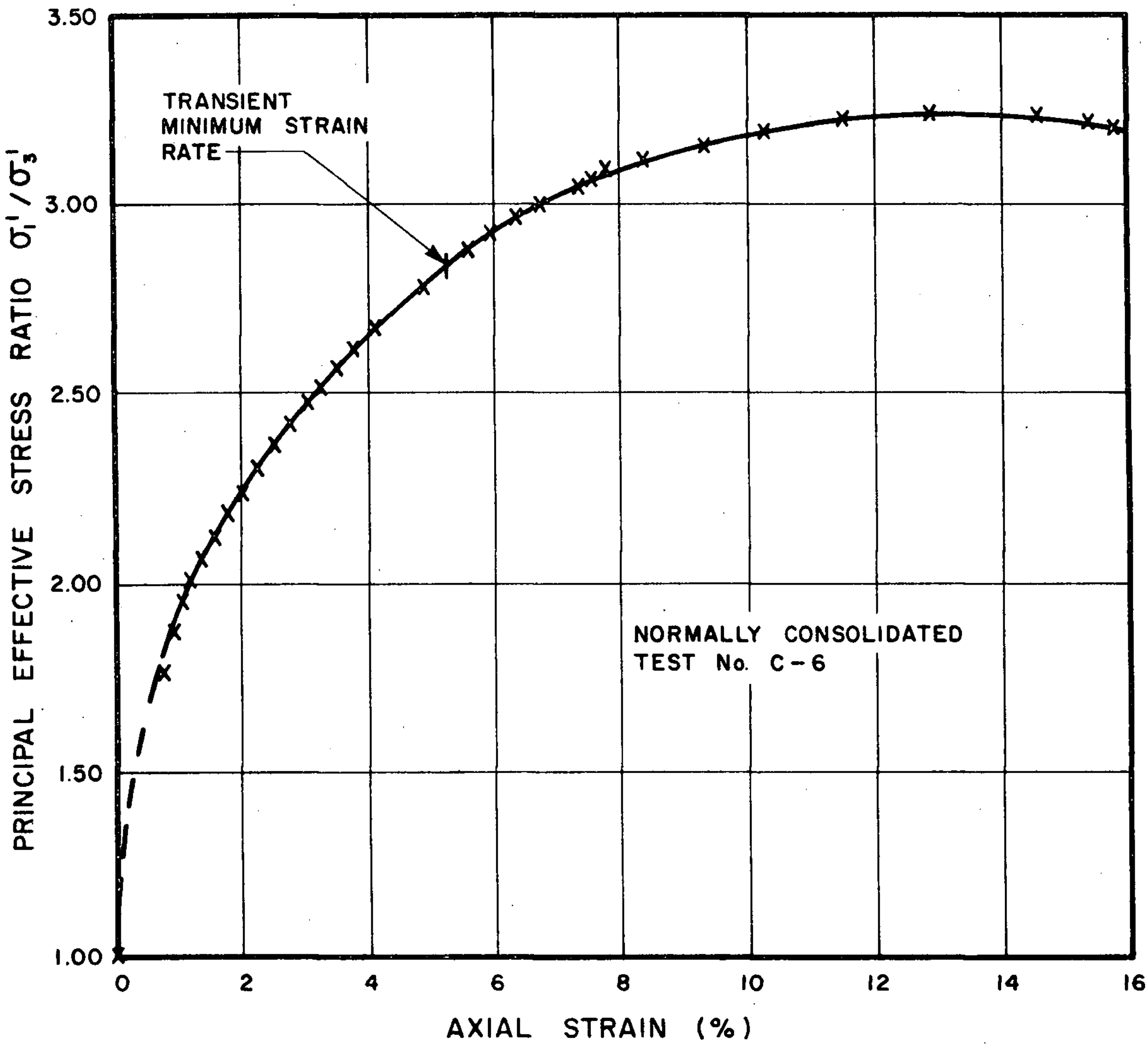


FIGURE 14 - DEVELOPMENT OF σ_1'/σ_3' DURING A CREEP TEST

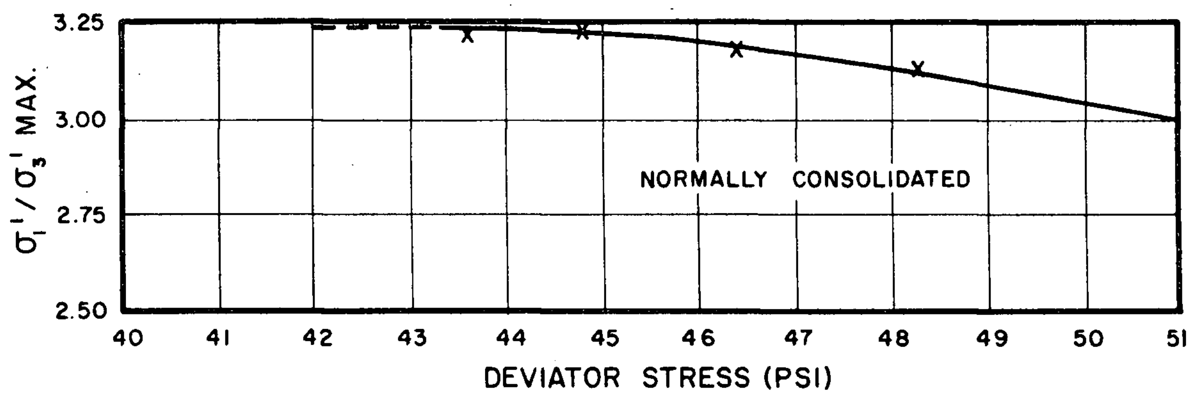


FIGURE 15 - MAXIMUM σ_1'/σ_3' OBTAINED FROM CREEP TESTS

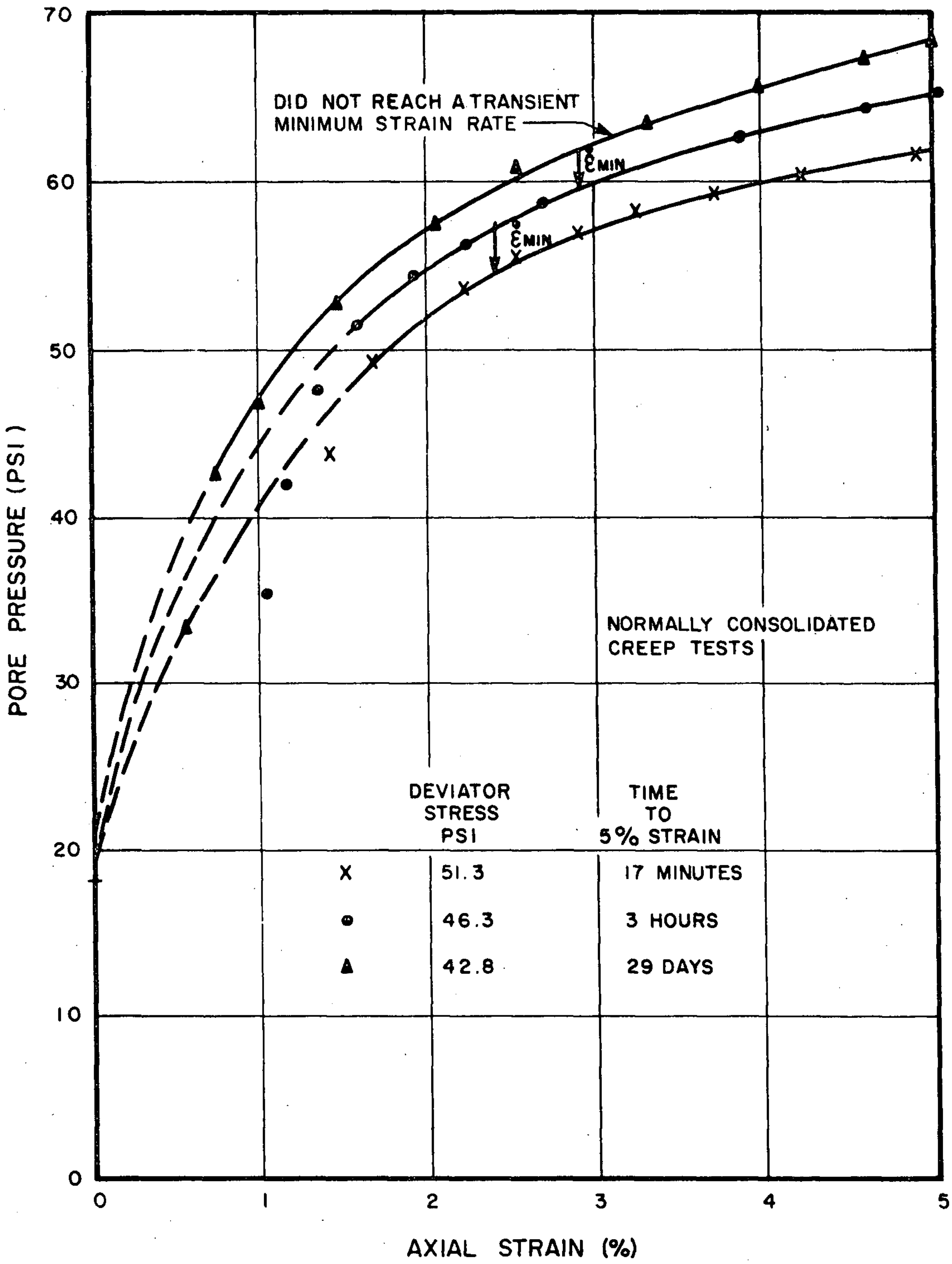


FIGURE 16 - EFFECT OF DEVIATOR STRESS ON PORE PRESSURE

longest duration test developed the largest pore pressure at five per cent strain, yet had not reached a transient minimum strain rate after 29 days when the test was stopped. The other two tests, although generating slightly smaller pore pressures at five per cent strain, resulted in creep rupture failures. Based upon these results, it can be seen that prediction of pore pressures and resulting creep failure is a complex problem.

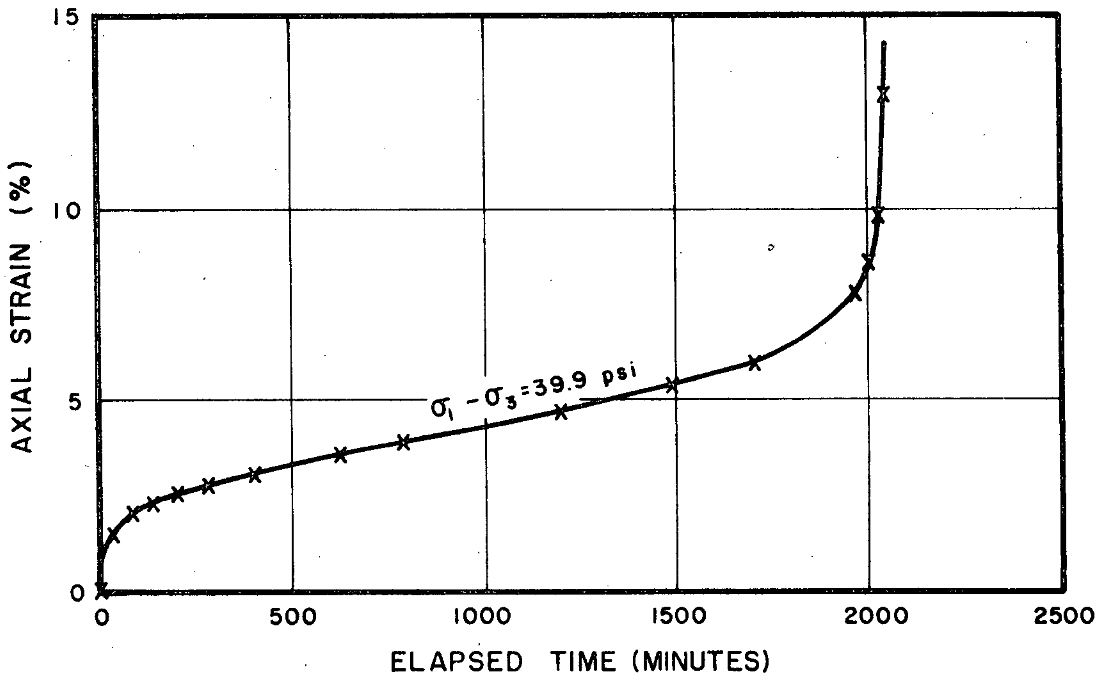
4.2 - Undrained Creep Rupture Tests on Overconsolidated Samples (Overconsolidation Ratio = 2)

All samples in this series were normally consolidated for 24 hours and then rebounded to an effective stress of 37.5 psi for 36 hours. The average water content before shearing was 33.5 per cent with a standard deviation of the average equal to ± 0.2 per cent. The average pore pressure rise during the 8-hour undrained stage before shearing was only ± 0.3 psi, although some tests actually registered a small decrease in pore pressure during this period. It is reasonable to expect that the pore pressure change during the undrained stage would be less for the overconsolidated samples, as compared with the normally consolidated samples, since the total consolidation time was longer and the effective stress had been reduced from 75 psi. This resulted in less tendency for secondary compression.

The incremental loading test which was performed yielded a maximum deviator stress of 46.5 psi at an axial strain of about 4.8 per cent. Using this value as a reference compressive strength, creep rupture tests were run at deviator stresses of 41.9, 39.9 and 38.9 psi.

Figure 17 shows the strain/time and pore pressure/time curves for Test C-15. Due to overconsolidation, the pore pressure did not increase as much as for normally consolidated samples; however, the shape of the strain/time curve is similar to those already presented. Figure 18 shows the similarity of strain rate/time behaviour to that obtained for normally consolidated samples.

Table III presents the values of parameters at the transient minimum strain rate for two creep rupture tests, as well as the results of the incremental loading test. The behaviour of this test series is similar to the test series on normally consolidated Haney clay and will not be commented on further at this point.



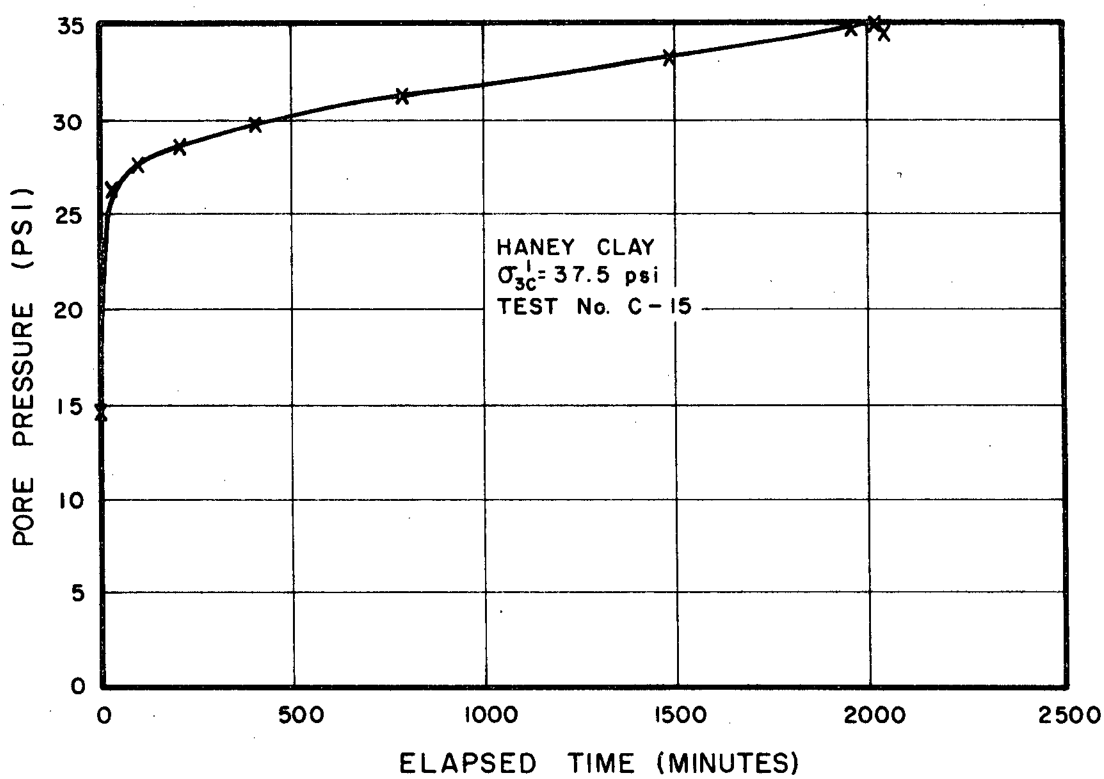


FIGURE 17 - UNDRAINED CREEP TEST
OVERCONSOLIDATION RATIO = 2

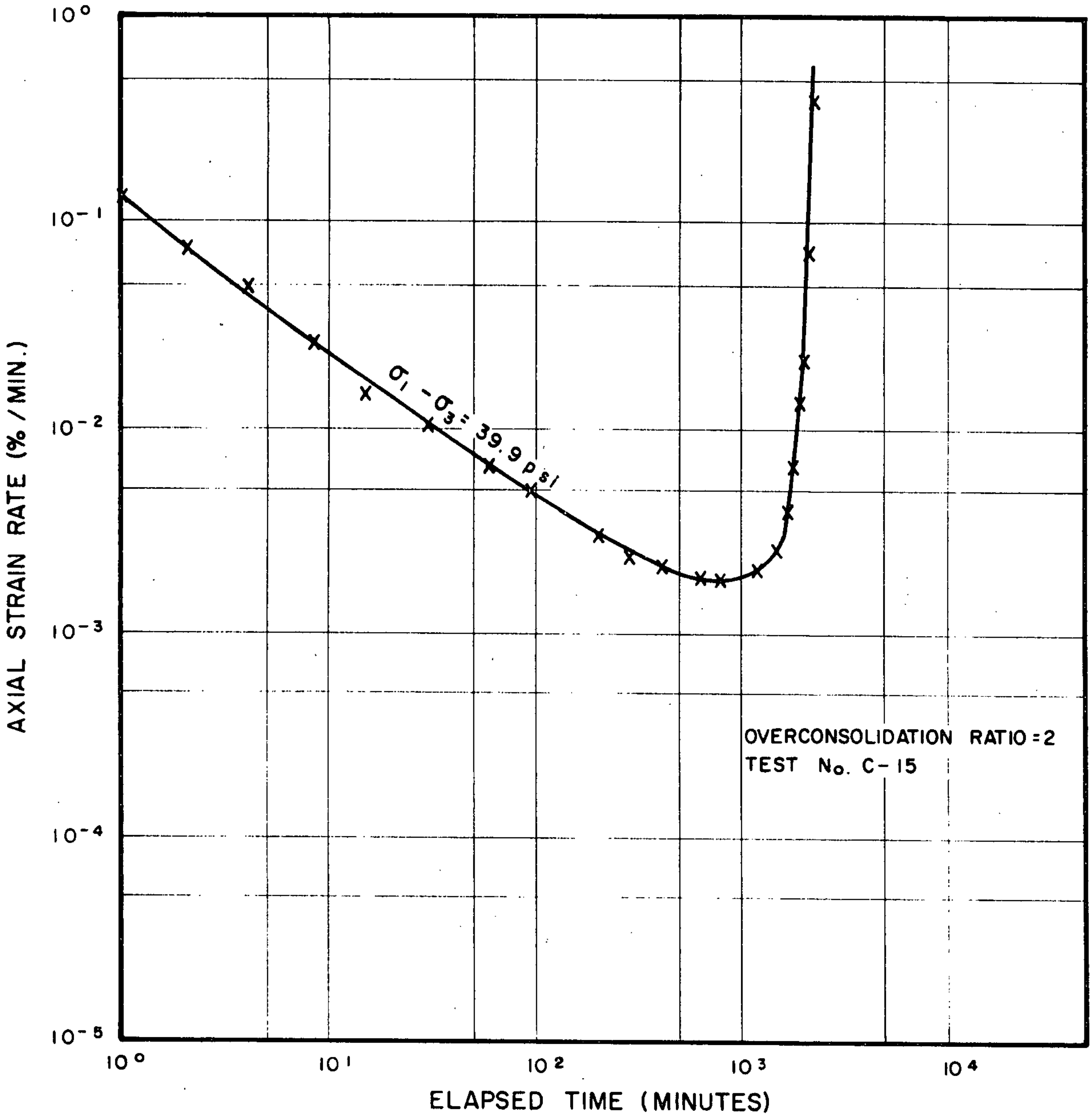


FIGURE 18 - LOG STRAIN RATE VERSUS LOG ELAPSED TIME ($\eta = 2$)

TABLE III

Overconsolidated Haney Clay (η=2)
Undrained Creep Rupture Tests

Deviator Stress (psi)	Transient Minimum Strain Rate (Per cent per Minute)	At Transient Minimum Strain Rate: σ_1'/σ_3'	Strain (Per cent)	Skempton "A"	Time to Transient Minimum Strain Rate (Minutes)
41.9	.018	2.79	2.9	.37	61
39.9	.0019	2.87	3.8	.42	790
38.9*	.000083	3.11	5.8	.50	12,950

*Did not reach a transient minimum strain rate--
parameters evaluated just before stopping test.

Incremental Loading Test*

Maximum Deviator Stress (psi)	Strain Rate (Per cent per Minute)	σ_1'/σ_3'	Strain (Per cent)	Skempton "A"	Total Elapsed Time (Minutes)
46.5	.29	3.12	4.4	.28	290

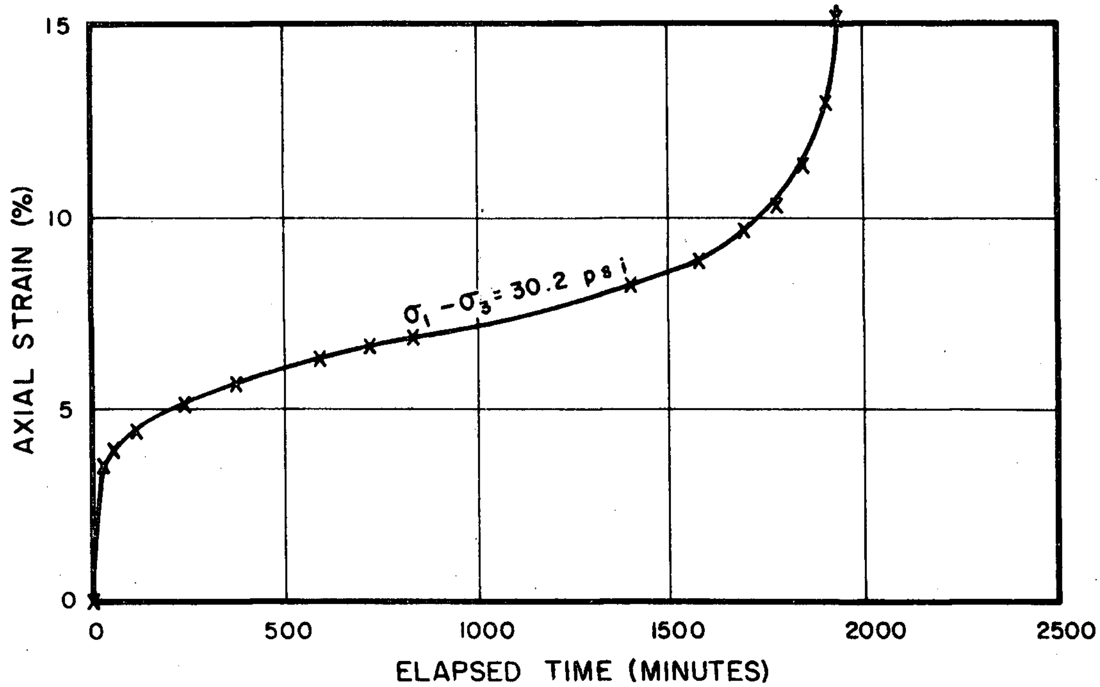
*Failure based on maximum deviator stress.

4.3 - Undrained Creep Rupture Tests on Overconsolidated Samples (Overconsolidation Ratio = 6)

All samples in this series were normally consolidated for 24 hours and then rebounded to an effective stress of 12.5 psi for 36 hours. The average water content before shearing was 34.5 per cent with a standard deviation of the average equal to ± 0.2 per cent. During the 8-hour undrained stage before shearing, the pore pressure dropped by an average of 0.4 psi.

An incremental loading test yielded a maximum deviator stress of 33.6 psi at an axial strain of about 6.0 per cent. Using this value as a reference compressive strength, creep tests were run at deviator stresses of 31.4, 30.2, 29.1 and 28.7 psi.

Figure 19 shows the strain/time and pore pressure/time curves for Test C-20. Note again the similarity in shape of the strain/time curve with the previous tests. The pore pressure during this test does not change appreciably and thus both total and effective stresses are nearly constant. Table IV presents the results of this test series in the same form as before. These results are similar to those



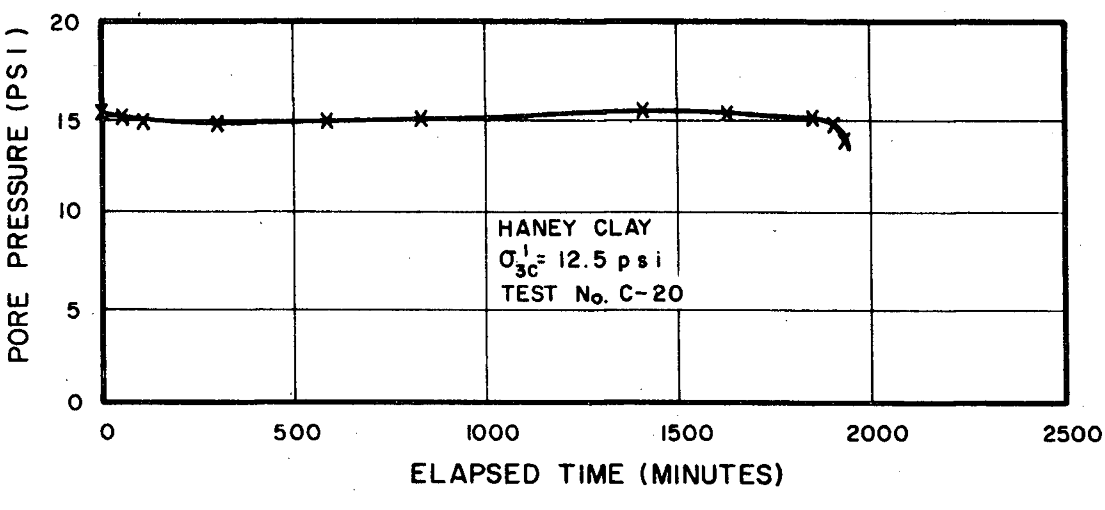


FIGURE 19 - UNDRAINED CREEP TEST
OVERCONSOLIDATION RATIO = 6

TABLE IV

Overconsolidated Haney Clay (η=6)
Undrained Creep Rupture Tests

Deviator Stress (psi)	Transient Minimum Strain Rate (Per cent per Minute)	At Transient Minimum Strain Rate σ_1'/σ_3'	At Transient Minimum Strain Rate Strain (Per cent)	At Transient Minimum Strain Rate Skempton "A"	Time to Transient Minimum Strain Rate (Minutes)
31.4	.018	3.42	4.8	.00	85
30.2	.0026	3.42	6.9	-.01	834
29.1	.0011	3.45	8.6	.04	3,060
28.7*	.000008	3.38	3.8	.03	11,800

*Did not reach a transient minimum strain rate--
parameters evaluated just before stopping test.

