CREEP OF CLAY SOIL UNDER STATIC LOAD

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Thesis
Submitted to the Department of Civil Engineering
in Partial Fulfillment of the Requirements
for the degree

Master of Science

Field of Civil Engineering

by

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Evanston, Illinois

June, 1962

ACKNOWLEDGEMENTS

The author wishes to express his gratitude to Dr. J. O. Osterberg for suggesting the topic of this investigation and for supervision and advice during preparation of this thesis. He also wishes to thank Dr. R. L. Kondner for the indispensable help and guidance in accomplishing this work.

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NOTATIONS

The following symbols have been adopted for use in this thesis :

w_t = water content of trimmings

w_o = initial water content of sample

w_e = final water content of sample

W_s = weight of solids of sample

W_o = initial weight of wet sample

S_o = initial degree of saturation

S_e = final degree of saturation

G = specific gravity

t = time

σ = strength

$\sigma_1 - \sigma_3$ = deviator stress

$J(t)$ = creep compliance

ε = strain

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

γ = shear strain

$\dot{\gamma}$ = rate of shear strain

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INTRODUCTION

In almost all soil mechanics phenomena, creep plays a part, hidden or open. An embankment, a slope or a cut can hold for weeks and even for years and then fail. Creep occurs in any loading test; it applies to the settlement of shallow foundations as well as pile foundations.

Often in soil mechanics, predictions are made based on a hypothesis, but any assumption needs experimental data to prove its validity. In their study of stress-strain characteristics of compacted clay under varied rates of strain, Osterberg and Perloff(1)* hypothesized that at constant stresses lower than the maximum, clay would yield with time and perhaps fail, because the strength decreases with decreasing rate of strain.

The aim of this thesis is partly to verify this hypothesis and to study the soil behavior under sustained loads less than ultimate loads. Stresses ranging from 60 to 95 percent of the ultimate strength are applied to the samples and then maintained constant.

Series of tests of two basic types were performed at various water contents. The first series is of the unconfined compression type, the others are of the consolidated undrained triaxial compression type. Since data are insufficient for comparison between different series, a special study is made of Series C which had given excellent results.

A rheologic analysis of the experimental results of this series is made to study the soil response during creep. Properties as linearity and viscosity are considered and a stress-strain-time relationship based on triaxial creep tests is developed and applied to the case of a triaxial compression test for a constant rate of strain

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* Numbers in ( ) refer to references listed at the end of this paper.

THEORETICAL ANALYSIS

Review

In 1931, Terzaghi[2] emphasized the importance of slow plastic flow of clays in the design of foundation structures. Based on the results of undrained compression tests, he noticed that clays had shown plastic deformations at a shear stress below the shearing strength determined by laboratory tests, and that they may deform for a long time.

In a series of tests on undisturbed samples, Casagrande and Wilson[3] have found that brittle clay and clay shales creep under a sustained load less than the load required to produce the ultimate compressive strength. They pointed out that sustained loads reduced the strength of saturated clays and explained the occurence of the slides along the Panama Canal by means of this theory. In a few tests, they found that the strain at failure was not influenced by the test duration whereas in some others an increase in failure strain with increasing test duration was observed. It was also noticed that for saturated clays, the strength decreases linearly with the logarithm of time.

On the other hand, Goldstein[4] gave the results of tests on samples having the same properties, i.e. size of samples, consistency of the clay, water content, stress history, etc. He stated that the samples failed after a different time under different loads, but the strain at failure stayed the same. He defined the failure strain as that at which the strain rate increased. He also found that it was independent of testing procedure, state of consolidation and test-duration. On this basis, Goldstein could determine the time of failure by extrapolating the strain-time curve inasmuch as the deformation at failure has the same magnitude.

Bjerrum-Simons-Torblaa[2] worked on a normally consolidated marine clay in its undisturbed state, using controlled-strain type of loading. They found a definite decrease in failure strain with increasing test deviation.

Creep Phenomenon

The problem of creep is first a problem of shear stress[5]. As soon as the critical shear stress is exceeded, a slow change in the structure of soil takes place. The resultant plastic deformation is therefore quite different from the elastic deformation.

Slow deformation is a function of time, and it can be seen (Fig. 1) that strength decreases to a value depending on the duration of loading, if there is no strengthening during test.

This decrease in strength is due principally to a structural rearrangement of particles. During deformation, particles move relative to each other. Some bonds are destroyed since they depend upon their relative arrangement and the distance between particles. Some new bonds are formed but it is believed that a long unloaded period is required to have a complete renewal. Therefore this slow and continuous deformation causes a gradual weakening of the soil.

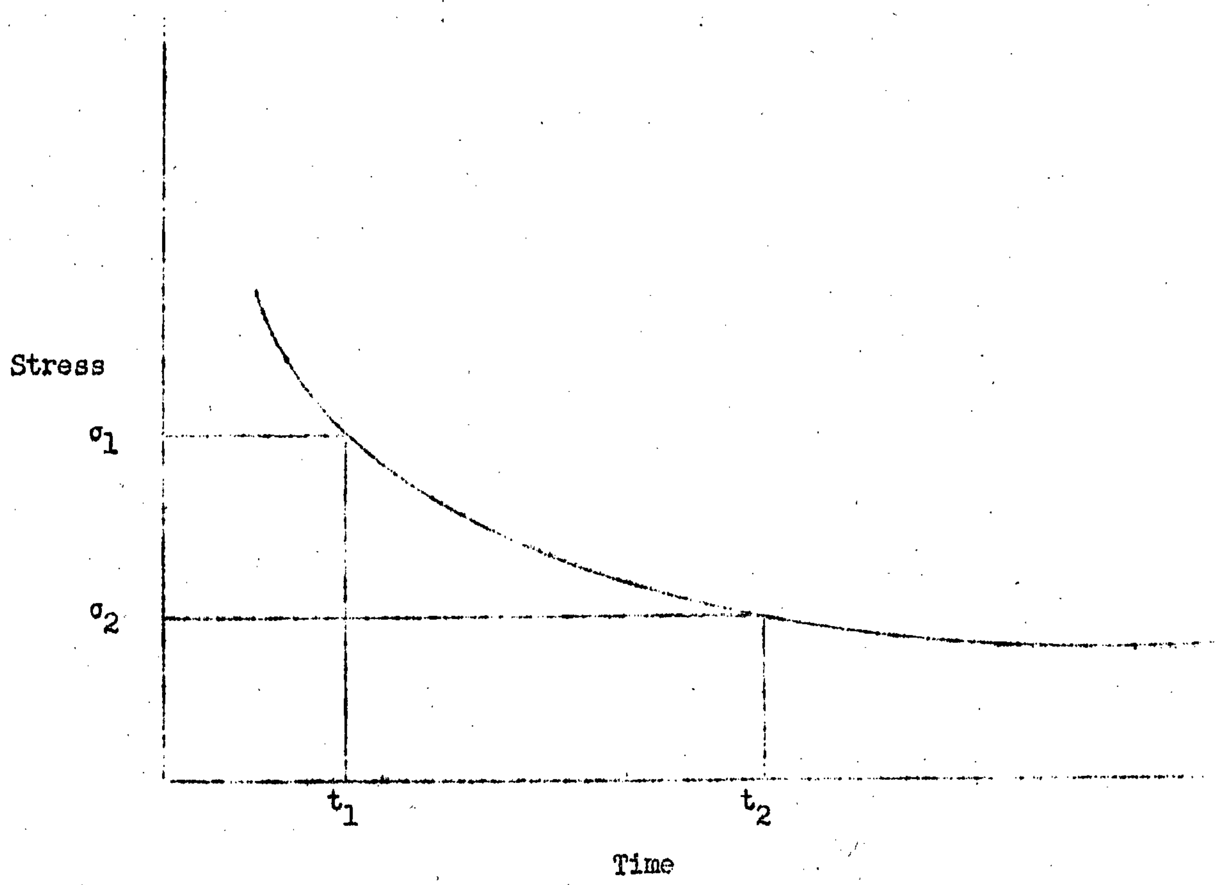


FIGURE 1

Stress-Time Curve

Types of Creep

In nature different types of plastic deformation may occur depending on the amount of load which is applied on the soil. In laboratory test, three types of creep can be observed depending on the type of clay and the stress conditions. When the applied load is small, the clay creeps very little and deformation stops. For a greater load, the soil can deform at a decreasing rate for an indefinite period of time, or it can deform at a nearly constant strain rate until the rate increases rapidly to cause failure.

EXPERIMENTAL INVESTIGATION

Types of Test

In this investigation, two types of test were performed :

1 - Unconfined Creep Strength Tests: For these tests, the specimens were encased in two thin membranes separated by a thin coating of silicone grease. The samples were set up in triaxial chambers and were subjected to air at atmospheric pressure. They were weighed both before and after test to determine the initial and final water contents. When the test lasted less than 5 days, the water content change was usually less than 0.6%, but it was about 1% for a test of 10 days.

2 - Consolidated-Undrained Triaxial Creep Strength Tests: The sample is first consolidated under an hydrostatic pressure and then, without permitting further consolidation it is subjected to a sustained load. For a lateral pressure of 1 kg $/cm^2$, only one membrane has been used. However, it was not sufficient to protect the sample from contamination by glycerin, the surrounding liquid. For a lateral pressure of 2 Kg $/ cm^2$ or more, two membranes separated by grease were used.

