

CREEP OF SATURATED SOIL
AT DIFFERENT TEMPERATURES

by

BALARKA GUPTA

B.Tech. (Hons), Indian Institute of Technology,
Kharagpur, India, 1960

A THESIS SUBMITTED IN PARTIAL FULFILMENT OF
THE REQUIREMENTS FOR THE DEGREE OF
M.A.Sc
in the Department
of
Civil Engineering

We accept this thesis as conforming to the
required standard

THE UNIVERSITY OF BRITISH COLUMBIA
September, 1964

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

Department of Civil Engineering

The University of British Columbia,
Vancouver 8, Canada

Date September 8th, 1964

ABSTRACT

A theoretical as well as laboratory investigation into the creep of soil at different temperatures is presented. Other observations such as the effect of temperature on pore pressure under constant stress are also reported.

An hypothesis explaining the phenomenon of creep of soil is proposed; experimental results verifying the same are also presented.

Experimental work consisted of the following two types of triaxial tests:

1. Unconsolidated undrained tests on cylindrical specimens without pore pressure measurements.
2. Consolidated undrained tests on cylindrical specimens with pore pressure measurements.

A triaxial machine equipped with a non-flow null indicating type pore pressure measuring device was used for all shear tests. An outer jacket surrounding the triaxial cell was designed for temperature control for all tests reported.

Observed data show that other factors being constant, strain rate increases with increase of temperature although the relationship between strain rate and absolute temperature is not linear at all times. It is also observed that pore

pressure increases with increase of temperature provided other factors are kept constant.

ACKNOWLEDGMENT

The author wishes to express his gratitude and indebtedness to Dr. W.D.L. Finn for his guidance and supervision. He also gratefully acknowledges the continued help given by Professor N.D. Nathan throughout the preparation of this thesis. He further wishes to express his appreciation to Professor J.K. Mitchell of the University of California for reviewing the manuscript and making valuable suggestions. Finally he extends his thanks to the staff, Department of Civil Engineering Workshop, for the help rendered in designing and constructing the equipment required for the experimental work.

TABLE OF CONTENTS

	<u>Page</u>
<u>CHAPTER I</u>	
INTRODUCTION	1
A. Description of the problem	1
B. Previous approaches to a solution	5
C. Aim of this thesis	9
<u>CHAPTER II</u>	
THEORETICAL INVESTIGATION	10
A. Strain of clay	10
B. Creep or flow behaviour of clay: Qualitative analysis	12
C. Quantitative analysis	18
D. Rheological model to explain the flow characteristics of clay	21
E. Equation relating strain rate and Absolute Temperature	22
<u>CHAPTER III</u>	
EQUIPMENT AND TESTING PROCEDURE	25
A. Undisturbed soil samples	25
B. Preparation of samples for testing	26
C. Testing procedure:	
(1) Arrangement for temperature control	27
(2) Cylindrical triaxial test	28
(3) Pore pressure panel	30
<u>CHAPTER IV</u>	
DISCUSSION OF THE RESULTS	33
A. Variation of strain with time at a particular temperature under a constant total stress	35
B. Comparison of the strain curves at different temperatures	39

TABLE OF CONTENTS (continued)

Page

C. Variation of pore pressure with time at a particular temperature and constant total stress	41
D. Comparison of the pore pressure curves at different temperatures	46
E. Experimental verification of the relationship between strain rate and absolute temperature	48

CHAPTER V

SUMMARY OF CONCLUSIONS	55
------------------------------	----

CHAPTER VI

RECOMMENDATIONS	56
BIBLIOGRAPHY	57
APPENDIX	58

LIST OF TABLES

No.		<u>Page</u>
I.	U-U Test at 70°F	59
II.	U-U Test at 80°F	60
III.	U-U Test at 90°F	61
IV.	U-U Test at 100°F	62
V.	U-U Test at 110°F	63
VI.	C-U Test at 70°F	64
VII.	C-U Test at 90°F	66
VIII.	C-U Test at 100°F	68
IX.	C-U Test at 110°F	70

NOTATIONS

τ_{∞} = Ultimate Continuous Shear Strength

σ = Total External Stress

σ_0 = Ultimate Instantaneous Strength; Lower Yield Point

$D = \frac{de}{dt}$ = Strain Rate

τ = Shear Stress

λ = Average Distance between two balanced positions of molecules.

k = Boltzmann's Constant

T = Absolute Temperature

h = Plank's Constant

E_0 = Energy of Activation

n = Number of Molecules with Activation in Series per unit length in the Direction of Stress

N = Number of molecules with activation in one unit area of cross section perpendicular to the direction of stress

$\frac{de_b}{dt}$ = Strain rate of bond

η = Coefficient of Viscosity

ϵ = Total strain in the rheological model

ϵ_1 = Strain in elastic spring element

ϵ_2 = Strain in modified voigt element

E_1 = Modulus of elasticity of spring element

E_2 = Modulus of elasticity of spring in voigt element

U = Pore water pressure

R = Electrical repulsion between particles

A = Electrical Attraction between particles

$\bar{\sigma}_{an}$ = Intergranular stress

CHAPTER I

INTRODUCTION

A. Description of the Problem

Clay is not a fully elastic material: its deformation under external load is not fully recoverable even after the external load has been completely removed. Since it is not fully elastic, its total deformation under a particular external stress does not take place instantaneously. When a clay sample is loaded, an instantaneous deformation is observed. The elastic deformation is only a part of its total deformation, however: as time passes, deformation continues under the same externally applied load. This part of the deformation which is time dependent is known as the "creep" or "flow". Thus the deformation of clay consists of two parts: elastic strain and creep.

"Creep" means the slow change in shape which takes place continuously as a consequence of constant or continuously changing shear stress. Due to creep the strain of soil increases with time even under the same external stress. Consequently the strain of a soil after a given time interval will be more than that given by classical laboratory short-term tests; its magnitude, however, will depend on various factors such as time, type and properties of soil, magnitude of deviator stress, and temperature. Thus in many practical cases the strain after a long time might be far more than is

acceptable although classical short-term tests might indicate safety against rupture. Hence the importance of studying creep behaviour.

Creep is particularly significant in cases of structures built on or against natural slopes, for creep movement affects and endangers the stability and safety of such structures. Creep also plays an important role in earth dams, foundations of concrete dams and in many other kinds of structures. The intention of this thesis was to study creep problems of cohesive soils, so emphasis will be given only to such soils.

In slides of natural slopes in cohesive soils, which often flatten towards the top, bowl-shaped sliding planes are observed. Vertical cracks are usually seen at the upper edge of the slope (Fig. 1). These cracks are formed due to the combined action of tensile and shear stresses at the top. When the tensile strength of the soil is overcome, the length along which cohesion acts is reduced due to the crack. This worsens the situation and progressive rupture takes place. But it should be remembered that creep or flow is a slow process: even after the tension cracks are formed, the slides do not take place rapidly but in most cases slow movement occurs over years, decades, or may be even centuries.

Reference to some actual cases of creep may be interesting in this connection.

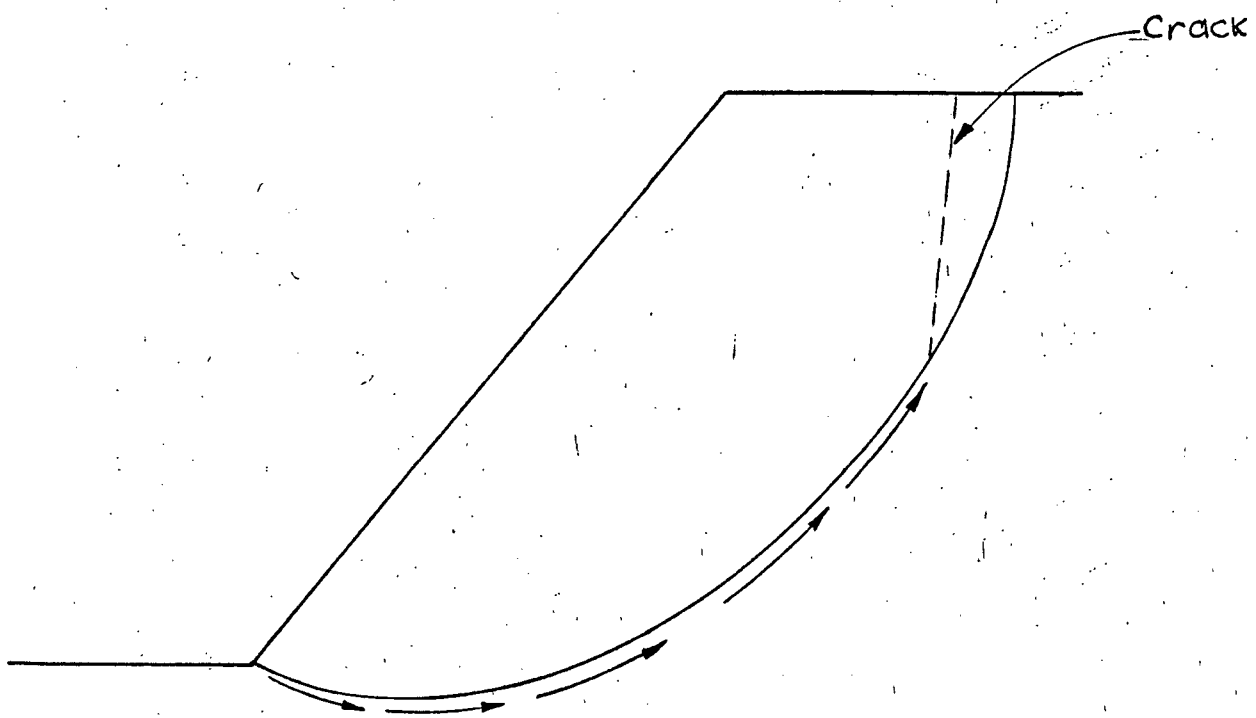


Figure 1

Professor R. Haefeli reports¹ that the soil movement at the southern Heinzenberg, resulting from a combination of creep and slide processes, covered an area of approximately 40 square kilometers consisting of "schistes lustrés". Such creep and slide movements often take place when the strata are parallel to the slope or, as in the mentioned region, when the inclination of the strata is flatter than that of the slope surface. In the latter case erosion at the foot of the slope brings a series of new sliding planes into action, one after another.

The surface movement measured in the years 1910-1931 at the southern Heinzenberg, which is exposed to marginal

erosion by the powerful Nolla torrent, varies between 10-26 centimeters at a mean slope of 25%. The thickness of the strongly distorted strata at the foot of the slope involved in the movement has been estimated as 50-150 meters. This is a case of particularly high creep movement covering a very large area.

In the case of the tunnel near Klosters¹ which covers a relatively much smaller area, the creep movement is rather deep seated and steady but relatively slow. The maximum rate of creep at the surface hardly exceeds a few centimeters a year. The deepest point of the tunnel involved in the movement lies at a depth of approximately 60 meters under the surface where creep velocity was measured as about 2 centimeters a year.

Similar movements can be found in cases of failure of buildings on slopes. Creep of backfill behind retaining walls and abutments causes a considerable engineering problem.

Thus creep movement occurs in small and large areas with high as well as low velocities; it may occur in situations which would be "safe" according to classical methods of analysis.

Rheology is that particular branch of science which deals with the deformation and flow of matter and as such covers both analytical and experimental studies. According to International Rheological nomenclatures, the term "creep" means deformations which are slowly recoverable after the removal of the applied load. Permanent or lasting deformations

are termed "flow"; but in soil mechanics the term "creep" is so widely used to denote permanent deformation that it might be misleading to replace it by the term "flow". At the same time it is realised that it is necessary to follow International Rheological nomenclature. So in the thesis the terms "creep" and "flow" will both be used to mean permanent deformation.

B. Previous Approaches to a Solution

Research workers in the past have attempted to study the creep behaviour of soil from various view points.

S.S. Vyalov and A.M. Skibitsky report² results of over 1000 experiments which were conducted to study the creep of both frozen and unfrozen soils.

Regarding the nature of creep curves in general they state that depending upon the value of constant external shear stress τ , deformations are either damping or non-damping: the former type is described as one where no failure takes place and the latter type the one which results ultimately in failure. Non-damping deformations first develop at a decelerating rate (shown in stage BC of Fig. A), passing afterwards into a stage of steady creep characterised by a constant rate of deformation represented by the linear section (stage CD of Fig. A) of the deformation versus time curve shown in Figure A.

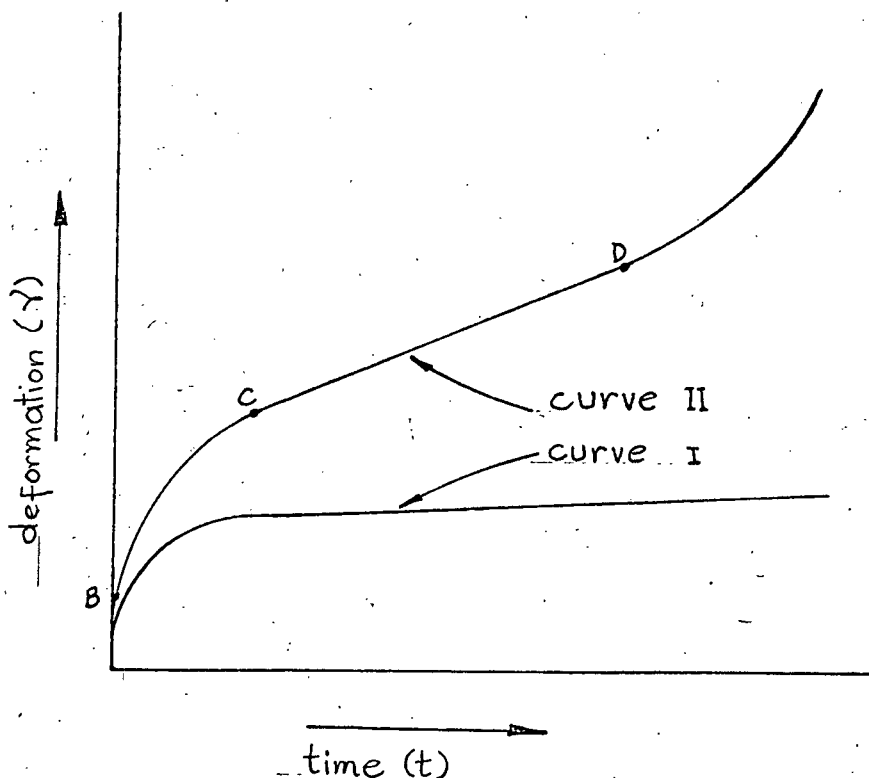


Figure A

Deformation Versus Time Curve

Curve I represents damping deformations.

Curve II represents non-damping deformations.

Further development of creep leads inevitably to failure. The higher the stress, the sooner failure will occur. If the applied shear stress τ is greater than the ultimate continuous shear strength τ_{∞} , non-damping deformations arise resulting in failure; on the other hand if $\tau < \tau_{\infty}$ the strain practically reaches a limit and failure does not occur. Although with $\tau < \tau_{\infty}$, slow flow is not impossible, it is of very little practical importance. This ultimate continuous strength, the authors claim, may be regarded as a very relevant and significant characteristics in calculating the continuous load carry-

ing capacity of soil.

The validity of this ultimate continuous strength was confirmed by rupture tests, in which all loads σ , greater than $.085 \sigma_0$ (where σ_0 is the ultimate instantaneous strength) resulted in failure; the load $\sigma = .085 \sigma_0$, however, did not result in failure, though the sample was under test for over 4 years.

In another paper³ which may be considered as an extension of that described above, the authors propose a method for calculating the creep and continuous strength of soil. They discuss the peculiarities that are encountered in cases of creep in dense clays. All these are explained in terms of a rheological model and a law of variation of viscosity is proposed.

M. Saito and H. Vezawa⁶ were concerned with the eventual failure of soils due to creep. They have expressed the view that the strain rate of the creeping soil in a slope may be taken as a good indicator of impending failure. They have reported that on a log-log graph, the creep rupture life (time to failure) is inversely proportional to the strain rate. They have suggested that an imminent failure could be forecast by observing a sudden rapidly increasing strain rate. Their most significant contribution is their proposal of an empirical formula relating the creep rupture life and strain rate which is independent of the kind of soil and testing procedure.

S. Murayama and T. Shibata⁴ have established theoretically the relation of strain characters of clay under constant external stress. They have stated that the strain is proportional to the logarithm of time - provided that the total applied stress is within a certain critical limit. This critical limit of stress corresponds to the point of first inflection in the graph of rate of strain versus shear stress (commonly known as the $D - \tau$ graph) designated as A in Fig. 2. This stress has been named the "Upper Yield Point" of soil. They claimed that if the total applied stress is below the upper yield point, no failure will occur even after long periods of straining. They have also proposed the existence of a restraining resistance in soil which in plain words means that for every soil there is a stress below which the strain rate is zero. They call this stress the "lower yield point" of soil (designated as B in Fig.2). Unless this stress is exceeded, there will be no viscous or retarded strain.

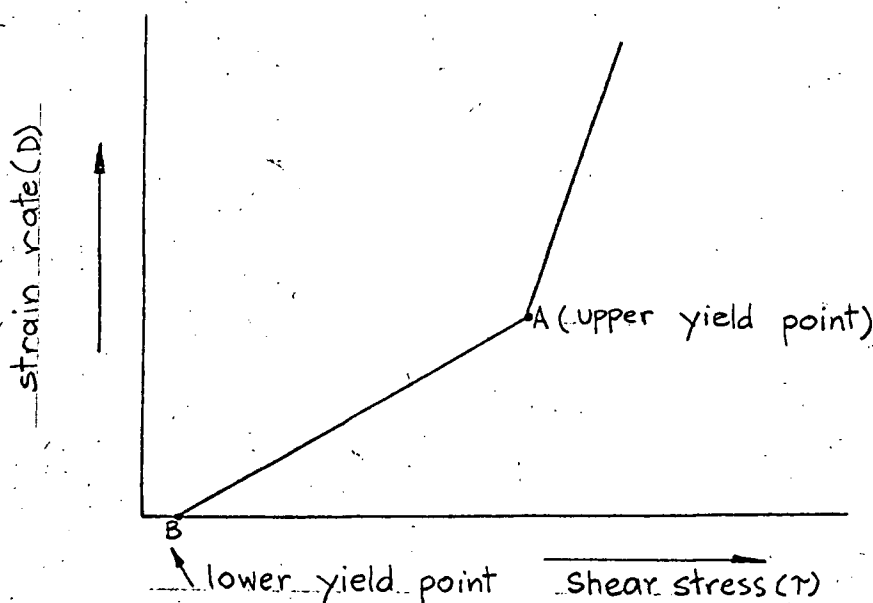


Figure 2

Another important feature of their paper is the study of thermal effect on the creep or flow of clay. They have deduced analytically and supported by experimental results a theory stating that the slope of the strain versus log-time graph bears a straight line relationship to absolute temperature. Thermal effect on the flow or creep characteristics of soil is a very important aspect as it is often encountered in practical problems.