Incremental Loading Test*

Maximum Deviator Stress (psi)	Strain Rate (Per cent per Minute)	σ_1'/σ_3'	Strain (Per cent)	Skempton "A"	Total Elapsed Time (Minutes)
33.6	.56	3.45	6.0	-.02	217

*Failure based on maximum deviator stress.

already presented. The most significant parameter measured during this test series was the pore water pressure. Since the pore pressure remained nearly constant during the creep tests, both the total and effective principal stress ratios were unchanged. However, the strain/time response was similar to the results of tests on samples with different consolidation histories which had variations in pore pressure during creep tests. Figure 20 shows the strain rate/time curve for Test C-20, which is again very similar to the results presented from other tests.

4.4 - Drained Creep Rupture Tests on Overconsolidated Samples (Overconsolidation Ratio = 25)

All samples in this series were normally consolidated for 24 hours and then rebounded to an effective stress of 3.0 psi for 44 hours. The average water content before shearing was 36.8 per cent with a standard deviation of the average equal to ± 0.25 per cent.

For comparative purposes, all strains quoted for drained tests are shear strains $(\varepsilon_1 - \frac{\Delta V}{3V})$. Therefore, the volumetric component is eliminated from strain rate calculations.

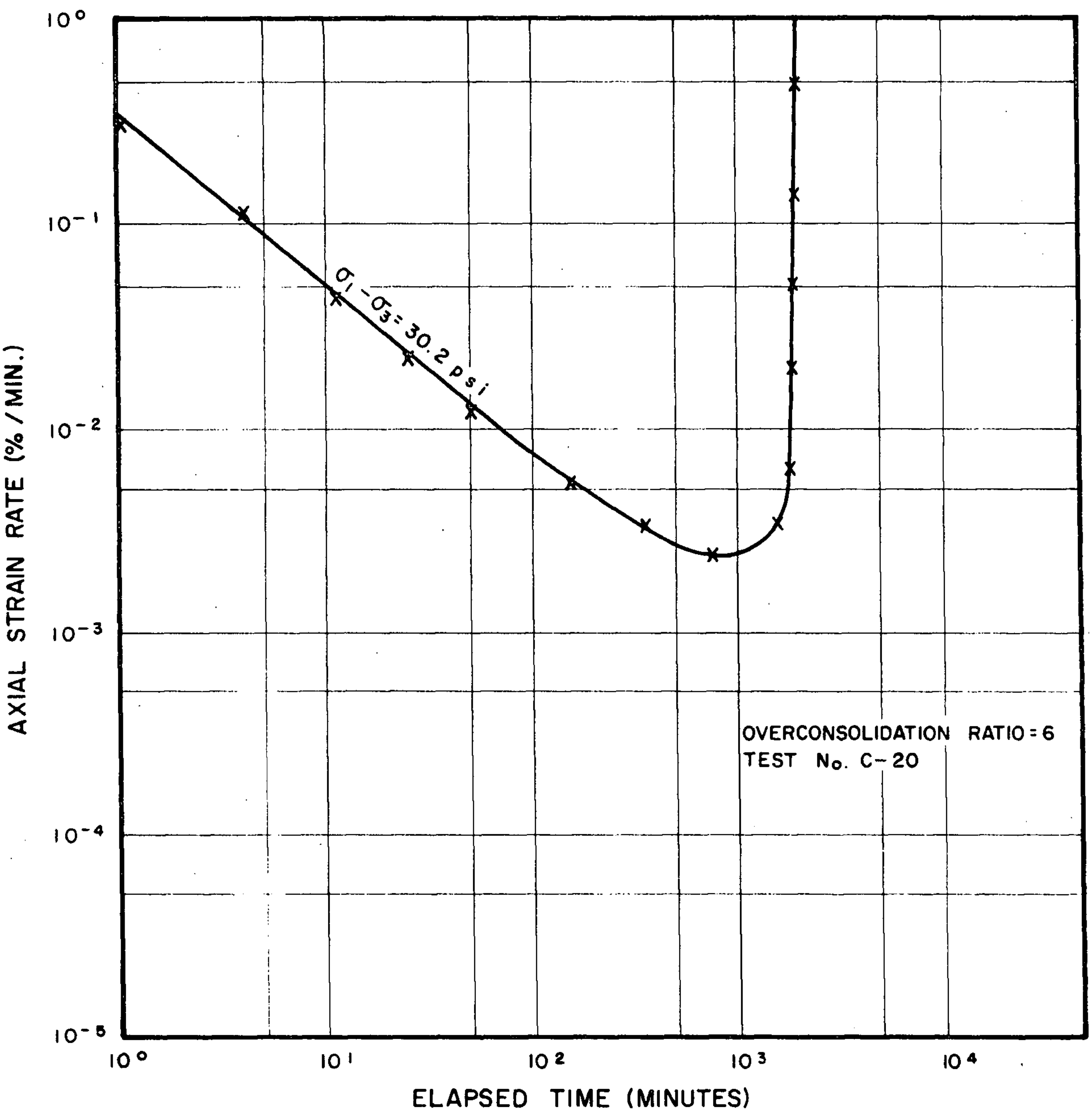
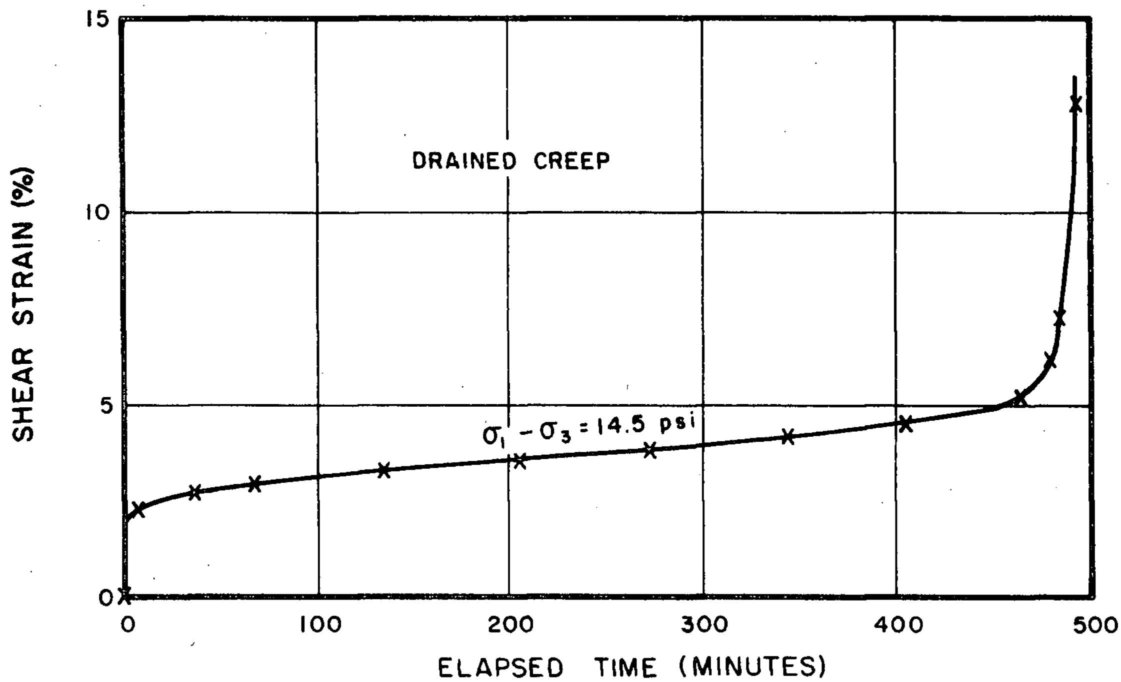


FIGURE 20 - LOG STRAIN RATE VERSUS
LOG ELAPSED TIME ($\eta = 6$)

A drained incremental loading test yielded a maximum deviator stress of 15.9 psi at a shear strain of 4.5 per cent. Using this value as a reference compressive strength, drained creep rupture tests were run at deviator stresses of 15.1, 14.5, 13.9 and 13.1 psi.

Figure 21 shows the shear strain/time and water content/time curves for Test C-22. The shape of the strain/time curve is similar to those already reported and, as anticipated, the sample was observed to swell during shear. Table V presents a summary of the results obtained from this test series. The compressive strength for these drained tests in noted to decrease for increasing times to failure.

Initially, it was thought that the decrease in strength during a drained test for heavily overconsolidated samples could be explained as the result of an increase in void ratio during shear and, therefore, by the resulting reduction in number or strength of interparticle bonds. However, a comparison of the shape of the strain rate/time curve for Test C-22, shown in Figure 22, indicates that the strain rate response is similar to that previously reported for undrained creep tests. To investigate this further, two undrained creep tests were performed with the same



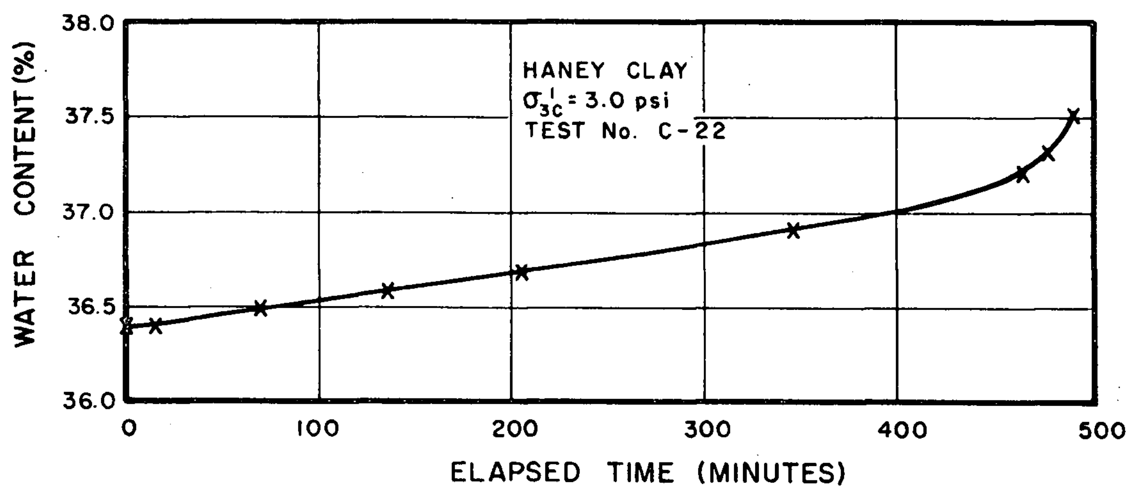


FIGURE 21 – DRAINED CREEP TEST
OVERCONSOLIDATION RATIO = 25

TABLE V

Overconsolidated Haney Clay (η=25)

Drained Creep Rupture Tests

Deviator Stress (psi)	Transient Minimum Strain Rate (Per cent per Minute)	At Transient Minimum Strain Rate: $\Delta V/V$ (Per cent)	At Transient Minimum Strain Rate: Shear Strain (Per cent)	Time to Transient Minimum Strain Rate (Minutes)
15.1	.011	.38	4.4	95
14.5	.0039	.50	3.5	205
13.9	.0036	.66	4.0	272
13.1*	.000012	.66	3.3	17,300

*Did not reach a transient minimum strain rate--parameters evaluated just before stopping test.

Drained Incremental Loading Test*

Maximum Deviator Stress (psi)	Strain Rate (Per cent per Minute)	$\Delta V/V$ (Per cent)	Shear Strain (Per cent)	Total Elapsed Time (Minutes)
15.9	.12	.43	4.5	680

*Failure based on maximum deviator stress.

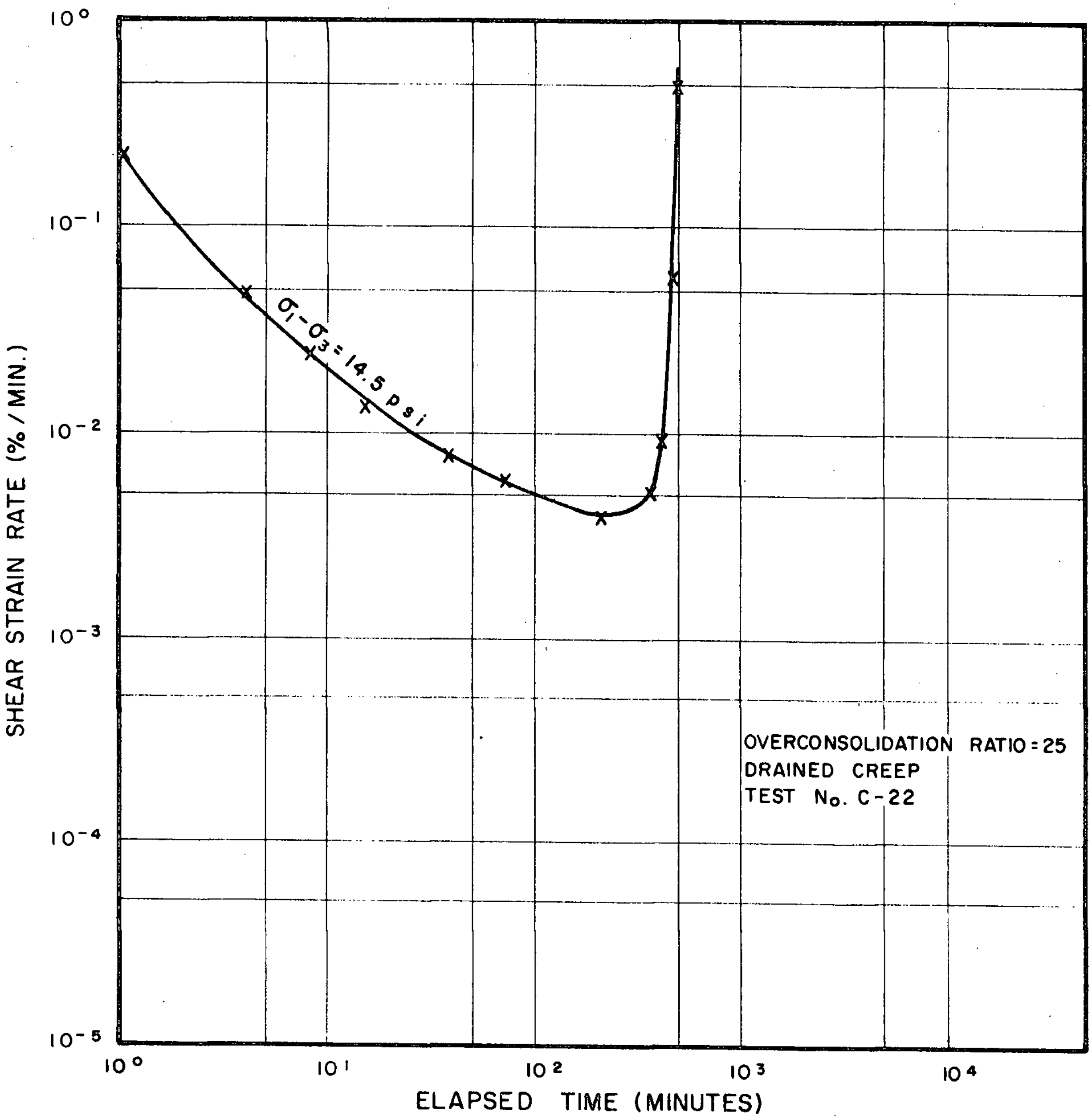


FIGURE 22 – LOG SHEAR STRAIN RATE VERSUS LOG ELAPSED TIME ($\eta = 25$)

consolidation history as for this test series. The results of these undrained tests are presented in the next section.

4.5 - Undrained Creep Rupture Tests on Overconsolidated Samples (Overconsolidation Ratio = 25)

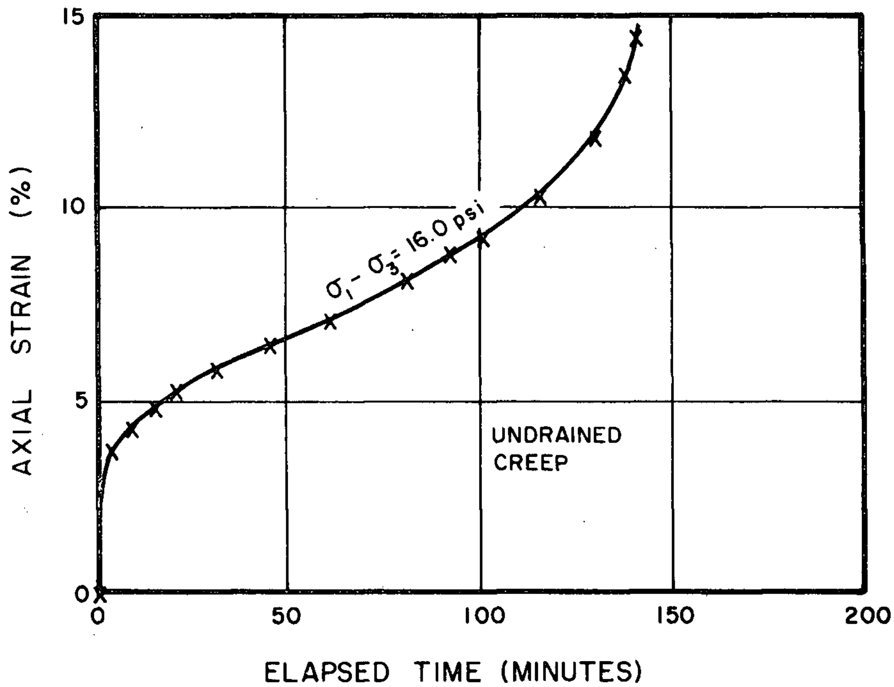
All samples in this series were normally consolidated for 24 hours and then rebounded to an effective stress of 3.0 psi for 36 hours. The three undrained tests run in this series were trimmed from a block of Haney clay which was obtained from a slightly different location than all the other tests reported in this thesis. As a result, the average water content before shearing was 1.7 per cent higher than for the samples used in the drained test series with the same consolidation history. During the 8-hour undrained stage before shearing, the pore pressure dropped by an average of 0.6 psi.

An incremental loading test yielded a maximum deviator stress of 17.6 psi at an axial strain of about 7.2 per cent. Using this value as a reference compressive strength, creep tests were run at deviator stresses of 16.9 and 16.0 psi.

Figure 23 shows the strain/time and pore pressure/time curves for Test C-35. Note that during this test, the pore pressure is continually decreasing and, therefore, the principal effective stress ratio is also decreasing throughout the test. Figure 24 shows the strain rate/time curve which is again similar to the results already presented for other test series. Table VI presents a summary of the tests performed in this series.

Based upon the results of these drained and undrained tests, it is obvious that the additional swelling during the drained test only decreases the compressive strength, in addition to the effect of time during the undrained test.

Since the water contents after consolidation for the drained and undrained test series varied by 1.7 per cent, it is not possible to realistically compare the compressive strengths obtained from the two test series. It would be expected that the undrained compressive strength would be larger. However, the higher water content at the start of the undrained tests would likely reduce the compressive strength and, therefore, the fact that the measured undrained compressive strengths are only slightly larger than the drained compressive strengths is understandable.



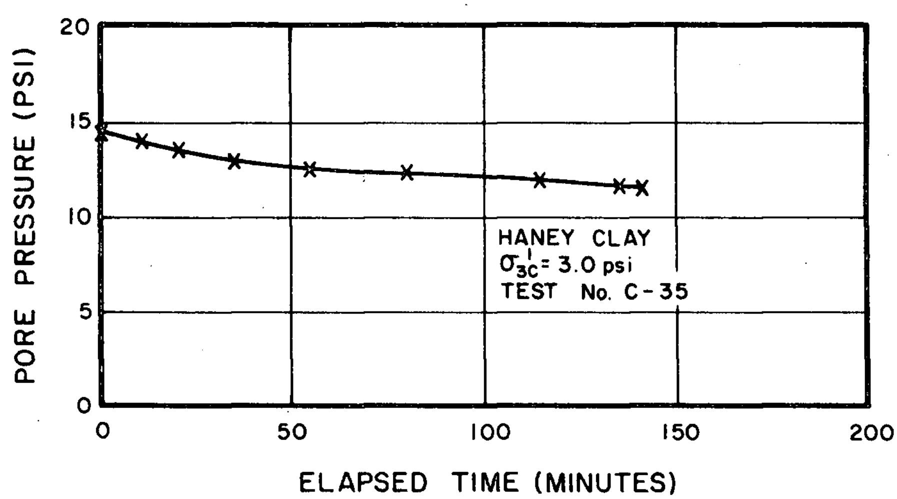


FIGURE 23 - UNDRAINED CREEP TEST
OVERCONSOLIDATION RATIO = 25

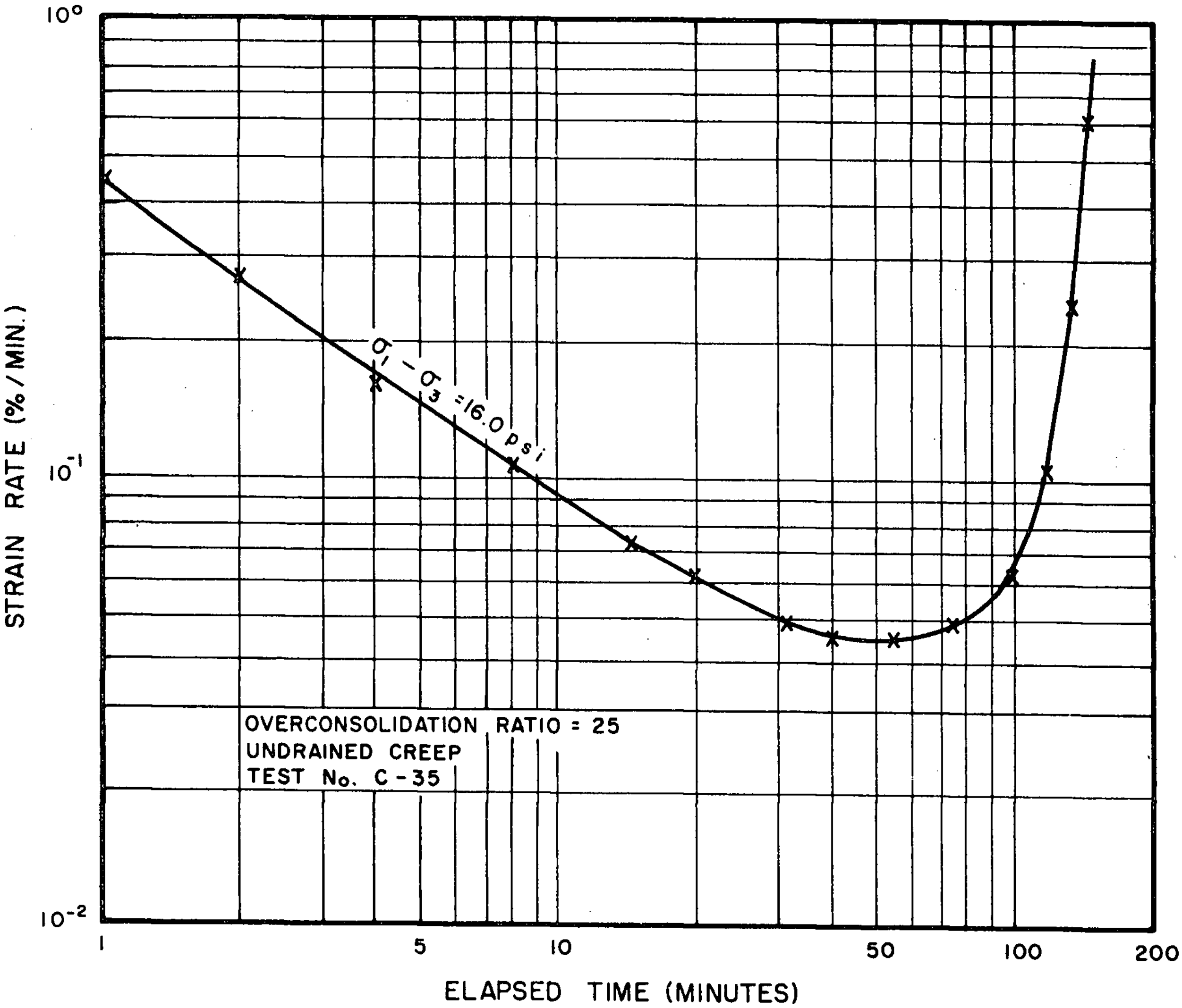


FIGURE 24 — LOG STRAIN RATE VERSUS LOG ELAPSED TIME ($\eta = 25$)

TABLE VI

Overconsolidated Haney Clay (η=25)
Undrained Creep Rupture Tests

Deviator Stress (psi)	Transient Minimum Strain Rate (Per cent per Minute)	At Transient Minimum Strain Rate			Time to Transient Minimum Strain Rate (Minutes)
		σ_1'/σ_3'	Strain (Per cent)	Skempton "A"	
16.9	.15	4.26	6.6	-.11	16
16.0	.046	3.95	6.9	-.13	55

Incremental Loading Test*

Maximum Deviator Stress (psi)	Strain Rate (Per cent per Minute)	σ_1'/σ_3'	Strain (Per cent)	Skempton "A"	Total Elapsed Time (Minutes)
17.6	.15	3.96	7.2	-.11	596

*Failure based on maximum deviator stress.

4.6 - Summary

The results of creep tests on Haney clay at stress levels large enough to eventually cause creep rupture show that, for various consolidation histories and drainage conditions, the strain rate during the initial stage of the test continually decreases until reaching a transient minimum value and then increases until rupture. As a result, it has been proposed that the onset of creep rupture be defined as occurring at the transient minimum strain rate. Also, as a result of the existence of a transient minimum strain rate, it is noted that Haney clay does not exhibit "secondary creep."

The strain rate/time behaviour for all creep rupture tests was noted to be similar, whether drainage was allowed, or whether pore pressures, and thus effective stresses, were observed to be increasing, decreasing or not changing during the test.

For normally consolidated Haney clay, creep rupture was noted to occur at approximately the same principal effective stress ratio, while the onset of creep rupture at the transient minimum strain rate could not be predicted on the basis of effective stress parameters.