Test Procedures

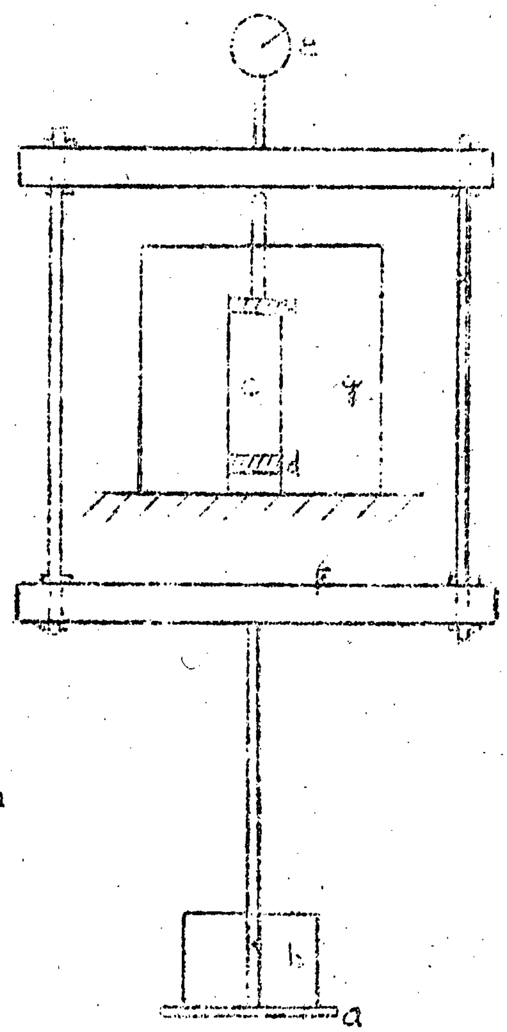
For all tests, the procedure of loading was similar. The specimen was subjected to incremental axial loading, the elapsed time between increments being kept constant. The size of load increment was one kilogram up to the final increment which depended upon the desired stress level. All loads were applied, quickly but without impact, by means of dead weights on the loading cradle supported by the piston. The load was then maintained to study the soil behavior under nearly constant stress at constant volume. The schematic diagram of the apparatus used is shown in Figure 1a.

The regular time interval between loads was 30 seconds. The deformation readings were taken half a minute after load increments. This is the reason why there is no sudden break in the strain-time curve. The analysis of the application of each load increment shows that a sudden deformation occurs followed by a creep deformation which decreases rapidly with time. Since the deformation readings are taken when the strain change decreases, the curve is smooth and a step strain-time curve is avoided.

Preparation of Samples

In order to run the above tests, similarity of samples is a highly desirable feature. They must be uniform and their stress history controlled. The only way to fulfill these requirements is to work with remolded samples. Undisturbed samples would be more useful in order to compare the results with what happens in nature, but the impossibility of getting uniform duplicated samples is sufficient not to use undisturbed ones. Moreover the samples must be prepared at as high a degree of saturation as practicable. This is required to eliminate the volume changes during testing and therefore eliminate strength variation.

The apparatus used for the preparation of the samples was a Vac-Aire extruder. It consists of an extrusion auger which orients the



a. loading platform

b. load

c. soil specimen

d. porous stones

e. indicator dial

f. hanger system

g. chamber

FIGURE 1a

Schematic Creep Apparatus

particles spirally. This method has the advantage of giving a uniform structure to the soil. The samples also pass through a vacuum chamber which prevents the formation of air voids and gives highly saturated samples.

The dry soil was first mixed with a certain amount of water to get a water content of 31%. Then the clay passed 5 times in the machine to get the best homogeneity possible. It was extruded as a circular bar of 3.5 cm in diameter and cut into 10-cm lengths. The specimens were sealed with 4 layers of wax and kept in storage in the humid room to prevent any loss of water. Since the influence of thixotropy is important, a special discussion of its effects is done at the end of the section: Results of tests.

Before testing a sample, the wax was carefully peeled off, and the length was reduced to 7.6 cm. The trimmings were used as a check up of the initial water content.

<u>Soil Tested</u>

The soil used was a clay of the Potapsco Formation, Potomac Group, Lower Cretaceous Period. It was obtained from approximately 6 miles north of Baltimore, Maryland[6].

The Atterberg limits are: Liquid Limit = 42%
Plastic Limit = 21%

The specific gravity is 2.68.

RESULTS OF TESTS

Series A

The first series of tests (Table 1) was of the unconfined compression type. At first, the compressive strength was determined. For a moisture content of about 31%, it was found to be 1.43 and 1.51 kg/cm^2 for tests A-1 and A-2. Bur later, 4 consolidated undrained compression tests with a lateral pressure of 1 kg lcm^2 and a water content of about 32% successively gave the following stresses: 1.51, 1.33, 1.40 and 1.34 kg lcm^2 for samples B-2, B-13, B-14 and B-15. It was decided to take 1.40 kg 1 cm^2 as the ultimate compression strength, which corresponded to a maximum load of 16 kilograms. The different dead loads to be applied were then calculated from this last value.

Seven tests were run with different applied stresses to study the soil behavior. The applied loads correspond to 95, 85, 80, 75, 70, 65 and 60 percent of the maximum load and the results are plotted in Figures 2 and 3. In Figure 2, the strain-time curves show that for the first three minutes, which corresponds to a loading up to seven kilograms, the strain varies very little. But from this time until the end of loading, the strain increases rapidly with time. The curves should coincide for that portion, but they spread somewhat. This is probably due to small changes in water content and thixotropic effects. It can be seen that for samples having lower water contents it takes longer to get the same strain than for specimens having higher water contents.

In Figure 4, the stress-strain curves show a similar sequence. The lower the water content, the steeper the modulus of deformation. This sequence can be better visualized in rearranging the results of tests in the following manner :

#Test	% Applied Load	w_e	Time in Min. at 2% Strain from Figure 2	Stress in Kg/cm at 2% Strain from Figure 3	Date of Test
A-3	95	31.8	3.5 min.	.75	4-20
A-6	75	31.5	4.0	.79	4-23
A-5	65	30.2	4.2	.82	4-20
A-4	85	30.3	4.7	.94	4-20
A-10	70	29.4	4.7	1.00	5-24
A-8	60	29.2	5.3	.95	4-27
A-9	80	29.3	5.8	1.15	5-24

The only discrepancy is for samples A-10 and A-9. But these were tested five weeks later than the others, and they were probably affected by strengthening due to aging as will be explained.

When loading was completed for the specimens which had not failed, the strain rate decreased rapidly and attained a constant value. It seems to be independent of the percentage of applied stress.

Only two samples failed, A-3 and A-4, which represented 95 and 85% of applied load. The first failure occured at the end of loading whereas the second one happened in less than one hour.

<u>Series B</u>

<u>Results</u>:

The tests of this series were of the consolidated undrained compression type. The samples were first consolidated to an hydrostatic pressure of 1 kg/cm^2. The compressive strengths of two samples were 1.08 and 1.05 kg/cm^2 (B-1, B-3), which correspond to a maximum load of twelve kilograms.

The results of eight tests are given in Table 2 and in Figures 4 and 5. Five out of eight samples failed between five and twenty-four hours, two failed after ten days and one did not fail.

<u>Problem</u>:

These results do not agree with those of the first series. Since the samples were consolidated before testing, they should have a lower water content and therefore a greater compressive strength. But it was found that the strength was smaller in the second case. Moreover seven out of eight specimens failed whereas only two samples with 85 and 95% of maximum load had failed in the first series. To explain such inconsistencies, a program of special types of tests was designed.

Swelling Test:

Generally the water content was higher at the end of the tests than at the beginning. Thus a swelling test was designed to determine the swelling properties of the clay. A sample was carefully weighed and measured and placed in the triaxial apparatus. Water was forced through and around the sample by means of a vacuum created at the top of the specimen. The volume increase was only 2.2% and thus could not be the cause of the discrepancy.

Glycerin Test:

In the series A of unconfined compression tests the samples were found uniform enough and the weight of solids was 109.50 ± 1.8 grams. In series B, the weight of solids was usually greater than 114 grams. The only reason for this increase in weight is that a non volatile material could have come into the sample during the test. Glycerin, used in the triaxial chamber, could have played a part because of its special properties. It was then decided to run a test to study the behavior of a sample when contaminated by glycerin.

A dry sample was soaked in glycerin overnight and then dried in the oven. The specimen was weighed at different times and the results can be summarized as follows

Weight of dry sample prior to test : 109.20 grams

Weight of soaked sample : 118.90 grams

Weight of dry sample after one day in the oven	:	115.51 grams
" " " " " two days " " "	:	114.92 "
" " " " " three " " " "	:	114.71 "

To discover how glycerin could contaminate the clay, three consolidated undrained compression tests were run in different ways. It has been suspected that the vacuum which helps the water going through the sample may favour the infiltration of glycerin and that the use of only one membrane may not be sufficient to protect the sample. A summary of results and procedures is the following :

# Test	W_s	Procedures
B-13	109.16	No vacuum - 1 membrane
B-14	108.49	Vacuum - 2 membranes
B-15	109.00	No vacuum - 2 membranes

For these three samples, none was affected by glycerin. The dangerous case is then when vacuum is used with only one membrane surrounding the specimen, as it was employed in the other tests of the Series B.