C. Aims of this Thesis

The aim of this thesis was to study the effect of temperature on the flow characteristics of clay. Thermal effects on pore pressure were also measured. Experiments were conducted to verify the theories proposed by Murayama and Shibata.

CHAPTER II

THEORETICAL INVESTIGATION

A. Strain of Clay

As mentioned in Chapter I of this thesis, deformation of clay consists of two parts:

Elastic and Retarded or viscous strains.

A brief explanation of these two types of deformations is given below.

a) Elastic deformation: This may be caused by the bending of individual platy soil particle, grain compression, or by interparticle bonds acting as elastic hinges.

b) Retarded or Viscous deformation: With the passage of time, soil goes from a less to a more dispersed structure which allows a relatively more parallel particle orientation and as a result soil particles move relative to each other in a time-dependent manner.

In case of saturated or partially saturated soil, the absorbed layers of water may be considered to exert a considerable influence on the viscous strain of clay as illustrated below.

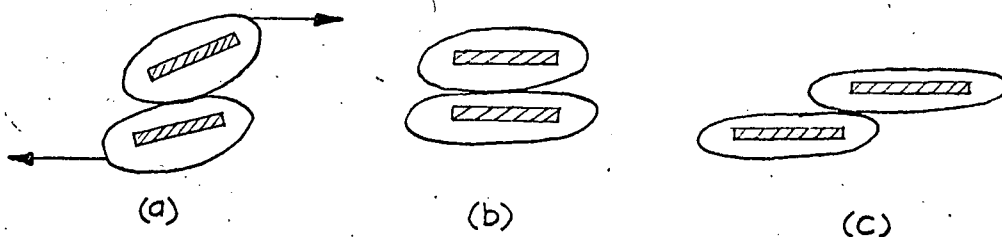


Figure 3

In a private communication to the author, J.K. Mitchell of the University of California asserts that creep has been observed even in the case of oven dried clays, implying that even with solid-solid contact, relative movement of grains can be time-dependent.

When a soil sample is sheared under an external load, the adsorbed water layers offer a sort of bonding effect against the smooth sliding of one clay particle with respect to another. But with time, the bound molecules of the adsorbed water layers are caused to move by shear stresses, and the adsorbed layers change shape, allowing movements or changes in orientation of the soil grains as shown in figure 3. Consequently soil shows viscous strain.

The combined strains can be shown schematically by the following figure 4.

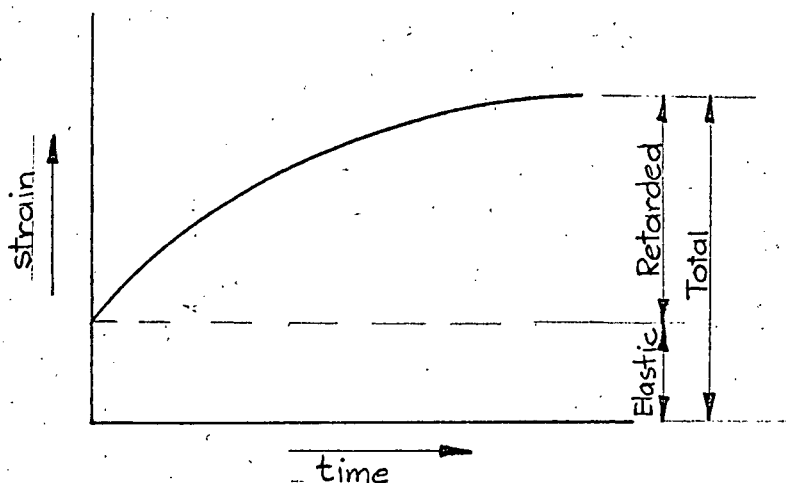


Figure 4

The above explanation only states qualitatively the nature of viscous strain but does not explain the flow behaviour in any detail. An attempt has been made in the following pages to develop an hypothesis explaining quali-

tatively the flow of soil. A quantitative analysis of the same, as developed by Murayama, is also presented.

B. Creep or Flow Behaviour of Clay: Qualitative analysis

Before mechanisms of flow involving quantitative analysis are discussed in detail, the author wishes to offer a physical explanation in a microscopic scale of what is actually happening in the soil mass. After that, a brief quantitative explanation will be given.

Particles of clay contain electrical charges on their surface. Pore water molecules, being polar in nature, are attached onto the surface of the clay particles forming what is known as the adsorbed water layers.

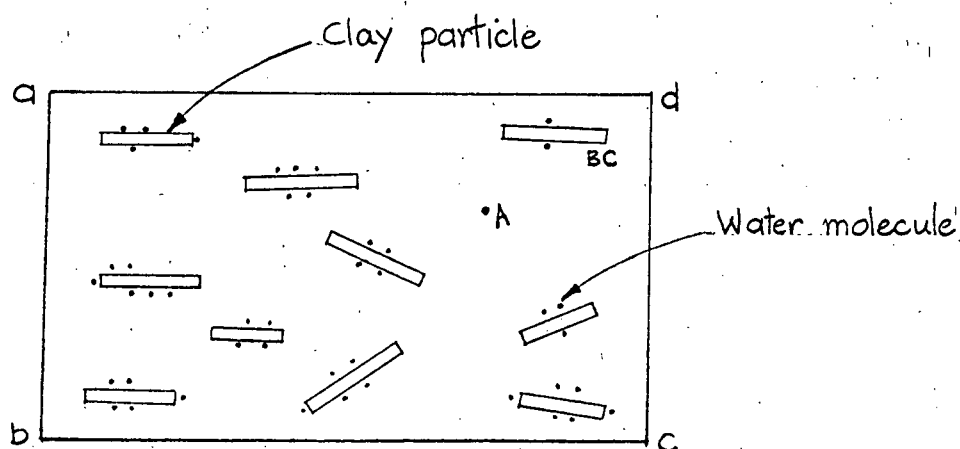


Figure 5

Consider an enclosed space abcd containing clay particles surrounded by adsorbed layers. The water molecules are represented schematically as dots in figure 5. The number of such water molecules that are attracted onto the soil surface varies under different environmental conditions. Consider an ideal case when the soil in the enclosed space abcd

has no external stress and the energies of the whole system is such that most of the water molecules are attached to the soil forming the adsorbed layers viz BC and all the molecules are at rest. However, some water molecules such as A exist as single molecules without any bond. The single letter A is used to signify no bond and two letters BC are used to denote the existence of only one bond.

Adsorbed layers of water consist of water molecules which are under some sort of bond or link with each other and also with the soil particle. Schematically this is shown in figure 6.

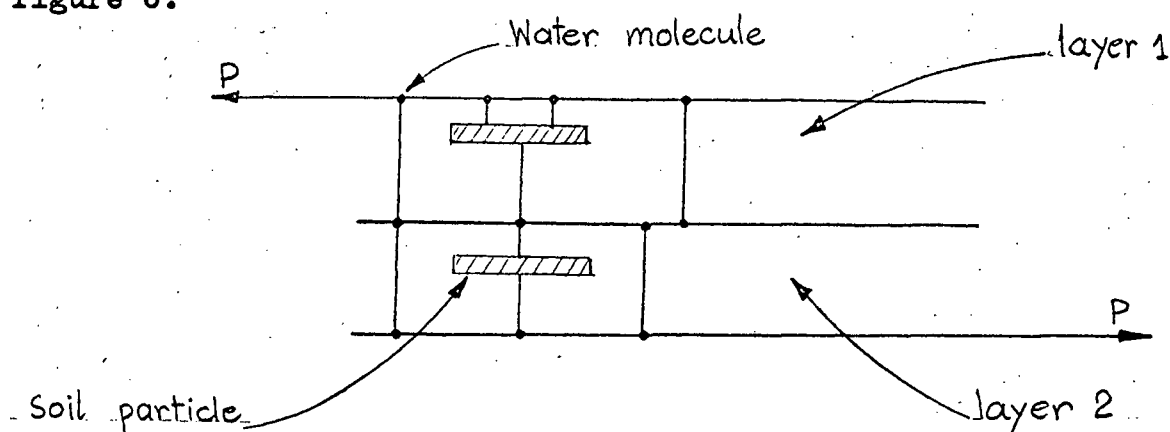


Figure 6

But a single free water molecule like A has no bond. If an external stress tending to cause shearing strain, is applied, the bonds present in the adsorbed layers will tend to resist the imposed shear and unless the bonds are broken partly or fully, the water molecules cannot be moved relative to one another. This can be illustrated by figure 6. Imagine a shearing force P tending to move the layer 1 relative to the layer 2. If P is not strong enough to break any one of the existing bonds, layer 1 will not move relative to layer 2.

On the other hand, if P is strong enough to break all the bonds, layer 1 will move away from layer 2 without any further resistance. In other words, under an external stress, molecules under bond will show less movement than those without any bond. So if an external stress is applied to the soil mass in abcd, it is expected that a free molecule such as A will tend to move first. Imagine that under an external stress A moves towards BC. As A comes closer to BC, the repulsion between A and BC increases resulting in the increase of energy of the whole system. Finally a stage may be reached when A comes very near BC and knocks off a molecule, say C, from BC. At that time all the molecules A, B and C are in some sort of unstable state such that any two may combine with one another to form a bond, viz A and B may combine to form AB and C will be released. Similarly B and C may combine to form BC leaving A free. The free molecule will again start moving to take part in the process of deformation. In the foregoing discussion only the interaction between A and BC, which may be considered as a unit process of deformation, was considered. In the whole system several such unit processes of deformations are taking place, bringing about the total deformation of clay.

If the external stress is greater, the reaction takes a somewhat different form. Continuing with the discussion of A and BC, if the external stress is beyond a certain limit, as A approaches BC, it knocks off either B or C in such a way that even in the final state no bond is reformed, i.e. there is no formation of units such as AB or AC or BC. A, B and C will remain separate without any bond. Since it was assumed

that in BC there was only one bond to start with, and that after one unit process of deformation only one bond was broken, there would be no bond left after the completion of the reaction. In a case where there might initially be more than one bond existing, even after the deformation process there may be still some bonds left in the unit. Thus the foregoing illustration does not necessarily mean that all the bonds formed by molecules in a unit such as BC are broken by one unit process of deformation; all it means is that in such a process of deformation, some bonds are broken and not all of them are reformed or repaired. The number of bonds breaking, however, depends on the amount of applied stress.

But one important fact should be mentioned here: when the external stress is such that no bond is broken permanently, creep of clay takes place only due to the exchange of position of molecules; a process which eventually changes the configuration of the clay particles somewhat. Nevertheless, the particles are still held by the bonds which prevent the sliding relative to each other in the medium of the adsorbed layers. But if all these bonds are broken due to very high external stress, there will not be anything to hold the particles against the external shear and that would mean failure of the soil.

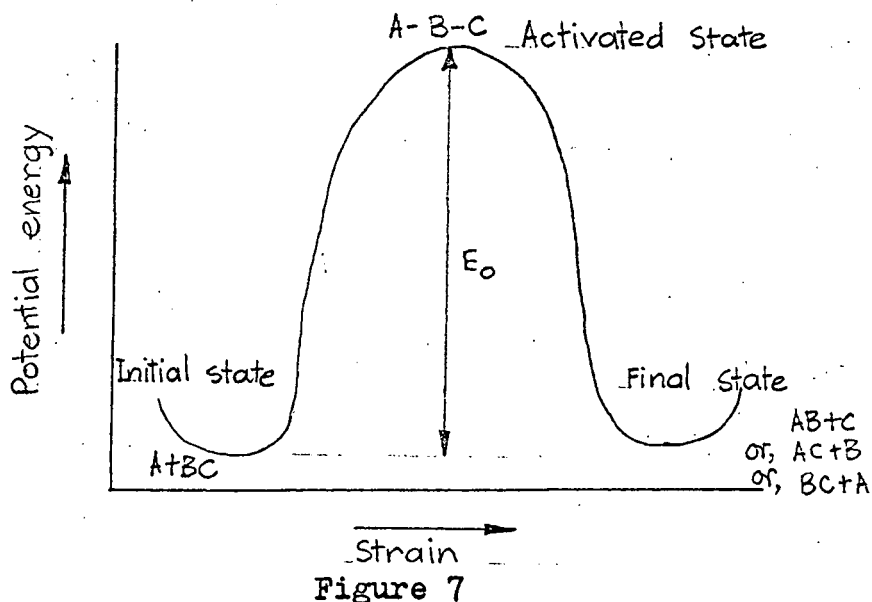
In the foregoing discussion one point was clear: irrespective of whether bonds were broken or not, molecules were in motion in order that the unit process of deformation might take place. In other words, if clay show retarded deformation or creep, the water molecules must be in motion.

Going back to the discussion involving A and BC, it was assumed that, to start with, the energy of the system was such that all the molecules were at rest and only when an external load was applied molecules started moving to take part in deformation process. This situation might call for an explanation from the point of view of the energy. At the beginning (i.e. before the application of external load) the molecules were at rest which meant that the energy of the system was so low that it could not keep the molecules in relative motion: some external energy was necessary to set the molecules in motion which was done by the application of an external force. But it might be misleading to think that the molecules could be set to motion only by an external force. All the force did was to supply sufficient energy to set the molecules in motion. In terms of physical chemistry, the molecules were activated or set in action by the external force. Similarly, molecules can be activated by supplying the necessary energy in some other form, viz: thermal energy. Thus it is possible to activate molecules by supplying thermal energy to the system. Therefore, in two identical creep tests if the temperature of one is increased keeping all other factors constant, the one with increased temperature is expected to show more strain - a fact supported by experimental results at the end of this thesis.

In macroscopic scale, it may be stated as follows. Soil particles move past each other through the medium of the adsorbed layers more easily if more and more bonds are broken,

resulting ultimately in the failure of the soil mass.

The whole reaction involving A and BC can be explained by means of a diagram of potential energy versus strain as shown in figure 7.



As explained before, with the approach of A to BC, the repulsion increases, thus increasing the potential energy of the whole system. When the potential energy reaches the peak as shown by the crest of the curve, the state is called the "activated state" and the difference of the energy between the activated state and the "initial state" is called the "energy of activation",⁵ denoted by E_0 . In terms of chemistry, activation energy is defined as the minimum energy that a system must acquire before undergoing appropriate changes. Physically it means the minimum energy that is necessary so that the interaction between A and BC may take place. In the activated state A-B-C are so disposed with reference to each other that B can be associated with either

A or C and a very slight displacement will result either in the reaction to form $AB+C$, $AC+B$ in the "final state" or return to the initial state with the formation of $A+BC$. The foregoing discussion, then, gives a graphical idea of one unit process of deformation, starting from the "initial state", passing through the "activated state", to the "final state".

C. Quantitative Analysis

Before the mathematical equations obtained by Murayama are discussed, an attempt will be made to summarize very briefly the theories proposed by Murayama and Shibata.⁴

Their observations, which they are attempting to explain, are:

1. Taking flow strain as the ordinate in natural scale and loading time as the abscissa in logarithmic scale, this relation results in a straight line and the tangent of the slope angle of this straight line increases linearly with applied stress.

2. If a stress larger than the upper yield value is applied, the flow strain increases and failure takes place after a certain time.

3. The slope of strain versus logarithm of time bears a straight line relationship to absolute temperature.

As mentioned in the earlier part of this thesis, Murayama and Shibata⁴ arrived at an equation giving the strain rate of clay. Their work is based on Henry Eyring's rate process theory.⁵

Henry Eyring arrived at the following equation for finding out the rate of strain of certain viscous liquids:

$$\text{Rate of Strain } \frac{de_1}{dt} = \frac{2\lambda n RT}{h} \exp \left(\frac{-E_0}{RT} \right) \sinh \left(\frac{\lambda}{2N} \frac{\sigma_1}{RT} \right) (1)$$

where,

R = Boltzmann's Constant

T = Absolute temperature

h = Plank's Constant

E_0 = Energy of Activation

λ = Average distance between two balanced positions of molecules.

n = number of molecules with activation in series per unit length in the direction of stress.

N = number of molecules with activation in one unit area of cross section perpendicular to the direction of stress.

In his calculations, Eyring assumed that number of molecules with activation and hence the number of unit process of deformation was independent of the applied stress .

Murayama and Shibata for finding out the rate of strain of clay made the following assumptions and accordingly modified the equation (1) to suit their assumptions.

Their assumptions were:

a) Unlike Eyring's theory where it is assumed that the number of molecules with activation is independent of the applied stress, Murayama assumed that n and N are functions of the total stress applied to the clay sample.

b) Clay possesses, inside its bonds, certain resistance to applied stress. They called it the "Restraining Resistance" and denoted it by σ_0 in their equation. Physically it means that only when the applied stress σ is greater than the restraining resistance σ_0 , does the process of viscous deformation start. In other words, if the applied stress is below σ_0 , no molecule is activated to take part in the unit process of deformation.