CHAPTER 5

CREEP RUPTURE CRITERIA

5.1 - Line of Minimum Strain Rates

In Chapter 4, results were presented showing the response of Haney clay with various consolidation histories to creep loadings. Except for the strain rate curves, all test results were plotted against the elapsed time on a linear scale. This was done to enable the reader to appreciate the changes taking place during a test based on a natural time scale. Unfortunately, elapsed times for creep tests vary over a wide range and thus the most convenient plot for presentation of all data is on a base of logarithm of elapsed time. Figure 25 shows the strain/logarithm time curves for all tests on normally consolidated samples. It is impossible to predict from the strain/logarithm time curve whether or not a sample is going to rupture, since upward curvature does not necessarily indicate increasing strain rates. The transient minimum strain rates are marked on the curves with a short vertical line and it is obvious that no indication of the transient minimum strain rate condition is available from this logarithm plot.

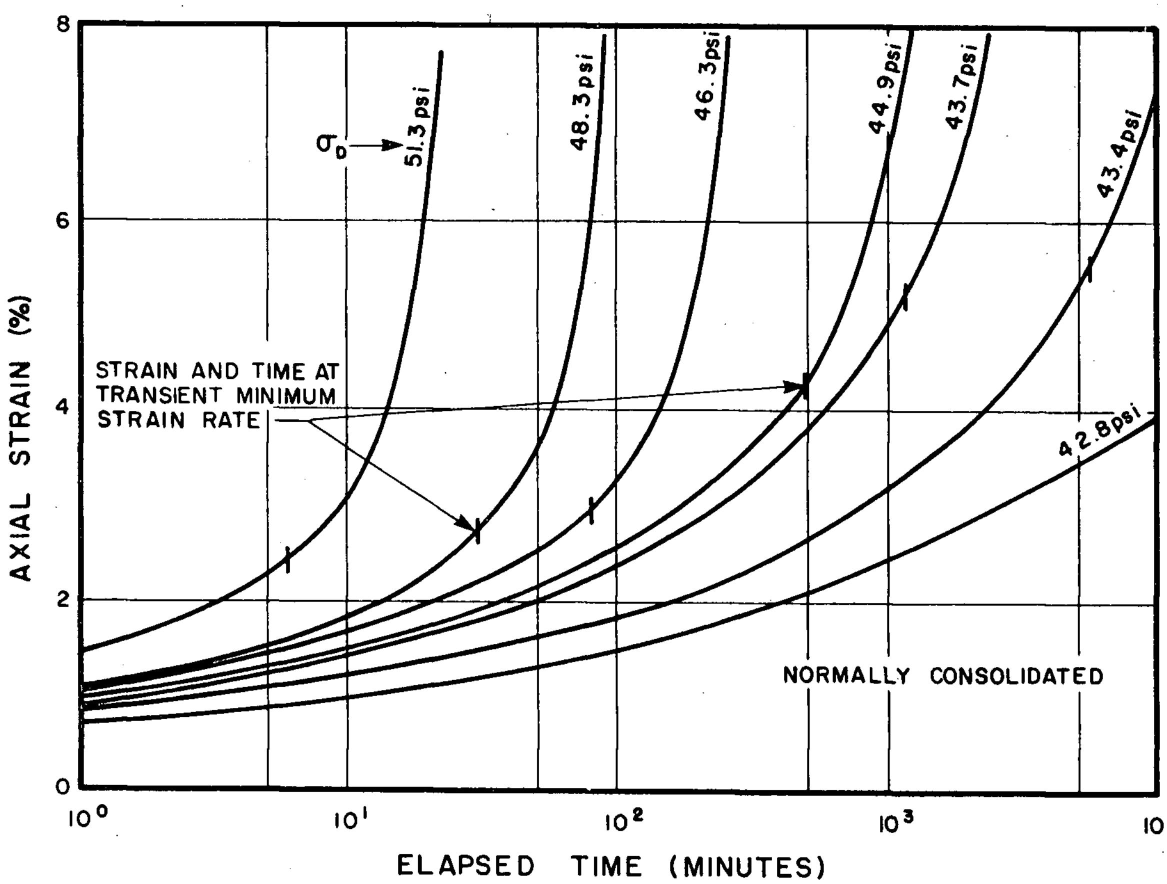


FIGURE 25 - STRAIN / TIME RESULTS
UNDRAINED CREEP OF HANEY CLAY

Figure 26 shows the log-log plot of strain rate against elapsed time. It is evident that the lower the deviator stress the longer the total rupture life. Based on these experimental results, it is seen that all transient minimum strain rates nearly lie on a straight line in this log-log plot. The equation of this line for normally consolidated specimens is:

$$\log_{10} t = -.142 - 1.15 \log_{10} \dot{\varepsilon}_m \pm .116 \qquad (8)$$

where: t = elapsed time until the transient minimum strain rate (in minutes)

$\dot{\varepsilon}_m$ = transient minimum strain rate (in per cent per minute)

The 95 per cent confidence limits of ± 0.116 are equal to twice the standard error of estimate (Arkin and Colton 1966) and the coefficient of correlation for this equation (defined as $1 - \frac{\text{standard error of estimate}}{\text{standard deviation}}$) is 0.942.

All strain rate/time curves start below the line of transient minimum strain rates and, if they proceed to it, failure will eventually take place. Otherwise, the sample will not fail in creep rupture. Unfortunately, this

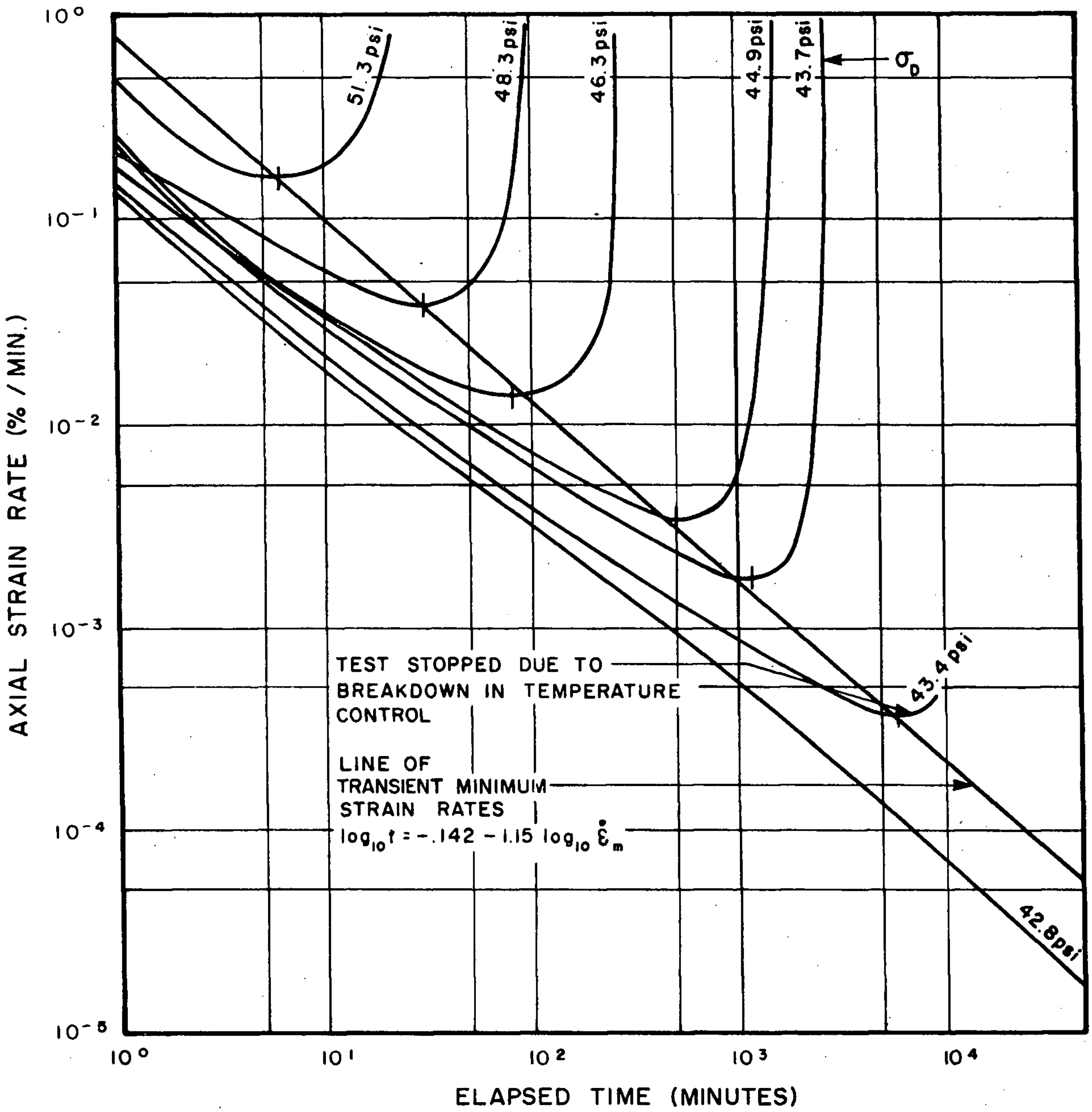


FIGURE 26 - STRAIN RATE / TIME RESULTS
UNDRAINED CREEP TESTS
NORMALLY CONSOLIDATED HANEY CLAY

criterion does not permit evaluation of the magnitude of strains which occur and these could be excessively large. In Figure 26, Test C-5, with a deviator stress of 42.8 psi, appears to be proceeding along a line parallel to the line of transient minimum strain rates in the latter stages of the test. It is unlikely, then, to ever reach the line of transient minimum strain rates and, therefore, it is reasonable to assume that this sample will not fail in creep at this stress level. This would indicate the existence of an upper yield strength for clays and its existence is in agreement with the hypotheses of Murayama and Shibata (1961) and Vialov and Skibitsky (1957), which state that an upper yield strength exists for soils, below which creep rupture failure will not occur. This upper yield compressive strength of 42.8 psi is 17.7 per cent below the reference compressive strength of 52.0 psi obtained from the incremental loading test.

For stresses below the upper yield strength, deformation rates will eventually become negligible after large elapsed times. This is based on the assumption that all test conditions remain the same; namely, total stresses, temperature and drainage conditions. Under field conditions, small changes in temperature or stresses in the soil will cause variations in the strain rate and, therefore, a slope known to be creeping very slowly, when loaded with additional weight from heavy rainfall or snowmelt, will certainly speed up and may actually fail.

Figure 27 shows the log-log plot of strain rate versus elapsed time for all creep tests on Haney clay with an overconsolidation ratio of two. Placed on this curve is the line of transient minimum strain rates obtained from the tests on normally consolidated clay (Equation 8). It is noted that the line of transient minimum strain rates fits these data, in addition to that of the normally consolidated tests; yet the consolidation history, water content and creep loadings are different. Test C-27 with a deviator stress of 38.9 psi does not appear to approach the line of transient minimum strain rates and, therefore, it is predicted that it would never fail. This deviator stress is 16.3 per cent below the reference strength of 46.5 psi obtained from the incremental loading test.

Figure 28 shows the log-log plot of strain rate against elapsed time for all creep tests on Haney clay with an overconsolidation ratio of six. The line of transient

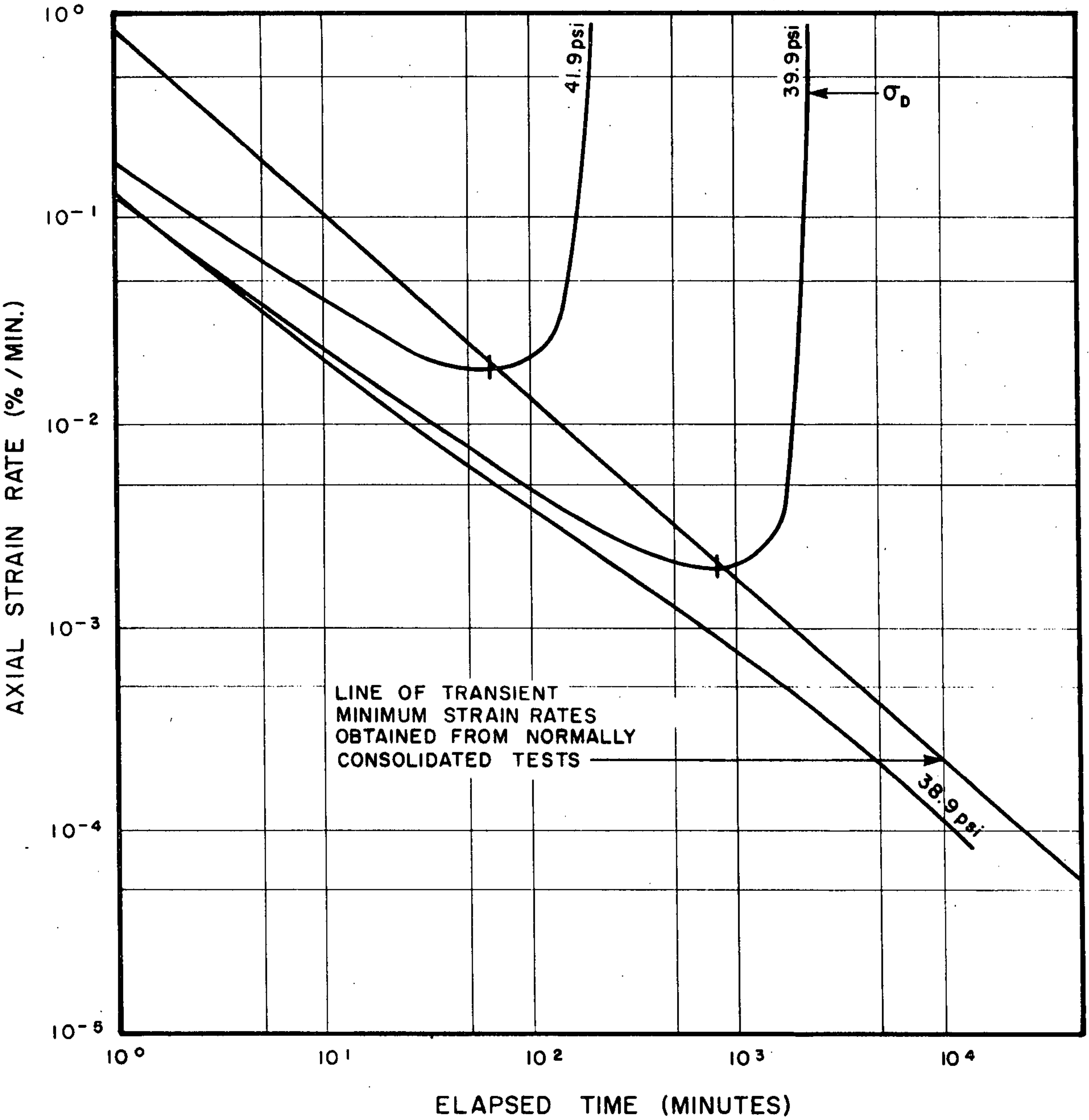


FIGURE 27 - STRAIN RATE / TIME RESULTS
UNDRAINED CREEP TESTS
OVERCONSOLIDATED HANEY CLAY ($\eta = 2$)

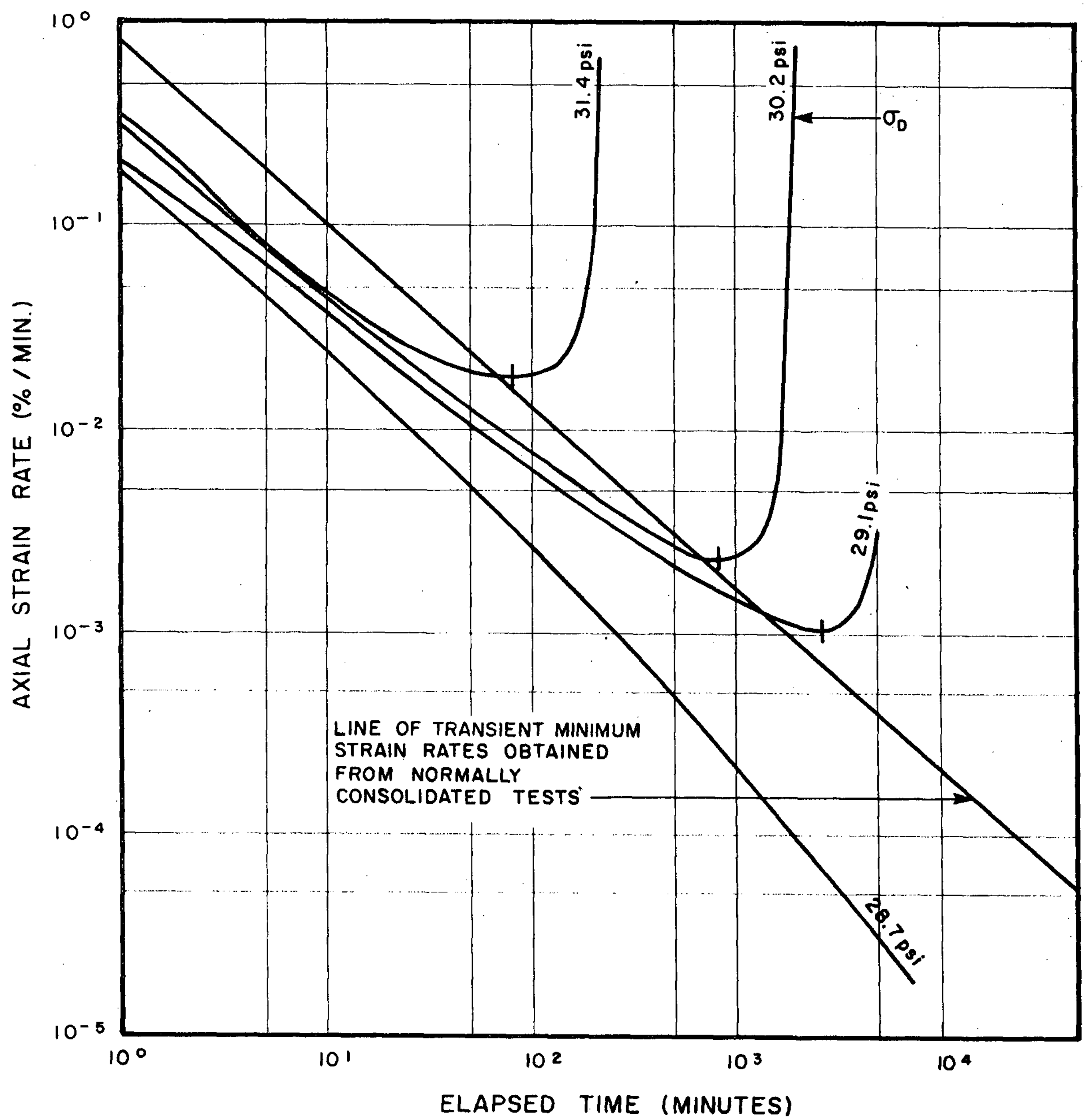


FIGURE 28 - STRAIN RATE / TIME RESULTS
UNDRAINED CREEP TESTS
OVERCONSOLIDATED HANEY CLAY (η=6)

minimum strain rates from normally consolidated tests is placed on this plot and, as for tests with an overconsolidation ratio of two, the line fits the test results with reasonable agreement. The test, with a deviator stress of 28.7 psi, is predicted to never fail and is at a stress level 14.6 per cent below the reference strength of 33.6 psi obtained from the incremental loading test.

Figure 29 shows the same relationship for the drained and undrained creep tests on Haney clay with an overconsolidation ratio of 25. Although there is a greater scatter of data from the line of transient minimum strain rates obtained from the normally consolidated tests, the relationship still appears to adequately represent the time at which the transient minimum strain rate occurs for these heavily overconsolidated samples. Some of the scatter of results about the line of transient minimum strain rates may be caused by inaccurate evaluation of strain rates, since they are calculated on the basis of uniform strain throughout the complete length of the sample. This assumption is reasonable for normally consolidated and lightly overconsolidated clays, but subject to error in heavily overconsolidated soils (Roscoe, Schofield and

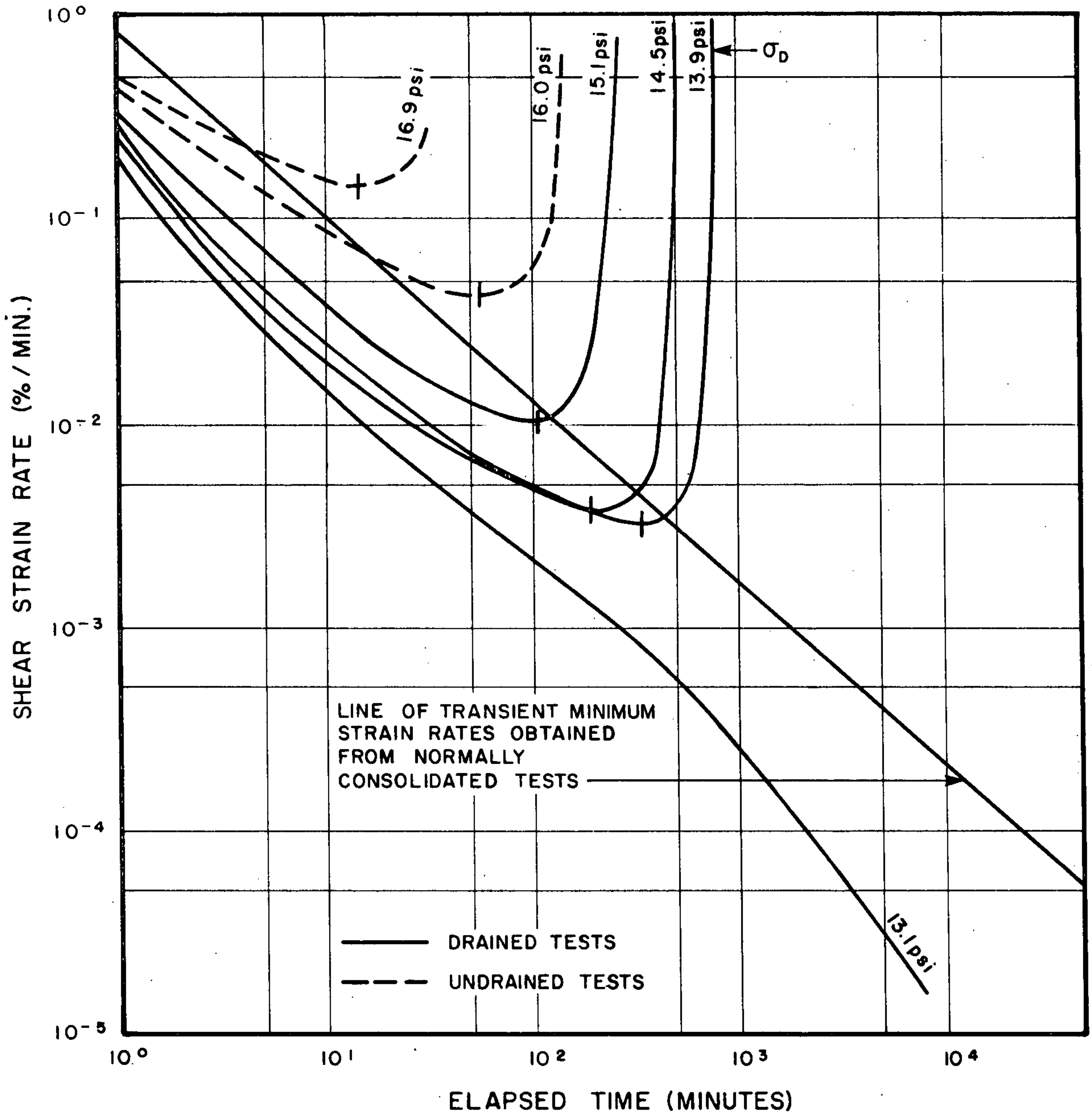


FIGURE 29 – STRAIN RATE / TIME RESULTS
UNDRAINED AND DRAINED CREEP TESTS
OVERCONSOLIDATED HANEY CLAY ($\eta = 25$)

Wroth 1958). For the drained tests, a decrease of 17.6 per cent is noted between the deviator stress which does not cause failure (13.1 psi) and the reference strength of 15.9 psi.

The agreement between the line of transient minimum strain rates from normally consolidated tests and those tests with different consolidation histories indicates that the same basic process governs the creep behaviour for all tests. It is, therefore, not surprising that the per cent reduction in strength between the reference strength and the upper yield strength is nearly the same (14.6 to 17.7 per cent) for all consolidation histories.