It can easily be seen that the contamination by glycerin causes an increase in the weight of solids and therefore changes the values of degrees of saturation, water contents, void ratios and volumes of solids and voids.

To get a certain degree of consistency in results, corrections must be made. Since the water content of trimmings is usually known, it is assumed that it represents the conditions of the entire sample. The calculations of the corrected weight of solids are done using this relationship :

$$w_t = \frac{W_w}{W_s} = \frac{W_o - W_s}{W_s}$$

or

$$W_s = \frac{W_o}{1 + w_t}$$

where W_s = Weight of solids

W_o = Weight of specimen Wet before test

w_t = Water content of trimmings.

However, for samples B-1, B-3 and B-5, the average value of 109.50 grams is taken since the water contents of trimmings had not been determined. The difference between the non corrected and corrected W_s is equal to the weight of glycerin in the sample. Substracting this value from the weight of specimen wet after test, the actual weight is obtained. The rest of calculations is done as usual. The results are shown in Table 2.

If the assumption of contamination is true, the discrepancy of lower strength at a lower water content may be easily explained by examining the properties of glycerin. Glycerin [7] possesses remarkable solvent powers and it is a good dispersing agent. In solution with water in a sample, it reduces the strength by changing the structure of soil. Moreover the specific gravity which is approximately 2.5 - 2.6 and the high boiling point (290°C) are the causes of an increase in weight when the soil is dry.

Series C

This series of tests is of the consolidated undrained compression type with a lateral pressure of 2 kg/cm^2. The maximum load was determined by means of 4 tests, C-4, C-5, C-6 and C-8 which gave an average compressive strength of 1.90 kg/cm^2. Ninety, eighty, seventy and sixty % of the maximum load were applied on four samples, but none failed as it is shown in Figure 6 and Table 3. Dead loads were added until failure, in order to verify the ultimate strength of each sample. The stress-strain curves in Figure 7 give a similar compressive strength of 2.56 ± 0.05 kg/cm^2. But this value is much higher than that determined by the earlier four tests. This is probably due to a lower water content and to thixotropic effects during tests.

It can be hypothesized that when the soil is under sustained loads, creep or slow plastic deformation may facilitate a new orientation of particles and bring the soil in a more stable condition. It would explain the strength increase during the creep tests.

Although we have only a limited number of tests for a single consistency, the strain-time curves in Figure 6 show that during creep, the strain rate is nearly independent of the percentage of applied stress. Moreover, under sustained loads, creep is proportional to the logarithm of time.

Series D

To determine what the strength is at a lower water content, one consolidated undrained compression test was made with an hydrostatic pressure of 4 kg/cm^2. The compressive strength was 3.21 kg/cm^2 for a final water content of 27.8%. The results are given in Table 3.

Thixotropy

Thixotropy is the phenomenon of strength gain with time of remolded soil. Mitchell[8] offers an explanation based on the assumption that the internal energy and stress conditions in a thixotropic soil immediately after remolding are not equilibrium conditions.

To verify if thixotropy could affect the results, two unconfined compression tests (A-11, A-12) were made two months later than the first two (A-1, A-2). An increase of 28% in strenght was found. A difference of 1% in water content may change considerably the strength, as it can be seen in Figure 8. The stress-water content curve shows that a decrease of 1% in moisture content corresponds approximately to a gain strength of 20%. Therefore thixotropy had a smaller influence on strength in the tests then the water content.

RHEOLOGIC ANALYSIS OF EXPERIMENTAL RESULTS*

Linearity

Since the first two series of tests gave inconclusive results, it was decided to concentrate the analysis on the third series of tests which was much more consistent. To study the soil response during creep, the results were plotted in various ways.

First, the strain-time curves in Figure 6 are replotted, the initial strain being taken as the last load increment is applied. In other words, the time $t = 0$ corresponds to the strain at the time of application of the last load increment, neglecting the past history of specimens. Such a plot is shown in Figure 9. The units now used are p.s.i. for easier comparaison with previous published works, Kondner (6), (9).

Normalizing the results, the creep compliance J (t) is plotted versus log. time in Figure 10. The creep compliance is defined as follows:

$J(t)$ = creep compliance function of time = $\frac{\epsilon(t)}{\sigma}$

where

$\epsilon(t)$ = strain at time t

σ = stress during creep

For linear materials, a unique relation exists between J (t) and t whatever the applied stress. As it can be seen in Figure 10, the soil is nonlinear since the four curves are well apart.

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* This analysis is based on the presently unpublished work by Kondner on creep response of a clay soil.

Stress-Strain-Time Relationship

The main problem in soil mechanics is to find a stress-strain-time relationship which can be applied to any soil under a variety of conditions. In order to anticipate the soil response, an equation is developed based on the results of creep tests and then applied and compared with results of triaxial tests.

To do so, data from Figure 9 is plotted as follows in Figure 11: $\log t/\epsilon$ in ordinate and $\log t$ in abcissa for the four nearly constant level stresses. What results is four parallel lines. The equation for any of these lines can then be written

$$\log t/\epsilon = \log C + B \log t$$

where

$\log C$ = intercept at time equal to one unit, which in this case is one minute.

B = slope of the lines.

Rearranging the terms, we get:

$$\log t/\epsilon - \log C = B \log t$$

$$\log t/\epsilon C = B \log t$$

$$t/\epsilon C = t^B$$

$$\epsilon = \frac{t}{C\ t^B}$$

$$\epsilon = \frac{t^{(1-B)}}{C} \qquad (1)$$

Let $\frac{1}{C} = A$

$1 - B = d$

Equation (1) becomes :

$$\epsilon = A\ t^d \qquad (2)$$

The value of the slope B is :

$$B = \frac{\Delta \log (\frac{t}{\epsilon})}{\Delta \log t} = 0.989$$

Then $d = 1 - B = 0.011$

To evaluate A, let t = 1 minute in equation (2) :

$$(\epsilon)_{t = 1} = A \qquad (3)$$

Since $\frac{t}{\epsilon} = C = f\ (\sigma)$ in Figure 11,

$$(\epsilon)_{t=1} = A = f\ (\sigma)$$

It must be noted that the factor A depends on the applied stress.

Plotting A/σ versus A (Fig.12) gives the following equation :

$$A/\sigma = a + b\ A$$

where

a = intercept on the $\frac{A}{\sigma}$ axis

b = slope of the line

$\sigma = (\sigma_1 - \sigma_3)$ = deviator stress

or $$A = a \left[\frac{\sigma}{1 - b\sigma}\right] \qquad (4)$$

The values in Figure 12 are taken from Figure 11 and are summarized in Table 4.

Substituting equation (4) into equation (2) gives :

$$\epsilon = A\ t^d = a \left[\frac{\sigma}{1 - b\sigma}\right] t^d \qquad (5)$$

which can be called the creep equation since it was determined from creep tests. Equation (5) may be applied to the case of a triaxial test for a constant rate of strain to give the following development:

Since the strain rate is constant, one may write:

Strain rate = constant = $\dot{\varepsilon} = k$

Therefore, $\dot{\varepsilon} = \frac{\varepsilon}{t} = k$

and $$t = \frac{\varepsilon}{k} \tag{6}$$

Substituting equation (6) into equation (5) gives:

$$\varepsilon = a \left[\frac{\sigma}{1 - b\,\sigma}\right]\left[\frac{\varepsilon}{k}\right]^d$$

$$\frac{\varepsilon}{\varepsilon^d} = \varepsilon^{1-d} = \varepsilon^B$$

$$\varepsilon^B = \frac{a}{k^d}\left[\frac{\sigma}{1 - b\sigma}\right]$$

Taking the $\frac{1}{B}$ root of both sides gives:

$$\varepsilon = \left[\frac{a}{k^d}\right]^{1/B}\left[\frac{\sigma}{1 - b\sigma}\right]^{1/B} \tag{7}$$

Let $$\left[\frac{a}{k^d}\right]^{1/B} = R\ , \tag{8}$$

and $$\frac{1}{B} = S \tag{9}$$

Substituting equations (8) and (9) into equation (7) gives :

$$\varepsilon = R\left[\frac{\sigma}{1 - b\sigma}\right]^S \tag{10}$$

and using $\sigma = \sigma_1 - \sigma_3$ gives :

$$\mathcal{E} = R \left[\frac{\sigma_1 - \sigma_3}{1 - b\,(\sigma_1 - \sigma_3)} \right]^s \tag{11}$$

which is the stress-strain-time equation for a triaxial compression test at constant strain rate as determined from the results of triaxial creep tests.

To verify the applicability of equation (11) the stress-strain response for the triaxial tests of Series C was calculated from equation (11) and compared with the actual experimental response.

The values are the stresses developed during the loading period for the test C-10. Thus, the strains computed can be compared directly with the strains measured in test C-10. Since ninety percent of maximum stress had been applied for test C-10, it represents a nearly complete triaxial test.