Writing these assumptions in mathematical form:

$$\begin{aligned} \text{when } 0 < \sigma < \sigma_0 \quad n \text{ or } N &= 0 \\ \sigma > \sigma_0 \quad n &= a (\sigma - \sigma_0) \\ N &= b (\sigma - \sigma_0) \end{aligned} \quad (2)$$

substituting equation (2) in equation (1), they obtained the following equation relating the rate of strain of bond $\frac{de_b}{dt}$ with the stress on the bond σ_b .

$$\begin{aligned} \text{Thus, } \frac{de_b}{dt} &= A_b (\sigma - \sigma_0) \sinh \left(\frac{B_b \sigma_b}{(\sigma - \sigma_0)} \right) \\ \text{where } A_b &= \frac{2\lambda a kT}{h} \exp \left(- \frac{E_0}{RT} \right) \\ \text{and } B_b &= \frac{\lambda}{2 b RT} \end{aligned} \quad (3)$$

It is known that for materials showing viscous strain, $\sigma = \eta \frac{de}{dt}$

where σ = applied stress

η = coefficient of viscosity

$\frac{de}{dt}$ = strain rate

Hence from equation (3) the apparent coefficient of viscosity is represented by

$$\eta_b = \frac{(\sigma - \sigma_0)}{\frac{d\epsilon_b}{dt}} = \frac{1}{A_b \sinh\left(\frac{B_b \cdot \sigma_b}{\sigma - \sigma_0}\right)} \quad (4)$$

D. Rheological Model to Explain the Flow Characteristics of Clay

In order to simulate the visco-elastic behaviour of clay, a model was proposed by Murayama. In order to represent the presence of the restraining resistance of clay, the slider element σ_0 was added to the ordinary three element model. In clay, as has already been discussed in the

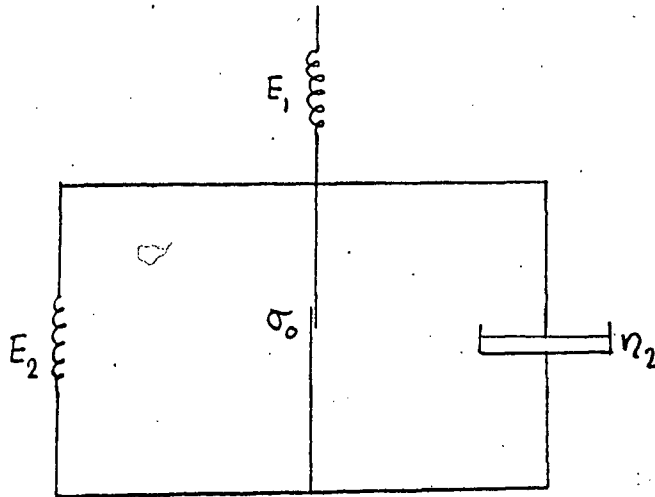


Figure 8

Rheological Model Proposed by Murayama and Shibata

beginning of this chapter, there are elastic as well as retarded or viscous deformations. When a load is applied, a sudden elastic deformation takes place and after that viscous deformation takes place slowly with time. The model resembles the visco-elastic behaviour in the following way:

When an external stress σ is applied, the spring element E_1 which resembles the elastic behaviour of soil is immediately depressed showing elastic deformation. But the load is not transferred to the spring element E_2 or the dashpot η_2 unless it is greater than σ_0 which, physically, means that unless the applied stress is more than the restraining resistance acting inside the bonds, no viscous deformations take place. If the total applied stress is more than σ_0 , the balance i.e. $(\sigma - \sigma_0)$ is transferred to E_2 and dashpot η_2 causing viscous strain.

Murayama and Shibata claimed that the apparent coefficient of viscosity η_2 of the voigt element in the proposed model is represented by the apparent coefficient of viscosity expressed in equation (4).

Thus the apparent coefficient of viscosity η_2 for this mechanical model is given by

$$\eta_2 = \frac{1}{A_2 \cdot \sinh\left(\frac{B_2 \cdot \sigma_2}{\sigma - \sigma_0}\right)} \quad (5)$$

E. Equation Relating Strain Rate and Absolute Temperature

Assuming total applied stress to be σ and the strains in the elastic spring element and modified voigt element to be ϵ_1 and ϵ_2 respectively, Murayama arrived at the following equation relating strain rate and absolute temperature. Referring to the rheological model,

$$\text{Total strain } \epsilon = \epsilon_1 + \epsilon_2$$

Total applied stress $\sigma = \epsilon_1 E_1$

$$\text{also } \sigma = \epsilon_2 E_2 + \frac{(\sigma - \sigma_0)}{B_2} \sinh^{-1} \left\{ \frac{1}{A_2 (\sigma - \sigma_0)} \frac{d\epsilon_2}{dt} \right\} + \sigma_0 \quad (6)$$

$$\text{If } 0 < \epsilon_2 < \frac{\sigma - \sigma_0}{2B_2 E_2} (2B_2 - 1)$$

ϵ is approximately given by

$$\begin{aligned} \epsilon &= \frac{\sigma}{E_1} + \frac{(\sigma - \sigma_0)}{E_2} + \frac{(\sigma - \sigma_0)}{B_2 E_2} \log \frac{A_2}{2} B_2 E_2 t \\ &= \frac{\sigma}{E_1} + \frac{(\sigma - \sigma_0)}{E_2} + \frac{(\sigma - \sigma_0)}{B_2 E_2} \log \frac{A_2 B_2 E_2}{2} + \frac{(\sigma - \sigma_0)}{B_2 E_2} \log t \end{aligned}$$

$$\text{or, } \frac{d\epsilon}{d \log t} = \frac{(\sigma - \sigma_0)}{B_2 E_2} \quad (7)$$

Comparing equations (4) and (5) and realising that η_2 in the model is expressed by equation (4), B_b of equation (3) can be substituted for B_2 in equation (7). Hence substituting the value of B_b in equation (3) for B_2 in equation (7) and neglecting σ_0 ,

$$\frac{d\epsilon}{d \log t} \approx \frac{2bkT\sigma}{\Lambda E_2}$$

$$\text{or } \frac{d\epsilon}{d \log t} \approx \frac{2bk\sigma}{\Lambda E_2} \times T \quad (8)$$

From the equation (8) Murayama and Shibata claimed that $\frac{d\epsilon}{d \log t}$ and absolute temperature T are directly propor-

tional if the applied stress σ is kept constant.

This conclusion would be valid if E_2 and b were independent of T . Discussion of the truth of these conditions will be given later in connection with the results of this thesis.

CHAPTER III

EQUIPMENT AND TESTING PROCEDURE

A. Undisturbed Soil Samples

The undisturbed soil samples were obtained from Haney, British Columbia; hence the soil is described as "Haney Clay." All the experiments presented in this thesis were performed with undisturbed Haney Clays.

The physical properties of the soil are as follows:

Specific gravity = 2.79

Grain size finer than 2 microns = 50%

Liquid limit = 50%

Plastic limit = 25%

Plasticity index = 25%

Natural water content = 45% - 49%

Degree of saturation = 100%

Maximum past pressure = 35 psi

From the point of view of visual inspection, the soil is greyish black, fairly stiff and quite sticky when rubbed between fingers. Samples consisted of handcut blocks. All the samples did not have the same water content. Even in the same block it was observed that the water content at the centre was different from that at the corners. While cutting smaller pieces from the big block for preparing specimens, sufficient care was taken so as not to expose the cut surface of the main block to the atmosphere for a long time. Imme-

mediately after a small piece was cut, the big piece as well as the small piece were coated with thick wax to keep the water content unchanged. This precaution was found to be necessary because it was found that if the sample was exposed to the atmosphere even for a small period, the water content changed.

B. Preparation of Samples for Testing

Soil samples were cut out in the form of cylinders of diameter 1-5/16" and length 2-7/8" by means of standard cutting tool as shown by figure 9. Cutting of soil samples was done in the manner described in the book "Soil Testing for Engineers"⁷ by T.W. Lambe - John Wiley & Sons. Only one porous stone was used, since volume change was not measured. The porous stone was heated and saturated to assure that no air bubbles were left inside. This was done before cutting the sample. Immediately after the sample was cut, the porous stone was placed on the pedestal. Sufficient care was taken to ensure that no air bubble was entrapped between the stone and the pedestal. Also every precaution was taken to remove any excess water on the pedestal. It was observed that if there was any excess water, it often softened the bottom of the sample which led to early failure in the case of undrained tests (Q test). With some practice and sufficient care it was found that the water could be removed quite satisfactorily. Rolling the rubber membrane over the sample was found to be fairly difficult. However, if sufficient care was taken, satisfactory results were obtained. "O" rings were rolled onto the membrane to seal the specimen.

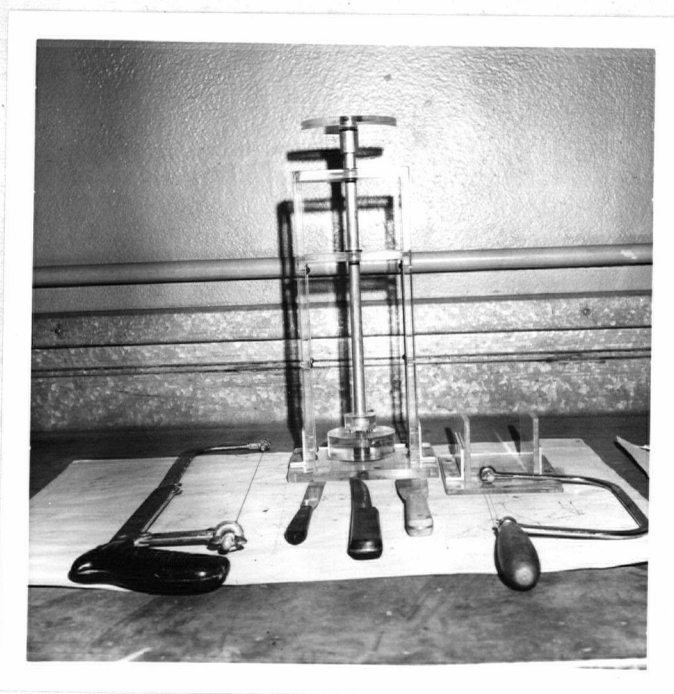


Fig. 9 Cutting Tools.

C. Testing Procedure

1. Arrangement for Temperature Control

The temperature was kept constant during a particular test. A rectangular jacket surrounding the standard triaxial cell was devised to act as a temperature bath as shown in figure 10. Two sides of the outer jacket or bath were made of lucite sheet to allow observation of the soil sample inside; the other two sides were made of steel. Inside the bath an immersion heater in series with a thermostat was installed to keep a particular temperature throughout the test. In order to avoid local concentration of heat, an automatic stirrer was used to keep the water in the outer bath in constant circulation. The thermostat was calibrated by running a temperature test with water at different temperatures. It was observed that up to about 100°F. there was almost no temperature drop in about 3 days. When the temperature was between 110°F. to 120°F. there was a drop of 1.5°F. during three days. While testing soil, care was taken above 100°F. by adjusting the thermostat at an interval of about 8 hours, to prevent any temperature drop in the soil. As a whole, the temperature control system worked very well during all the tests. Attempts were made to heat the chamber fluid (which was water) to within 5°F. of the intended test temperature before admitting it to the chamber. Some difficulty was encountered here, however, since considerable heat was lost in the balancing reservoir. By trial and error the temperature of input water was so set that, after loss in the balancing

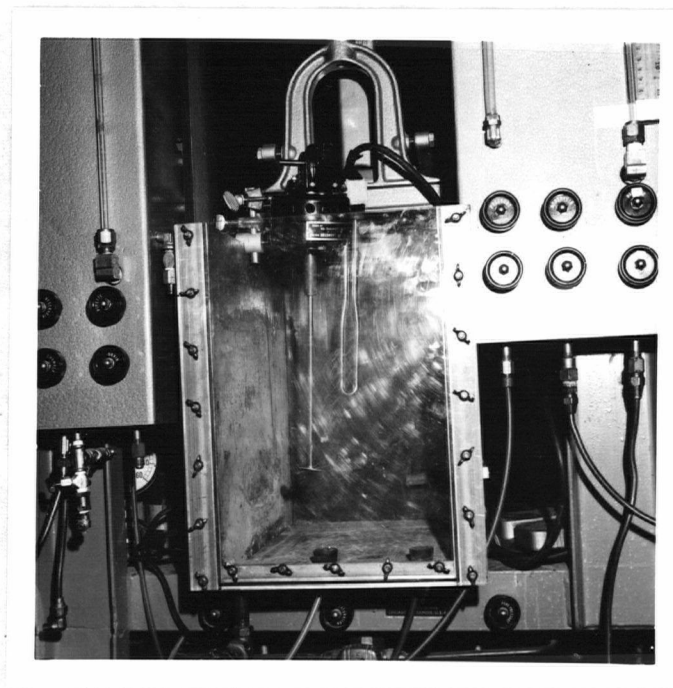
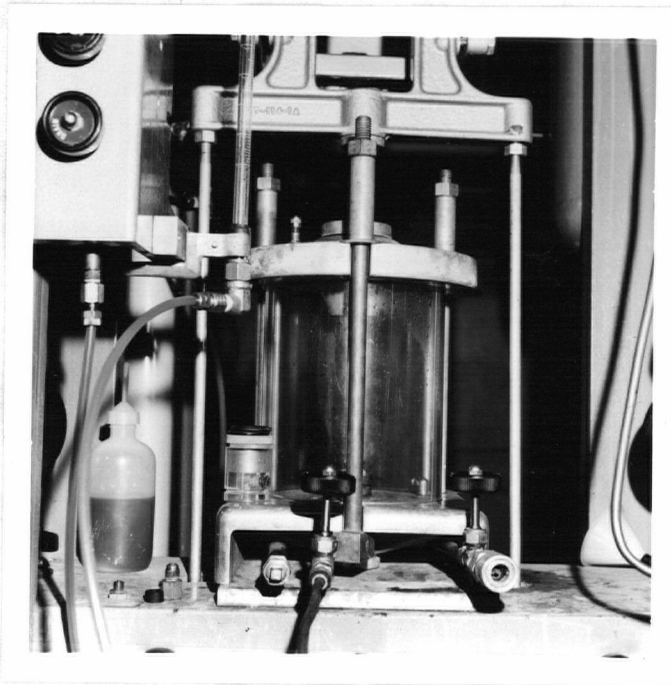


Fig. 10 Triaxial Cell
 and
outer jacket for temperature control.

reservoir, the water in the triaxial cell would have approximately the same temperature as that of the test. For example if it was desired to maintain 80°F. inside the triaxial cell, it was found that if water at about 92°F. was admitted to the balancing reservoir, it would end up at about 80°F. inside the cell. Room temperature in the laboratory had a considerable effect on the heat loss and proper allowance had to be made for that.

2. Cylindrical Triaxial test

Two types of tests were done in the triaxial machine on similar types of soil sample:

- a) Undrained.
- b) Consolidated undrained.

Detailed procedures of testing are discussed under two different heads.

a) Undrained test. After the soil sample had been properly placed on the pedestal of the triaxial cell, as described earlier, hot water was admitted to the cell at a temperature corresponding to that of the test. In that condition the sample was kept for about 3 hours so that the soil could be heated up to the desired temperature. No drainage was permitted during this time. Then the cell pressure was applied in six equal increments at the rate of 5 psi every 5 minutes. Thus it took half an hour to build up 30 psi, the chamber pressure used in tests. No drainage was permitted during the application of the all round pressure. The uplift on the piston due to the all round pressure was counter-

balanced by dead weight. Then the necessary deviator stress was applied by means of additional dead weights. A dial gauge reading to $\frac{1}{1000}$ in was used to measure deformations. Readings were taken at intervals of 1 minute for the first five minutes, intervals of 5 minutes from 5 up to 60 minutes and then at the rate of one every 10 minutes up to 100 minutes. After 100 minutes the recording time interval was increased.

It was desired that the applied deviator stress be such that the soil sample would fail in about a day. This creep rupture time or time to failure was chosen arbitrarily to give a fairly long and well defined curve.

For a given water content, the critical deviator stress to cause failure in a day was found by trial and error, and kept constant for all the tests of a set. Several experiments had to be performed to arrive at the required deviator stress because if the water content of one sample was a few percent different from that of another, there was a considerable change in the failure time under the same deviator stress even at the same temperature. Since the water content of the samples could not be totally controlled in spite of care, several experiments had to be carried out and the result of those samples having almost same water content were recorded for comparison. To get the water content of the samples almost the same, all the samples for a set of tests were taken from the same block of soil. Even from the same block, only the central portion was taken because at the edge or side the water content often changed as noted before.

b) Consolidated Undrained Tests. The apparatus, preparation and cutting of the samples, the size of the samples and the general method of testing were the same as described in the undrained tests. Only the changes are noted here.

The maximum past pressure of the soil under investigation was found to be about 30 - 35 psi. In order to avoid the effect of precompression, the all round pressure was raised to 50 psi. Drainage was permitted during the consolidation period, which was about $2\frac{1}{2}$ days. Consolidation was done at that particular creep test temperature. Since after $2\frac{1}{2}$ days of drainage, all the samples came up to almost the same water content, irrespective of the initial water content, the latter was less important in this case. Thus in this set of tests, fewer tests were necessary. One test at each temperature was performed and the results were quite consistent. Pore pressures were recorded at each temperature. Time intervals for recording the strains were exactly same as described in the case of undrained tests. Pore pressures were measured simultaneously with the strains.