It is proposed that for Haney clay a line of transient minimum strain rates exists which is independent of consolidation history, stress level and drainage conditions, assuming that the total stresses remain constant. Figure 30 shows the transient minimum strain rates obtained by a least square fit of all the test results. The equation of this line is:

$$\log_{10} t = .037 - 1.08 \log_{10} \dot{\varepsilon}_m \pm .322 \qquad (9)$$

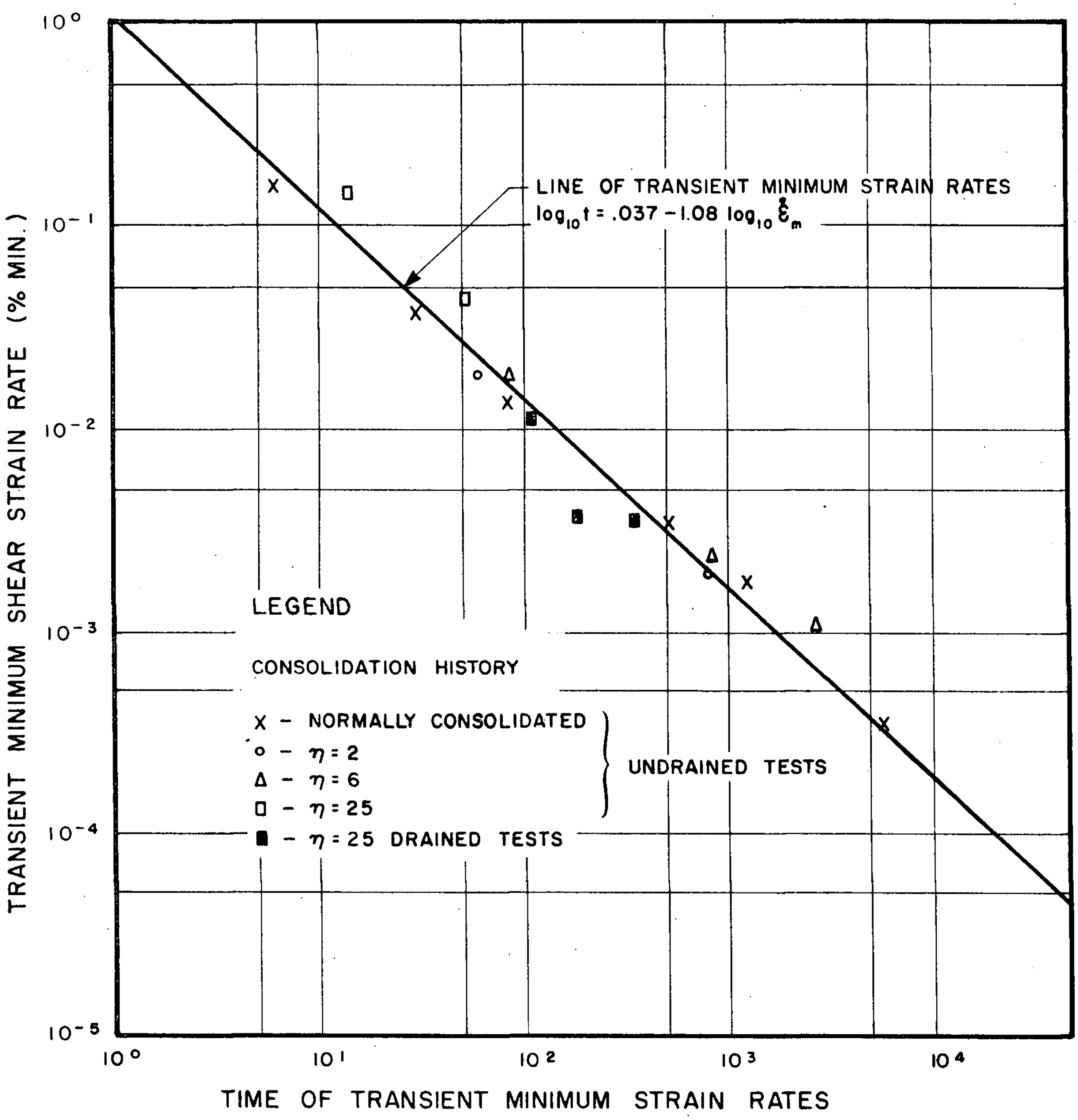


FIGURE 30 - LINE OF TRANSIENT MINIMUM STRAIN RATES HANEY CLAY

The slope of this line is nearly the same as in Equation 8 (the line of transient minimum strain rates obtained from only normally consolidated tests), but the ordinate when $\dot{\varepsilon}_m = 1.0$ is slightly greater. The 95 per cent confidence limits of Equation 9 are larger than for Equation 8, and the coefficient of correlation for Equation 9 is 0.819. Therefore, the line of transient minimum strain rates, which best fits all test data, does not correlate as well as the line of transient minimum strain rates obtained for the normally consolidated test series only. However, the accuracy of the fit is still considered adequate, such that the line of transient minimum strain rates (Equation 9) can be said to be independent of consolidation history, stress level and drainage conditions.

5.2 - Total Rupture Life Criterion

Saito and Uezawa (1961) proposed that a linear relationship exists between the logarithm of the "secondary strain rate" and the logarithm of the total rupture life, independent of the type of soil. As already discussed in Chapter 4, a "secondary strain rate" does not exist for Haney clay. However, for tests which eventually rupture, the "secondary strain rate" evaluated from a strain/time curve is nearly equal to the numerical value of the transient minimum strain rate. Therefore, to compare the results of creep rupture tests on Haney clay with the theory proposed by Saito and Uezawa, it is only necessary to plot the transient minimum strain rate against the total rupture life.

Figure 31 shows the rupture criterion of Saito and Uezawa, together with their 95 per cent confidence limits of ± 0.59. The equation for the relationship is:

$$\log_{10} t_r = .498 - .916 \log_{10} \dot{\varepsilon}_s \pm .59 \qquad (10)$$

where: t_r = total rupture life (in minutes)

$\dot{\varepsilon}_s$ = "secondary strain rate" (in per cent per minute)

Data shown falling inside these confidence limits include all creep rupture tests on Haney clay, one undrained creep rupture test on remoulded illite at 110 degrees F (Campanella 1965) and the results of Sherif (1965) for undrained tests on an overconsolidated Seattle clay.

The equation of the best straight line obtained from the test results for Haney clay is:

$$\log_{10} t_r = .751 - .92 \log_{10} \dot{\varepsilon}_m \pm .272 \qquad (11)$$

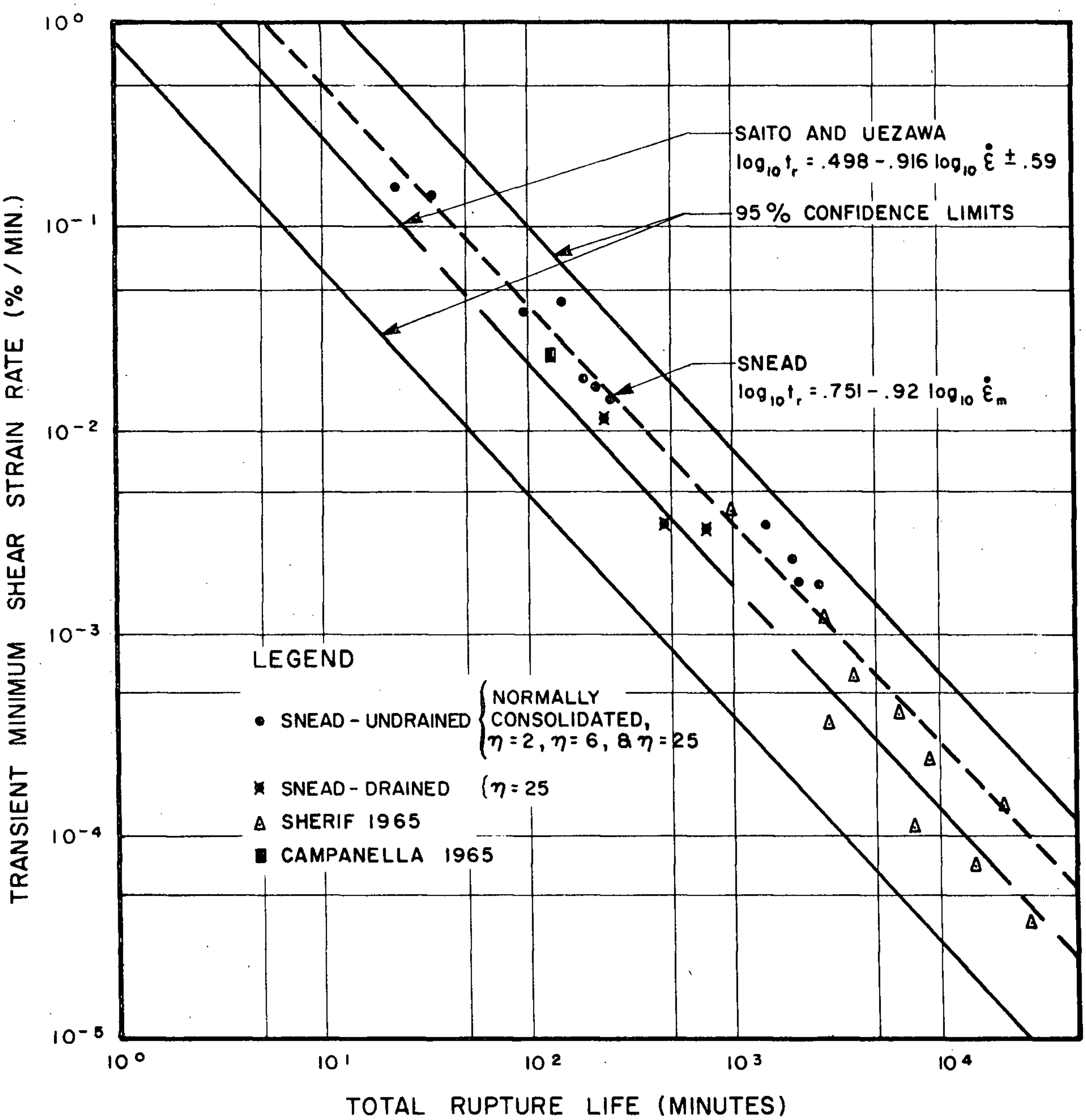


FIGURE 31 - TOTAL RUPTURE LIFE OF LABORATORY CREEP TESTS

The slope of this line is the same as that obtained by Saito and Uezawa, but the ordinate when $\dot{\varepsilon}_m = 1.0$ is slightly greater. The 95 per cent confidence limits for Haney clay are smaller than those obtained by Saito and Uezawa. Therefore, it is suggested that each soil may have its own relationship between the transient minimum strain rate and total rupture life which is similar to that of other soils. However, this relationship may not necessarily be unique for all soils as suggested by Saito and Uezawa.

Small variations in samples, which are inevitable, cause a considerable difference in rupture times. Therefore, the 95 per cent confidence limits for the rupture criterion obtained from laboratory studies are fairly large. For example, the Saito and Uezawa equation (Equation 10) predicts that the 95 per cent confidence limits of a predicted failure in 10 days range between 2-1/2 to 39 days. Although this is a large range in natural time, it at least allows evaluation of whether failure may take place in minutes, a few hours or many years. Likewise, the 95 per cent confidence limits of Equation 11 for a predicted failure in 10 days range between 5-1/2 and 19 days. Only with further research on additional soils will it be possible to determine whether each individual soil should be considered to have its own rupture equation or whether the scatter caused by sample variation creates such a wide error band that the inclusion of all soils into a single rupture equation, as proposed by Saito and Uezawa, is acceptable.

5.3 - Prediction of Creep Rupture Failures in the Laboratory

In order to predict the time of a creep rupture failure, it is necessary to determine some relationship between the strain rate measured during a test and the time remaining until creep rupture occurred. Therefore, it was decided to investigate the laboratory strain rates as a function of the time to rupture. At this point, it is necessary to emphasize the difference between the time to rupture and the total rupture life. The total rupture life is the elapsed time from initial loading until rupture, while the time to rupture is the remaining time to rupture from any stage of a test. Therefore, any rupture criteria based upon the time to rupture would not require the knowledge of the initial time of loading.

Figure 32 shows a plot of the logarithm of time to rupture against the logarithm of the strain rate for Test C-6. Note that with increasing elapsed time, the test progresses from right to left on this plot. There are two occasions when the strain rate has a value of 0.10 per cent per minute--once during the decreasing strain rate stage, 2,570 minutes before rupture, and once during the increasing strain rate stage, 150 minutes before rupture. Therefore, for this test, if the strain rate is known to be 0.01 per cent per minute, rupture is a <u>minimum</u> of 150 minutes away.

Figure 33 shows results from all creep rupture tests on Haney clay when the strain rate is increasing. Therefore, each point shown on Figure 33 is a reading taken after the transient minimum strain rate. It can be seen that for Haney clay there is a linear relation between the logarithm of the strain rate and the logarithm of the time to rupture, which is independent of stress level, consolidation history and drainage conditions. Therefore, knowing the existing accelerating strain rate in a test, a <u>minimum</u> time to rupture can be predicted. If the strain rate is decreasing, then no information regarding time to rupture is obtained and, in fact, the sample may not fail. As

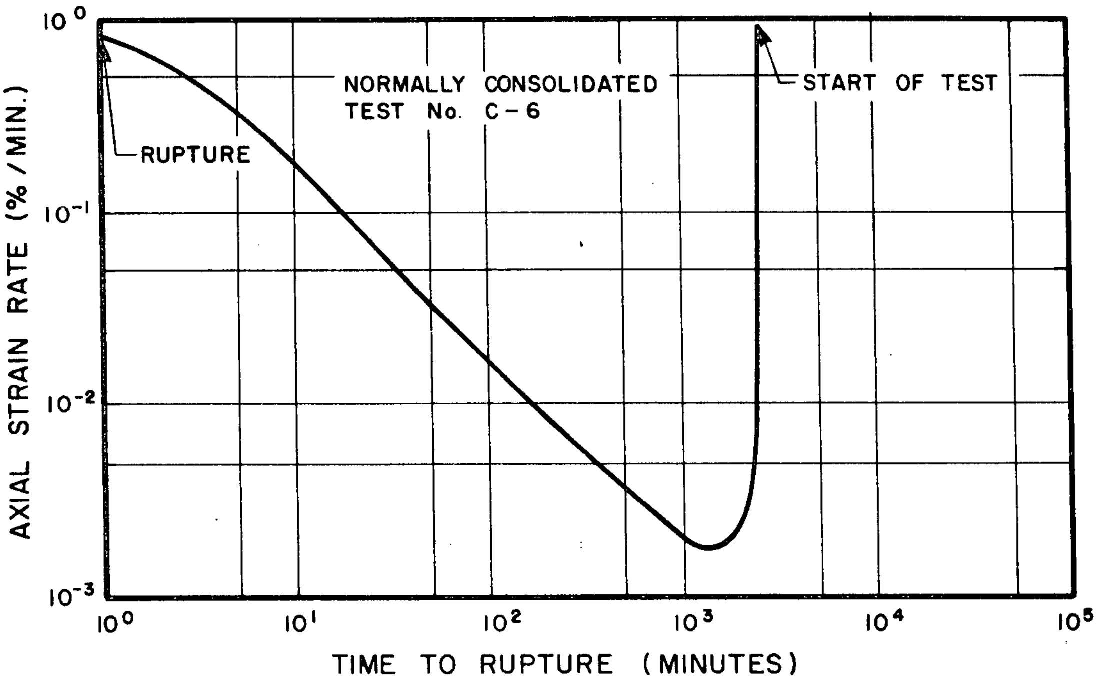


FIGURE 32 - TIME TO RUPTURE / STRAIN RATE CURVE

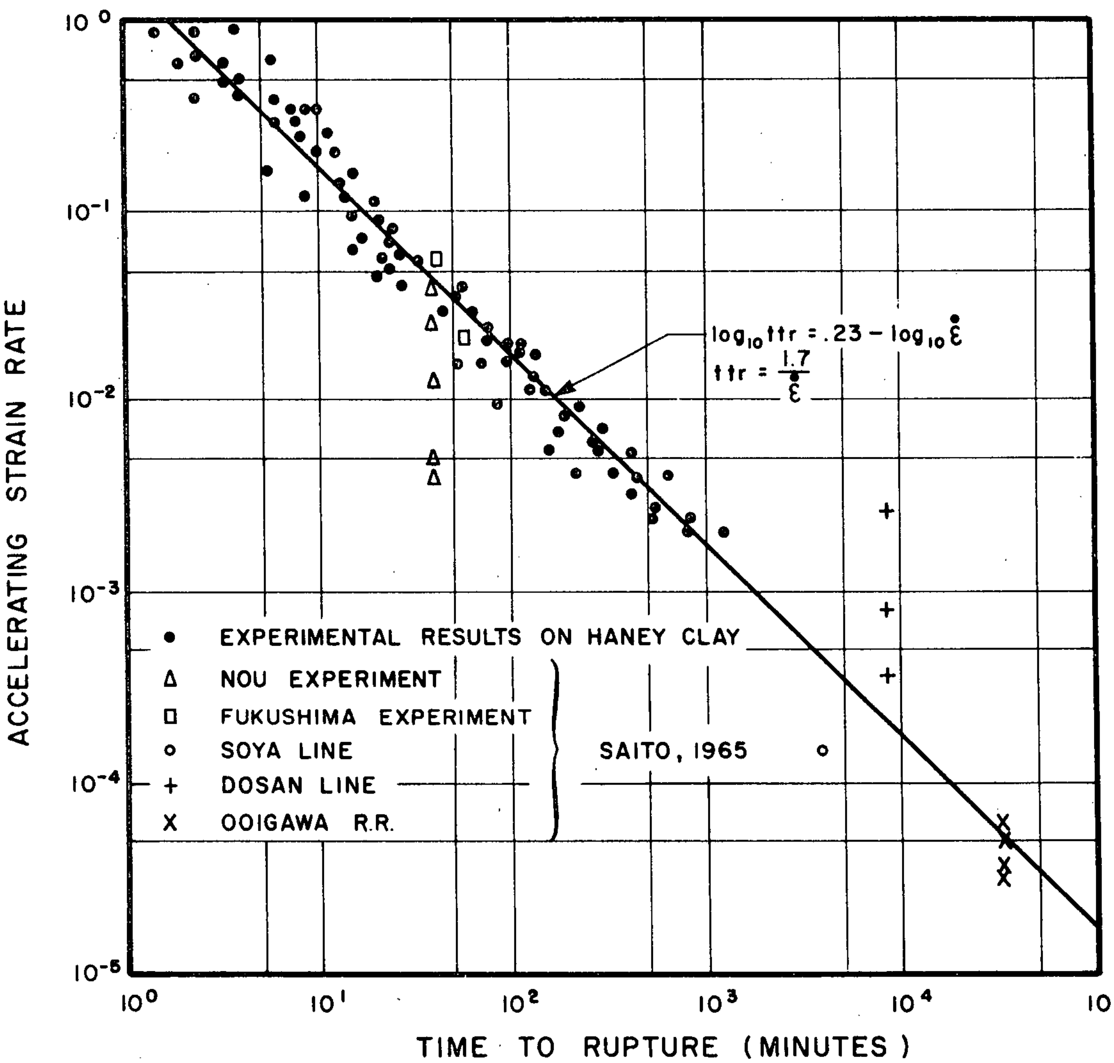


FIGURE 33 - TIME TO RUPTURE / STRAIN RATE RELATIONSHIP

pointed out in Chapter 4, until a transient minimum strain rate has been reached it is impossible to predict whether a failure will take place.

The equation of the relationship for Haney clay between time to rupture and accelerating strain rate is:

$$\log_{10} t_{tr} = .23 - \log_{10} \dot{\varepsilon} \qquad (12)$$

which can be reduced to:

$$t_{tr} = \frac{1.7}{\dot{\varepsilon}} \qquad (13)$$

where: t_{tr} = time to rupture from time t (in minutes)

$\dot{\varepsilon}$ = accelerating strain rate at time t (in per cent per minute)

If this strain rate/time to rupture relationship for Haney clay is applicable to other soils, then Equation 13 can be used to predict slope failures. This must be confirmed by extensive testing conducted on various soils throughout the world. Of course, the application of this relationship would require that the soil have a transient minimum strain rate when subjected to a creep loading above the upper yield strength.

In Chapter 2, reference was made to a method of forecasting slope failures by Saito (1965). This method, although being subject to criticism, has successfully predicted slope failures. Data from these slope failures have been plotted on Figure 33 and it can be seen that the time to rupture criterion for Haney clay would have adequately predicted impending failure. Saito's prediction of the time of failure was based on calculations from Equation 10, the use of which this author believes to be fundamentally incorrect. The prediction of the time to rupture must be based upon parameters which are independent of the initial time of loading. However, it should be noted that Equation 10 is numerically similar to Equation 12 and, therefore, this similarity may explain Saito's success.

5.4 - Prediction of Creep Rupture Failures in the Field

In Section 5.3, a relationship between the time to rupture and the accelerating strain rate has been presented which permits the prediction of the time of creep rupture for laboratory creep tests on Haney clay. Limited field results indicate that this relationship would predict failures observed and reported by Saito (1965). This relationship should, therefore, be investigated, both in the laboratory and the field, to determine if this simple relationship is independent of soil type, consolidation history, drainage conditions and stress level. The following discussion will assume that such an all-encompassing relationship exists. The following procedure could then be used to predict the time when a slope failure might occur.

By monitoring the deformations of a slope and converting these results into strain rates, it would be possible to continually predict a minimum length of time until a rupture might be expected. If the strain rates increased due to heavy rainfall, for example, the predicted time to rupture would decrease and, if this time to rupture dropped below a critical value, say 24 hours, then the affected area could be closed off to protect the public. If the strain rates later decreased, without a failure occurring, then the predicted time to rupture would again increase and the affected area could once again be opened to the public.

Subject to further research to confirm the hypothesis presented in this section, a general practical approach to the prediction of slope failures has been proposed. As already discussed, detectable movements are known to take place long before some failures occur and, if monitored, these movements can be used to predict the time of the failure.

CHAPTER 6

STRESS/STRAIN/STRAIN RATE THEORY

6.1 - Introduction

In Chapter 5, the existence of an upper yield strength for Haney clay was shown. This upper yield strength was evaluated by performing a series of creep rupture tests, each test at a decreased deviator stress, until a failure did not take place. It was assumed that failure would not occur if the logarithm of strain rate/logarithm of time curve did not appear to approach the line of transient minimum strain rates. Since performance of a series of creep tests is a very tedious and time-consuming procedure, it is of interest to determine whether equivalent information can be obtained by some easier means. For example, could constant strain rate tests be used to evaluate the upper yield strength? These tests are easier to perform, since the time at which readings should be taken can be predetermined and corrections to axial loads are not required to maintain constant stress conditions. If correlation of the results obtained from creep tests and constant strain rate tests is to be attempted, a hypothesis must be presented to predict the interrelation between these tests.

6.2 - Deviator Stress/Strain/ Strain Rate Relationship

In an attempt to reduce the large number of variables which affect the stress/strain behaviour of soils, and thereby permit a possible correlation of results from different types of triaxial tests, the following limitations will apply to all discussions in this section:

(1) - Samples must not be heavily overconsolidated. This will ensure that strain measurements are representative of the actual strains throughout the sample (Roscoe, Schofield and Wroth 1958).

(2) - All tests must be undrained. This will eliminate effects due to volume change for which no theory is proposed in this thesis.

(3) - The duration of the test must be sufficient to ensure pore water pressure equalization throughout the sample.

(4) - All tests must be performed in such a manner that the axial compressive strain is continually increasing and, therefore, the effect of strain reversals will not be discussed.

(5) - The initial consolidation history must be the same for all samples.

(6) - As previously stated, all tests must be performed at constant temperature.

These restrictions, therefore, exclude from discussion all tests on heavily overconsolidated clay, but include the results of all normally consolidated and overconsolidated (η =2 and η =6) undrained creep rupture, incremental loading and constant strain rate tests.

Within the limitations already presented, it is proposed that the deviator stress, during triaxial tests on samples with the same initial consolidation history, can be expressed as a function of the current strain and current strain rate.

To investigate the validity of this hypothesis, the results of creep rupture, incremental loading and constant strain rate tests on Haney clay are shown in Figures 34, 35 and 36. Plotted on these figures are the strain rate/strain paths for the normally consolidated, overconsolidated (η =2) and overconsolidated (η =6) triaxial tests, respectively. On the paths of the constant strain

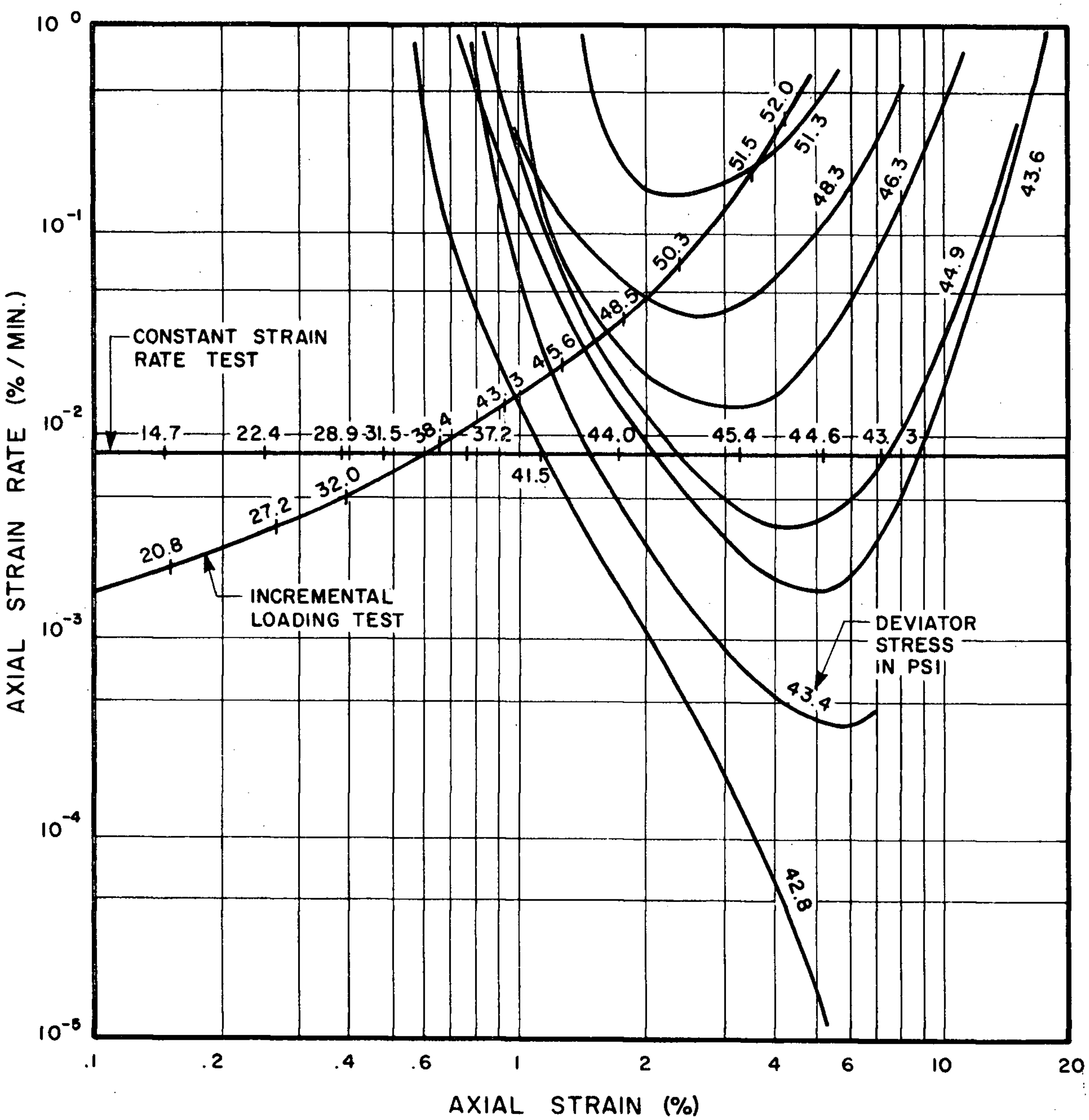


FIGURE 34 - STRAIN RATE / STRAIN CURVES
NORMALLY CONSOLIDATED HANEY CLAY

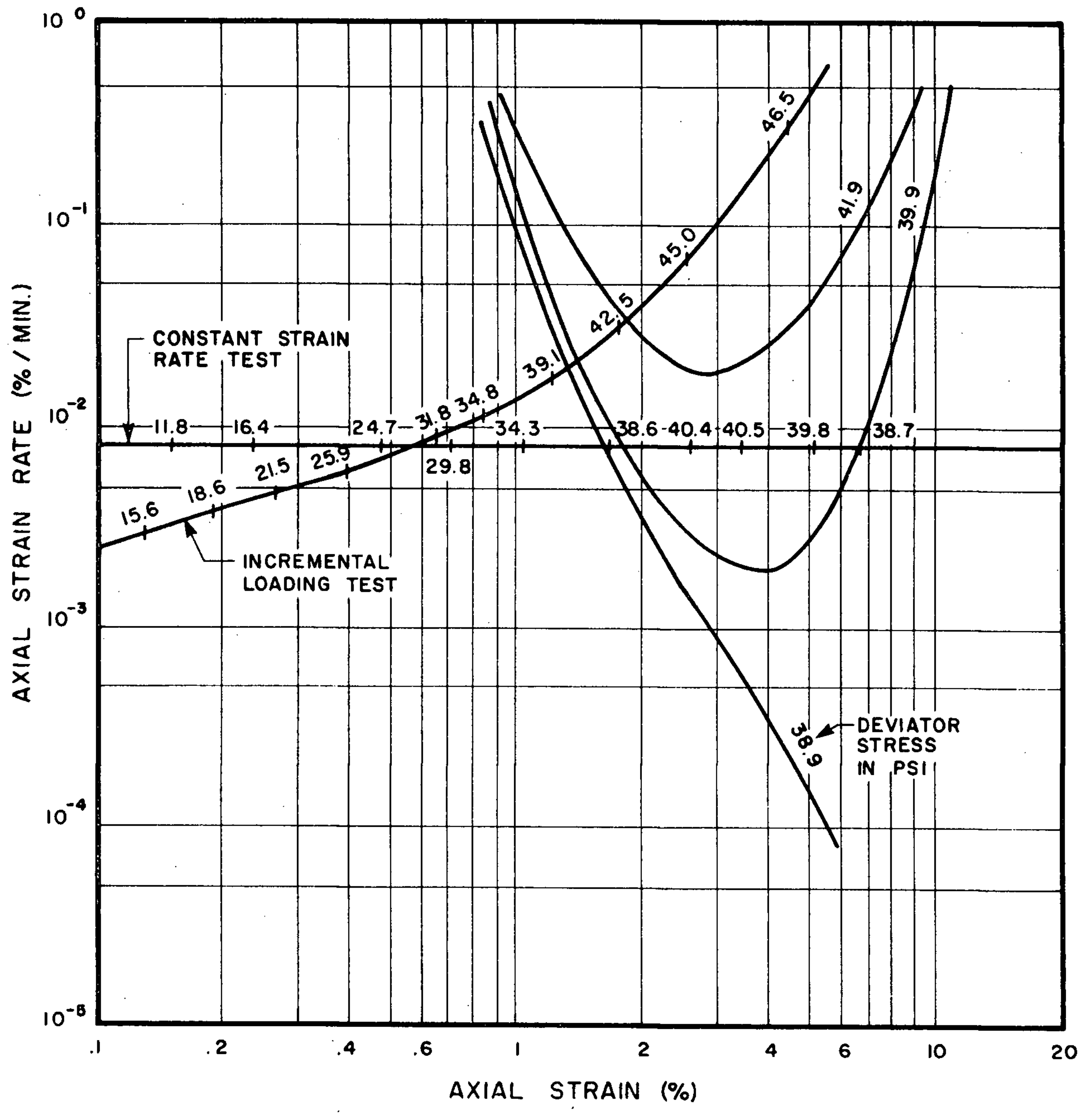


FIGURE 35 - STRAIN RATE / STRAIN CURVES OVERCONSOLIDATED HANEY CLAY ($\eta = 2$)