To use equation (11) the strain rate must be constant. Since the loading procedure was incremental, an overall average strain rate must be assumed.

Assuming the strain rate k = 0.01 in. per in. per minute gives an average value for R:

$$R_{av} = \left[\frac{a}{k^d} \right]^{1/B} = \left[\frac{1.13 \times 10^{-3}}{(0.01)^{0.011}} \right]^{1.011} \approx 0.001$$

where $a = 1.13 \times 10^{-3}$ = intercept on the $\frac{A}{\sigma}$ axis in Figure 12,

$$\frac{1}{B} = \frac{1}{0.989} = 1.011 ,$$

$$d = 1 - B = 0.011 .$$

Using stress values indicated and the average loading time, strain values were computed by means of equation (11). The results are summarized in Table 5 and the experimental and theoritical stress-strain curves are shown in Figure 13.

As it can be seen in Gigure 13, both the theoretical and experimental curves coincide for strains greater than five percent. For smaller strain, the discrepancy is due to the assumption of constant strain rate. By the load incremental method, the strain rate at small strain is not constant, but as the soil is more susceptible to deformation with increasing load, it deforms more constantly after a certain amount of time. If a controlled constant strain rate had been used during the triaxial compression test, the two curves would have probably been closer to each other.

However, the stress-strain-time relationship is valid only for triaxial tests similar to those of Series C. The equation changes if the lateral pressure, soil, or the water content is different.

Viscosity

The maximum value of shear stress in the specimen for a particular value of deviator stress $(\sigma_1 - \sigma_3)$ is

$$\tau_{max} = \frac{\sigma_1 - \sigma_3}{2}$$

Since the creep test is undrained, there is no volume change in the specimen. Therefore the first invariant of the strain tensor, θ, is zero and one can write

$$\theta = \varepsilon_1 + \varepsilon_2 + \varepsilon_3 = 0$$

and because symmetry

$$\varepsilon_r = \varepsilon_2 = \varepsilon_3 \quad .$$

Therefore,

$$\varepsilon_1 = - 2\, \varepsilon_3$$

and

$$\varepsilon_3 = - \frac{\varepsilon_1}{2} \tag{12}$$

On the other hand, plotting the shear strain versus the axial strain in the Mohr strain space gives

$$\gamma_{max} = \frac{\varepsilon_1 - \varepsilon_3}{2} \tag{13}$$

Therefore $\gamma_{max} = \frac{3}{4}\, \varepsilon_1$

and

$$\dot{\gamma}_{max} = \frac{3}{4}\, \dot{\varepsilon} \tag{14}$$

where $\dot{\gamma}_{max}$ = maximum rate of shear strain

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

To see how the viscosity acts during creep tests, a graph of shear stress, τ, versus rate of shear strain, $\dot{\gamma}$, has been plotted at different times. Figure 14 shows that there is no constant relation between shear stress and rate of shear strain. Since the viscosity η is defined as the slope of the curve of τ versus $\dot{\gamma}$, it can be seen that the viscosity is not constant during creep tests, which is another proof of non linearity of the material. The viscosity depends on time, water content and applied stresses, but it is impossible to determine how it varies because the data are insufficient.

Quasi-Bingham Material

A Newtonian material has a linear relationship between stress and strain rate, which passes through the origin while non Newtonian is any other curve. A Bingham response is also linear, but deformation occurs when a certain amount of stress is developed, that is, when a "yield value" has been exceeded.

Rearranging the terms of equation 7, we get

$$\dot{\varepsilon} = \left[\frac{a}{\varepsilon^{B}}\right]^{\frac{1}{d}} \left[\frac{\sigma}{1 - b\,\sigma}\right]^{\frac{1}{d}} =$$

$$\dot{\varepsilon} = \left[\frac{a}{\varepsilon^{B}}\right]^{\frac{1}{d}} \left[\frac{\sigma_1 - \sigma_3}{1 - b\,(\sigma_1 - \sigma_3)}\right]^{\frac{1}{d}} \tag{15}$$

where $a = 1.13 \times 10^{-3}$

$b = 0.0301$

$\frac{1}{d} = \frac{1}{.011} = 91$.

Let $B = 1$

and $\varepsilon = 10\%$

and equation (15) becomes

$$\dot{\varepsilon}^{\frac{1}{91}} = (1.13 \times 10^{-2}) \left[\frac{\sigma}{1 - b\,\sigma}\right] \qquad (16)$$

Several values of $\dot{\varepsilon}^{\frac{1}{91}}$ are calculated and given in Table 5. As it can be easily seen, to calculate the values of $\dot{\varepsilon}$ one must take the 91 power of $\dot{\varepsilon}^{\frac{1}{91}}$ and this gives difficulties. But one can make a qualitative analysis and estimate the approximative curve.

For $\dot{\varepsilon}^{\frac{1}{91}} = 1$, $\dot{\varepsilon} = 1$. For smaller values, $\dot{\varepsilon}$ decreases very rapidly. Therefore, the relationship between strain rate and stress will indicate a Quasi-Bingham behavior. It means that the strain rate increases with the increase in stress level in a non-linear manner. Figure 15 shows the different relationships between strain rate and stress for the various types of viscous response.

CONCLUSIONS

The hypothesis, that at constant stresses lower than the ultimate strength clay would yield with time and perhaps fail, is only partly verified. Under sustained load, creep occurs and this slow plastic deformation may be of great importance for the structure foundations.

The few samples which failed show a similar behavior. They creep for a while at a constant rate and then the rate of strain increases rapidly to cause failure. This increase in the rate of creep coincides with the reversal of the slope on the strain-time curve.

From the tests in which the samples did not fail, it can be concluded that:

1 - creep is approximately proportional to the logarithm of time under sustained loads.

2 - the strain rate is nearly independent of the amount of the applied stress.

Hvorslev[2] noticed that investigations of the long-term rheological properties of clays were by nature time consuming, but he pointed out that basic research into the problem would lead to short-term tests or methods for making reliable estimates. The results of the tests of the Series B, even if they are not reliable with the results of other series, prove that creep may be accelerated by the use of a dispersing agent.

The difficulties in creep tests are the changing conditions during tests. The soil may be affected by a dissipation of pore water and by new bonds created when there is a reorientation of particles. The use of a dispersing agent may avoid these difficulties since it reduces the strength and causes failure sooner. A relationship between time of failure and stress and strain which has not been found yet, might possibly be determined.

If glycerin is used in triaxial chambers, adequate membranes should be used and special care taken to avoid contamination of samples.

A rheologic analysis was made of the experimental results of Series C. A creep equation was developed to give :

$$\varepsilon = a\left[\frac{\sigma_1 - \sigma_3}{1 - b(\sigma_1 - \sigma_3)}\right] t^d$$

Then this equation was applied to the case of a triaxial test for a constant rate of strain. The following stress-strain-time equation resulted :

$$\varepsilon : R\left\{\frac{(\sigma_1 - \sigma_3)}{1 - b\,(\sigma_1 - \sigma_3)}\right\}^s \qquad (27)$$

where the parameter R contains the time (strain rate) dependency.

This equation was verified with data obtained from a triaxial test. There was good agreement between the predicted and actual response with almost perfect coincidence for strains greater than five percent. For strains smaller than five percent, the discrepancy is probably due to the assumption of constant strain-rate since the actual loading was incremental. Time can then be taken into account in the development of the equation of the stress-strain curve. However the relationship is valid only for triaxial tests similar to those of Series C, i.e. for the soil of the Potapsco Formation and for samples subjected to an hydrostatic pressure of 2 Kg/cm^2 and having a water content of 29.0 percent.

No unique relation has been found between creep compliance and time for these four creep tests. Creep compliance depended on the applied stress and this proved that the soil was not linear.

By definition, the slope of the curve shear stress versus rate of shear strain is the viscosity. Curves were plotted for times equal to 50, 500 and 3000 minutes and they showed that the viscosity was not constant during creep tests.

A qualitative analysis of the relationship between stress and strain rate indicated that the soil had a Quasi-Bingham behavior. It means that the material had a linear response only after a certain amount of stress had been developed.

Generally thixotropy have affected the results to a certain extent. In the unconfined compression tests made one day and two months after molding the samples, and increase of ten percent in strength was due to this phenomenon. Moreover in Series C, a strengthening was noticed during the creep tests which lasted seven and nine days. Being loaded until failure, the four samples failed at the same ultimate strength, but at a much higher value than predicted.