3. Pore Pressure Panel

The pore pressure panel in which all the tests were performed is based on the M.I.T. design as shown in figure 11. The pore pressure measurement is based on applying a back pressure to the pore water to attain a "null" condition so that there is no movement of water from or into the specimen. The back pressure thus recorded is used to represent the pore

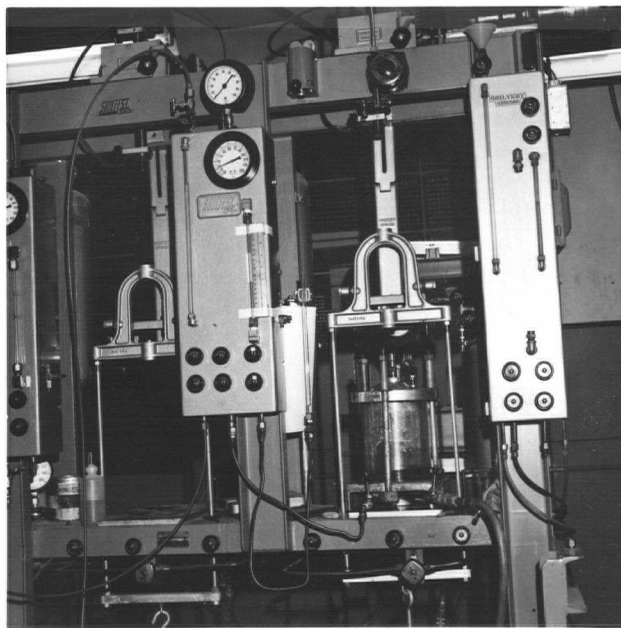
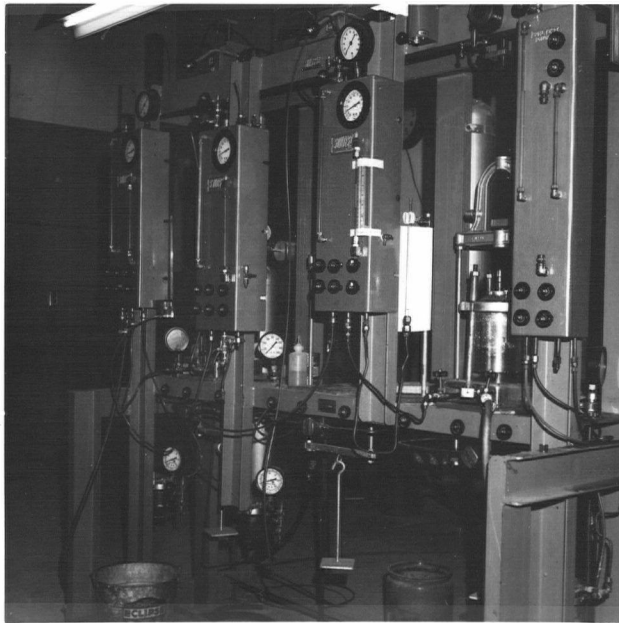


Fig. 11 Pore Pressure Panel.

pressure at the bottom of the sample. In this whole process of measuring pore water pressure, it must be ensured that the whole system from the end of soil sample to the panel is de-aired at all times, otherwise the pore pressures obtained would be erroneous.

a) De-airing of Pore Pressure Panel. The complete layout for the de-airing of the apparatus is shown diagrammatically in figure 12. The procedure for de-airing is outlined below.

1) With valves 1, 2, 3, 4, and 6 open and the rest of the valves shut, a vacuum of 30 inches of mercury was applied to the system. This was maintained until the movement of water in the glass tube (1.5 mm. I.D.) ceased.

2) Even with a vacuum of 30 ins. of mercury it was noticed that some air always became entrapped in the long tube connecting the outlet of the de-aired water tank to the pedestal of the triaxial cell. This air was driven away by opening the valve X momentarily. This momentary opening of valve X forced the entrapped air within the transparent tube into the pore pressure panel which was kept under vacuum. By repeating this process once or twice it could be clearly observed whether or not there was still any air in the long transparent tube. This was repeated until all the air bubbles were driven out.

3) Valve 4 was then shut, and by opening the valve 5, water was driven from the de-aired water tank, through the whole system into the reservoir of the pore-pressure panel. While water was being drawn through the system, rapid closing

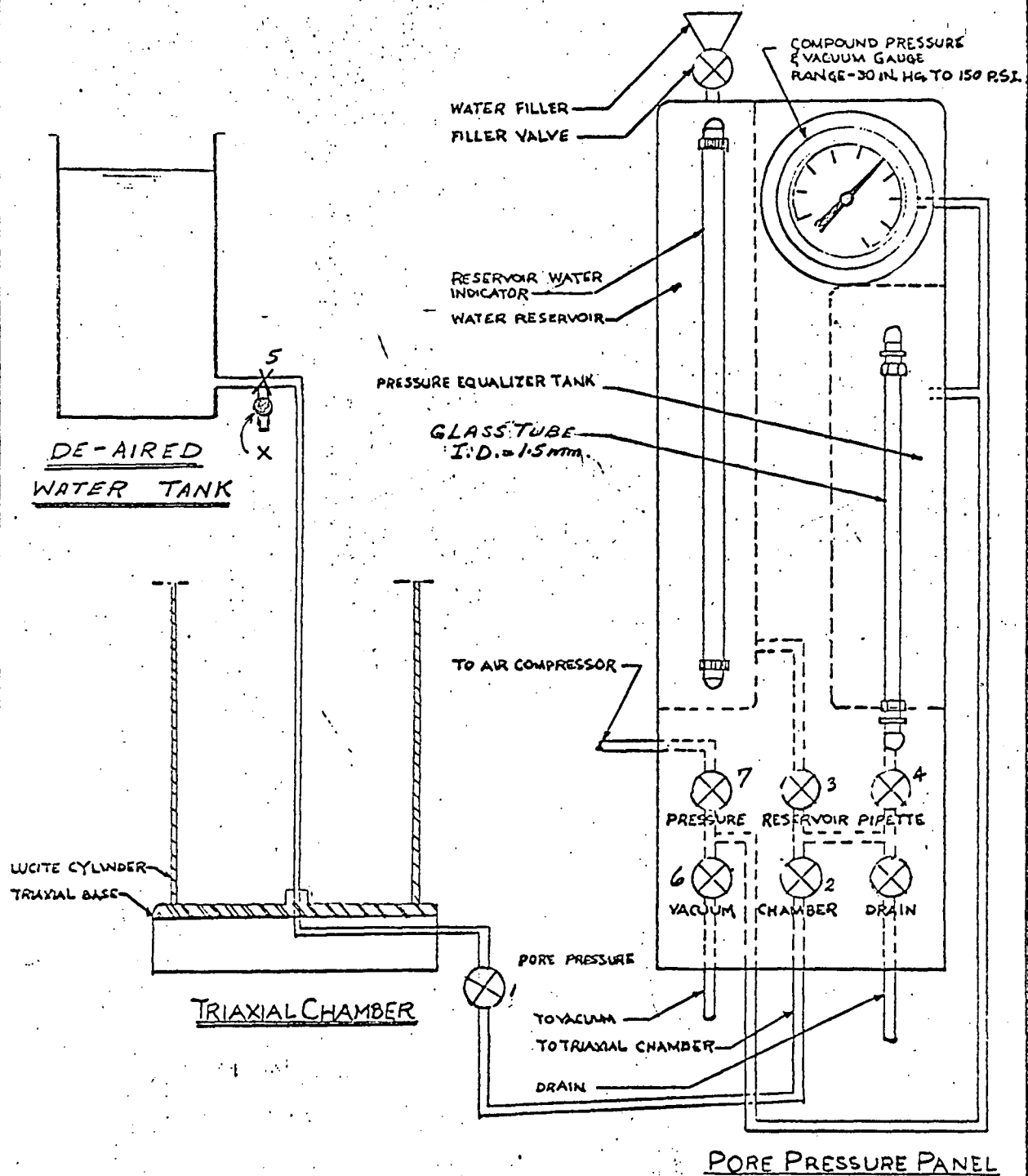


Figure-12. A Schematic Diagram To Show The Layout
For De-airing of
Pore Pressure Measuring Apparatus.

and opening of valves 2 and 3 helped the removal of air from the valves themselves. This rapid opening and closing of valves was found to be very useful.

4) When the reservoir was full, valve 3 was closed. Again rapid opening and closing of valve 4 removed air from the valve itself.

5) To check that the whole system was de-aired, valves 1 and 6 were closed and valve 4 was opened. Valve 7 was temporarily opened to get a convenient level of water in the glass tube while there was a positive pressure in the pressure equalizer tank.

6) The pressure was increased by opening valve 7. A drop in the water level in the glass tube indicates either expansion of the apparatus, compression of the Teflon Washer, compression of the air bubbles, or leakage. When valve 7 is adjusted to maintain constant pressure in the pressure equalizer tank, steady creep of the water level indicates leakage. A large drop which is not fully recoverable indicates air bubbles. Sometimes, due to the compression of the Teflon Washer, a drop was observed in the water column in the tube which was not all due to air. This difficulty was, however, overcome by keeping some pressure overnight before starting the experiment. With a pressure of 50 psi a drop of $\frac{1}{2}$ " could be expected.⁸ It was observed during the experiments that if the pressure of 50 psi was kept for about 2 days, a drop of less than $\frac{1}{4}$ " could be obtained.

CHAPTER IV

DISCUSSIONS OF THE RESULTS

As has been reported in Chapter III, there were two sets of tests: undrained tests without pore pressure measurements; and consolidated undrained tests with pore pressure measurements. The chamber pressure in the first set was 30 psi, the chamber pressure was raised to 50 psi during the second set of tests to ensure that the soil was normally loaded.

(a) Choice of deviator stress in undrained tests: Since the total stress was kept constant for all temperatures, it was desired to find a stress under which the soil would fail at approximately 1000 minutes at 70°F. The choice of the failure time of about 1000 minutes carries no special significance excepting that it would give long creep time to study the creep curves. Also it was felt that since time in minutes would be plotted to a log scale for some curves, 1000 minutes would be a good figure to select.

The particular deviator stress which would give a creep time of about 1000 minutes was found by trial and error. It was found that for a particular water content, if the deviator stress was increased or decreased even by 1 psi, creep time changed considerably. This fact is clearly shown by figure 13. Referring to curves in figure 13, it can be

seen that with 14 psi, the samples failed in about 3 minutes, with 13 psi in 20 minutes, with 12 psi in 60 minutes and with 9 psi it did not fail for about 2 days. Finally it was found that under a stress of 10.5 psi the sample failed in about 1300 minutes at 70°F. Hence 10.5 psi was chosen as the deviator stress for all temperatures for the first set of undrained tests.

(b) In the consolidated undrained tests, the deviator stress was selected arbitrarily to get a fairly long creep curve. Samples were not tested up to failure. The main aim of these tests was to study the behaviour of the pore pressures with temperature. Also strain-time - temperature curves were plotted to see if they were consistent with the first set of experiments. A deviator stress of 23 psi was used and that was kept constant for all temperatures.

In general the following features were observed and they are discussed one by one.

1. Variation of strain with time at a particular temperature under a constant total stress.
2. Comparison of strain curves at different temperatures.
3. Variation of pore pressure with time at a particular temperature and constant total stress.
4. Comparison of the pore pressure curves at different temperatures.
5. Experimental verification of the fact that the slope of the strain versus log-time curve bears a linear relationship with absolute temperature.

A. Variation of Strain with Time at a Particular Temperature under a Constant Total Stress

For the first set of experiments, the strain-time curves were plotted for five different temperatures viz: 70°F., 80°F., 90°F., 100°F. and 110°F. (Fig. 14), the total stress of 10.5 psi was kept constant for all five temperatures.

For the second set i.e. the consolidated undrained tests, experiments were carried out at 70°F., 90°F., 100°F. and 110°F. the total stress of 23 psi was constant again for all temperatures (Fig. 15).

This section will begin with a physical description of the creep curves. After that an attempt will be made to explain the curves in the light of creep theory. Since for a particular test, say 70°F., 80°F. or 100°F. the temperature was kept constant throughout, the explanation will be given in relation to the variation of strain with time at any constant temperature. Since the nature of variation of the creep curves is of the same kind for any temperature, the explanation should hold good for any of the curves presented.

1. Physical Description of the Creep Curves

Referring to figures 14 and 15, it can be clearly seen that the flow or creep curve at any temperature has an elastic part and a visco-elastic part. In the experiments, the first recorded strain was obtained approximately 1 minute after the beginning of the load application. They would therefore contain some creep strain but are presumed to be

representative of the elastic or instantaneous strains.

After the elastic strain, viscous deformation continues non-linearly with time. S.S. Vialov and A.M. Skibitsky³ report that when the applied stress is greater than the ultimate continuous stress, (that is, the stress above which failure will occur in a sample stressed for a long time) the flow curve consists of the following stages as illustrated by the accompanying figure.

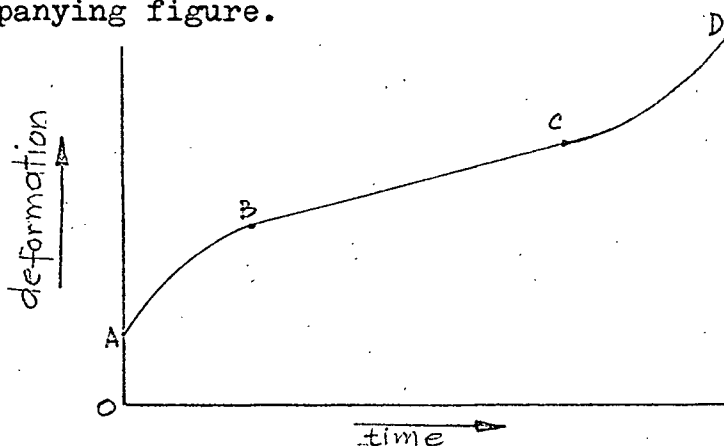


Figure 12(a)

- 1) Elastic deformation - OA
- 2) Variable - AB
- 3) Steady plastic flow at a constant rate - BC
- 4) Progressive flow at an increasing rate - CD

In the first set of experiments where the samples failed in each case, the flow curve consisted of the following stages (Refer to figure 14, graph at 90°F.)

- 1) Elastic deformation - OA
- 2) Plastic flow at a variable rate - AB
- 3) Plastic flow at a less variable rate than that in stage 2 but not quite at constant rate - BC
- 4) Progressive flow ending in failure - CF. Just

before the sample failed, very high strain rate was detected in each case by the dial gauge, before any failure plane was apparent to the naked eye. In the creep curve in figure 14, failure is shown by a very steep slope and marked F to indicate that just before failure the slope became very high.

In the second set, the deviator stress was relatively low as it did not cause failure of any sample within about 3000 minutes.

Flow curves in this case (the second set of tests) consisted in general the following stages: (Refer to figure 15, curve at 110°F.)

- 1) An elastic part - OA
- 2) Plastic flow at variable rate - AB
- 3) Plastic flow at almost constant rate - BC

2. Explanation of the Flow Curves in the Light of

Flow Theory

It is clear from the flow curves that at any particular temperature, the strain increases with time. Consider the strain-time curves at 70°F. for both undrained and consolidated undrained tests. Immediately after the stress is put on, there is an elastic deformation which may be mainly due to the bending of the plate like soil particles accompanied by some structural changes.

After the elastic stage, viscous or retarded strain becomes markedly predominant. To start with, the soil mass has a potential energy consistent with equilibrium at 70°F. When the load is put on, additional energy is supplied and the molecules in the adsorbed layer are activated to a state

out by the fact that the maximum strain at the end of about 2000 minutes was only 3.5%. The strain increased with time in the second set possibly due to the mutual exchange of position of the molecules causing weakening of the existing bond but possibly no major bond breakage.

From the point of view of soil structure, it may be explained as follows: With the passage of time, as the bonds become weaker, even under the same external stress, the soil is subjected to more and more shear strain. This shear strain causes the soil particles to be aligned in a more parallel array and consequently, the structure becomes more and more dispersed with time. This becomes apparent in the form of increased strain as shown by the creep curves.

The differences due to different starting temperatures will be discussed in the next section.

B. Comparison of the Strain Curves at Different Temperatures

During each test the temperature was kept constant throughout; but several tests were performed at different temperatures with the same constant total stress to observe the influence of temperature.

If two different creep tests are conducted with the same soil (initial structure presumed the same) at the same temperature and the same total stress under identical conditions, it is expected that (neglecting experimental and instrumental errors), after any given time interval, the strain should be the same for both tests.

Creep tests presented in this thesis were conducted at different temperatures. Thus any difference in strain of one test from that of another after any given interval of time, may be attributed to the difference in temperature.

Referring to figure 14 and comparing the strains of different creep curves after any interval of time, say 100 minutes, it is clear that the strain is maximum at 110°F, being less at lower temperatures and finally minimum at 70°F. Similar situation exists in the second set of tests (Refer to figure 15).

This observed fact can be explained in the light of creep theory in the following way.

At 70°F. the soil mass and its adsorbed layers have certain amount of energy in the whole system; the same soil at 110°F. however, will have more energy in the system because of higher temperature. If now an equal external stress is applied separately on soils at 70°F. and at 110°F., it may be expected that the energies of both the systems will be increased equally so that after the application of the external stress, the total energy of the soil at 110°F. will be more than that at 70°F.

Thus considering the creep tests, it can be said that after any given time interval, soil at 110°F. will have more energy than that at 70°F. and this, in the light of creep theory would mean that relatively a greater number of molecules will be activated at 110°F. than those at 70°F.

Activation of more molecules means more weakening

of the bond and consequently more strain.

Thus the observed fact that after any given time interval, the same soil under the same total stress but at a higher temperature (in a different test) would show relatively more strain, is quite consistent with the theory of creep.

C. Variation of Pore Pressure with Time at a Particular Temperature and Constant Total Stress.

As mentioned earlier, pore pressure was measured only in the second set of tests, i.e. the consolidated undrained tests. Consolidation was done under an all-round pressure of 50 psi for over $2\frac{1}{2}$ days at creep test temperature. No filler paper side drain was used. Then a deviator stress of 23 psi was put on and the soil was sheared without drainage. Tests were conducted at four different temperatures but during each test the temperature was kept constant throughout. Since the nature of variation of pore pressure curves is almost the same for all the tests, the explanation regarding the variation of pore pressure with time will be given in this section. The effect of temperature on the pore pressure will be discussed as the next item.

When the deviator stress was put on, it was expected, as is generally the case for a normally loaded clay, that the pore pressure at the beginning would reach a magnitude corresponding to an A value of .5 to 1.0. But that was not the case. The A value was as low as .2. The value of A was considerably low for all the experiments: there was only a slight but insignificant difference in value from one experiment to another.

Following is a discussion of the possible reasons why such low A values were observed.

It was at first suspected that the soil in question was very impermeable and that the reason for the low value of pore pressure at the beginning was simply a lag in the response of the measuring system. But this explanation had to be abandoned because even after considerable time - say 50 minutes - the pore pressure was still very low. Impermeability could be the cause of a time lag but could not be the cause of a permanent low reading.