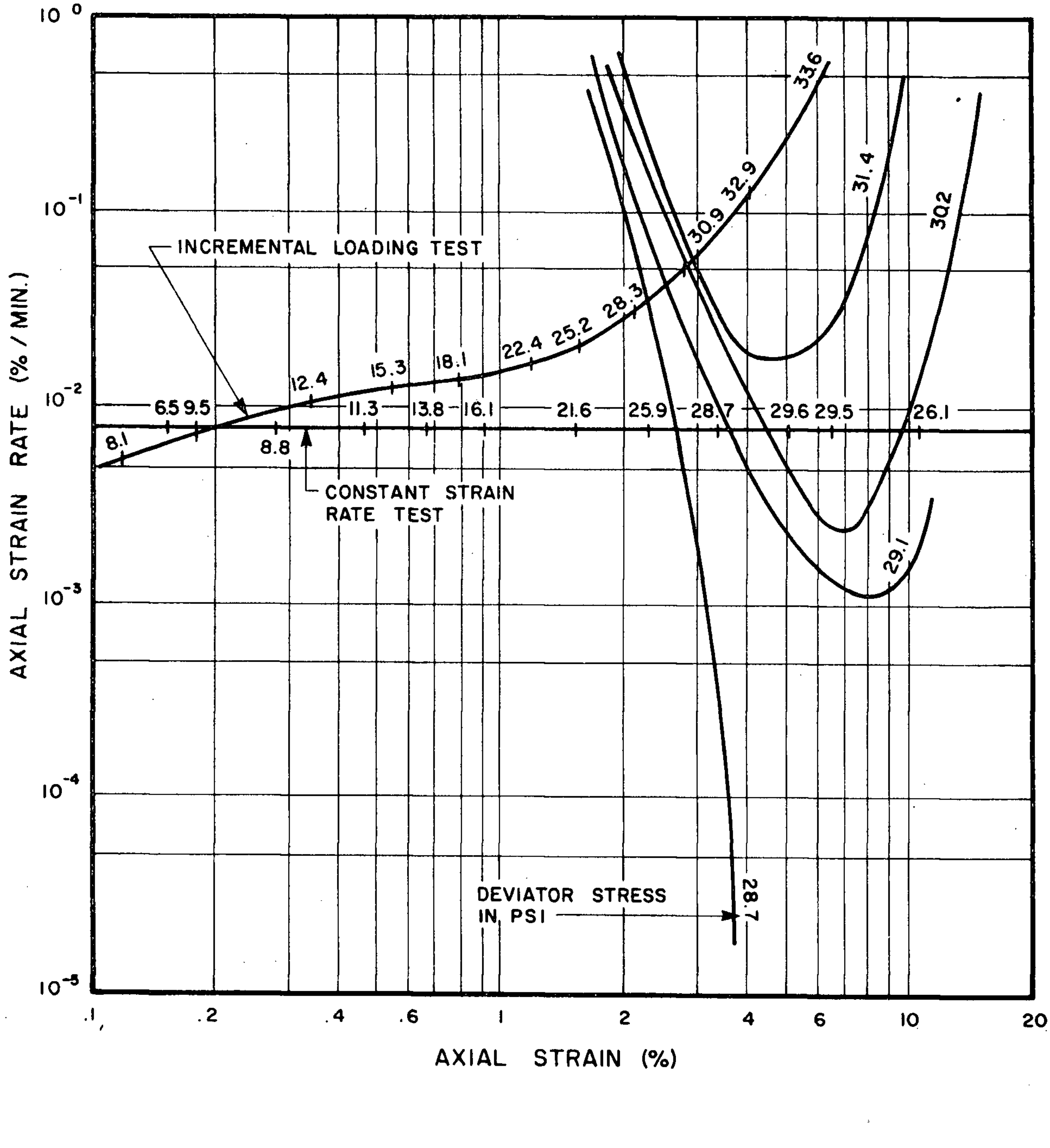


FIGURE 36 – STRAIN RATE / STRAIN CURVES OVERCONSOLIDATED HANEY CLAY ($\eta=6$)

rate and incremental loading tests, the values of deviator stress mobilized at that strain and strain rate are shown, while for the creep tests the constant deviator stress is shown on the creep curve. For the hypothesis to be valid, the deviator stress at any strain and strain rate must be unique and independent of the type of test. Investigation of these results indicates reasonably good agreement between the three types of tests. This relationship has, at present, only been observed to apply to Haney clay. It is hoped that similar research will be undertaken by others to determine whether this applies to most undisturbed clays.

No attempt has been made in this investigation to evaluate a mathematical relationship between stress, strain and strain rate, but rather a discussion of the interrelation of these parameters is presented to enable evaluation of the behaviour of one type of test based upon the results of another type. For example, Figures 37, 38 and 39 show the actual stress/strain curves obtained for incremental loading and constant strain rate tests on Haney clay. Plotted on these figures are points predicted from the creep rupture tests for both the incremental loading and constant strain rate tests. It should be noted that, for a constant strain

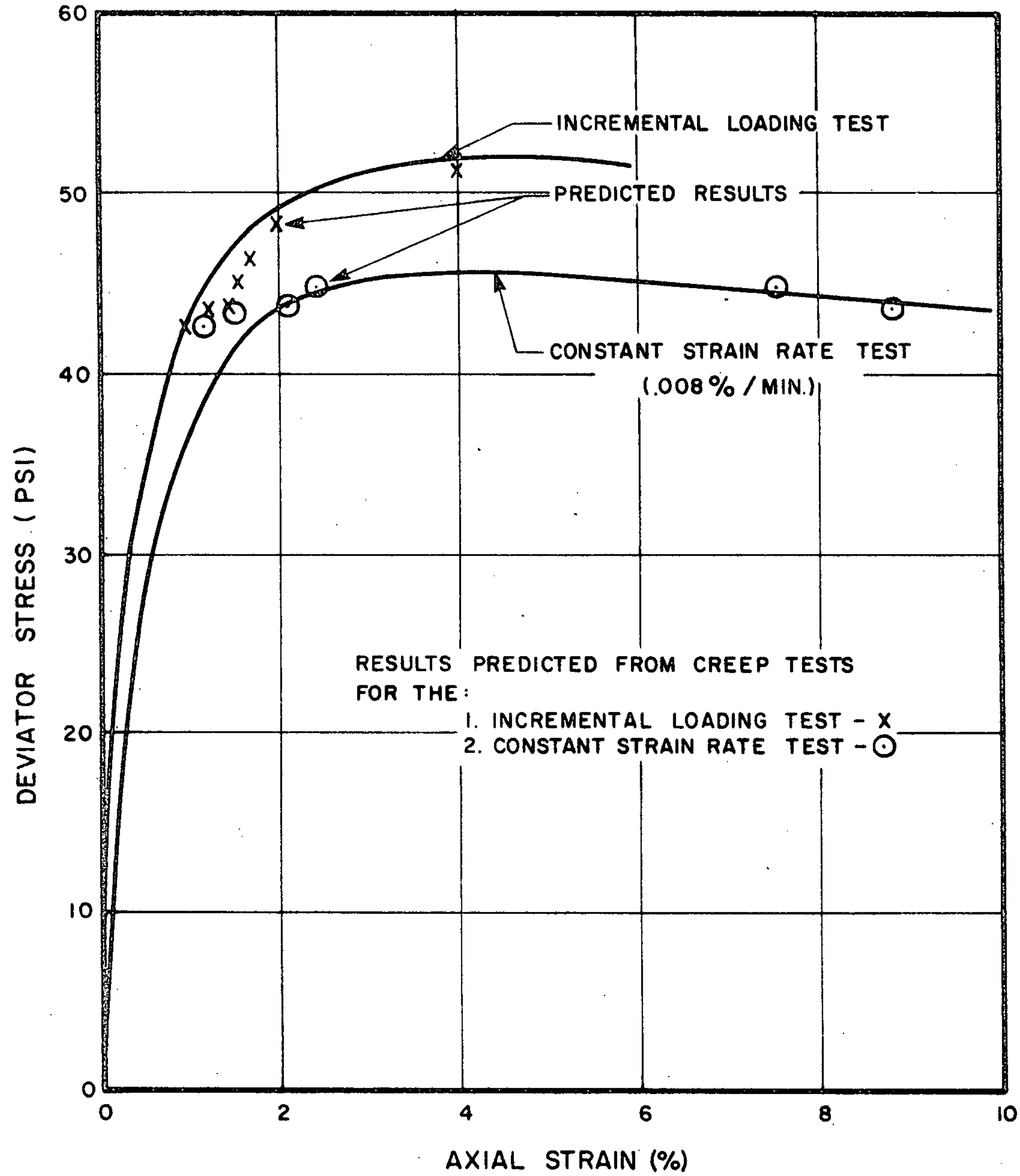


FIGURE 37 - PREDICTED AND ACTUAL STRESS/STRAIN CURVES FOR NORMALLY CONSOLIDATED HANEY CLAY

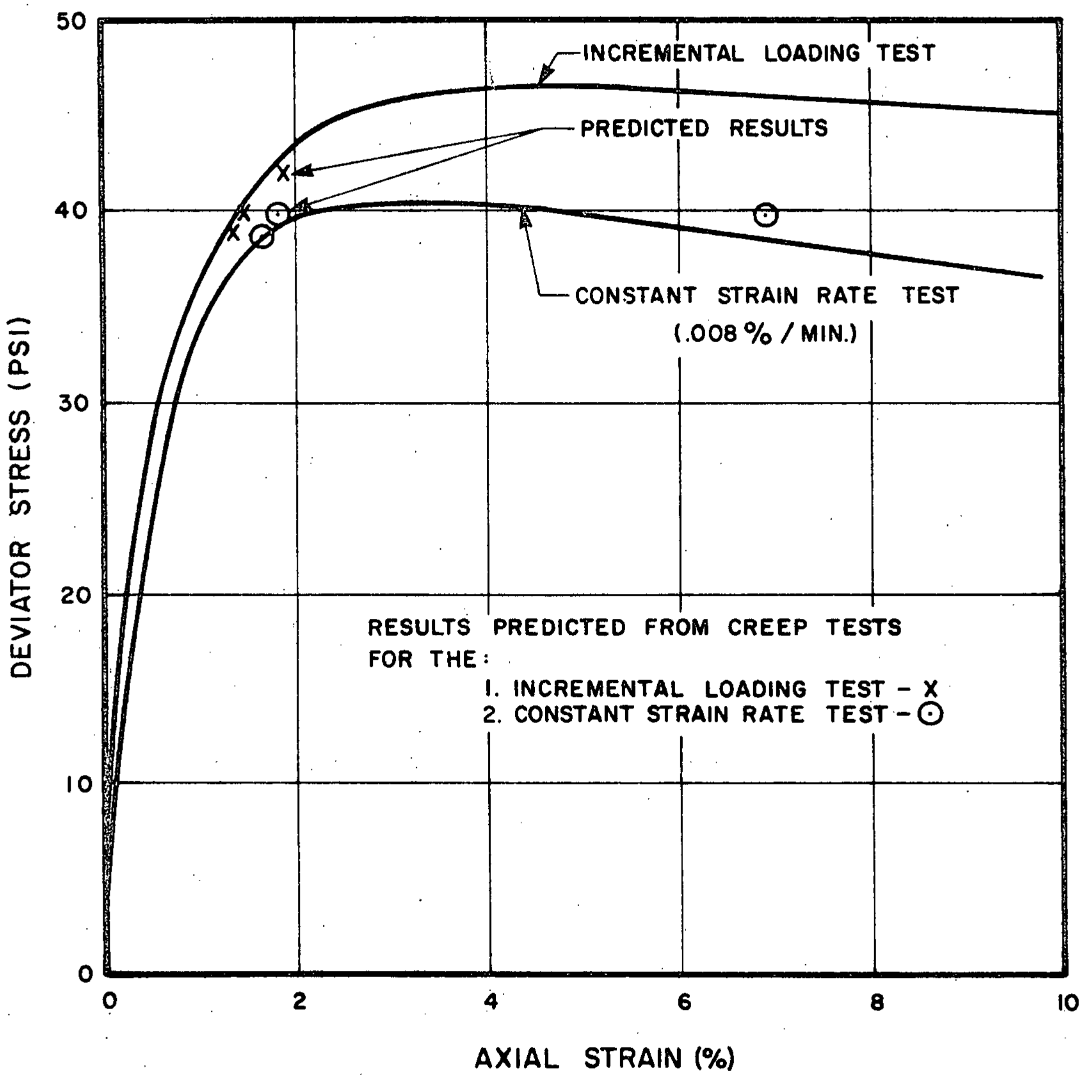


FIGURE 38 - PREDICTED AND ACTUAL STRESS/STRAIN CURVES FOR OVERCONSOLIDATED HANEY CLAY ($\eta = 2$)

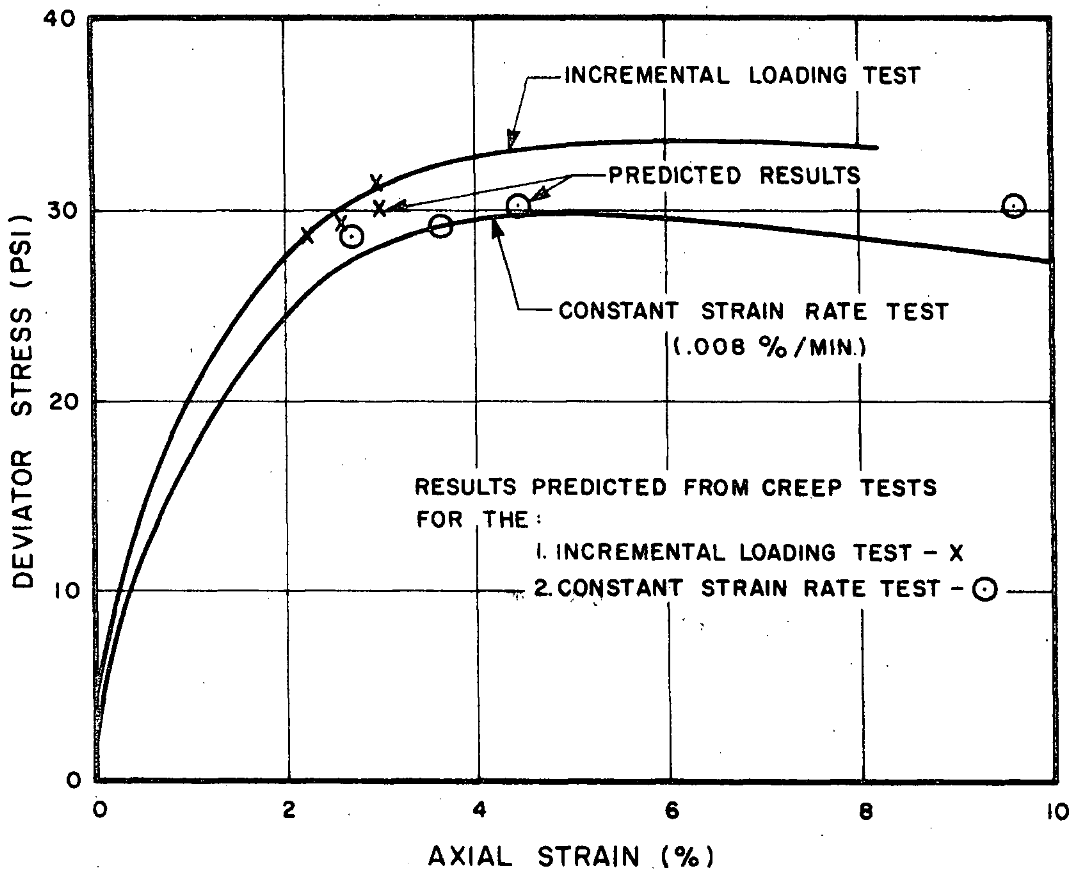


FIGURE 39 - PREDICTED AND ACTUAL STRESS / STRAIN CURVES FOR OVERCONSOLIDATED HANEY CLAY ($\eta = 6$)

rate test, the stress/strain curve can be predicted directly from a series of creep tests, since the strain rate/strain path is known; whereas, for incremental loading tests, neither the stress, strain nor strain rate remains constant and, therefore, prediction of the stress/strain curve can only be attemtped once the strain rate/strain path is known.

Consider now, the results of a constant strain rate test performed at a strain rate equal to the transient minimum strain rate of a creep rupture test. As a result of the relationship between deviator stress, strain and strain rate, the maximum deviator stress of the constant strain rate test should be equal to the deviator stress applied in the creep test (see Figures 34, 35 and 36) and should occur at the same strain as the transient minimum strain rate of the creep rupture test. This is a most important prediction, since it explains the interrelation between the transient minimum strain rates obtained from creep tests and the maximum deviator stresses obtained from constant strain rate tests.

The stress/strain/strain rate relationship also allows prediction of creep behaviour on the basis of the shape of stress/strain curves from constant strain rate tests. For instance, if the deviator stress continues to increase or levels off without decreasing with increasing strain, then a creep test would not reach a transient minimum strain rate, since this would be incompatible with the results of the constant strain rate test. Therefore, for soils which conform to this stress/strain/strain rate relationship and continually strain harden during constant strain rate tests, it is predicted that a creep rupture failure will not occur. However, any soil for which a maximum deviator stress is reached, followed by a decrease in the deviator stress with further strain would be expected to fail in a creep test if subjected to the maximum deviator stress of the constant strain rate test.

As reported in Chapter 4, the maximum deviator stress of incremental loading tests was about 15 per cent above the upper yield strength of Haney clay. The maximum deviator stress of the incremental loading test is mobilized at about the same strain as the minimum strain rate in creep tests and, therefore, the difference in strength can be related to the difference in strain rates at failure. At failure, the strain rate for the incremental loading test is very large, while the transient minimum strain rate for the upper yield strength is theoretically zero. The difference in strain rate at failure also explains the reason why the maximum deviator stress from the incremental loading test is larger than that obtained from the constant strain rate tests reported in this thesis.

6.3 - Evaluation of the Upper Yield Strength

In Chapter 5, the upper yield strength was determined by performing a series of creep rupture tests at decreasing stress levels until a failure did not occur. This procedure may require many tests before the upper yield strength is determined and, therefore, a method is proposed which enables the prediction of the upper yield strength based upon a minimum of two creep rupture or two constant strain rate tests. The proposed relation used to determine the upper yield strength is as follows:

$$\sigma_D = U.Y. + K\,\dot{\varepsilon}^{1/n} \qquad (14)$$

where: σ_D = maximum deviator stress (constant strain rate test) or deviator stress (creep test)

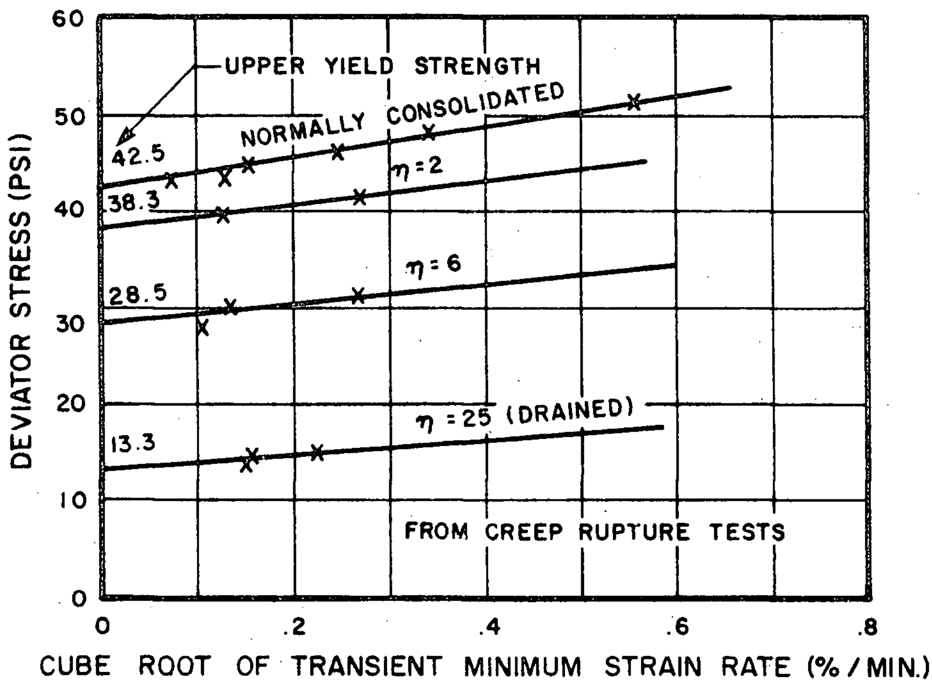
U.Y. = upper yield strength

K, n = constants

$\dot{\varepsilon}$ = strain rate (constant strain rate test) or transient minimum strain rate (creep test)

For Haney clay, it has been found that taking $n = 3$ gives a reasonably good straight line relation between σ_D and the cube root of $\dot{\varepsilon}$. Figure 40 shows the results of both creep rupture and strain rate controlled tests used to predict the upper yield strength of Haney clay for four consolidation histories. Twelve of the constant strain rate tests shown in Figure 40 were run on 1.4-inch by 2.8-inch samples. The maximum deviator stress in these tests has been reduced by 1.2 psi, the suggested correction to allow for the difference in side drain and membrane effects between 1.4-inch by 2.8-inch samples and 2.5-inch by 5.0-inch samples (Bishop and Henkel 1962, Duncan and Seed 1965). Table VII lists the upper yield strengths obtained from the creep tests and the constant strain rate tests. It can be seen that both the creep rupture and constant strain rate tests predict essentially the same upper yield strength for similar drainage and consolidation histories.