TABLE 1

#Test	Date	w_t in %	w_o in %	w_e in %	W_s in gms	S_o in %	S_e in %	$\sigma_1 - \sigma_3$ in kg/cm^2	% of ultimate Strength	Time to failure in min.
A-1	4-18		30.6	31.3	109.84	101.7	104.4	1.43		
A-2	4-19		31.4	30.9	109.27	101.0	103.1	1.51		
A-3	4-20		32.4	31.8	107.75	100.9	105.5	1.30	95%	7
A-4	4-20		30.3	30.3	110.71	101.8	99.6	1.30	85	30-190
A-6	4-23	32.9	32.0	31.5	107.64	100.0	104.6	1.11	75	
A-5	4-20		30.6	30.2	110.16	101.5	104.4	0.99	65	
A-8	4-27	31.2	30.9	29.2	109.99	101.9	106.0	0.94	60	
A-9	5-24	30.4	30.1	29.3	111.33	101.2	103.2	1.27	80	
A-10	5-24	30.8	30.7	29.4	110.34	104.6	101.8	1.11	70	
A-11	6-10	30.1	30.2	30.0	110.26	104.1	103.8	1.78		
A-12	6-11	29.9	30.0	29.9	110.73	102.6	105.2	1.83		

Results of Tests of Series A of the Unconfined Compression Type

TABLE 2

#Test	Date	w_t in %	w_o in %	W_e in %	W_s not corr. in gms	W_s Corr. in gms	S_o in %	S_e in %	$\sigma_1 - \sigma_3$ in kg/cm^2	% of Ultimate Strength	Time to failure in min.
B-1	4-19		30.1	30.0	111.96	109.50	97.0	101.0	1.08		
B-3	4-23		31.3	29.3	114.85	109.50	102.1	95.6	1.05		
B-2	4-21		31.6	32.2	109.80	109.80	102.7	99.9	1.51		
B-7	4-25	30.7		27.0	119.06	110.30	103.3	97.0	1.05	90	28-1120
B-6	4-25	31.0	30.9	26.7	109.59	109.59	100.1	97.6	0.94	80	12 days
B-11	5-3	30.8		29.2	117.95	110.20	98.8	101.1	0.94	80	600-1300
B-9	4-28	32.3		26.7	118.94	103.00	103.5	94.0	0.89	75	300-1740
B-5	4-21		31.7	28.8	114.55	109.50	103.2	101.0	0.85	70	300-1410
B-10	4-30	31.9		28.6	117.26	109.00	106.0	106.5	0.86	70	1530
B-12	5-3	30.8	30.5	25.0	110.49	110.49	104.1	99.9	0.81	65	12 days
B-4	4-25	31.2		25.4	118.12	110.00	102.7	99.7	0.72	60	
B-13	5-11	31.6	31.4	31.9	109.16		105.0	106.9	1.32		
B-14	5-19	31.5	31.9	32.4	108.49		103.8	106.9	1.40		
B-15	5-21	31.8	31.7	31.2	109.00		104.8	103.3	1.34		

Results of Tests of Series B of the Consolidated Undrained Compression Type. Hydrostatic Pressure of 1 Kg/cm^2.

TABLE 3

#Test	Date	w_t in %	w_o in %	w_e in %	W_s in gms	S_o in %	S_e in %	$\sigma_1-\sigma_3$ in kg/cm^2	% of Ultimate Strength
C-4	5-24	31.9	30.6	30.3	110.36	104.4	104.0	1.93	
C-5	5-25	31.5	31.6	31.2	109.01	103.0	103.0	1.85	
C-6	5-26	31.4	31.6	31.1	108.67	101.0	104.0	1.84	
C-8	5-29	31.3	31.2	30.6	108.65	99.8	107.6	1.90	
C-11	6-1	30.4	30.2	29.0	111.15	100.1	107.8	1.29	60
C-7	5-31	30.4	30.0	28.7	111.72	102.1	99.8	1.47	70
C-9	5-31	30.4	30.2	28.8	110.90	102.2	104.4	1.64	80
C-10	6-1	31.6	30.9	29.2	109.45	103.4	104.0	1.81	90
D-3	6-11	29.8	29.7	27.8	111.97	103.0	104.8	3.21	

Results of Tests of Series C and D of the Consolidated Undrained Compression Type. Hydrostatic Pressures of 2 and 4 Kg/cm^2.

TABLE 4

σ in p.s.i.	A	$A/\sigma \times 10^{-3}$
17.8	0.0379	2.13
20.3	0.0630	3.10
22.8	0.0806	3.54
25.2	0.1100	4.37

Summary of Values of Figure 12.
Relationship between Factor A and Deviator Stress.

TABLE 5

$\sigma_1 - \sigma_3$ in kg/cm^2	$\sigma_1 - \sigma_3$ in p.s.i.	$b(\sigma_1-\sigma_3)$	$1-b(\sigma_1-\sigma_3)$	$\frac{\sigma_1 - \sigma_3}{1-b(\sigma_1-\sigma_3)}$	$R\left[\frac{\sigma_1-\sigma_3}{1-b(\sigma_1-\sigma_3)}\right]^s$	$1/91$ ε
0.424	5.90	0.178	0.822	7.19	.0073	.0814
0.528	7.35	0.221	0.779	9.45	.0097	.1066
0.635	8.83	0.261	0.734	12.03	.0124	.1360
0.740	10.30	0.310	0.690	14.9	.0154	.1685
0.842	11.70	0.352	0.648	18.0	.0187	.2040
0.945	13.10	0.394	0.606	21.1	.0219	.2440
1.05	14.55	0.438	0.562	26.1	.0270	.2950
1.14	15.90	0.479	0.521	30.5	.0319	.3450
1.23	17.20	0.520	0.480	35.8	.0370	.4050
1.33	18.50	0.557	0.443	41.7	.0430	.4720
1.41	19.70	0.594	0.406	48.5	.0505	.5480
1.50	20.80	0.626	0.374	55.7	.0580	.6300
1.58	22.00	0.662	0.338	65.2	.0690	.7360
1.66	23.00	0.693	0.307	75.0	.0781	.8490
1.73	24.00	0.723	0.277	86.6	.0900	.9790
1.80	25.00	0.753	0.247	101.3	.1060	1.142

Strain Values Computed by Means of the Stress-Strain-Time Equation.
Qualitative Analysis of Rate of Strain.