It was next suspected that the soil might not be fully saturated. In the case of a partially saturated soil, the development of pore pressure due to an applied deviator stress is lower because the pore fluid is compressible. Trapped air is reduced in volume in accordance with Boyle's Law and is partially dissolved into pore water in accordance with Henry's law. Consequently the pore pressure developed in partially saturated soil would be less than that in a fully saturated one by an amount dependent on various factors such as the degree of saturation, the value of the applied deviator stress, etc. Thus to check the saturation, the B value was measured. Chamber pressure was raised in five increments at the rate of 10 psi and after each increment the B value was measured. In general for the soil in question the B values were as follows:

10 psi - $B = .8$

20 psi - $B = .8$

30 psi - $B = .8$ or $.9$

40 psi - $B = .9$

$$50 \text{ psi} - B = .9$$

The B values as recorded above were not very consistent. The amount of air that was present in the sample at 10 psi to yield $B = .8$ should have been dissolved before 50 psi, so that B would then be unity. But from the values obtained, B was not equal to 1 even at 50 psi. This would suggest an instrument error. So it may be assumed that even if the soil was not saturated at the beginning, at 50 psi it had reached full saturation. Thus the suggestion that the A value was low because of incomplete saturation could no longer be considered as valid.

A value of 1.0 for normally loaded clays⁹ are probably for marine clays. For fresh river deposits the value might be quite different. Generally the A value is lower for clays with a more dispersed structure. Thus a fresh water deposit might have a lower A value than that of marine clays: it may be less by an amount depending upon how dispersed a structure it has. Added to this, disturbance of the sample during cutting, placing, etc. would probably tend to make the structure more dispersed, reducing further the value of A. Therefore, this was a probable reason for the low A value observed at the beginning of the tests.

Regarding the general trend of the pore pressure curve, it is quite clear that pore pressure increased with time. When a deviator stress is applied to a normally loaded clay, the clay sample tends to reduce in volume. But in the case of a saturated sample with no drainage permitted, no volume change

takes place. Thus the whole process of shearing takes place at a constant void ratio.

It was found by Rutledge¹¹ that the void ratio plotted against, first the minor principal stress during consolidation, and second, the minor principal stress at failure, will yield two parallel straight lines on a semi-logarithmic plot.

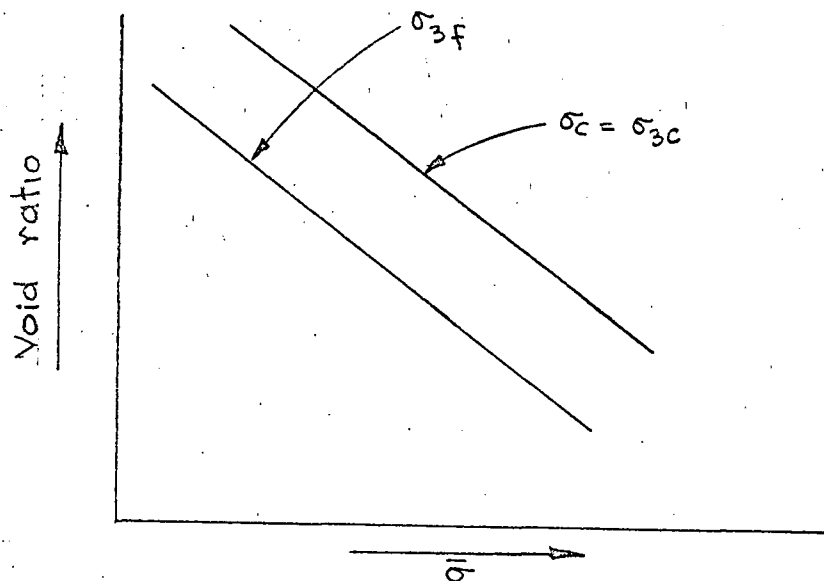


Figure 12(b)

$\bar{\sigma}$ = Normal Effective Stress (Logarithmic Scale)

It has been suggested¹¹ that the difference between these two lines is due to a difference in structure. In consolidation stage, the soil has a random structure, while at failure, shearing strains have produced a dispersed structure. As structure goes from a less to a more dispersed one, average interparticle distance increases, causing a reduction in the "net interparticle repulsion". In a saturated, dispersed clay the "net interparticle repulsion" is equal to the effective stress.¹⁰ Thus a dispersed structure at a given effective

stress corresponds to a smaller void ratio.

Now, if the structural change takes place at a constant void ratio, it is clear that it must be accompanied by a reduction in effective stress; and, at constant total stress, this means an increase in pore pressure. It is postulated here, then, that the application of the deviator stress is accompanied by immediate strains and immediate change of structure, followed by a long term strain and a long term change of structure.

If these were true, the curves for the tests described herein might be superimposed on those of figure 12b as shown in figure 12c.

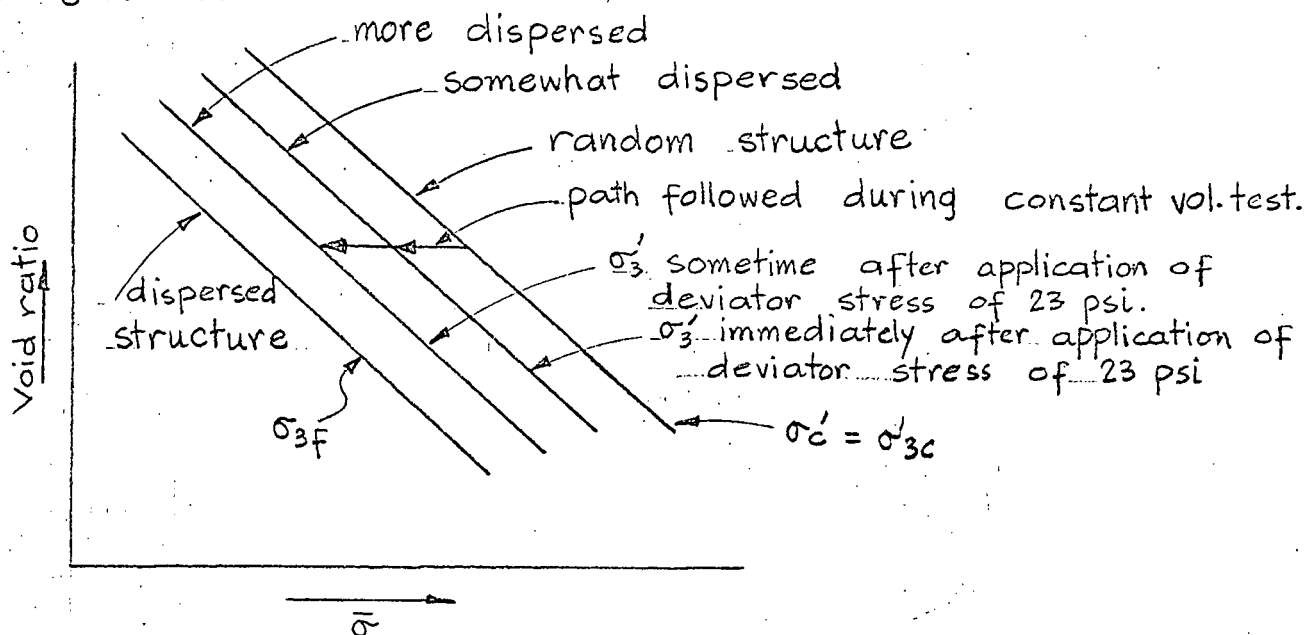


Figure 12(c)

$\bar{\sigma}$ = Normal Effective Stress (Log. Scale)

This hypothesis would explain the increases of pore pressure with time under constant volume, temperature, and deviator stress that were observed in the tests, in a manner consistent with the foregoing theory.

D. Comparison of the Pore Pressure Curves at Different Temperatures

Since temperature was kept constant throughout one single test the effect of a change of temperature on pore pressure cannot be observed unless pore pressure curves at different temperatures are compared. Since all other factors were kept the same, the difference in the value of pore pressure after any interval of time may be attributed to the difference in temperature.

It is clearly seen from figure 16 that at a particular time pore pressures are greater for tests at higher temperature, than those at lower temperature. Possible explanations with some discussion are offered below.

Professor Lambe of M.I.T. has put forward the following equations in his paper¹⁰ on the mechanistic picture of shear strength of clay: He stated that, in the case of a highly plastic, saturated, dispersed clay having no air-mineral contact or mineral-mineral contact, total stress

$$\sigma = U + (R - A) \dots\dots(a)$$

where,

σ = total stress

U = pore water pressure

R = electrical repulsion between particles

A = electrical attraction between particles

Although the assumption of no air-mineral or mineral-mineral contact may be controversial, it is true that in a saturated soil there is no free air present. Then if mineral-

mineral contact is assumed, equation (a) becomes, according to Lambe,

$$\sigma = U + (\bar{\sigma}_{am} + R - A) \dots (b)$$

where, $\bar{\sigma}_{am}$ is the intergranular stress.

Before equations (a) and (b) are discussed to interpret the change in pore pressure, an explanation regarding the structure of soil, for tests at different temperatures, is necessary.

Research workers in the past have shown experimentally and to some extent quantitatively that an increase in temperature tends to cause flocculation in a colloidal medium.¹⁰

Since all other factors except temperature were kept constant in the tests presented herein, it is expected that after a particular time interval, the samples at higher temperature will tend to have a relatively more flocculated structure than those at lower temperatures. But in actual case, the structure of the samples at higher temperature, after any time interval, remains equally dispersed as those at lower temperature. In other words, samples at higher temperatures, in spite of their tendency towards a relatively more flocculated structure, have the same amount of dispersed structure as those at lower temperatures. Thus, compared to the samples at lower temperature, those at higher temperatures correspond to a relatively more dispersed structure.

If a soil structure becomes more dispersed, its average interparticle spacing is increased eventually reducing A as well as R. But A is relatively insensitive to environment

(Ref. 10, page 565) and so more dispersion would mean reduction in (R-A).

Considering both the equations (a) and (b) it is clearly seen that a decrease in (R - A) with constant σ , means an increase in U if static equilibrium is to be maintained.

Thus samples at higher temperatures, corresponding to a relatively more dispersed structure, also correspond to a relatively lower value of (R - A) and hence to a relatively higher value of pore pressure than those at lower temperatures.

Thus the pore pressure curves, showing higher values of pore pressures with rise of temperature at any interval of time, since the application of the deviator stress, are quite consistent with the theory.

E. Experimental Verification of the Fact that the Slope of the Strain Versus Logarithm of Time Curve Bears a Linear Relationship with Absolute Temperature

As has already been mentioned in Chapter II of this thesis, Murayama and Shibata derived theoretically and verified experimentally the fact that the slope of the strain versus logarithm of time bears a linear relationship to absolute temperature. It was a part of this thesis to determine experimentally the relationship between $\frac{d\varepsilon}{d \log t}$ and absolute temperature T and hence verify the equation $\frac{d\varepsilon}{d \log t} = C.T.$ where C is a constant.

The above point will be discussed with respect to the first and second sets of experiments respectively.

Considering figure 18 showing the first set, it can be seen that the above straight line relationship is well maintained. But this curve (fig. 18) is based on results up to about the first twenty minutes. After this, the strain-log time curves (fig. 17) do not have constant slopes and $\frac{d\epsilon}{d \log t}$ versus T curves no longer remain straight lines. Since the $(\epsilon - \log t)$ curves, after about twenty minutes, change slope constantly, it is inconvenient to draw $(\frac{d\epsilon}{d \log t} - T)$ curves beyond this point. Instead $\frac{d\epsilon}{d \log t}$ versus time curves are drawn to illustrate the relationship between $\frac{d\epsilon}{d \log t}$ and T at different times.

Consider the graphs in figure 19. From these curves it is clear that in the initial period of about 20 minutes there is a constant rate of strain for each temperature.

After this initial period there is a complicated transition stage which lasts up to about 160 minutes. This stage is quite indeterminate. But after about 160 minutes, this transition stage gives way to a fairly steady state where again the curves (excepting that at 100°F.) approximate horizontal straight lines. However, the relationship between these constant rates of strain and their corresponding temperatures is not the same as was the case during the first twenty minutes; there does appear to be a correlation, although there is not a good linear relationship. After 300 minutes, the rates of strain again diverge, and, prior to failure, become very erratic.

The above discussion can possibly be simplified in the following way to obtain a less complicated relationship. Each $(\epsilon - \log t)$ curve in figure 17 can be approximated reasonably to two straight lines, such that each curve consists of only two slopes. One curve of $\frac{d\epsilon}{d \log t}$ versus T is drawn for the initial slopes (shown in dotted line fig. 20) and another curve of $\frac{d\epsilon}{d \log t}$ versus T (shown in full line fig. 20) is drawn for the slopes after the transition stage. These two curves are put on the same diagram (Fig. 20). Although from figure 20 it is quite clear that the linear relationship is maintained during the first period (dotted curve), it shows that after the transition stage the rate of strain $\frac{d\epsilon}{d \log t}$ still increased with temperature. From these tests, however, the relationship between rate of strain and temperature was no longer linear.

In the second set of experiments also an attempt was made to determine the relationship between $\frac{d\epsilon}{d \log t}$ and absolute temperature T . Considering figure 21, it can be clearly seen that the strain and logarithm has a constant slope at all the temperatures considered. In the first set this linear relationship is maintained only up to about the first 20 minutes, after that the curves become non-linear. This may be due to the fact that the principal stress ratio is much higher in the first set than that in the second.

According to Murayama if the applied stress is larger than a certain limit (upper yield point) consistent with the particular clay and testing procedures in question, then

$\epsilon - \log t$ curve becomes non-linear after a certain time. Probably that is the reason why $\epsilon - \log t$ in the first set is non-linear after the initial period.

It is expected that results of consolidated undrained tests to be in better accord with Murayama's theory than those of unconsolidated undrained tests since the C-U tests were carried out at a deviator stress more likely to be less than upper yield point.

In the second set $\epsilon - \log t$ curves (Fig. 21) are straight lines throughout conforming to Murayama's theory.

Returning to the question of rate of strain as a function of temperature, however, although there was a constant rate of strain throughout each test of the second set, figure 22 shows that the rates were not linearly related to temperature. An initial straight line followed by an upward curve was obtained as in the previous case - see figure 20.

It might be interesting in this connection to discuss the validity of the assumptions made by Murayama in arriving at the conclusion that strain rate bears a linear relationship to absolute temperature. The relationship of strain rate ($\frac{d\epsilon}{d \log t}$) to absolute temperature T , as obtained by Murayama, is given by the following equation:

$$\frac{d\epsilon}{d \log t} \approx \frac{2bR\sigma}{\lambda E_2} T \quad (\text{refer to page 23 of this thesis})$$

From the equation it is clear that $\frac{d\epsilon}{d \log t}$ bears a linear relationship with T provided E_2 and b are independent of T .

Although E_2 may be independent of T , independence of b to T requires investigation.

Recalling equation (2) page 20 of this thesis, it may be seen that,

$$N = b(\sigma - \sigma_0) \quad (9)$$

where N is the number of molecules with activation in a unit area of cross section perpendicular to the direction of stress.

Since it has been observed that even with the same external stress, strains as well as strain rates change with change in temperature, it may be said that N is not independent of T .

If N is not independent of T , considering equation (9) it may be seen that b also is not independent of T since $(\sigma - \sigma_0)$ may be assumed as constant if external stress σ remains unchanged.

Thus the linear relationship of strain rate to absolute temperature as claimed by Murayama is open to question.

Although it is difficult to define exactly the nature of the curves in figures 20 (full line) and 22, the following observation may be made. From both figure 20 and 22, it appears that up to 100°F, all the points are on a single straight line, but that at 110°F. lies above the line. The increase of $\frac{d\epsilon}{d \log t}$ with temperature is thus uniform up to 100°F. and more rapid thereafter.

Physically it may mean that the soil in question is not as sensitive to temperature changes up to 100°F. as it is above 100°F. It is realised that this 100°F. temperature is

only critical for the soil under investigation (if at all) and is not necessarily an absolute figure for soils in general. The important point is that there may be some critical temperature consistent with the nature of a soil, the environmental conditions, and the testing procedures, above which a soil is more sensitive to temperature with regard to its deformation characteristics. Regarding pore pressure changes also it is clearly seen from figure 16 that the increase of pore pressure with temperature after 100°F. is more rapid than that up to 100°F.

The following hypothesis may be put forward, to explain the observed results, in the light of creep theory.

It has been discussed before that if energy of the soil mass is increased, more molecules are activated; number of such molecules activated maybe proportional to the amount of external energy supplied to the system. This is possibly true until the total energy of the system reaches a certain critical limit consistent with a soil and its environment. If, however, this limit of energy is exceeded, activation of molecules would take place at a faster rate and possibly significant bond breakage will begin.

In the tests presented, maybe the energy corresponding to the temperature of 100°F. is critical for the soil under investigation.

Thus below 100°F, the rate of activation of the molecules and hence the strain rate is proportional to the tem-

perature but above 100°F. strain rate is higher as shown by figure 20 and 22.

Further research is necessary to arrive at a more definite conclusion; nevertheless, figures 20 and 22 bring out this important point: there may exist a temperature corresponding to a particular clay under a particular environment above which the soil is more sensitive in regard to its deformation characteristics.

CHAPTER V

SUMMARY OF CONCLUSIONS

Based on the experimental data and discussions presented in this thesis, following conclusions can be made for the soil under investigation:

1. Strain of clay increases with temperature under a constant total stress.
2. If the principal stress ratio is low, strain bears a straight line relationship with logarithm of time: If the principal stress ratio is quite high this straight line relationship is not maintained for a long time.
3. The relationship between $\frac{d\varepsilon}{d \log t}$ and absolute temperature T is not linear for all temperatures.
4. There may be a temperature corresponding to a soil, its environment and testing procedures, above which that soil is more sensitive to temperature with regard to its deformation characteristics. This point requires verification.
5. Under a constant total stress, pore pressure increases with the rise of temperature.

CHAPTER VI

RECOMMENDATIONS

Further investigations are needed on every phase of the work reported in this thesis. Time and testing were limited, resulting in data that need verification by investigators in future.

The following are the recommendations regarding changes in the equipment and testing procedure, which may improve the quality of results to a great extent:

1. Measurement of sample temperature instead of measuring the temperature of water in the triaxial cell.
2. Use of filter paper side drains.
3. Use of temperature-controlled bath from which water at required temperature can be directly pumped into the triaxial cell instead of passing the water through the balancing reservoir where considerable heat is lost.
4. Use of ceramic and plates instead of porous stones.