Sherif (1965) suggested that a flow rule existed for clays, which predicted that the deviator stress in a



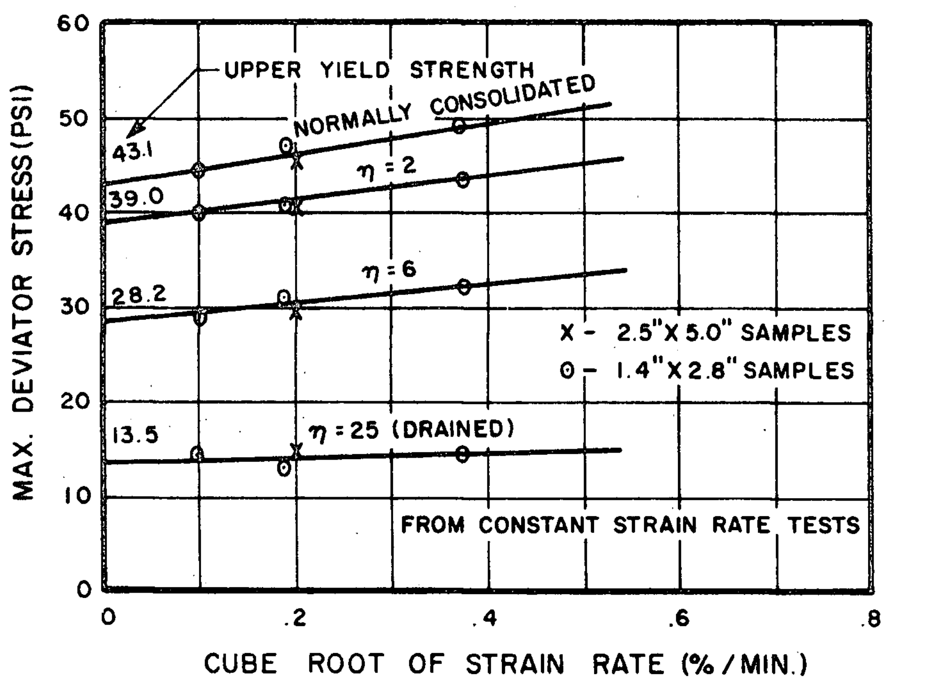


FIGURE 40 - DETERMINATION OF UPPER YIELD STRENGTH USING PROPOSED METHOD

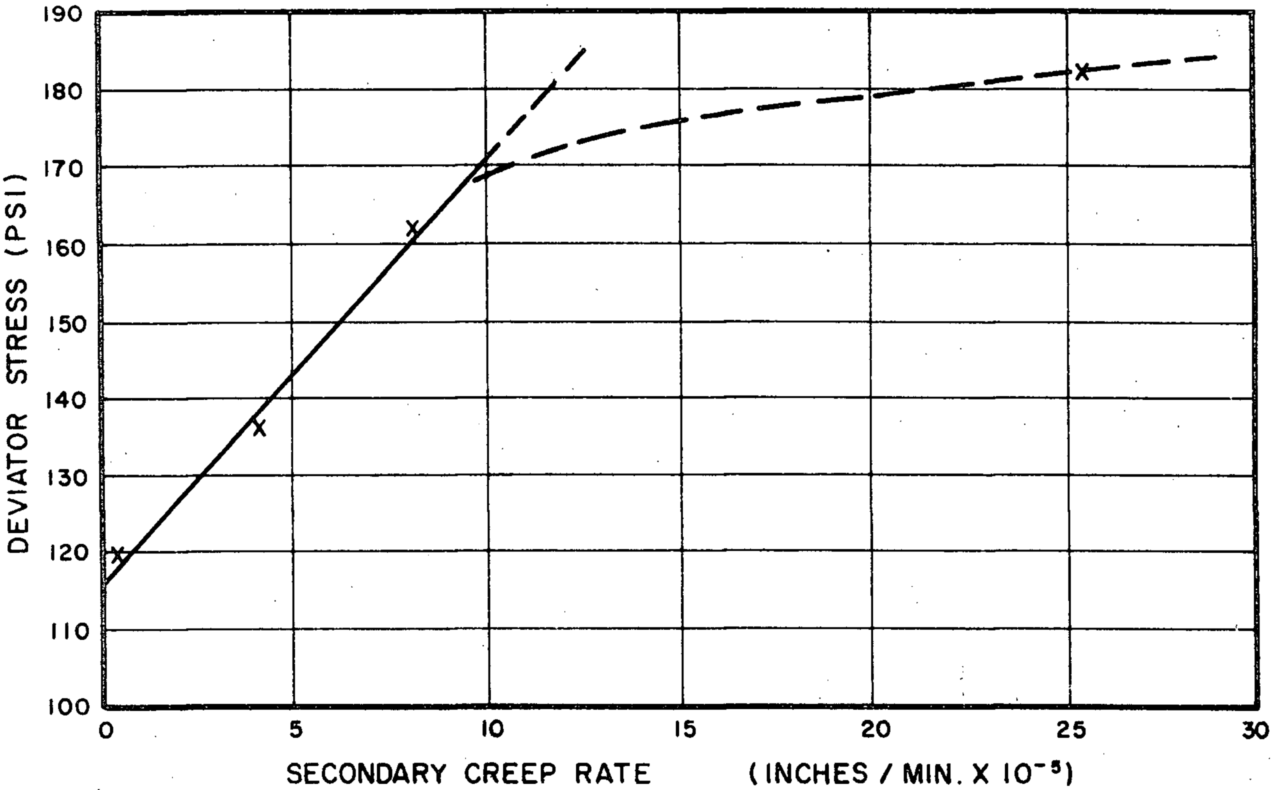
TABLE VII

Upper Yield Strength for Haney Clay

Consolidation History	Condition	Upper Yield Strength: Obtained from Creep Rupture Tests (psi)	Upper Yield Strength: Obtained from Constant Strain Rate Tests (psi)
Normally consolidated	Undrained	42.5	43.4
$\eta = 2$	Undrained	38.5	38.8
$\eta = 6$	Undrained	28.6	28.7
$\eta = 25$	Drained	13.5	13.1

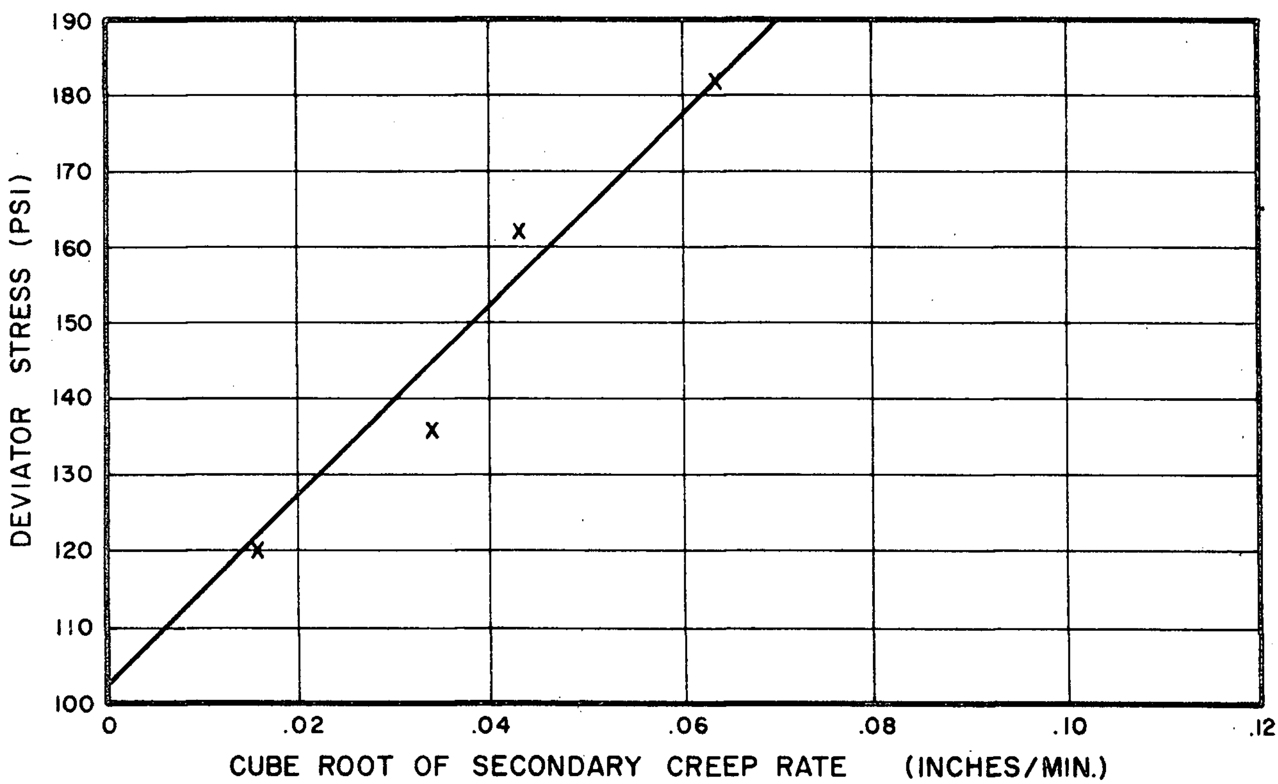
creep test was equal to the upper yield strength plus a constant times the secondary creep rate. This is equivalent to Equation 14 with $n = 1$ and predicts a straight line relation between deviator stress and strain rate. Sherif postulated that this relation holds for stress levels slightly above the upper yield strength, but is invalid at higher stress levels. Figure 41 shows Sherif's results for a Seattle clay. The three creep tests at the lower stress levels are shown to lie on a straight line. The projection of this line to zero strain rate gives the upper yield strength. The creep test at the highest stress level is shown to be considerably off the straight line. In Figure 42, all four test points are shown to approximately define a straight line when plotted against the cube root of the strain rate. Therefore, the upper yield strength for Seattle clay can be predicted equally well by Equation 14 with $n = 3$. Future research on other soils will, hopefully, provide proof of the applicability of Equation 14 with $n = 3$ for the determination of upper yield strengths.

Murayama and Shibata (1961) proposed that a linear relation exists between $\frac{d\varepsilon}{d \log t}$ and σ up to the upper yield strength. The strain/time equation for their rheological



(AFTER SHERIF, 1965)

FIGURE 41 - STRESS VERSUS SECONDARY CREEP RATE



(AFTER SHERIF, 1965)

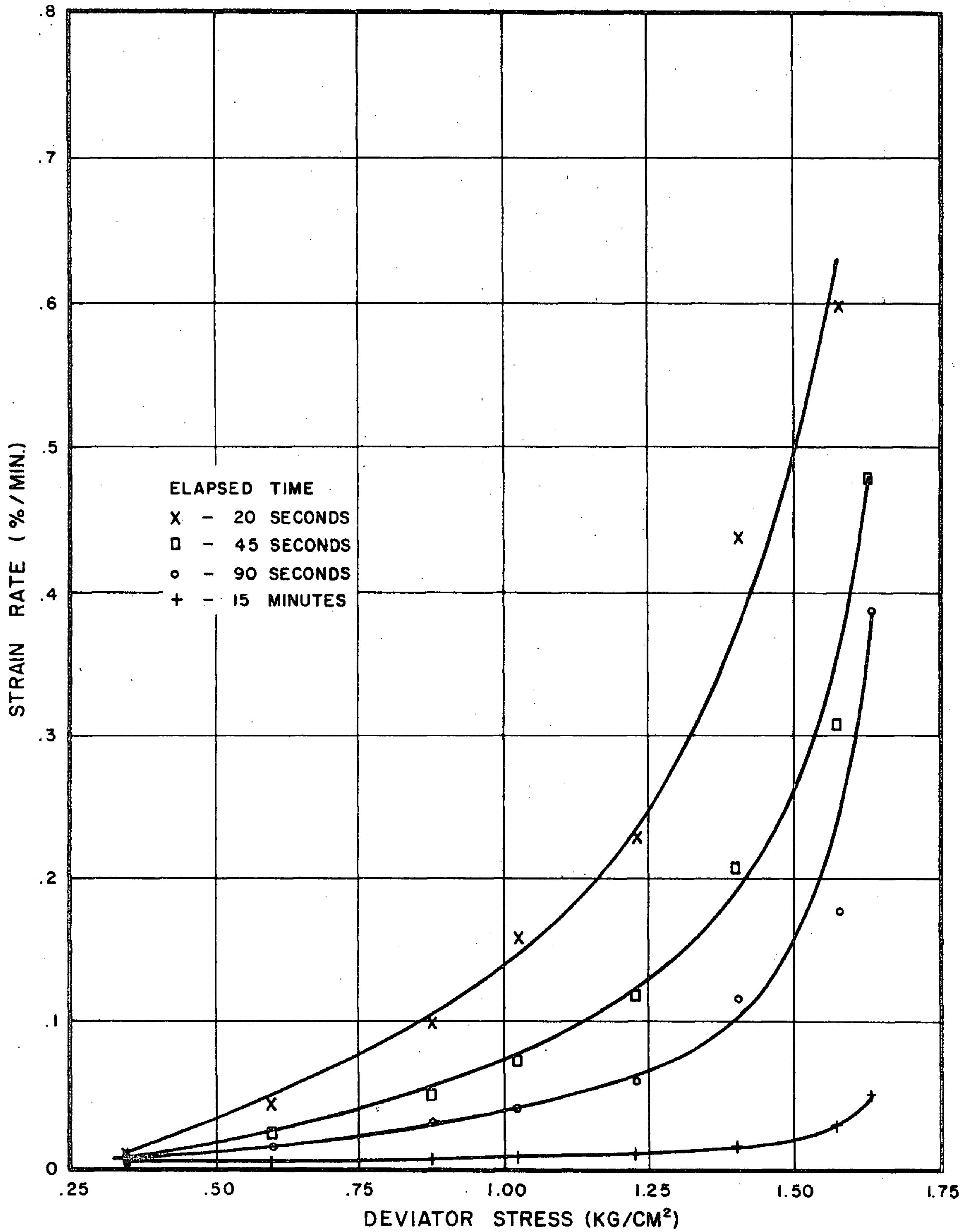
FIGURE 42 - STRESS VERSUS CUBE ROOT OF SECONDARY CREEP RATE

model is presented again for convenience:

$$\varepsilon = \frac{\sigma}{E_1} + \frac{\sigma - \sigma_o}{E_2} + \frac{\sigma - \sigma_o}{B_2 E_2} \log At \tag{6}$$

Murayama and Shibata (1964) differentiated Equation 6 by time instead of the logarithm of time and thereby predicted a linear relation between $d\varepsilon/dt$ and σ for constant elapsed times and for stresses below the upper yield strength. This method, proposed by Murayama and Shibata, for obtaining the upper yield strength cannot be checked for Haney clay, since nearly all creep tests on Haney clay were run at stress levels above the upper yield strength. However, results of Campanella (1965) for the undrained creep of illite are at stress levels below the upper yield strength and, as a result, this data can be used to check the Murayama and Shibata theory. Figure 43 shows the strain rate at constant elapsed times against the stress level. The strain rate/stress results are not linear, but curved, and no sharp discontinuity in the curve exists to permit evaluation of the upper yield strength. However, the maximum stress level is above the upper yield strength, since a creep rupture failure occurred for the test with σ_D = 1.63 kg/cm^2.

Murayama and Shibata (1964) also proposed that the upper yield strength could be obtained from an incremental



(AFTER CAMPANELLA, 1965)

FIGURE 43 - RESULTS OF UNDRAINED CREEP OF ILLITE

stress test. Figure 44 shows the results of four incremental tests on Haney clay which indicate that no sharp discontinuity exists in the $\log \sigma / \log \varepsilon$ curve to permit evaluation of the upper yield strength as proposed by Murayama and Shibata. It must, therefore, be assumed that the Murayama and Shibata methods for determination of the upper yield strength are not applicable for Haney clay or remoulded illite.

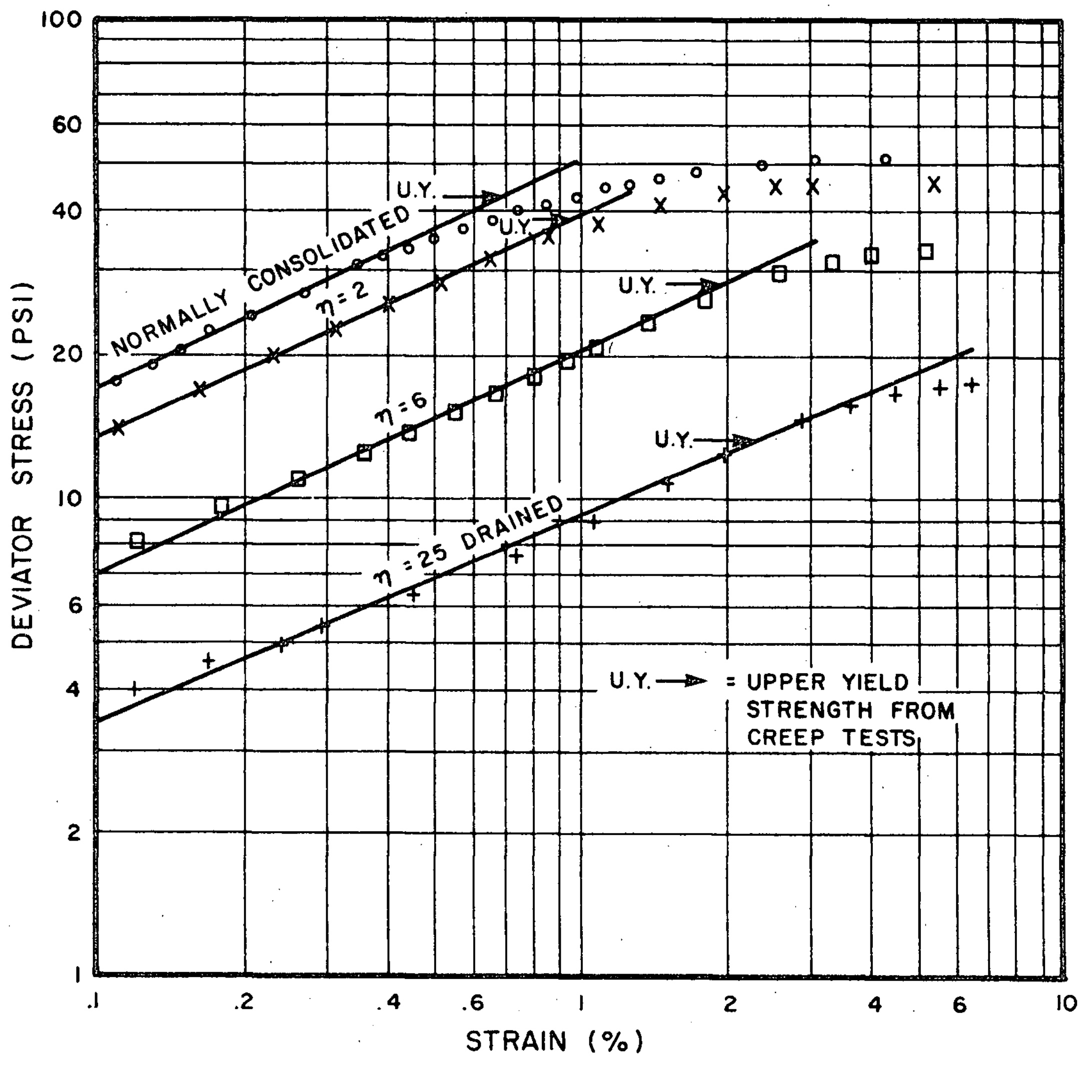


FIGURE 44 - INCREMENTAL LOADING TESTS ON HANEY CLAY

CHAPTER 7

CONCLUSIONS

The following is a list of conclusions and hypotheses derived from the analysis of data from creep, constant strain rate and incremental loading triaxial tests:

(1) - A secondary or constant creep strain rate does not exist for Haney clay subjected to stresses above the upper yield strength. The creep strain rate during creep rupture tests initially decreased with elapsed time, reached a transient minimum strain rate and then increased until rupture. Once a transient minimum strain rate had been reached, the sample inevitably ruptured. Therefore, the onset of failure can be considered to occur at the transient minimum strain rate.

(2) - The pore water pressure in normally consolidated samples of Haney clay continually increased during all creep tests. Thus, the effective stresses were decreasing and the effective stress ratio increasing throughout the complete test. For a creep test with an overconsolidation ratio of six, the pore pressure

barely changed during a creep rupture test. Since the shape of the strain/time curve is similar for both normally consolidated and overconsolidated undrained tests, the onset of creep rupture cannot be explained in terms of effective stresses.

(3) - Based upon the results of creep tests on Haney clay, a relationship exists between the transient minimum strain rate and the elapsed time of loading, which is independent of consolidation history, stress level and drainage conditions. Confirmation of this fact is not available for other soils and should be investigated.

(4) - Results of creep rupture tests on Haney clay confirmed a relationship between the total creep rupture life and the transient minimum strain rate. This is similar to the hypothesis of Saito and Uezawa.

(5) - For Haney clay, a relationship exists between the deviator stress, strain and strain rate for tests in which the strain is continually increasing, the temperature remains constant, adequate time is available for pore pressure equalization within the sample, and the consolidation history is the same. This

relationship enables correlation of stress/strain data between creep rupture and constant strain rate tests. At present, this relationship is restricted to undrained conditions and to soil that is not heavily overconsolidated. It is anticipated that this relationship will hold true for other saturated cohesive soils.

(6) - Results of creep rupture tests on Haney clay show that the strain rate is inversely proportional to the time to rupture during the final stage of a test in which the strain rate is increasing. This relationship is independent of stress level, consolidation history and drainage conditions. This relationship could be applied to field measurements to predict the minimum time until an anticipated failure.

(7) - A method is proposed by which the upper yield strength of a cohesive soil can be evaluated by either a series of creep rupture or constant strain rate tests. The upper yield strength, evaluated by this method, resulted in the same upper yield strength being predicted by creep rupture and constant strain rate tests for each of four consolidation histories.

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APPENDIX A

TEST EQUIPMENT AND TESTING PROCEDURES

A1 - Stress-controlled Apparatus

The frame used for incremental loading and creep tests was described by Glynn (1960); however, all pore pressure and volume change measuring apparatus used by Glynn was removed and replaced with newer components and the frame was mounted on rubber pads to minimize transmission of building vibrations to the sample. All tests performed with this apparatus used 2.5-inch by 5.0-inch triaxial samples. An O-ring seal was installed in the machined bushing for the loading ram to reduce leakage during long tests. This stopped all leakage from the cell, but increased the piston friction slightly. Deviator stresses were applied by weights placed on a hanger beneath the table of the frame. An additional hanger was constructed which enabled the placing or removal of 250 pounds of weights "instantaneously." In actual fact, large deviator stresses were not placed instantaneously, but rather applied continuously over about a 10-second period to avoid impact effects.

The chamber pressure was applied by regulated compressed air acting on an air/water interface. A Fairchild Stratos pressure regulator (0-100 psi) was used to regulate compressed air to $\pm$0.1 psi. Figure 45 shows a schematic layout of the stress-controlled equipment.

The pore water pressure and the chamber pressure were measured with an electrical transducer manufactured by Data Sensors Inc. which measured absolute pressures over a range of 150 psi. Being an absolute pressure gauge, it was necessary to make small corrections due to changes in barometric pressure. Its rated compliance for 150 psi was 4×10^{-4} cubic inches. Output from the transducer was run through a switching and balancing unit and then into a Budd strain indicator, which gave a digital readout. Calibration of the readout enabled accurate pressure measurements of $\pm$0.1 psi.

All pore water lines external to the cell used during pore pressure measurements were 1/8-inch copper tubing, while the saturation spiral for the top drainage lead was heavy-walled nylon. Drainage and consolidation were performed against a back pressure of 15 psi, which was used to ensure 100 per cent saturation. The back pressure was maintained by a

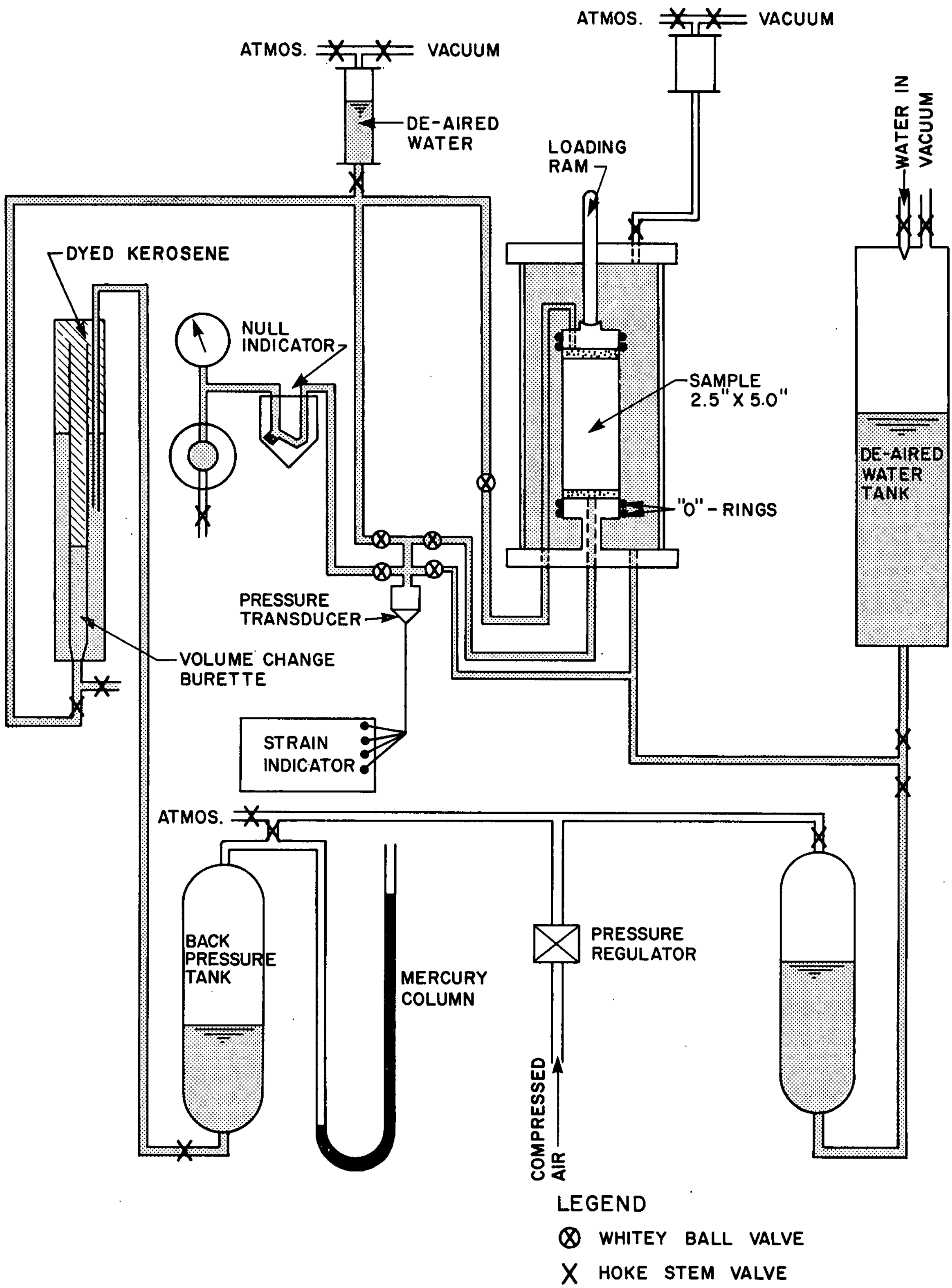

FIGURE 45 - SCHEMATIC LAYOUT OF STRESS-CONTROLLED APPARATUS

mercury column. The volume change apparatus was similar to that described by Bishop and Henkel (1962). Kerosene (dyed red with Sudan III analine dye) was used as the second fluid in the apparatus, which could measure a total volume change of 100 cc. Drainage lines were 1/4-inch outside diameter saran tubing, as the compliance of these lines was not important. The pore pressure measuring system was connected to a (Bishop and Henkel type) "Perspex" null indicator, which was used to check that the system was de-aired and to measure compliance (Bishop and Henkel 1962, p 207).