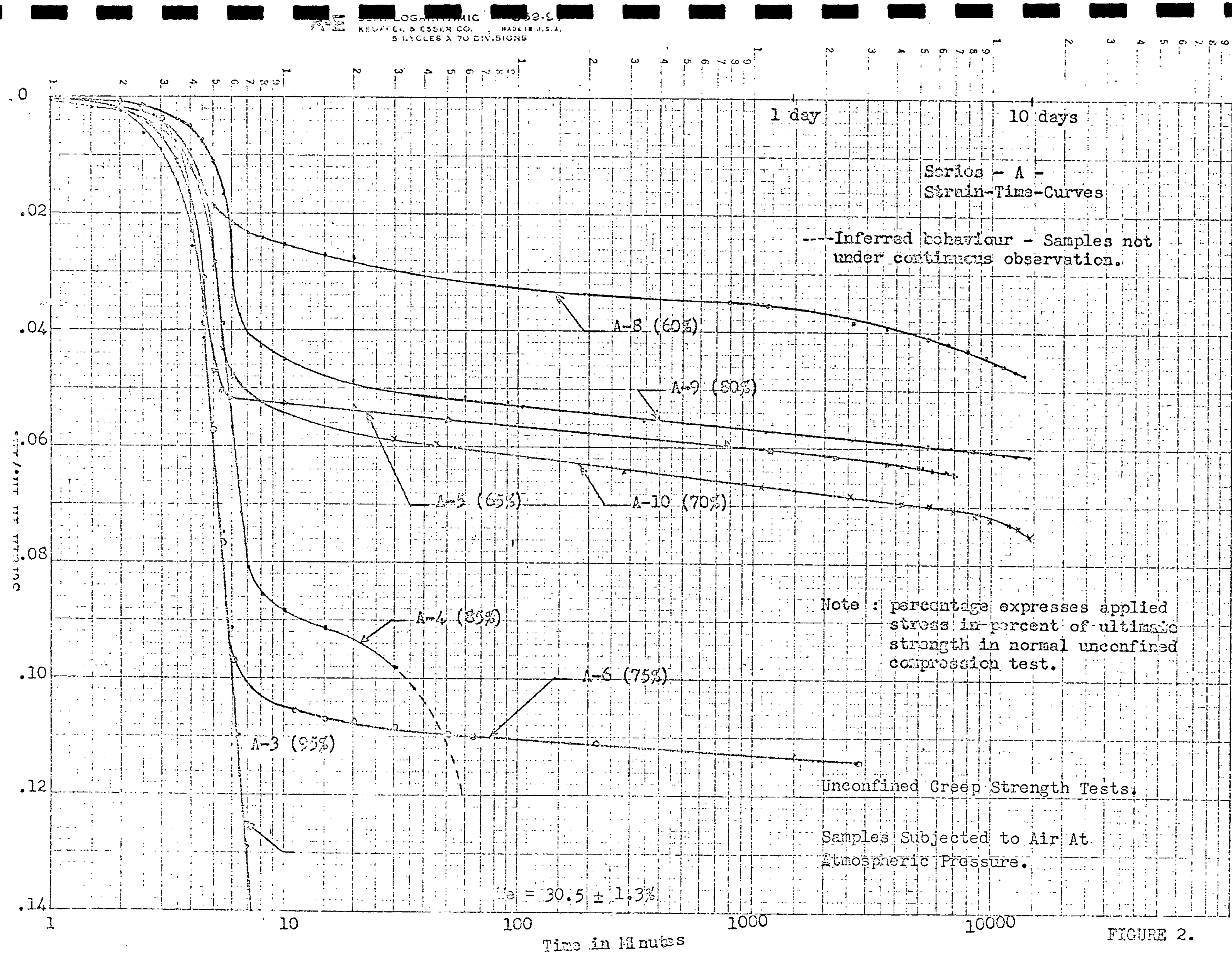
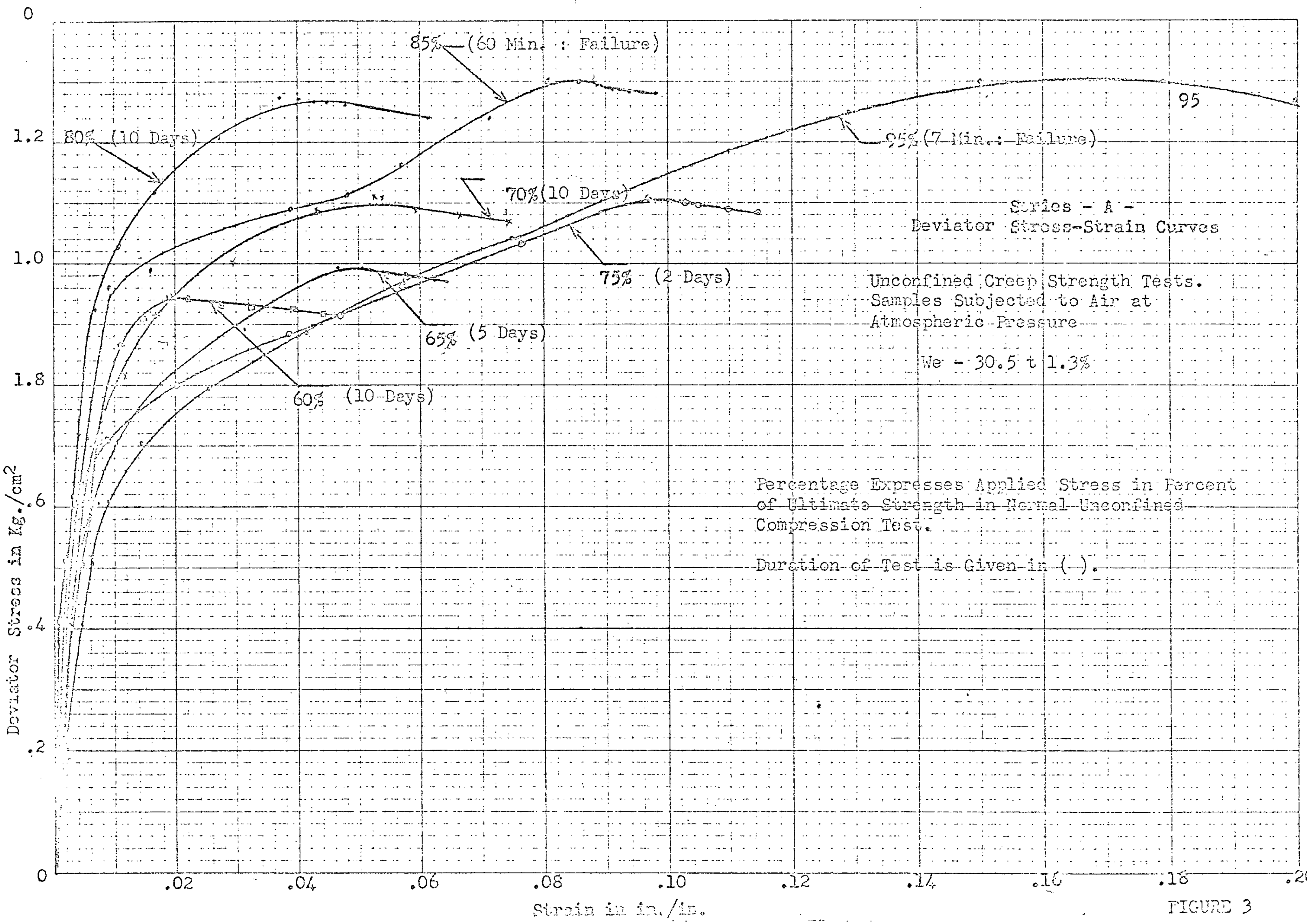

1 day
10 days
Series - A -
Strain-Time-Curves
----Inferred behaviour - Samples not under continuous observation.
A-8 (60%)
A-9 (80%)
A-5 (65%)
A-10 (70%)
A-4 (85%)
A-6 (75%)
A-3 (95%)
Note : percentage expresses applied stress in percent of ultimate strength in normal unconfined compression test.
Unconfined Creep Strength Tests.
Samples Subjected to Air At Atmospheric Pressure.
We = 30.5 ± 1.3%
.0
.02
.04
.06
.08
.10
.12
.14
1
10
100
1000
10000
Time in Minutes

FIGURE 2.



85% (60 Min. : Failure)
80% (10 Days)
95
95% (7 Min. : Failure)
70% (10 Days)
Series - A -
Deviator Stress-Strain Curves
75% (2 Days)
Unconfined Creep Strength Tests.
Samples Subjected to Air at
Atmospheric Pressure
65% (5 Days)
We - 30.5 t 1.3%
60% (10 Days)
Percentage Expresses Applied Stress in Percent
of Ultimate Strength in Normal Unconfined
Compression Test.
Duration of Test is Given in ( ).
Deviator Stress in Kg./cm²
0
1.2
1.0
1.8
.6
.4
.2
0
.02
.04
.06
.08
.10
.12
.14
.16
.18
.20
Strain in in./in.