BIBLIOGRAPHY

1. Haefeli, R., (1953), "Creep problems in soils, snow and ice". 3rd International Conference on Soil Mechanics and Foundation Engineering. Proceedings, Vol. III.
2. Vyalov, S.S., Skibitsky, A.M., (1957), "Rheological processes in frozen soils and dense clays", 4th International Conference on Soil Mechanics and Foundation Engineering, Proceedings, Vol. I.
3. Vyalov, S.S., Skibitsky, A.M., "Problems of the Rheology of Soils". 5th International Conference on Soil Mechanics and Foundation Engineering. Proceedings.
4. Murayama, S., Shibata, T., "Rheological properties of clays". 5th International Conference on Soil Mechanics and Foundation Engineering. Proceedings.
5. Eyring, Glasstone, Laidler., Theory of Rate Processes, N.Y. McGraw Hill.
6. Saito, M., Vezawa, H., "Failure of Soil due to Creeps", Soil Mechanics Laboratory, Railway Technical Research Institute, Japanese National Railways.
7. Lambe, T.W., (1962), "Soil Testing for Engineers", N.Y., John Wiley.
8. Mittal, H.K., (1963), "Effect of Plain Strain on Pore pressure parameters", M.A.Sc Thesis, U.B.C.
9. Taylor, D. W., (1948), "Fundamentals of Soil Mechanics", N.Y., John Wiley, p. 371.
10. Lambe, T.W., (1960), "A mechanistic picture of shear strength in clay", Research Conference on shear strength of cohesive soils. pp. 555-580.
11. Scott, R.F., (1963), "Principles of Soil Mechanics", Massachusetts, Addison Wesley Publishing Co. Inc.
12. Bishop, A.W., and Henkel, D.G., (1957), "The Measurement of soil properties in the triaxial Test". London, Arnold.

APPENDIX

EXPERIMENTAL RESULTS

Physical Properties of Soil:

Specific Gravity	2.79
Grain Size finer than 2 microns	50%
Liquid Limit	50%
Plastic Limit	25%
Plastic Index	25%
Natural Water Content ...	47% \pm 2%
Degree of Saturation	100%
Maximum Past Pressure ...	35 psi.

Dimensions of test specimen:

Diameter	1.31 in.
Area of Cross Section ...	1.35 sq. in.
Height	2.875 in.
Volume	3.88 cu. in.

SET # 1

Unconsolidated Undrained Tests.

Water Content = 45.1% Chamber Pressure = 30 psi

Temperature = 70°F Initial dial reading = 315

Deviator Stress = 10.5 psi

TABLE 1

Time in mins	Dial reading in 10^3 in	Vertical deformation in 10^3 in	Area A in in^2	Time in minutes	Dial reading in 10^3 in	Vertical deformation in 10^3 in	Area A in in^2
1	350.0	35.0	1.350	60	377.4	62.4	1.380
2	356.5	41.5		70	383.0	68.0	
3	359.0	44.0		80	387.0	72.0	
4	360.0	45.0		100	393.2	78.2	1.388
5	361.1	46.1		150	414.8	99.8	
10	365.0	50.0		200	428.9	113.9	
15	367.1	52.1		300	440.4	125.4	1.411
20	368.7	53.7		400	451.0	136.0	
25	370.1	55.1		500	459.0	144.0	
30	371.2	56.2	1.377	600	463.6	148.6	1.423
35	372.6	57.6		700	469.0	154.0	
40	373.2	58.2		800	475.0	160.0	
45	374.0	59.0		900	481.2	166.2	
50	374.5	59.5		1000	491.0	176.0	1.438
55	375.2	60.2		1300	510.2	195.2	
					SAMPLE FAILED		

Water Content = 45.2%

Chamber Pressure = 30 psi

Temperature = 80°F

Initial dial reading = 300.00

Deviator stress = 10.5 psi

TABLE II

Time in mins	Dial reading in 10^3 in	Vertical def. in 10^3 in	Area A in in ²	Time in minutes	Dial reading in 10^3 in	Vertical def. in 10^3 in	Area A in in ²
1	338.0	38.0	1.350	65	393.0	93.0	
2	344.8	44.8		70	395.8	95.8	
3	351.0	51.0		80	402.0	102.0	
4	353.5	53.5		90	406.0	106.0	
5	355.5	55.5		100	410.5	110.5	1.404
10	360.0	60.0		150	429.0	129.0	
20	366.0	66.0		200	440.2	140.2	
25	369.4	69.4		300	455.8	155.8	1.427
30	372.0	72.0	1.385	400	463.5	163.5	
35	374.2	74.2		500	470.0	170.0	
40	376.5	76.5		600	477.5	177.5	1.439
45	380.0	80.0		700	487.6	187.5	
50	383.0	83.0		800	492.3	192.3	
55	386.2	86.2		900	496.4	196.4	
60	390.0	90.0	1.394	1000	500.4	200.4	1.451
					SAMPLE FAILED.		

Water Content = 44.8% Chamber Pressure = 30psi
 Temperature = 90°F Initial dial reading = 300.00
 Deviator stress = 10.5 psi

TABLE III

Time in mins	Dial reading in 10^3 in	Vertical def. in 10^3 in	Area A in in ²	Time in minutes	Dial reading in 10^3 in	Vertical def. in 10^3 in	Area A in in ²
1	343.8	43.8	1.35	60	402.0	102.0	1.40
2	355.5	55.5		65	405.0	105.0	
3	360.5	60.5		70	408.0	108.0	
4	364.0	64.0		80	412.0	112.0	
5	366.0	66.0		100	421.0	121.0	1.41
10	372.5	72.5		200	450.0	150.0	
15	377.0	77.0		300	462.0	162.0	1.43
20	380.0	80.0		400	470.0	170.0	
25	382.5	82.5		500	476.2	176.2	
30	385.0	85.0	1.39	600	480.0	180.0	1.44
35	388.5	88.5		700	483.4	183.4	
40	392.0	92.0		800	488.6	188.6	
45	396.0	96.0		900	491.1	191.1	1.45
50	398.0	98.0		960	494.0	194.0	
55	400.0	100.0			SAMPLE FAILED		

Water Content = 45.3% Chamber Pressure = 30 psi
 Temperature = 100°F Initial dial reading = 300.00
 Deviator Stress = 10.5 psi

TABLE IV

Time in mins	Dial reading in 10^{-3} in	Vertical def. in 10^{-3} in	Area A in in ²	Time in minutes	Dial reading in 10^{-3} in	Vertical def. in 10^{-3} in	Area A in in ²
1	346.0	46.0	1.35	55	420.0	120.0	
2	360.0	60.0		60	423.5	123.5	1.41
3	367.2	67.2		65	426.5	126.5	
4	371.5	71.5		70	429.8	129.8	
5	373.0	73.0		90	438.0	138.0	
10	382.1	82.1		100	441.5	141.5	1.42
15	387.8	87.8		150	455.5	155.5	
20	394.0	94.0		200	465.0	165.0	
25	398.0	98.0		300	478.0	178.0	1.44
30	402.5	102.5	1.40	400	486.8	186.8	
35	407.0	107.0		500	494.4	194.4	
40	410.8	110.8		600	506.8	206.8	1.45
45	415.0	115.0		680	516.0	216.0	
50	417.5	117.5			SAMPLE FAILED		

Water content = 45.1% Chamber Pressure = 30psi
 Temperature = 110°F Initial dial reading = 300.00
 Deviator stress = 10.5 psi

TABLE V

Time in min	Dial reading in 10 ³ in	Vertical def. in 10 ³ in	Area A in in ²	Time in minutes	Dial reading in 10 ³ in	Vertical def. in 10 ³ in	Area A in in ²
1	345.0	45.0	1.35	70	448.6	148.6	
2	361.0	61.0		80	454.0	154.0	
3	368.2	68.2		90	457.8	157.8	
4	372.8	72.8		100	462.0	162.0	1.43
5	375.0	75.0		150	475.4	175.4	
10	385.5	85.5		200	483.0	183.0	
15	394.0	94.0		300	494.0	194.0	1.436
20	401.2	101.2		400	499.4	199.4	
25	406.4	106.4		500	504.0	204.0	
30	413.0	113.0	1.41	600	508.6	208.6	1.443
35	417.8	117.8		700	512.5	212.5	
40	424.0	124.0		800	516.7	216.7	
45	429.7	129.7		900	518.8	218.8	1.452
50	434.0	134.0		1010	530.5		
60	441.5	141.5	1.42		SAMPLE FAILED		

SET No 2

Test 1

CONSOLIDATED UNDRAINED TESTS

Water content = 49.3%

Initial dial reading after

Temperature 70°F

Consolidation

- 571

Deviator Stress ($\sigma_1 - \sigma_3$) - 23 psi

Initial height of sample

Chamber Pressure - 50 psi

after consolidation

- 2.79"

TABLE VI

ELAPSED TIME IN MINS.	DIAL READING IN 10 ⁻² INS.	VERTICAL DEFORMATION IN 10 ⁻² INS.	STRAIN E IN %	PORE PRESSURE IN P.S.I.	AREA A IN INS. ²
1	593.5	22.5	0.806	2.2	1.360
2	597.0	26.0	0.924	2.4	
3	599.0	28.0	1.000	2.8	
4	601.0	30.0	1.070	2.8	
5	601.8	30.8	1.100	2.9	
10	606.0	35.0	1.250	3.0	
15	607.6	36.6	1.31	3.2	
20	609.0	38.0	1.360	3.5	
25	610.0	39.0	1.40	3.5	
30	611.0	40.0	1.430	3.7	1.367
40	613.0	42.0	1.500	3.8	
50	614.0	43.0	1.540	3.9	
60	616.0	45.0	1.610	4.0	
70	617.4	46.4	1.66	4.5	
80	618.0	47.0	1.680	5.0	
90	618.0	47.0	1.680	5.3	

TABLE VI CONTD

ELAPSED TIME IN MINS.	DIAL READING IN 10 ⁻³ INS.	VERTICAL DEFORMATION IN 10 ⁻³ INS.	STRAIN ϵ IN %	PORE PRESSURE IN P.S.I.	AREA A IN INS. ²
100	618.5	47.5	1.70	5.6	1.371
120	618.5	47.5	1.70	5.9	
140	618.7	47.7	1.71	6.2	
150	619.0	48.0	1.72	6.4	
180	620.5	49.5	1.77	6.8	
250	624.0	53.0	1.90	7.4	1.377
500	627.0	56.0	2.00	9.2	
750	629.0	58.0	2.08	10.1	
1000	631.0	60.0	2.15	11.2	
1200	632.4	61.4	2.20	11.6	
1500	634.5	63.5	2.28	12.2	
2000	636.0	65.0	2.32	14.0	

Test 2

Water content	47.5%	Initial dial reading after	
Temperature	90°F	Consolidation	495
Deviator Stress ($\sigma_1 - \sigma_3$)	23 psi	Initial height of sample	
Chamber Pressure	50 psi	after consolidation	2.80"

TABLE VII

ELAPSED TIME IN MINS.	DIAL READING IN 10 ⁻³ INS.	VERTICAL DEFORMATION IN 10 ⁻³ INS.	STRAIN ϵ IN %	PORE PRESSURE IN P.S.I.	AREA A IN INS. ²
1	519.0	24.0	0.86	2.3	
2	523.0	28.0	1.00	2.3	
3	526.0	31.0	1.10	2.3	
4	527.5	32.5	1.16	2.4	
5	529.0	34.0	1.21	2.5	
6	530.0	35.0	1.25	2.5	
7	531.0	36.0	1.28	2.7	
8	532.0	37.0	1.32	2.8	
9	533.0	38.0	1.35	3.0	
10	533.0	38.0	1.35	3.2	
15	536.0	41.0	1.46	3.4	
20	537.8	42.8	1.53	3.5	
30	540.0	45.0	1.60	3.8	
40	542.0	47.0	1.68	4.1	
50	543.0	48.0	1.71	4.4	
60	544.5	49.5	1.77	4.7	

TABLE VII CONTD.

ELAPSED TIME IN MINS.	DIAL READING IN 10 ⁻³ INS.	VERTICAL DEFORMATION IN 10 ⁻³ INS.	STRAIN ϵ IN %	PORE PRESSURE IN P.S.I.	AREA A IN INS. ²
80	546.0	51.0	1.82	5.2	
90	547.0	52.0	1.85	5.5	
100	547.8	52.8	1.88	5.7	
150	550.0	55.0	1.96	6.5	
200	552.0	57.0	2.04	7.5	
250	553.4	58.4	2.08	8.0	
300	554.0	59.0	2.10	8.5	
400	556.0	61.0	2.18	9.5	
500	557.5	62.5	2.24	10.0	
600	558.5	63.5	2.27	10.5	
1000	562.0	67.0	2.40	11.7	
1400	5650	70.0	2.50	12.7	
1800	5678	72.8	2.60	13.3	
2000	569.3	74.3	2.65	13.6	

Test 3

Water content 47%

Temperature 100°F

Deviator stress ($\sigma - \sigma_3$) 23 psi

Chamber Pressure 50 psi

Initial dial reading after

Consolidation

520

Initial height of sample

after consolidation

2.80"

TABLE VIII

ELAPSED TIME IN MINS	DIAL READING IN 10 ⁻³ INS	VERTICAL DEFORMATION IN 10 ⁻³ INS	STRAIN ϵ IN %	PORE PRESSURE IN P.S.I.	AREA A IN INS ²
1	545.0	25.0	0.89	2.0	1.360
2	549.0	29.0	1.04	2.4	
3	552.0	32.0	1.14	2.8	
4	553.0	33.0	1.18	3.0	
5	555.0	35.0	1.25	3.2	
10	560.0	40.0	1.43	3.4	
15	562.0	42.0	1.50	3.5	
20	564.1	44.1	1.58	3.7	
25	564.8	44.8	1.60	3.8	
30	567.5	47.5	1.70	3.9	1.373
40	568.2	48.2	1.72	4.0	
50	569.0	49.0	1.75	4.5	
60	571.0	51.0	1.82	5.0	
70	571.0	51.0	1.82	5.2	
80	571.8	51.8	1.83	5.5	
90	572.8	52.8	1.86	5.7	

TABLE VIII CONTD.

ELAPSED TIME IN MINS.	DIAL READING IN 10^{-3} INS.	VERTICAL DEFORMATION IN 10^{-3} INS.	STRAIN ϵ IN %.	PORE PRESSURE IN P.S.I.	AREA A IN INS.^2
100	574.4	54.4	1.32	6.0	1.377
120	574.6	54.6	1.95	6.3	
140	576.5	56.5	1.98	7.5	
160	576.6	56.6	2.02	7.7	
250	580.0	60.0	2.14	9.4	1.380
300	582.0	62.0	2.21	9.6	
450	583.8	63.8	2.22	10.5	
750	587.8	67.8	2.35	11.5	
1000	590.2	70.2	2.49	12.03	1.384
1335	590.6	70.6	2.5	13.0	
1500	593.3	73.3	2.59	13.3	
1650	593.4	73.4	2.60	13.4	
1700	594.0	74.0	2.64	13.5	
2000	596.0	76.0	2.71	13.6	

Test 4

Water content 48.8%

Temperature 110°F

Deviator stress ($\sigma_1 - \sigma_3$) 23 psi

Chamber Pressure 50 psi

Initial dial reading after

consolidation 426

Initial height of sample

after consolidation 2.80"

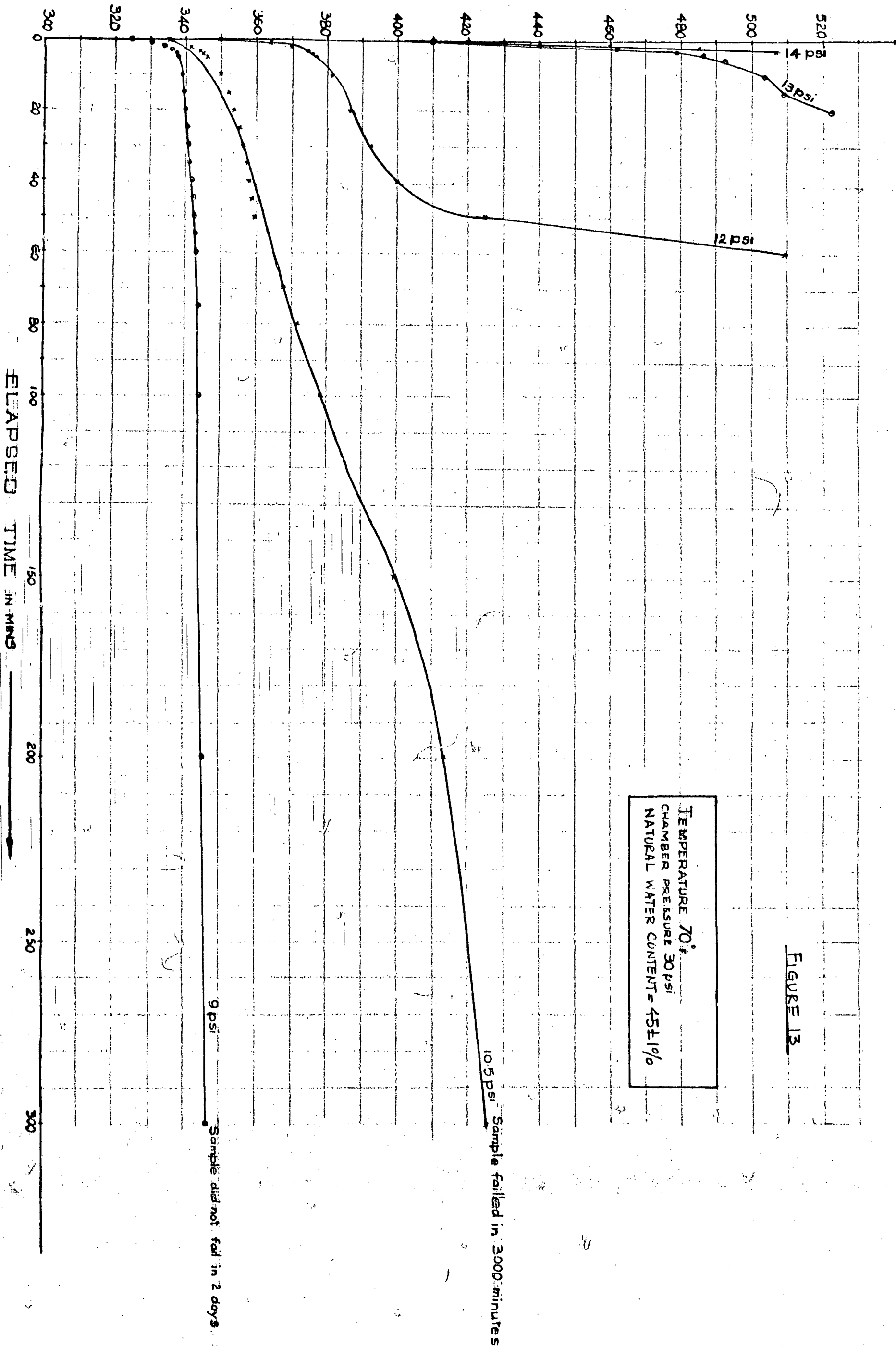
TABLE IX

ELAPSED TIME IN MINS	DIAL READING IN 10 ⁻³ INS	VERTICAL DEFORMATION IN 10 ⁻³ INS	STRAIN E IN %	PORE PRESSURE IN P.S.I	AREA A IN IN. ²
1	447.0	27.0	0.964	2.0	1.360
2	452.0	32.0	1.140	2.3	
3	456.0	36.0	1.290	2.5	
4	458	38	1.36	2.8	
5	460.0	40.0	1.430	2.9	
6	460.0	40.0	1.43	2.9	
7	462.0	42.0	1.500	3.0	
10	467.0	47.0	1.680	3.0	
15				3.5	
20	471.0	51.0	1.820	4.0	
30	475.0	55.0	1.960	4.3	1.377
40	476.0	56.0	2.00	4.5	
50	479.0	59.0	2.110	4.8	
60	482.0	62.0	2.210	5.0	
80	483.0	63.0	2.250	5.7	
100	485.0	65.0	2.320	6.5	1.380

TABLE IX CONTD.