A2 - Constant Strain Rate Apparatus

All constant strain rate tests were performed on a Wykeham Farrance bench model testing frame. Constant deformation rates between 0.065 inches per minute and 0.000028 inches per minute were possible with this equipment. The triaxial cell, in which both 2.5-inch by 5.0-inch and 1.4-inch by 2.8-inch specimens were tested, was built in the UBC Civil Engineering workshop. The bushing for the 1/4-inch diameter hardened steel ram was composed of two Thomson ball bushings used as a guide with an O-ring seal to stop leakage. Figure 46 shows a

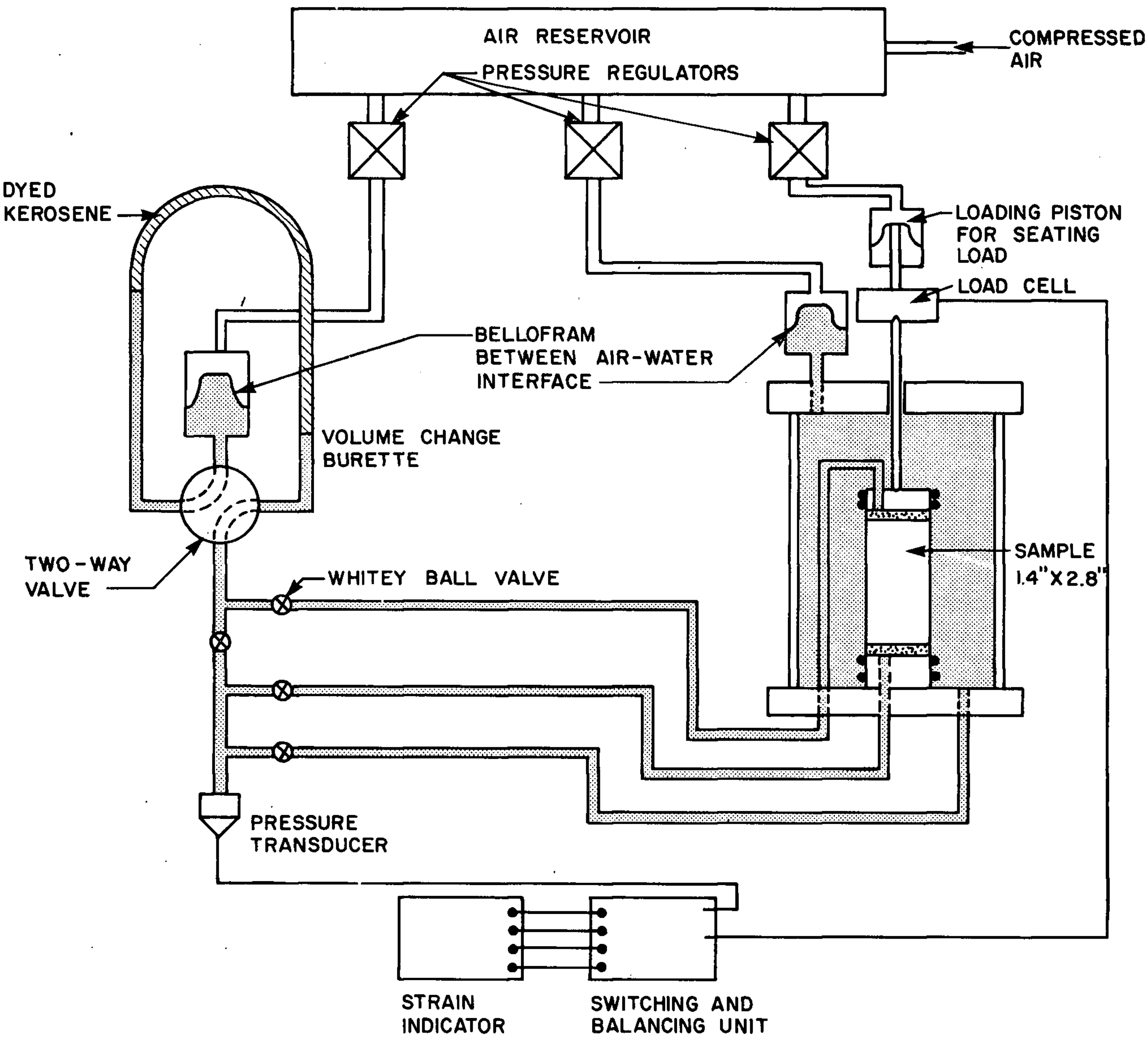

FIGURE 46 - SCHEMATIC LAYOUT OF CONSTANT STRAIN RATE APPARATUS

schematic diagram of the equipment layout. Pore water pressure and chamber pressure were measured with a Data Sensors transducer, which was incorporated in a volume change unit similar to that described by Chan and Duncan (1967). Figure 47 shows a photograph of the volume change unit. Connection between this unit and the triaxial cell was made using short flexible tubing. The bottom drainage and chamber pressure leads were 1/8-inch outside diameter saran tubing, while the top drainage lead was a continuation of the heavy-walled nylon saturation spiral from within the chamber.

Regulated air pressure acting on a rubber Bellofram was used to provide the chamber and back pressure. The Bellofram was used to reduce the amount of air dissolved in the water under pressure.

Axial loads were measured using a circular load cell made of beryllium copper. It had a thin central diaphragm with strain gauges attached to it. Since the diaphragm deflection was small when loaded, the strain rate during a controlled strain rate test was constant. This is in contrast to the strain rates which occur when the axial load is measured using a "soft" proving ring. In this case, the

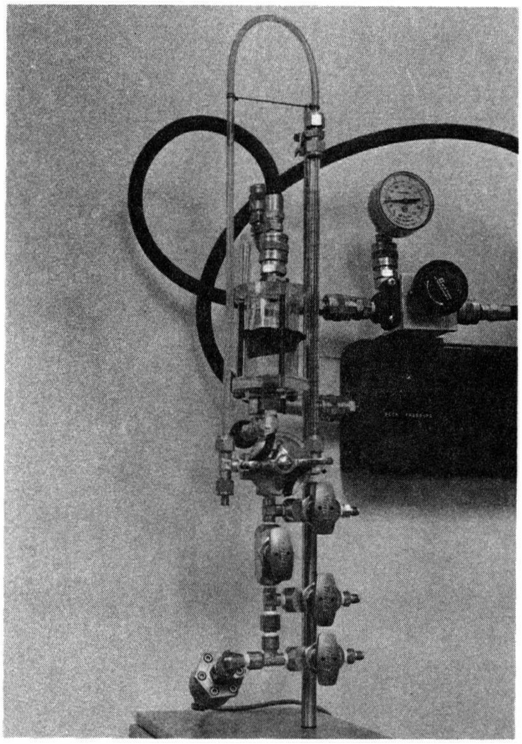

FIGURE 47 - VOLUME CHANGE AND
PORE PRESSURE MEASURING UNIT

actual strain rate may vary by a factor of three during a "constant" strain rate test.

Both pore pressures and axial loads were measured using a switching and balancing unit in conjunction with a Budd strain indicator. A small loading piston was installed above the load cell to provide a seating load of 0.5 psi during consolidation of samples. This was required to ensure that the top cap of the sample maintained alignment with the loading rod during consolidation.

A3 - Testing Procedures - Stress-controlled Apparatus

Firstly, the system was checked to ensure that it was de-aired. Continuous filter paper side drains were then cut from Whatman's No. 54 filter paper with slits every 1/4-inch. The porous stones were boiled before every test to ensure removal of entrapped air and, while cooling, the filter paper was placed in the water to saturate.

Since Haney clay is fairly firm in its natural state, it was decided to roll the membranes up over the sample, thus entrapping a minimum of air. Two membranes, with a layer of Dow Corning silicone grease between them, were used to enclose the sample. The membranes used were standard 2.5-inch by 9-inch triaxial membranes, 0.012-inch thick. To preroll these membranes, they were pulled over a powdered 2.5-inch diameter mandril and rolled around a 2.5-inch inside diameter O-ring. The membranes were transferred from the mandril to the base pedestal, ready for rolling up around the sample.

The clay sample was carefully trimmed using taut piano wire and a "soil lathe." Water content specimens were taken from the sides to obtain the average initial water content. After measuring the dimensions of the trimmed specimen and getting the initial wet weight, the sample was carefully placed on the base pedestal between the saturated stones. The filter paper side drains were then carefully placed around the sample, the inner membrane rolled up, the silicone grease applied and the outside membrane rolled up. Figure 48 shows a sample being installed in the apparatus. The ends of the sample were sealed with two O-rings top and bottom, these being dropped in position from individual expanders previously placed on the saturation spiral. After the triaxial cell was assembled, chamber water was admitted from a de-aired water

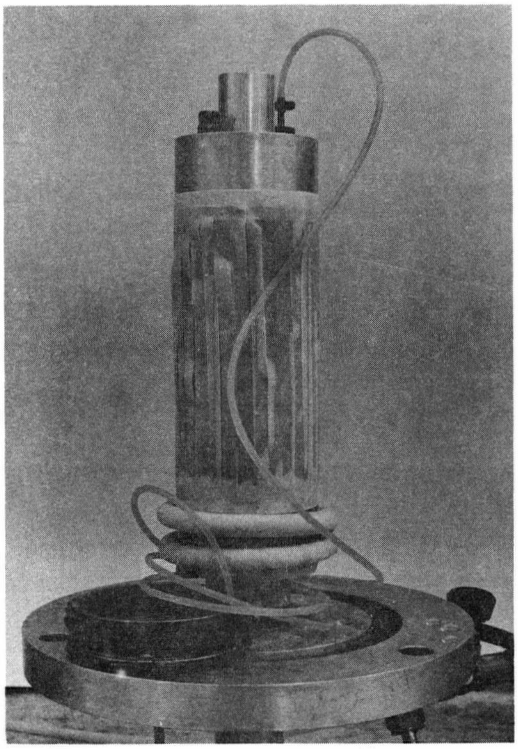

FIGURE 48 – SAMPLE BEING INSTALLED IN STRESS-CONTROLLED APPARATUS

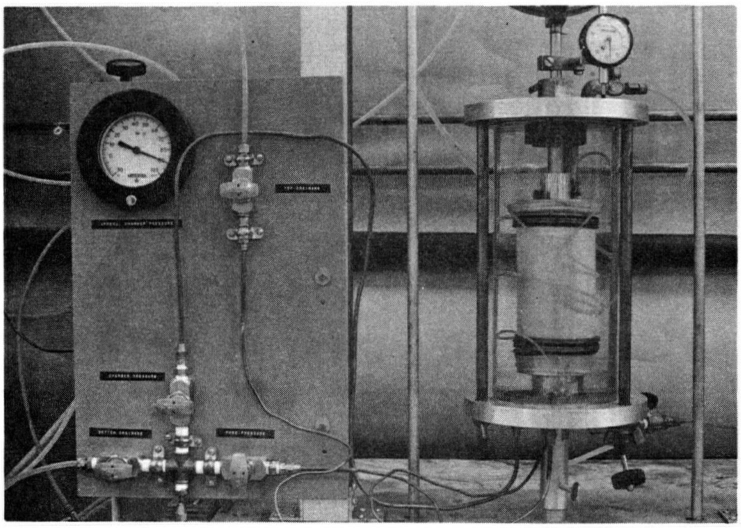

FIGURE 49 – CREEP TEST IN PROGRESS

reservoir and, when the chamber was filled, the chamber pressure line was connected to the pressure tank. As the chamber pressure was being raised in increments, appropriate weights were placed on the loading pan to compensate for uplift on the loading ram. After each increment of chamber pressure, the pore water pressure in the sample was measured. Incremental values of Skempton "B" were calculated to check for saturation and leakage.

The sample was then allowed to consolidate to an effective stress of 75 psi with a small seating load of 0.5 psi to ensure that the loading piston remained in contact with the sample. If the sample was to be tested normally consolidated, it was allowed to drain for 36 hours; however, if it was to be tested after overconsolidation, it was allowed to drain for 24 hours and then rebounded for 36 hours. Since consolidation usually started around noon, samples were finished consolidating around midnight, one or two days later, which left about eight hours until the next morning, at which time shearing would actually be started. For undrained tests, the sample was left for these eight hours in an undrained condition, which provided an exceptionally good check on possible leakage. For the heavily overconsolidated drained tests, swelling was allowed to continue for an additional eight hours, although the volume change was exceptionally small. Figure 49 shows a test in progress.

For creep tests, the creep load was applied in one increment and, as the sample strained, corrections were applied to keep the deviator stress constant. This was difficult to do at the beginning of the test and near creep rupture, due to the high strain rates and resulting area changes. However, during most of the tests the deviator stress was held constant to within ± 0.1 psi.

At the end of the test, the sample was back drained to an effective stress of 1.0 psi. This ensured that a large negative pore pressure did not exist in the sample and reduced the possibility of absorption of free water by the sample while being removed from the apparatus. The water content of the whole sample after the test was determined and compared with the water content based upon initial side trimmings of clay at the start of the test, allowing for the ensuing volume changes. Most tests were found to have up to ± 0.5 per cent additional water after the test; most of this additional water having been supplied by the filter paper during installation. In general, for the undrained tests, the pore pressure change during the eight hours before shear was used to indicate whether small leaks in the membrane or around the O-rings existed.

A4 - Testing Procedures - Constant Strain Rate Apparatus

As many testing procedures as possible were maintained the same for constant strain rate and stress-controlled tests. Most constant strain rate tests were performed on 1.4-inch by 2.8-inch samples, although some 2.5-inch by 5.0-inch samples were also used. For the small samples, two commercial "Sheik" membranes with silicone grease in between were used together with filter paper side drains. Since no de-aired water supply was available near the testing frame, the triaxial cell, together with the installed sample, had to be carried from the water supply. To maintain saturation in the drainage leads from the cell during this operation, small individual reservoirs were attached to each drainage line. Figure 50 shows a sample being installed in the cell with the individual reservoirs attached to the cell's vertical posts.

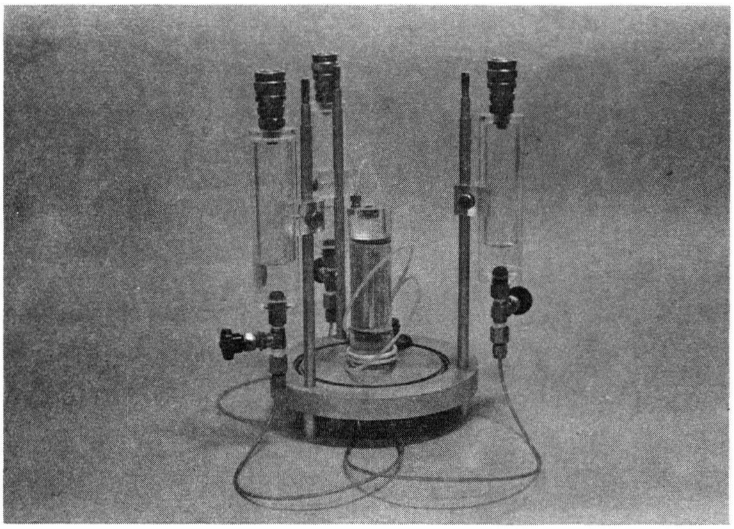

FIGURE 50 - SAMPLE BEING INSTALLED IN CONSTANT STRAIN RATE APPARATUS

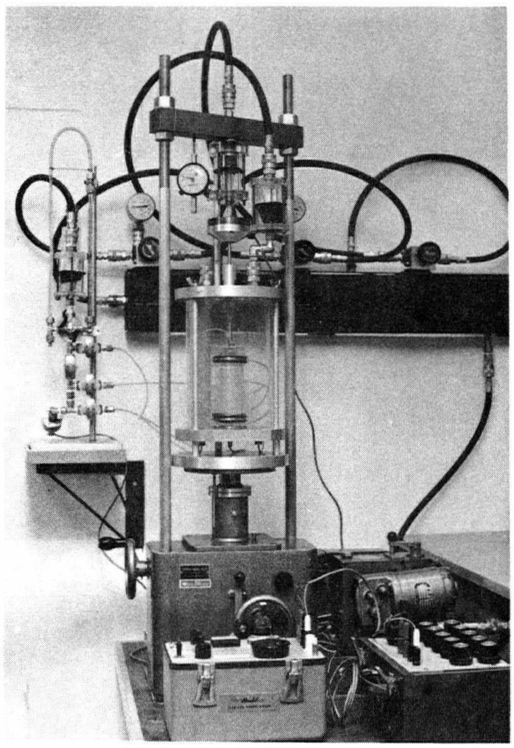

FIGURE 51 - CONSTANT STRAIN RATE TEST IN PROGRESS

After the triaxial cell was positioned in the testing frame, a small chamber pressure was applied. The drainage leads were then carefully disconnected from the individual reservoirs and connected to the volume change device. The connection was made with a "bead" of water present at the end of the leads to ensure that no air was trapped while making the connection. No difficulty was encountered with this procedure during the testing program.

Before consolidation, a seating load of 0.5 psi was applied by the loading piston. Consolidation pressures and times were kept the same as for those of the stress-controlled tests. Figure 51 shows a test in progress.

APPENDIX B

TYPICAL RESULTS FROM CREEP RUPTURE TESTS

All data obtained from stress and strain rate controlled triaxial tests were punched onto computer cards, to be processed by the UBC computer (IBM 7040). Not only did this save tedious calculation, but it left the data in a form which could easily be used in any way desirable at a later date.

The following list explains the various columns of the computer output:

(1) - Elapsed Time (t)

The total time in minutes since shearing was initiated.

(2) - Deviator Stress (σ_D)

The total load in pounds acting on the sample divided by the cross-sectional area in square inches corrected for both axial strain and volume change, assuming the sample deforms as a right circular cylinder. In other words:

$$A = \frac{A_o L_o - \Delta V}{L_o - \Delta L}$$

where: A = average cross-sectional area

A_o = initial cross-sectional area

L_o = initial length

ΔV = change in volume

ΔL = change in length

The first column of computer output under the heading, "Deviator Stress" gives the stress at the time of the reading, while the second column gives the stress just after the reading. The difference in stress is due to small additional loads placed on the sample to maintain the deviator stress "constant."

(3) - Per cent Strain

The shear strain in per cent calculated by:

$$\varepsilon = \left(\frac{\Delta L}{L_o} - \frac{\Delta V}{3V}\right) \times 100\,\%$$

(4) - Pore Pressure (u)

The pore water pressure in psi calculated from the transducer calibration; for drained tests this column is used for water content.

(5) - Major Principal Effective Stress (σ_1')

The major principal effective stress in psi defined by:

$$\sigma_1' = \sigma_3 + \sigma_D - u$$

(6) - Minor Principal Effective Stress (σ_3')

The minor principal effective stress in psi defined by:

$$\sigma_3' = \sigma_3 - u$$

(7) - Principal Effective Stress Ratio (σ_1' / σ_3')

(8) - Mean Normal Effective Stress (p)

The mean normal effective stress in psi defined by:

$$p = \frac{1}{3}\,(\sigma_1' + 2\sigma_3')$$

(9) - Roscoe q/p

The ratio q/p defined by:

$$q/p = (\sigma_1' - \sigma_3')/p = \sigma_D/p$$

This ratio can be related to the principal effective stress ratio.

(10) - Skempton "A"
<u>Pore Pressure Parameter</u>

Since all samples were 100 per cent saturated, Skempton "A" was calculated by:

$$\text{Skempton "A"} = \frac{\Delta u}{\Delta \sigma_D}$$

where: Δu = pore pressure change from the start of test

$\Delta \sigma_D$ = change in deviator stress from start of test

This column is blank for drained tests.

(11) - <u>Strain Rate ($\dot{\varepsilon}$)</u>

The strain rate in per cent per minute was calculated from shear strain and elapsed time data, as discussed in the following. Referring to Figure 52, consider three observed points on the strain/time curve. It is desired to obtain the strain rate at time t_2. Assuming a linear variation of strain with time, the strain rate at time $\frac{t_1 + t_2}{2}$ is $\frac{\varepsilon_2 - \varepsilon_1}{t_2 - t_1}$, and the strain rate at time $\frac{t_2 + t_3}{2}$ is $\frac{\varepsilon_3 - \varepsilon_2}{t_3 - t_2}$. Now assuming a linear variation of strain rate from time $\frac{t_1 + t_2}{2}$ to time $\frac{t_2 + t_3}{2}$, then/

$$\dot{\varepsilon}_2 = \frac{t_3 - t_2}{t_3 - t_1} \cdot \frac{\varepsilon_2 - \varepsilon_1}{t_2 - t_1} + \frac{t_2 - t_1}{t_3 - t_1} \cdot \frac{\varepsilon_3 - \varepsilon_2}{t_3 - t_2}$$

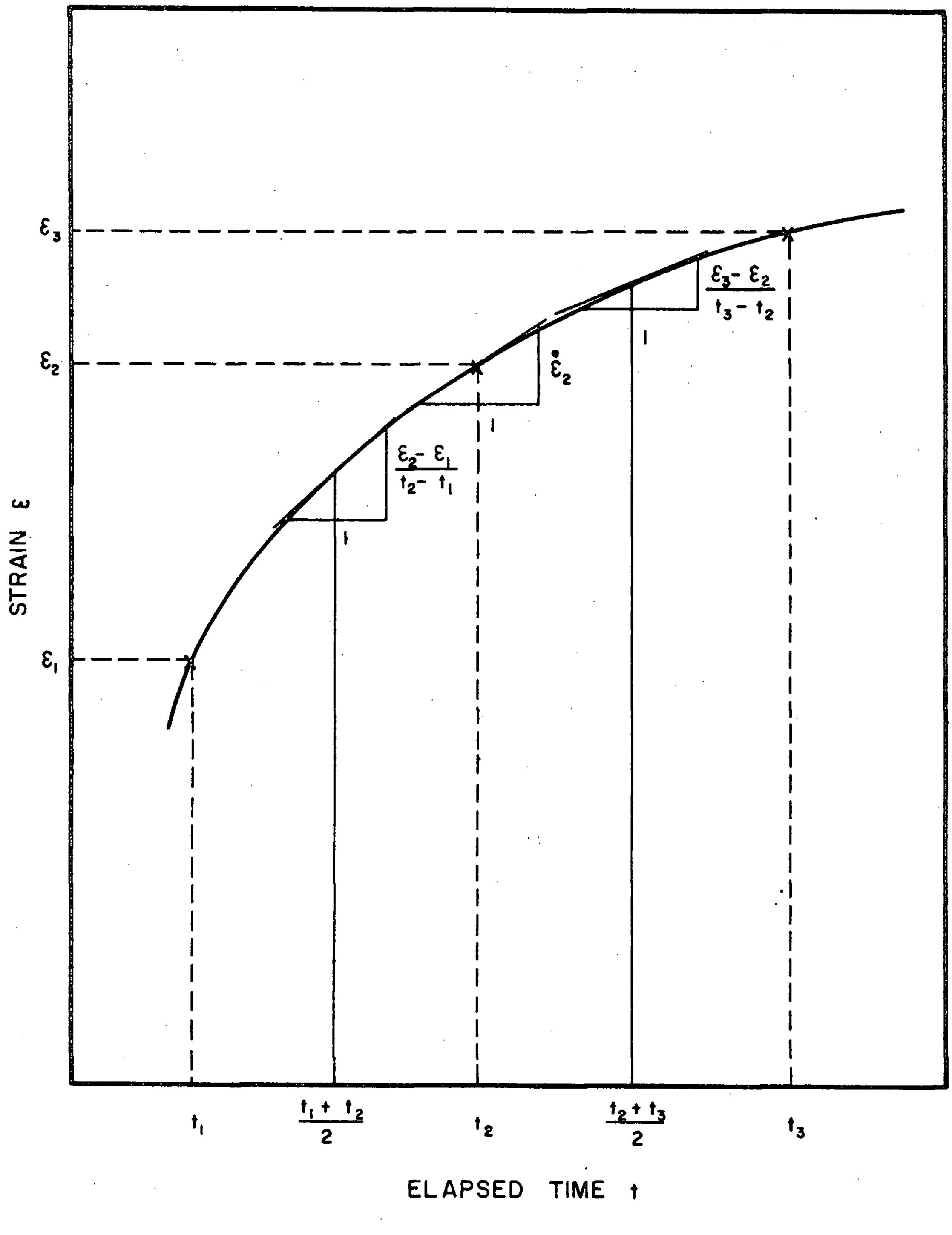

FIGURE 52 - CALCULATION OF STRAIN RATES

Since the calculation of a strain rate by this method requires one set of readings before and after the time at which the strain rate is being obtained, no value of strain rate is calculated for the first and last readings of any test.

(12) - Time to Rupture (ttr)

The duration of time in minutes remaining until the sample actually ruptured.

$$ttr = t_r - t$$

where: t_r = rupture time

If this column is blank, the sample did not rupture.