FIGURE 3

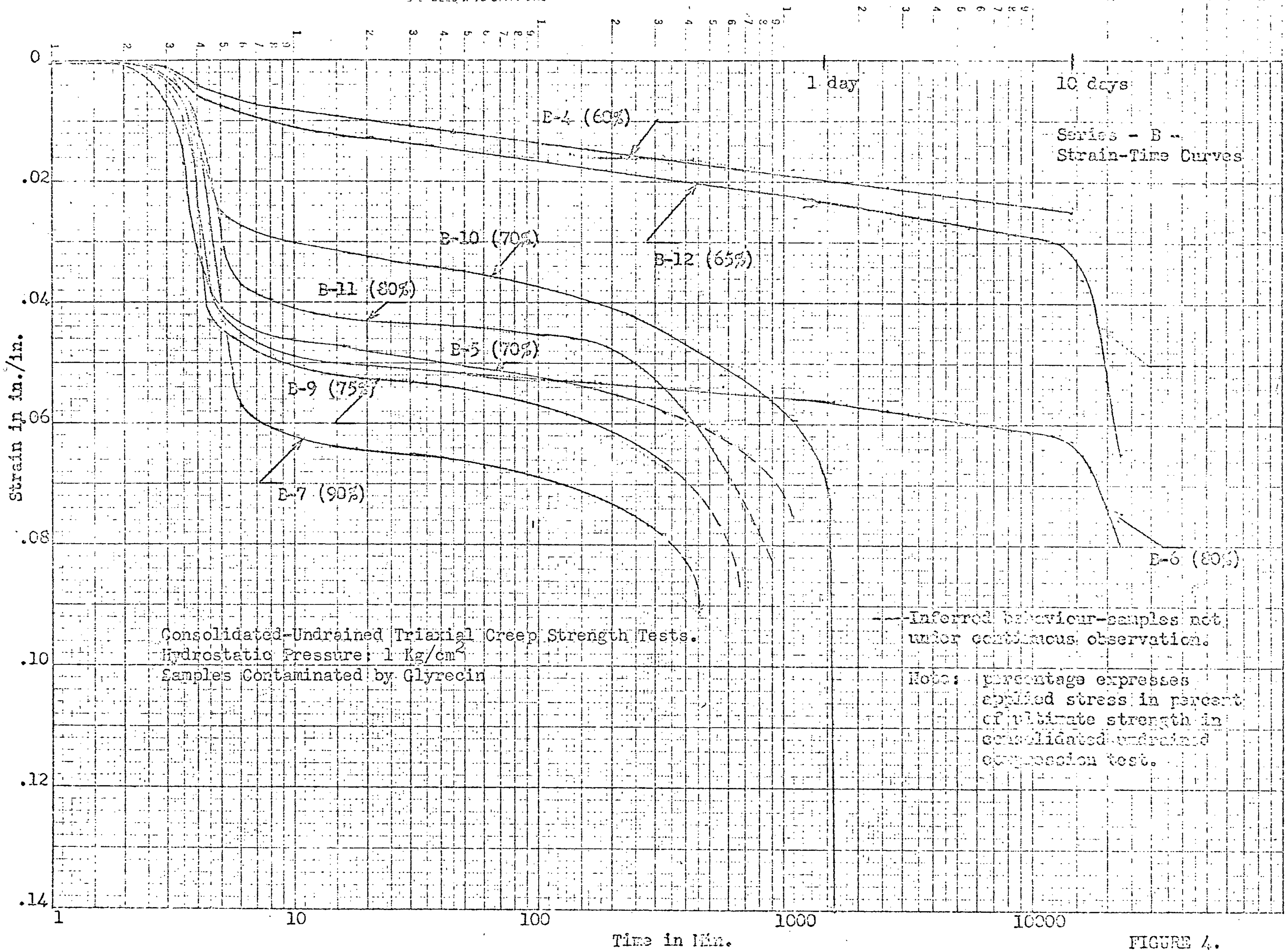
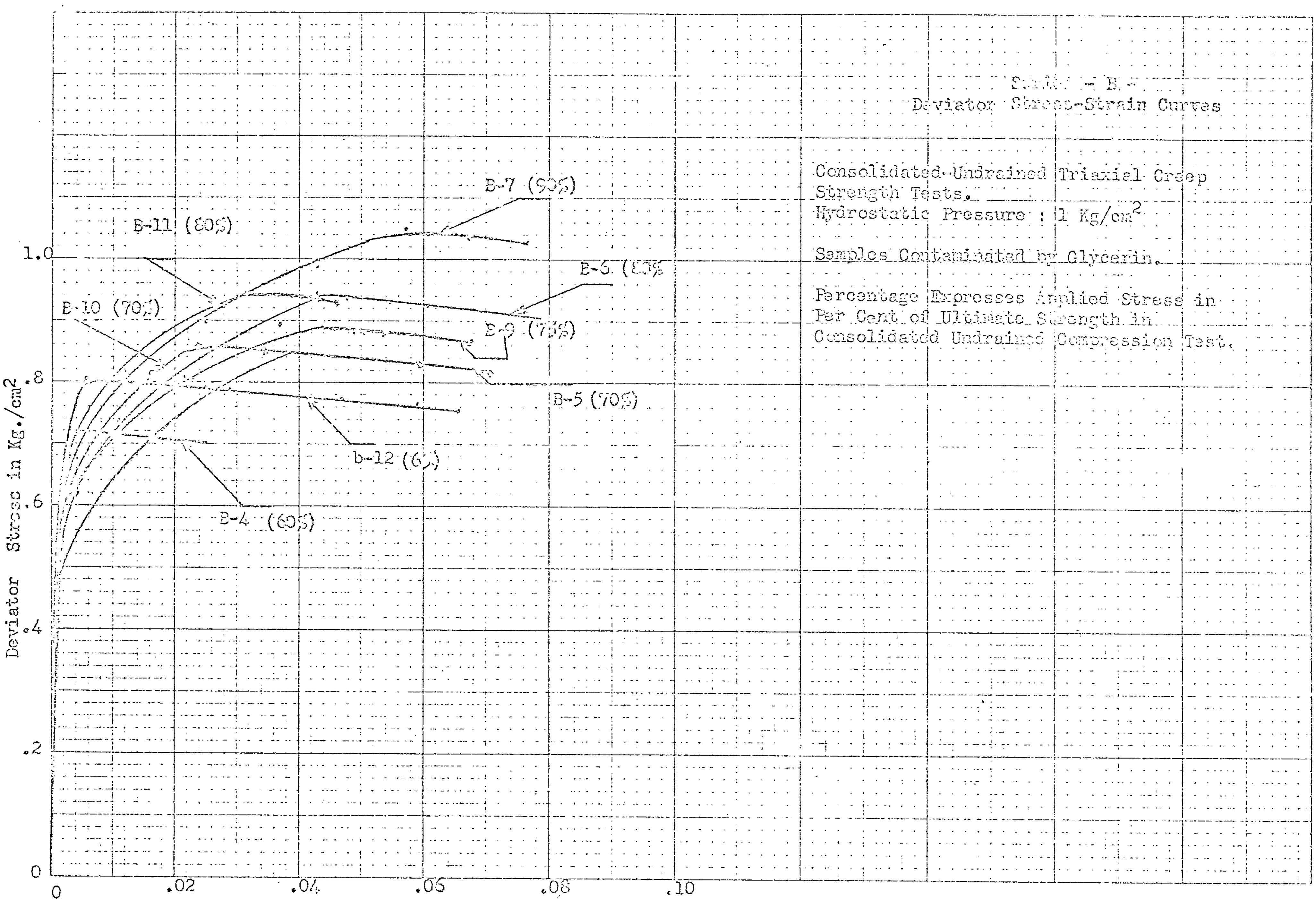

Series - B -
Strain-Time Curves
1 day
10 days
B-4 (60%)
B-12 (65%)
B-10 (70%)
B-11 (80%)
B-5 (70%)
B-9 (75%)
B-7 (90%)
B-6 (80%)
Consolidated-Undrained Triaxial Creep Strength Tests.
Hydrostatic Pressure: 1 Kg/cm^2
Samples Contaminated by Glyrecin
---Inferred behaviour-samples not under continuous observation.
Note: percentage expresses applied stress in percent of ultimate strength in consolidated undrained compression test.
Strain in in./in.
Time in Min.
0
.02
.04
.06
.08
.10
.12
.14
1
10
100
1000
10000

FIGURE 4.



Deviator Stress-Strain Curves
Consolidated-Undrained Triaxial Creep Strength Tests.
Hydrostatic Pressure : 1 Kg/cm2
Samples Contaminated by Glycerin.
Percentage Expresses Applied Stress in Per Cent of Ultimate Strength in Consolidated Undrained Compression Test.
B-7 (90%)
B-11 (80%)
B-10 (70%)
B-9 (75%)
B-5 (70%)
b-12 (6
B-4 (60%)
1.0
.8
.6
.4
.2
0
0
.02
.04
.06
.08
.10
Deviator Stress in Kg./cm2
Strain in in./in.

FIGURE 5

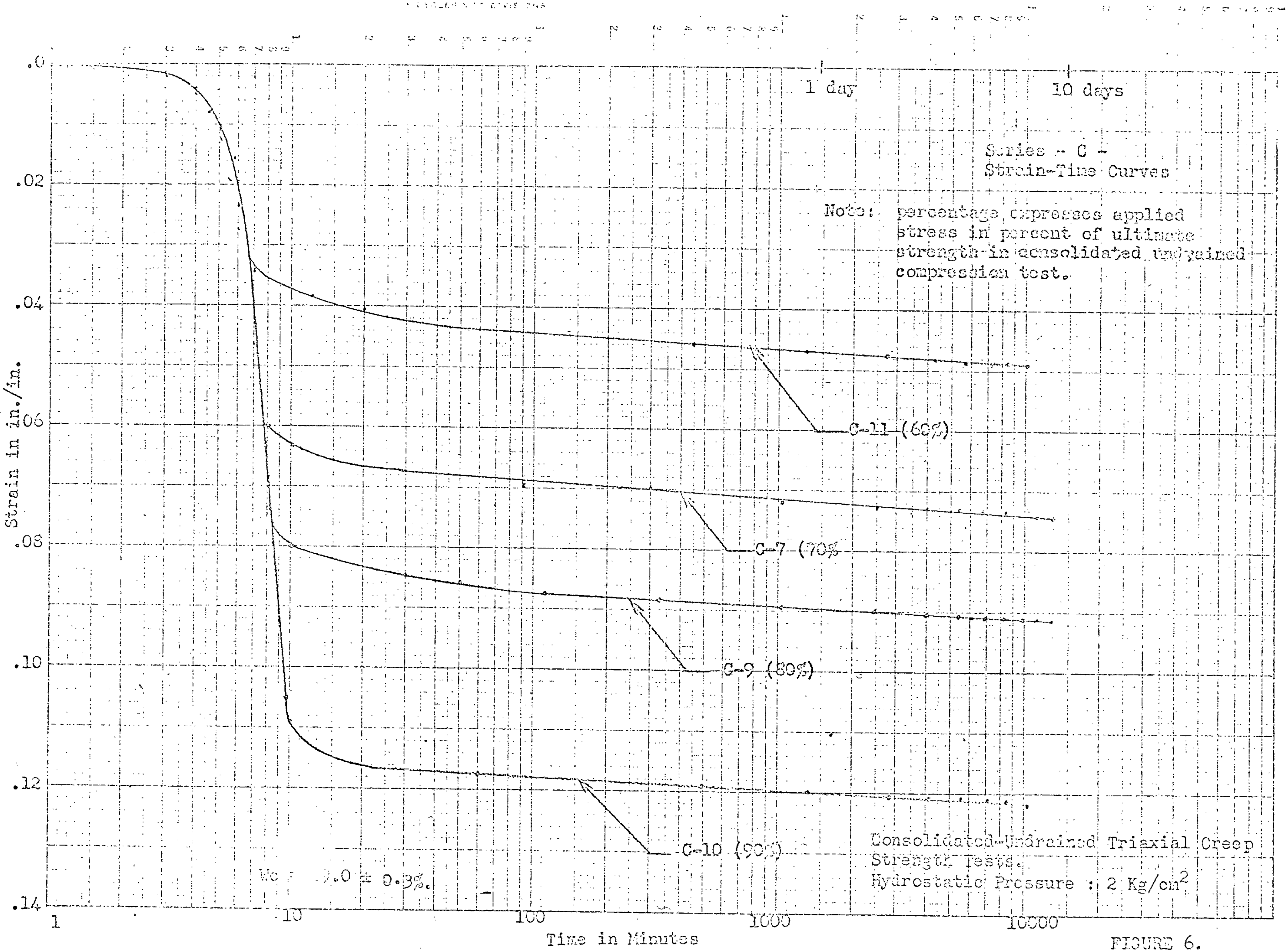

Series - C -
Strain-Time Curves
Note: percentage expresses applied stress in percent of ultimate strength in consolidated undrained compression test.
1 day
10 days
C-11 (60%)
C-7 (70%
C-9 (80%)
C-10 (90%)
Consolidated-Undrained Triaxial Creep Strength Tests.
Hydrostatic Pressure : 2 Kg/cm^2
Strain in in./in.
Time in Minutes
.0
.02
.04
.06
.08
.10
.12
.14
1
10
100
1000
10000

FIGURE 6.