ELAPSED TIME IN MINUTES	DIAL READING IN 10^3 IN	VERTICAL DEFORMATION IN 10^3 IN	STRAIN %	PORE PRESSURE IN P.S.I	AREA A IN IN^2
120	486.5	66.5	2.33	7.5	1.39
200	492.5	72.5	2.54	9.8	
350	495.5	75.5	2.65	11.0	
500	498.5	78.5	2.75	11.8	
750	502.0	82.0	2.93	12.5	
1000	505.0	85.0	3.04	14.0	
1500			3.16	18.0	

DIAL GAUGE READING
IN 10^{-3} INCHES



TEMPERATURE 70°
CHAMBER PRESSURE 30 psi
NATURAL WATER CONTENT = 45.1%

FIGURE 13

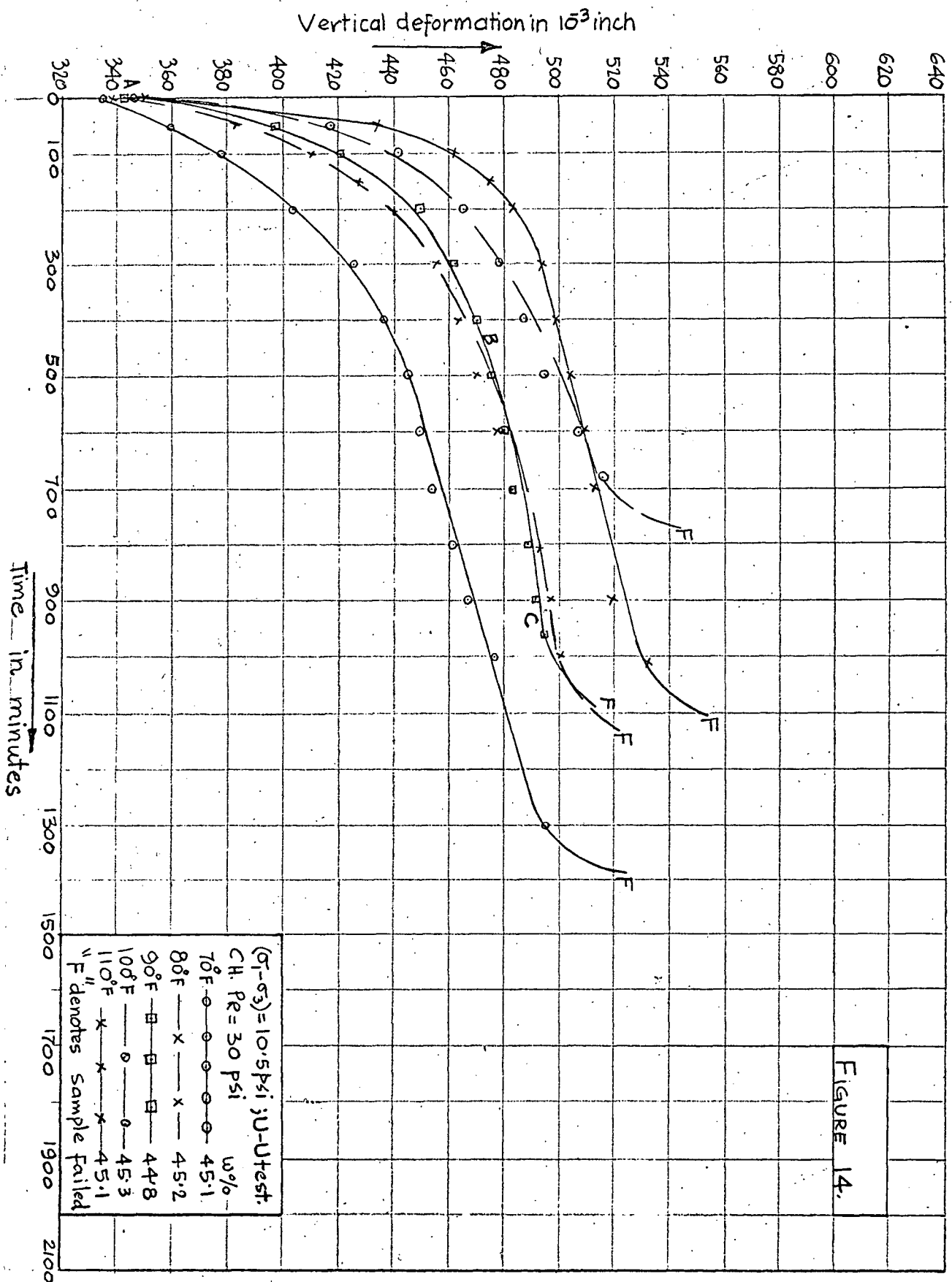


FIGURE 14.

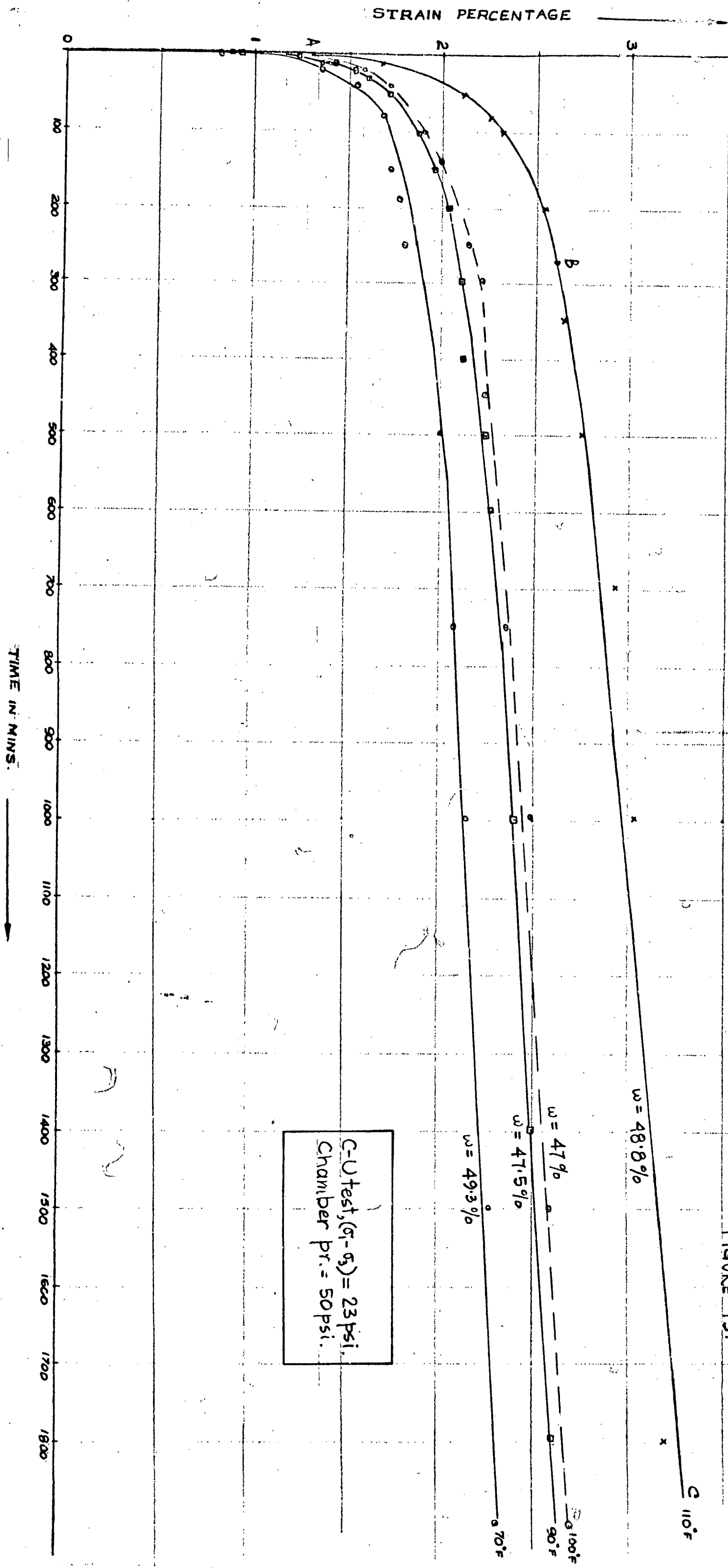


FIGURE 15.

FIGURE 16

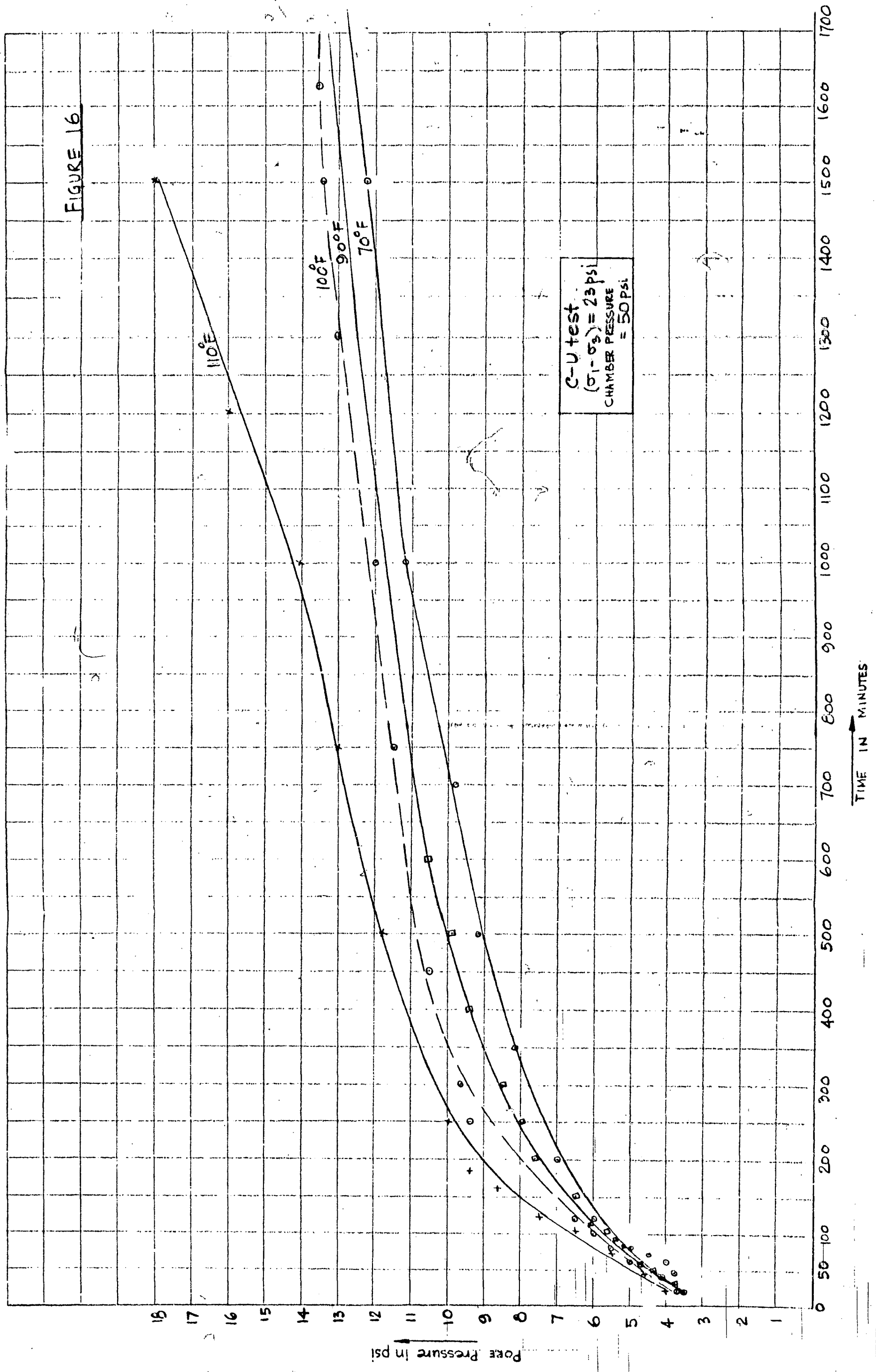


FIGURE 17

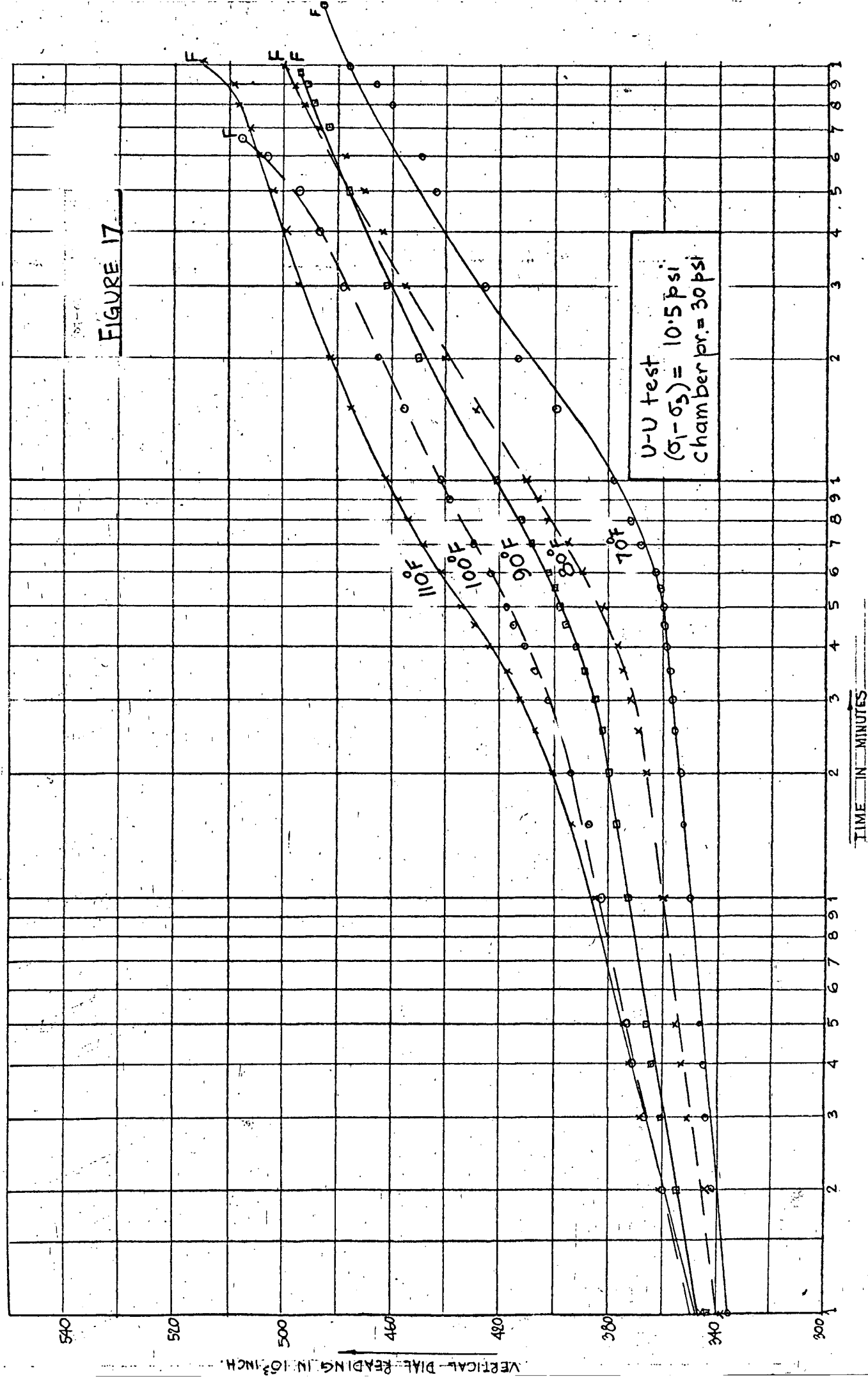


FIGURE 18.