NORMALLY CONSOLIDATED UNDRAINED CREEP TEST NO. C- 6

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		PERCENT STRAIN	PORE PRESSURE (PSI)	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	SKEMPTON A	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
0.0	0.6	43.8	0.00	17.2	73.4	72.8	1.01	73.0	0.01	0.00	0.00000000	2619.0
0.5	43.5	44.0	0.75	32.5	101.0	57.5	1.76	72.0	0.60	0.36	0.90211528	2618.5
1.0	43.9	43.9	0.90	39.5	94.4	50.5	1.87	65.1	0.67	0.52	0.25162449	2618.0
2.0	43.8	43.9	1.03	43.8	90.1	46.2	1.95	60.9	0.72	0.61	0.11267800	2617.0
4.0	43.9	43.9	1.18	46.6	87.3	43.4	2.01	58.1	0.76	0.68	0.06152358	2615.0
8.0	43.8	43.8	1.34	48.8	85.0	41.2	2.06	55.8	0.79	0.73	0.03447740	2611.0
16.0	43.7	43.8	1.53	51.0	82.7	39.0	2.12	53.6	0.82	0.76	0.02138130	2603.0
30.0	43.7	43.8	1.75	53.1	80.6	36.9	2.18	51.5	0.85	0.83	0.01363411	2589.0
50.0	43.7	43.8	1.97	54.7	79.0	35.3	2.24	49.8	0.88	0.87	0.00961565	2569.0
80.0	43.7	43.8	2.21	56.3	77.4	33.7	2.30	48.3	0.91	0.91	0.00721540	2539.0
127.0	43.7	43.8	2.49	57.9	75.7	32.1	2.36	46.6	0.94	0.95	0.00537103	2492.0
185.0	43.7	43.8	2.76	59.2	74.4	30.8	2.42	45.3	0.96	0.98	0.00428327	2434.0
245.0	43.7	43.8	2.99	60.3	73.4	29.7	2.47	44.3	0.99	1.00	0.00372860	2374.0
307.0	43.7	43.8	3.21	61.1	72.6	28.9	2.51	43.4	1.01	1.02	0.00335572	2312.0
387.0	43.7	43.8	3.46	61.9	71.7	28.1	2.56	42.6	1.03	1.04	0.00294906	2232.0
485.0	43.7	43.8	3.73	62.9	70.8	27.1	2.61	41.7	1.05	1.06	0.00258248	2134.0
620.0	43.7	43.8	4.05	63.8	69.9	26.2	2.66	40.8	1.07	1.08	0.00223769	1999.0
1025.0	43.5	43.8	4.82	65.5	68.0	24.5	2.77	39.0	1.11	1.13	0.00186994	1594.0
1425.0	43.5	43.8	5.55	66.7	66.7	23.3	2.87	37.7	1.15	1.16	0.00201167	1194.0
1620.0	43.6	43.8	5.96	67.3	66.3	22.7	2.92	37.3	1.17	1.16	0.00228966	999.0
1760.0	43.6	43.8	6.30	67.7	66.0	22.3	2.96	36.9	1.18	1.17	0.00255139	859.0
1910.0	43.6	43.8	6.70	68.1	65.5	21.9	2.99	36.5	1.20	1.18	0.00287543	709.0
2105.0	43.5	43.8	7.31	68.6	64.9	21.4	3.04	35.9	1.21	1.20	0.00374658	514.0
2160.0	43.7	43.8	7.53	68.8	64.9	21.2	3.06	35.8	1.22	1.20	0.00408338	459.0
2210.0	43.7	43.8	7.74	68.9	64.8	21.1	3.08	35.6	1.23	1.20	0.00450227	409.0
2255.0	43.7	43.8	7.96	69.1	64.6	20.9	3.09	35.5	1.23	1.20	0.00492365	364.0
2295.0	43.7	43.8	8.16	69.2	64.5	20.8	3.10	35.3	1.24	1.21	0.00543639	324.0
2330.0	43.7	43.8	8.36	69.3	64.4	20.7	3.11	35.3	1.24	1.21	0.00610298	289.0
2365.0	43.7	43.8	8.59	69.4	64.3	20.6	3.12	35.1	1.24	1.21	0.00687326	254.0
2400.0	43.7	43.8	8.84	69.6	64.1	20.4	3.14	35.0	1.25	1.22	0.00777374	219.0
2460.0	43.5	43.8	9.36	69.8	63.8	20.2	3.15	34.8	1.25	1.22	0.01086167	159.0
2480.0	43.7	43.8	9.59	69.9	63.8	20.1	3.17	34.7	1.26	1.22	0.01240347	139.0
2495.0	43.7	43.8	9.78	70.0	63.7	20.0	3.18	34.6	1.26	1.22	0.01375639	124.0
2510.0	43.7	43.8	10.00	70.0	63.7	20.0	3.18	34.6	1.26	1.22	0.01554232	109.0
2523.0	43.7	43.8	10.22	70.1	63.6	19.9	3.19	34.5	1.27	1.23	0.01804931	96.0
2534.0	43.7	43.8	10.43	70.1	63.6	19.9	3.20	34.5	1.27	1.23	0.02027115	85.0
2543.0	43.7	43.8	10.62	70.2	63.5	19.8	3.21	34.4	1.27	1.23	0.02258170	76.0
2552.0	43.7	43.8	10.84	70.2	63.4	19.8	3.21	34.3	1.27	1.23	0.02624998	67.0
2559.0	43.7	43.8	11.03	70.3	63.4	19.7	3.22	34.3	1.27	1.23	0.02952736	60.0
2573.0	43.6	43.8	11.49	70.4	63.2	19.6	3.22	34.1	1.28	1.24	0.03881593	46.0
2582.0	43.6	43.8	11.87	70.5	63.1	19.5	3.23	34.1	1.28	1.24	0.04833172	37.0
2590.0	43.6	43.8	12.30	70.5	63.1	19.5	3.23	34.0	1.28	1.24	0.06208523	29.0
2598.0	43.5	43.8	12.87	70.5	63.0	19.5	3.23	34.0	1.28	1.24	0.08787847	21.0
2606.0	43.4	43.8	13.71	70.5	62.9	19.5	3.23	33.9	1.28	1.25	0.14451793	13.0

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		PERCENT STRAIN	PORE PRESSURE (PSI)	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	SKEMPTON A	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
2611.0	43.3	43.8	14.55	70.6	62.8	19.4	3.23	33.9	1.28	1.25	0.23499433	8.0
2613.0	43.5	43.5	15.08	70.5	63.0	19.5	3.22	34.0	1.28	1.24	0.29724862	6.0
2614.0	43.3	43.3	15.39	70.5	62.9	19.5	3.22	34.0	1.27	1.25	0.33181250	5.0
2615.0	43.2	43.4	15.74	70.4	62.7	19.6	3.20	34.0	1.27	1.25	0.38780594	4.0
2617.0	42.9	42.9	16.67	70.3	62.6	19.7	3.17	34.0	1.26	1.26	0.59795380	2.0
2618.0	42.5	42.5	17.34	70.2	62.4	19.8	3.14	34.0	1.25	1.26	0.78805470	1.0
2619.0	42.1	42.1	18.25	70.0	62.1	20.0	3.10	34.1	1.23	1.27	0.00000000	0.0

CREEP RUPTURE FAILURE

OVER CONSOLIDATED UNDRAINED CREEP TEST NO. C- 15 (OVER CONSOLIDATION RATIO = 2)

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		PERCENT STRAIN	PORE PRESSURE (PSI)	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	SKEMPTON A	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
0.0	0.6	40.0	0.00	14.7	38.4	37.8	1.02	38.0	0.02	0.00	0.00000000	2045.0
0.5	39.6	39.6	0.95	29.4	62.8	23.1	2.71	36.3	1.09	0.38	1.02384866	2044.5
1.0	39.6	39.6	1.02	28.2	63.9	24.3	2.63	37.5	1.05	0.34	0.13294956	2044.0
2.0	39.6	40.0	1.11	27.2	64.9	25.3	2.56	38.5	1.03	0.32	0.07709704	2043.0
4.0	40.0	40.0	1.23	26.7	65.8	25.8	2.55	39.1	1.02	0.30	0.05002741	2041.0
8.0	39.9	40.0	1.36	26.2	66.3	26.3	2.52	39.6	1.01	0.29	0.02808869	2037.0
15.0	39.9	40.0	1.50	26.1	66.3	26.4	2.51	39.7	1.01	0.29	0.01729822	2030.0
30.0	39.9	40.0	1.68	26.3	66.1	26.2	2.53	39.5	1.01	0.30	0.01060700	2015.0
58.0	39.9	40.0	1.89	26.8	65.6	25.7	2.55	39.0	1.02	0.31	0.00680086	1987.0
93.0	39.9	40.0	2.09	27.4	65.0	25.1	2.59	38.4	1.04	0.32	0.00512002	1952.0
144.0	39.9	40.0	2.31	27.9	64.5	24.6	2.62	37.9	1.05	0.34	0.00363962	1901.0
202.0	39.9	40.0	2.51	28.5	63.9	24.0	2.66	37.3	1.07	0.35	0.00319201	1843.0
292.0	39.9	40.0	2.76	29.2	63.2	23.3	2.71	36.6	1.09	0.37	0.00265933	1753.0
407.0	39.9	40.0	3.04	29.8	62.6	22.7	2.75	36.0	1.11	0.38	0.00236078	1638.0
624.0	39.8	40.0	3.51	30.6	61.7	21.9	2.82	35.2	1.13	0.41	0.00205899	1421.0
790.0	39.9	40.0	3.84	31.2	61.2	21.3	2.87	34.6	1.15	0.42	0.00194667	1255.0
1200.0	39.7	40.0	4.61	32.3	59.9	20.2	2.96	33.5	1.19	0.45	0.00212673	845.0
1492.0	39.7	40.0	5.28	33.1	58.1	19.4	3.05	32.6	1.22	0.47	0.00287573	553.0
1706.0	39.7	40.0	5.98	33.7	58.5	18.8	3.11	32.0	1.24	0.49	0.00426227	339.0
1852.0	39.7	40.0	6.70	34.2	58.0	18.3	3.16	31.6	1.26	0.50	0.00705417	193.0
1974.0	39.5	40.0	7.78	34.8	57.2	17.7	3.23	30.9	1.28	0.52	0.01592123	71.0
1986.0	39.9	40.0	7.98	34.8	57.7	17.7	3.25	31.1	1.29	0.51	0.01836962	59.0
2000.0	39.9	40.0	8.27	34.8	57.6	17.7	3.25	31.0	1.29	0.51	0.02362803	45.0
2012.0	39.9	40.0	8.58	34.9	57.4	17.6	3.27	30.9	1.29	0.51	0.03153631	33.0
2023.0	39.8	40.0	8.98	35.0	57.4	17.5	3.27	30.8	1.29	0.52	0.04698402	22.0
2027.0	39.9	40.0	9.19	35.0	57.4	17.5	3.28	30.8	1.30	0.52	0.05604669	18.0
2032.0	39.9	40.0	9.50	35.1	57.3	17.4	3.29	30.7	1.30	0.52	0.07774808	13.0
2036.0	39.8	40.0	9.86	35.2	57.2	17.3	3.30	30.6	1.30	0.52	0.11099525	9.0
2039.0	39.9	40.0	10.24	35.1	57.2	17.4	3.29	30.7	1.30	0.52	0.16586690	6.0
2042.5	39.7	40.0	10.98	35.1	57.1	17.4	3.27	30.7	1.29	0.52	0.40397511	2.5
2044.5	39.5	39.8	12.01	34.7	57.3	17.8	3.22	31.0	1.28	0.51	1.61595386	0.5
2045.0	39.3	39.3	12.95	34.2	57.7	18.3	3.15	31.4	1.25	0.50	0.00000000	0.0

CREEP RUPTURE FAILURE

OVER CONSOLIDATED UNDRAINED CREEP TEST NO. C- 20 (OVER CONSOLIDATION RATIO = 6)

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		PERCENT STRAIN	PORE PRESSURE (PSI)	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	SKEMPTON A	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
0.0	0.6	30.3	0.00	15.2	12.9	12.3	1.05	12.5	0.05	0.00	0.00000000	1925.5
0.5	29.7	30.2	2.02	19.3	37.9	8.2	4.61	18.1	1.64	0.14	0.21132226	1925.0
1.0	30.1	30.3	2.21	18.6	39.0	8.9	4.38	18.9	1.59	0.11	0.31746032	1924.5
2.0	30.2	30.3	2.41	17.6	40.1	9.9	4.05	20.0	1.51	0.08	0.17780502	1923.5
4.0	30.2	30.3	2.68	16.8	41.0	10.7	3.81	20.8	1.45	0.05	0.10968049	1921.5
7.0	30.2	30.3	2.90	16.2	41.6	11.3	3.67	21.4	1.41	0.03	0.06428036	1918.5
11.0	30.2	30.3	3.10	15.8	42.0	11.7	3.58	21.8	1.39	0.02	0.04394032	1914.5
17.0	30.2	30.3	3.31	15.5	42.3	12.0	3.51	22.1	1.37	0.01	0.03102099	1908.5
25.0	30.2	30.3	3.52	15.2	42.5	12.3	3.46	22.4	1.35	0.00	0.02274224	1900.5
35.0	30.2	30.3	3.71	15.1	42.7	12.4	3.43	22.5	1.34	-0.01	0.01588126	1890.5
50.0	30.2	30.3	3.92	15.0	42.8	12.5	3.42	22.6	1.34	-0.01	0.01289701	1875.5
76.0	30.2	30.3	4.19	14.8	42.9	12.7	3.39	22.7	1.33	-0.01	0.00929909	1849.5
103.0	30.2	30.3	4.41	14.8	42.9	12.7	3.39	22.7	1.33	-0.01	0.00741801	1822.5
157.0	30.2	30.3	4.74	14.8	42.9	12.7	3.38	22.8	1.33	-0.01	0.00538536	1768.5
230.0	30.2	30.3	5.06	14.8	42.9	12.7	3.37	22.8	1.33	-0.02	0.00418313	1695.5
298.0	30.2	30.3	5.33	14.8	42.9	12.7	3.39	22.7	1.33	-0.01	0.00369388	1627.5
368.0	30.2	30.3	5.57	14.9	42.9	12.6	3.39	22.7	1.33	-0.01	0.00333577	1557.5
433.0	30.2	30.3	5.78	14.9	42.9	12.6	3.40	22.7	1.33	-0.01	0.00308917	1492.5
584.0	30.2	30.3	6.21	14.9	42.7	12.6	3.40	22.6	1.33	-0.01	0.00272161	1341.5
718.0	30.2	30.3	6.56	15.0	42.7	12.5	3.41	22.6	1.34	-0.01	0.00265025	1207.5
834.0	30.2	30.3	6.87	15.0	42.7	12.5	3.42	22.5	1.34	-0.01	0.00259480	1091.5
1400.0	29.9	30.3	8.16	15.2	42.2	12.3	3.43	22.3	1.34	-0.00	0.00340346	525.5
1462.0	30.2	30.3	8.38	15.2	42.6	12.3	3.45	22.4	1.35	-0.00	0.00379457	463.5
1522.0	30.2	30.3	8.62	15.2	42.5	12.3	3.45	22.4	1.35	-0.00	0.00428221	403.5
1574.0	30.2	30.3	8.85	15.2	42.6	12.3	3.45	22.4	1.35	-0.00	0.00480084	351.5
1630.0	30.2	30.3	9.14	15.2	42.5	12.3	3.45	22.4	1.35	-0.00	0.00579695	295.5
1668.0	30.2	30.3	9.38	15.1	42.7	12.4	3.43	22.5	1.34	-0.01	0.00671164	257.5
1698.0	30.2	30.3	9.59	15.1	42.6	12.4	3.44	22.4	1.35	-0.00	0.00739151	227.5
1728.0	30.2	30.3	9.82	15.1	42.6	12.4	3.44	22.5	1.35	-0.00	0.00832227	197.5
1756.0	30.2	30.3	10.07	15.1	42.6	12.4	3.43	22.5	1.34	-0.01	0.00979047	169.5
1776.0	30.2	30.3	10.28	15.1	42.7	12.4	3.43	22.5	1.34	-0.01	0.01091307	149.5
1794.0	30.2	30.3	10.48	15.1	42.7	12.4	3.43	22.5	1.34	-0.01	0.01197797	131.5
1814.0	30.2	30.3	10.74	15.0	42.7	12.5	3.42	22.5	1.34	-0.01	0.01414719	111.5
1830.0	30.2	30.3	10.98	15.0	42.7	12.5	3.42	22.5	1.34	-0.01	0.01669051	95.5
1842.0	30.2	30.3	11.19	15.0	42.7	12.5	3.42	22.5	1.34	-0.01	0.01852987	83.5
1852.0	30.2	30.3	11.39	15.0	42.7	12.5	3.42	22.6	1.34	-0.01	0.02050282	73.5
1863.0	30.2	30.3	11.63	15.0	42.7	12.5	3.41	22.6	1.34	-0.01	0.02360412	62.5
1872.0	30.2	30.3	11.85	15.0	42.8	12.5	3.41	22.6	1.34	-0.01	0.02690920	53.5
1881.0	30.2	30.3	12.11	14.9	42.8	12.6	3.40	22.7	1.33	-0.01	0.03199623	44.5
1888.0	30.2	30.3	12.35	14.9	42.8	12.6	3.40	22.7	1.33	-0.01	0.03819838	37.5
1893.0	30.2	30.3	12.56	14.9	42.8	12.6	3.40	22.7	1.33	-0.01	0.04394032	32.5
1898.0	30.2	30.3	12.79	14.9	42.9	12.6	3.39	22.7	1.33	-0.01	0.05012832	27.5
1902.0	30.2	30.3	13.00	14.8	42.9	12.7	3.39	22.7	1.33	-0.01	0.05793992	23.5
1907.0	30.2	30.3	13.33	14.8	42.9	12.7	3.38	22.7	1.33	-0.01	0.07357448	18.5

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		PERCENT STRAIN	PORE PRESSURE (PSI)	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	SKEMPTON A	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
1912.0	30.2	30.3	13.74	14.8	42.9	12.7	3.37	22.8	1.32	-0.02	0.09707746	13.5
1917.0	30.1	30.3	14.30	14.7	42.9	12.8	3.35	22.8	1.32	-0.02	0.13815654	8.5
1922.0	30.0	30.3	15.12	14.6	42.9	12.9	3.32	22.9	1.31	-0.02	0.48840179	3.5
1925.0	29.6	29.6	17.17	13.7	43.3	13.8	3.15	23.6	1.25	-0.05	4.37184272	0.5
1925.5	28.7	28.7	19.66	11.8	44.4	15.7	2.83	25.3	1.13	-0.12	0.00000000	0.0

CREEP RUPTURE FAILURE

OVER CONSOLIDATED UNDRAINED CREEP TEST NO. C- 35 (OVER CONSOLIDATION RATIO = 25)

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		PERCENT STRAIN	PORE PRESSURE (PSI)	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	SKEMPTON A	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
0.0	0.6	16.0	0.00	14.5	4.1	3.5	1.16	3.7	0.15	0.00	0.00000000	141.5
0.5	15.6	15.6	2.79	16.1	17.4	1.9	9.37	7.0	2.21	0.11	3.04400232	141.0
1.0	15.5	16.0	3.04	16.1	17.4	1.9	9.03	7.1	2.18	0.10	0.44203333	140.5
2.0	16.0	16.0	3.37	15.8	18.2	2.2	8.16	7.5	2.11	0.08	0.28564733	139.5
4.0	15.9	16.0	3.79	15.2	18.8	2.8	6.60	8.2	1.95	0.04	0.17580871	137.5
6.0	16.0	16.0	4.07	14.7	19.2	3.3	5.88	8.6	1.86	0.01	0.12407072	135.5
8.0	16.0	16.0	4.29	14.4	19.6	3.6	5.45	8.9	1.79	-0.01	0.10273926	133.5
11.0	16.0	16.0	4.56	14.0	19.9	4.0	5.03	9.3	1.72	-0.03	0.08371844	130.5
14.0	16.0	16.0	4.79	13.8	20.2	4.2	4.79	9.5	1.68	-0.05	0.07233273	127.5
17.0	16.0	16.0	5.00	13.6	20.4	4.4	4.63	9.7	1.64	-0.06	0.06764450	124.5
20.0	16.0	16.0	5.20	13.4	20.5	4.6	4.49	9.9	1.61	-0.07	0.06195165	121.5
23.0	16.0	16.0	5.37	13.3	20.7	4.7	4.42	10.0	1.60	-0.08	0.05556513	118.5
27.0	16.0	16.0	5.58	13.2	20.8	4.8	4.31	10.1	1.57	-0.09	0.05224030	114.5
31.0	16.0	16.0	5.79	13.1	20.9	4.9	4.23	10.3	1.55	-0.09	0.04947761	110.5
35.0	16.0	16.0	5.98	12.9	21.0	5.1	4.15	10.4	1.54	-0.10	0.04768603	106.5
40.0	16.0	16.0	6.21	12.8	21.2	5.2	4.07	10.5	1.52	-0.11	0.04681534	101.5
45.0	16.0	16.0	6.45	12.7	21.3	5.3	4.02	10.6	1.50	-0.12	0.04581072	96.5
50.0	16.0	16.0	6.67	12.6	21.3	5.4	3.97	10.7	1.49	-0.12	0.04560981	91.5
55.0	16.0	16.0	6.90	12.6	21.4	5.4	3.95	10.7	1.49	-0.13	0.04621258	86.5
60.0	16.0	16.0	7.13	12.5	21.5	5.5	3.91	10.8	1.48	-0.13	0.04641350	81.5
65.0	16.0	16.0	7.37	12.4	21.5	5.6	3.87	10.9	1.47	-0.13	0.04641350	76.5
70.0	16.0	16.0	7.60	12.4	21.6	5.6	3.84	10.9	1.46	-0.14	0.04721720	71.5
75.0	16.0	16.0	7.84	12.4	21.6	5.6	3.83	11.0	1.46	-0.14	0.04942736	66.5
80.0	16.0	16.0	8.09	12.3	21.7	5.7	3.79	11.0	1.45	-0.14	0.05096778	61.5
84.0	16.0	16.0	8.30	12.3	21.7	5.7	3.78	11.1	1.44	-0.15	0.05274262	57.5
88.0	16.0	16.0	8.52	12.2	21.7	5.8	3.77	11.1	1.44	-0.15	0.05324492	53.5
92.0	16.0	16.0	8.72	12.2	21.8	5.8	3.75	11.1	1.43	-0.15	0.05450071	49.5
96.0	16.0	16.0	8.95	12.2	21.8	5.8	3.73	11.2	1.43	-0.15	0.05927266	45.5
100.0	16.0	16.0	9.20	12.1	21.8	5.9	3.72	11.2	1.43	-0.15	0.06283667	41.5
103.0	16.0	16.0	9.39	12.1	21.9	5.9	3.71	11.2	1.42	-0.16	0.06530036	38.5
106.0	16.0	16.0	9.59	12.1	21.9	5.9	3.70	11.2	1.42	-0.16	0.06898399	35.5
109.0	16.0	16.0	9.80	12.1	21.9	5.9	3.70	11.2	1.42	-0.16	0.07266762	32.5
112.0	16.0	16.0	10.03	12.0	21.9	6.0	3.68	11.3	1.42	-0.16	0.07668608	29.5
115.0	16.0	16.0	10.26	12.0	21.9	6.0	3.67	11.3	1.41	-0.16	0.08639743	26.5
118.0	16.0	16.0	10.54	12.0	21.9	6.0	3.67	11.3	1.41	-0.16	0.09242517	23.5
121.0	16.0	16.0	10.82	12.0	22.0	6.0	3.65	11.4	1.41	-0.17	0.09610673	20.5
123.0	16.0	16.0	11.02	12.0	22.0	6.0	3.65	11.4	1.41	-0.17	0.10548520	18.5
125.0	16.0	16.0	11.24	11.9	22.0	6.1	3.64	11.4	1.40	-0.17	0.11804301	16.5
127.0	16.0	16.0	11.49	11.9	22.1	6.1	3.62	11.4	1.40	-0.17	0.13110310	14.5
129.0	16.0	16.0	11.76	11.9	22.1	6.1	3.61	11.4	1.40	-0.17	0.14667472	12.5
131.0	16.0	16.0	12.08	11.9	22.1	6.1	3.60	11.5	1.39	-0.17	0.17329714	10.5
133.0	16.0	16.0	12.46	11.8	22.1	6.2	3.58	11.5	1.39	-0.18	0.20845887	8.5
135.0	15.9	16.0	12.91	11.8	22.2	6.2	3.56	11.5	1.38	-0.18	0.25869000	6.5
137.0	15.9	16.0	13.49	11.7	22.2	6.3	3.53	11.6	1.37	-0.18	0.35412899	4.5

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		PERCENT STRAIN	PORE PRESSURE (PSI)	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	SKEMPTON A	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
139.0	15.9	16.0	14.33	11.6	22.2	6.4	3.50	11.6	1.36	-0.19	0.62537670	2.5
141.0	15.7	15.7	15.99	11.5	22.2	6.5	3.41	11.7	1.34	-0.20	0.00000000	0.5

CREEP RUPTURE FAILURE

OVER CONSOLIDATED DRAINED CREEP TEST NO. C- 22 (OVER CONSOLIDATION RATIO = 25)

ELAPSED TIME (MIN)	DEVIATOR STRESS (PSI)		SHEAR STRAIN	WATER CONTENT	SIG 1 EFF (PSI)	SIG 3 EFF (PSI)	PRINCIPAL STRESS RATIO	MEAN NORMAL STRESS	ROSCOE Q/P	STRAIN RATE (PERCENT/MIN)	TIME TO RUPTURE
0.0	0.6	14.6	0.00	36.4	3.6	3.0	1.19	3.2	0.18	0.00000000	493.0
0.5	14.3	14.6	1.87	36.4	17.3	3.0	5.78	7.8	1.84	2.02020198	492.5
1.0	14.6	14.6	2.02	36.4	17.6	3.0	5.85	7.9	1.85	0.23141508	492.0
2.0	14.6	14.6	2.13	36.3	17.6	3.0	5.86	7.9	1.86	0.09481671	491.0
4.0	14.6	14.6	2.25	36.3	17.6	3.0	5.86	7.9	1.86	0.04983165	489.0
8.0	14.6	14.6	2.37	36.3	17.6	3.0	5.86	7.9	1.86	0.02587109	485.0
16.0	14.6	14.6	2.51	36.4	17.6	3.0	5.86	7.9	1.86	0.01483981	477.0
38.0	14.6	14.6	2.70	36.4	17.6	3.0	5.86	7.9	1.85	0.00794586	455.0
69.0	14.6	14.6	2.91	36.5	17.6	3.0	5.85	7.9	1.85	0.00595027	424.0
135.0	14.5	14.6	3.21	36.6	17.5	3.0	5.84	7.8	1.85	0.00420942	358.0
205.0	14.5	14.6	3.48	36.7	17.5	3.0	5.83	7.8	1.85	0.00392151	288.0
273.0	14.5	14.6	3.75	36.8	17.5	3.0	5.85	7.8	1.85	0.00422649	220.0
347.0	14.5	14.6	4.08	36.9	17.5	3.0	5.84	7.8	1.85	0.00518646	146.0
407.0	14.5	14.6	4.43	37.0	17.5	3.0	5.84	7.8	1.85	0.00932299	86.0
465.0	14.5	14.6	5.17	37.2	17.5	3.0	5.82	7.8	1.85	0.04260687	28.0
471.0	14.5	14.6	5.44	37.2	17.5	3.0	5.85	7.8	1.85	0.05989649	22.0
475.0	14.5	14.6	5.72	37.2	17.5	3.0	5.85	7.8	1.85	0.08410245	18.0
479.0	14.5	14.6	6.12	37.3	17.5	3.0	5.84	7.8	1.85	0.12408945	14.0
481.0	14.5	14.6	6.39	37.3	17.5	3.0	5.85	7.8	1.85	0.16095811	12.0
483.0	14.5	14.6	6.76	37.3	17.5	3.0	5.84	7.8	1.85	0.20926693	10.0
485.0	14.5	14.6	7.23	37.3	17.5	3.0	5.84	7.8	1.85	0.27257431	8.0
487.0	14.5	14.6	7.85	37.4	17.5	3.0	5.83	7.8	1.85	0.35221933	6.0
489.0	14.5	14.6	8.64	37.4	17.5	3.0	5.82	7.8	1.85	0.49363349	4.0
491.0	14.4	14.5	9.83	37.4	17.4	3.0	5.80	7.8	1.85	1.40054964	2.0
492.5	14.0	14.0	12.83	37.5	17.0	3.0	5.68	7.7	1.83	7.49729664	0.5
493.0	13.3	13.3	17.50	37.5	16.3	3.0	5.43	7.4	1.79	0.00000000	0.0

CREEP RUPTURE FAILURE