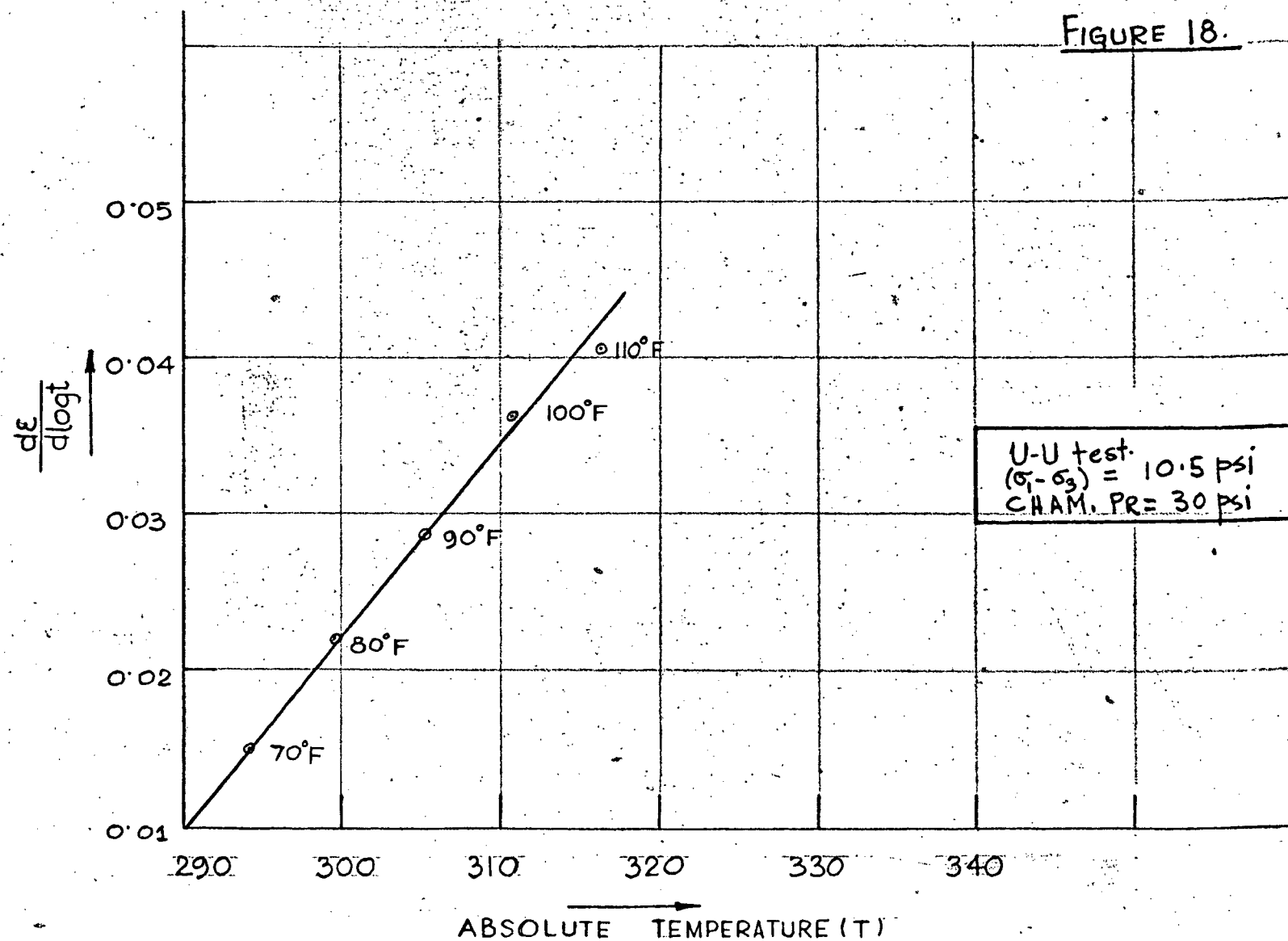
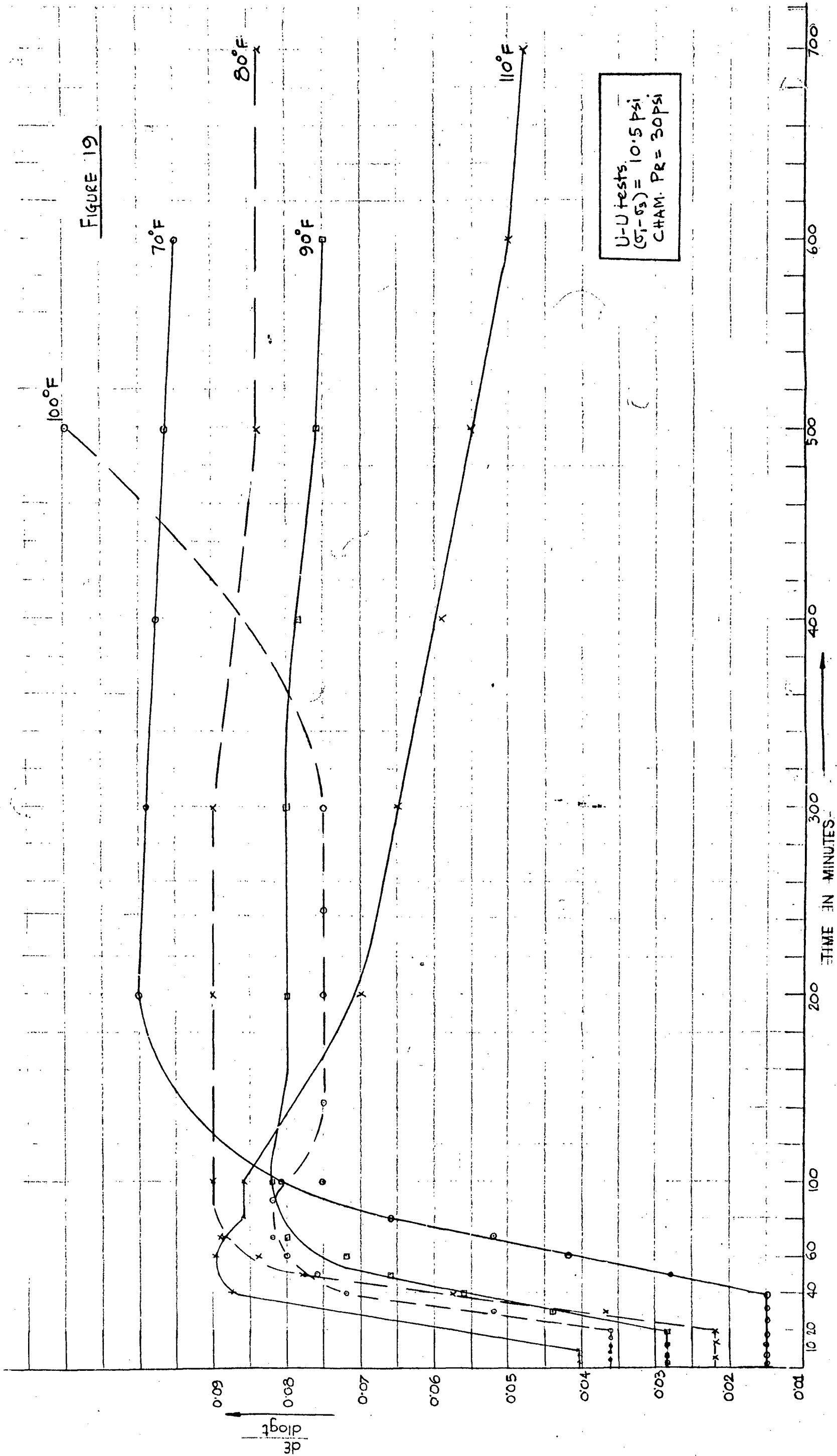
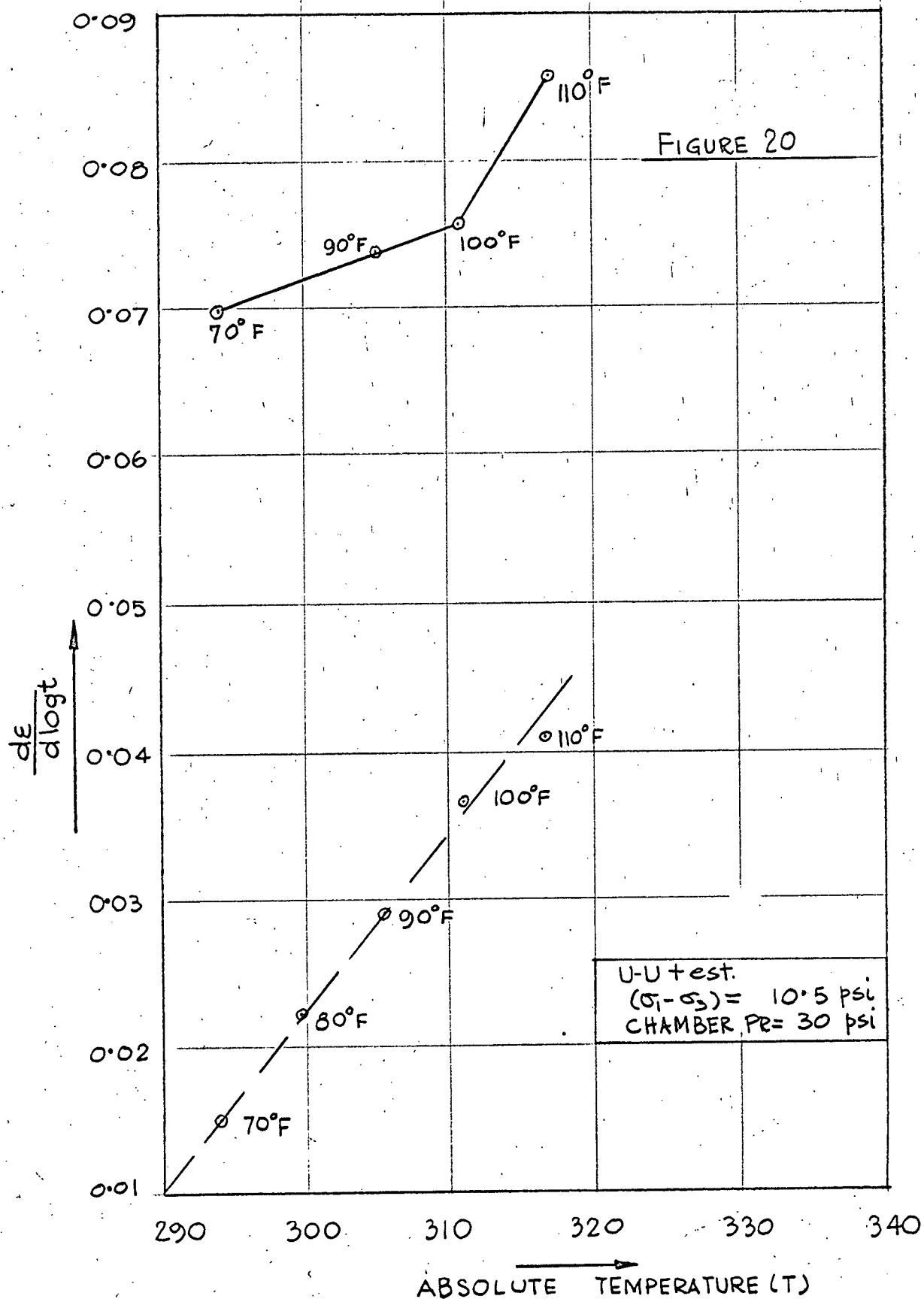


FIGURE 19





C-U TEST.

CHAMBER PRESSURE 50 psi

DEVIATOR STRESS 23 psi

FIGURE 21

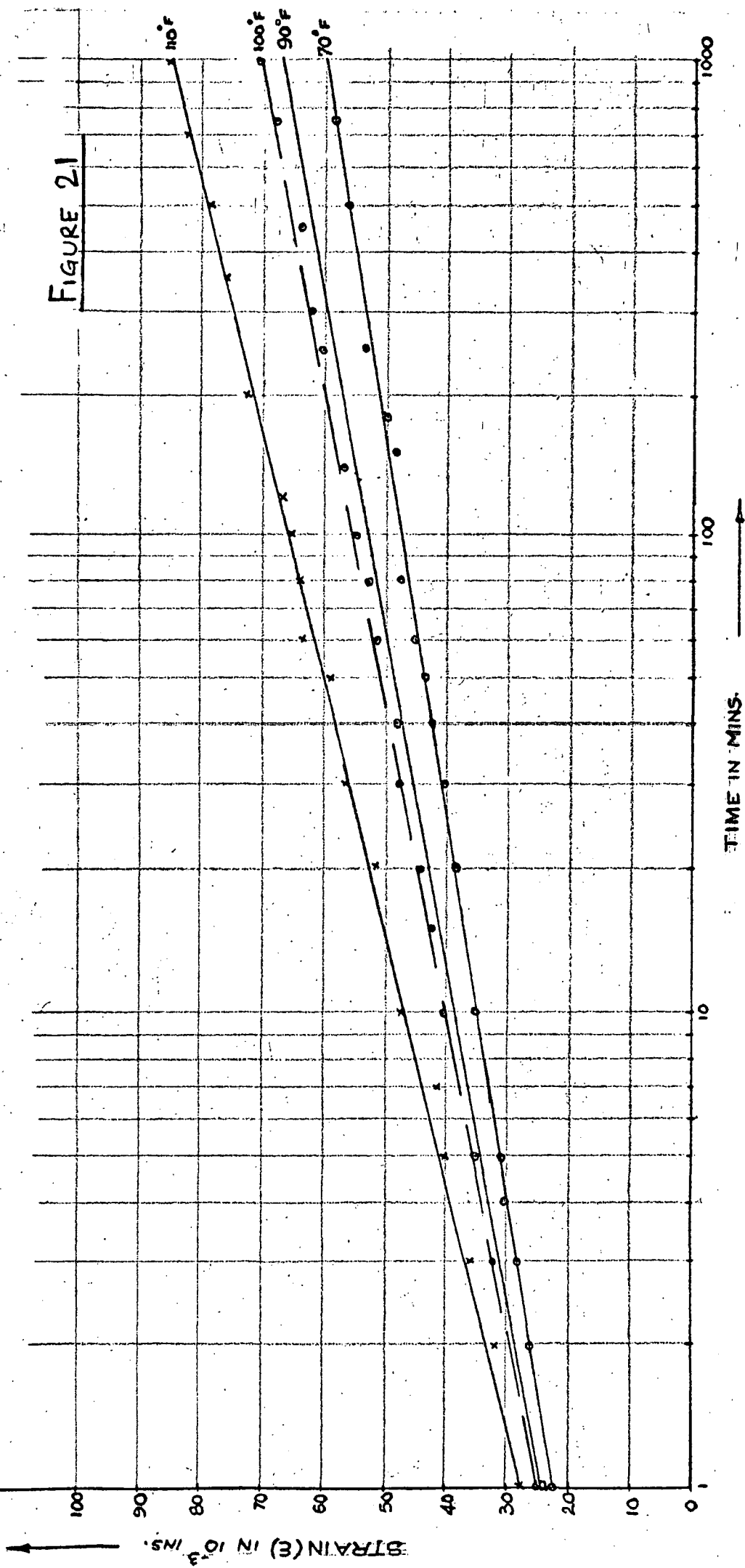
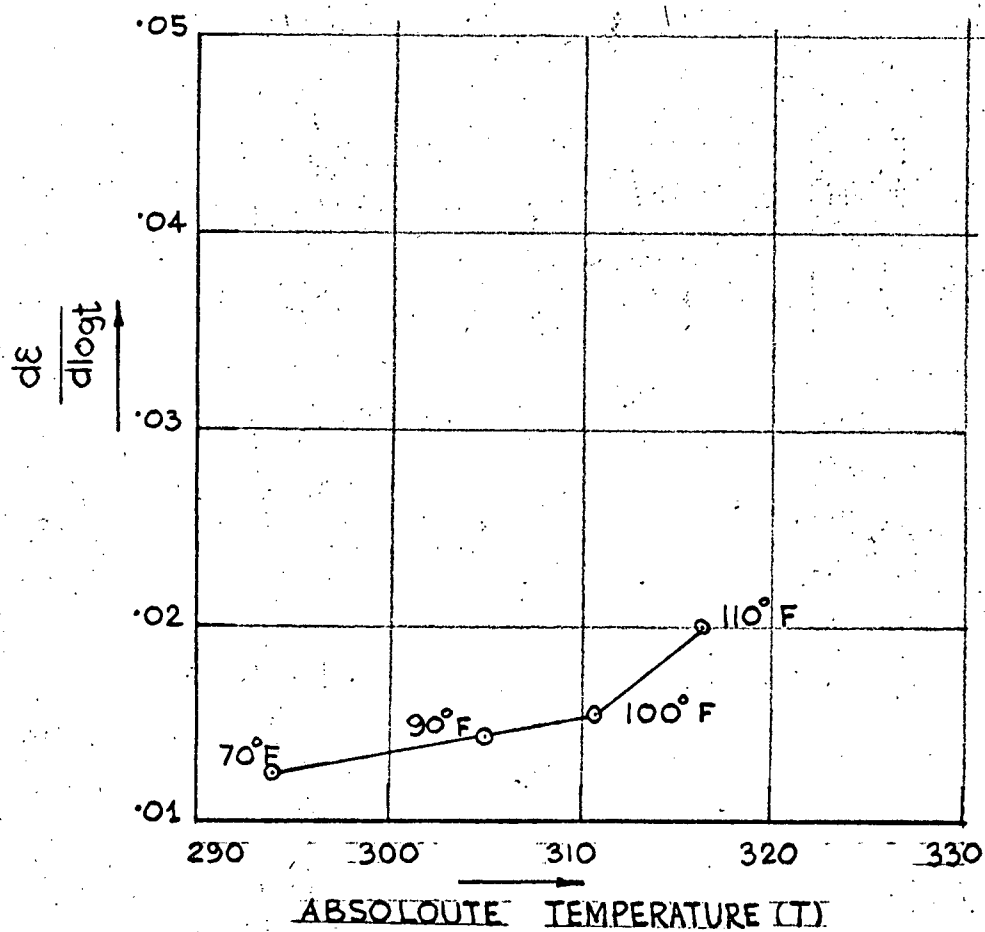


FIGURE 22.



C-U test.
 $(\sigma_1 - \sigma_3) = 23 \text{ psi}$
CHAM. PR = 50 psi