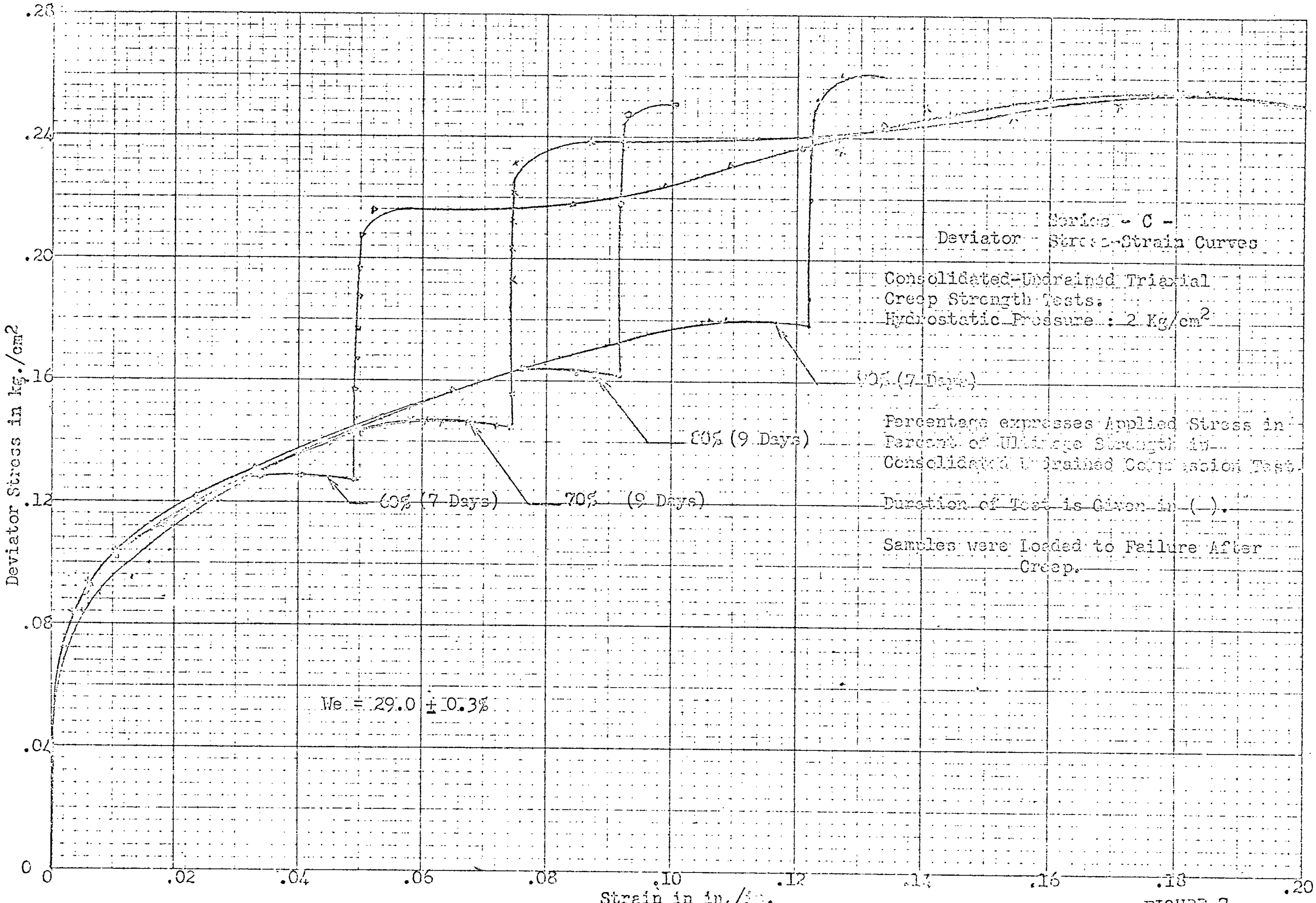

Series - C -
Deviator Stress-Strain Curves
Consolidated-Undrained Triaxial
Creep Strength Tests.
Hydrostatic Pressure : 2 Kg/cm²
90% (7 Days)
80% (9 Days)
70% (9 Days)
60% (7 Days)
Percentage expresses Applied Stress in
Consolidated Undrained Compression Test.
Duration of Test is Given in ( ).
Samples were Loaded to Failure After
Creep.
We = 29.0 ± 0.3%
Deviator Stress in kg./cm²
Strain in in./in.
0
.04
.08
.12
.16
.20
.24
.28
0
.02
.04
.06
.08
.10
.12
.14
.16
.18
.20

FIGURE 7.

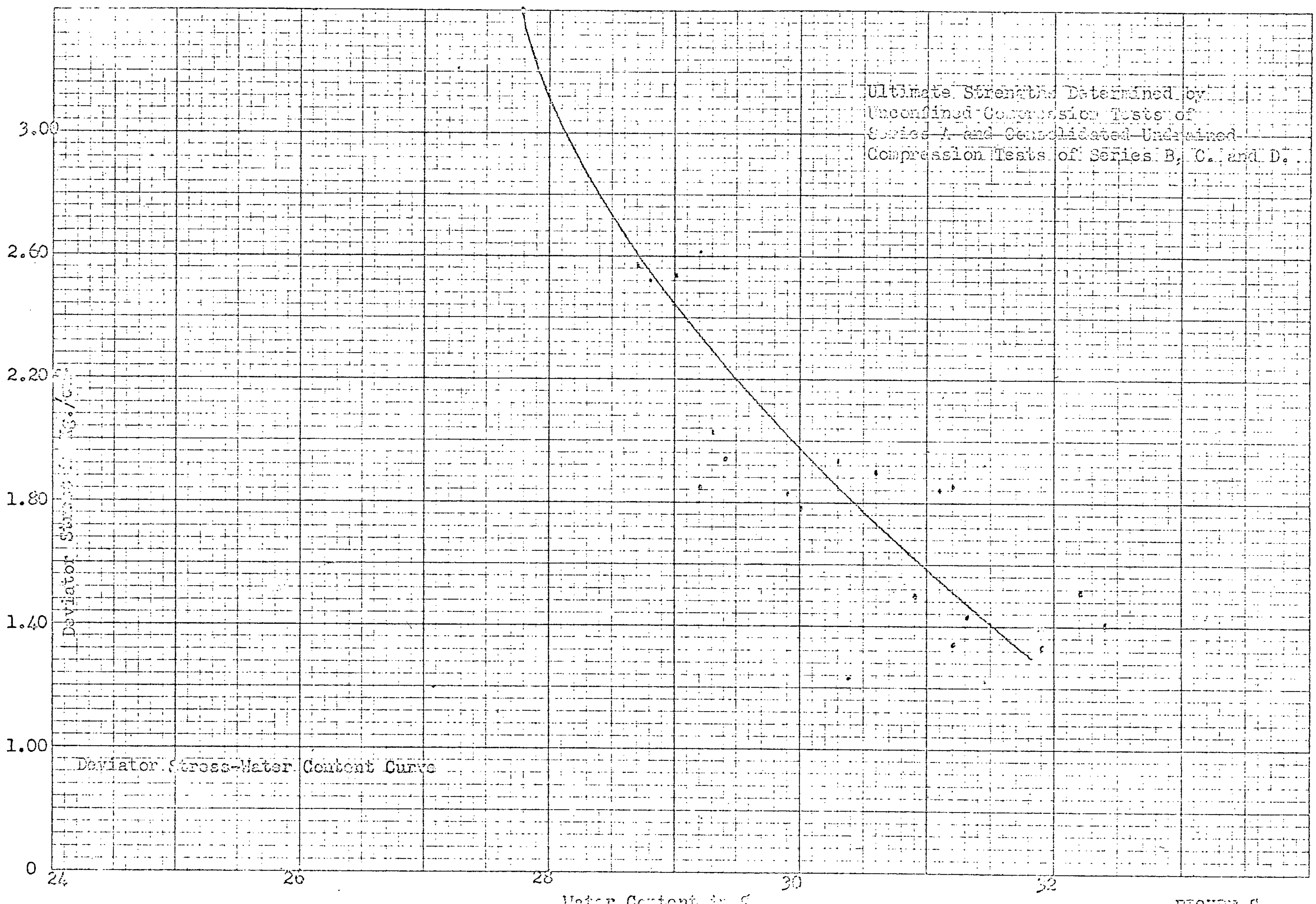

Ultimate Strengths Determined by
Unconfined Compression Tests of
~~Series A and Consolidated Undrained~~
Compression Tests of Series B, C. and D.
Deviator Stress-Water Content Curve
Deviator Stress Kg./cm²
Water Content in %
3.00
2.60
2.20
1.80
1.40
1.00
0
24
26
28
30
32

FIGURE 8.

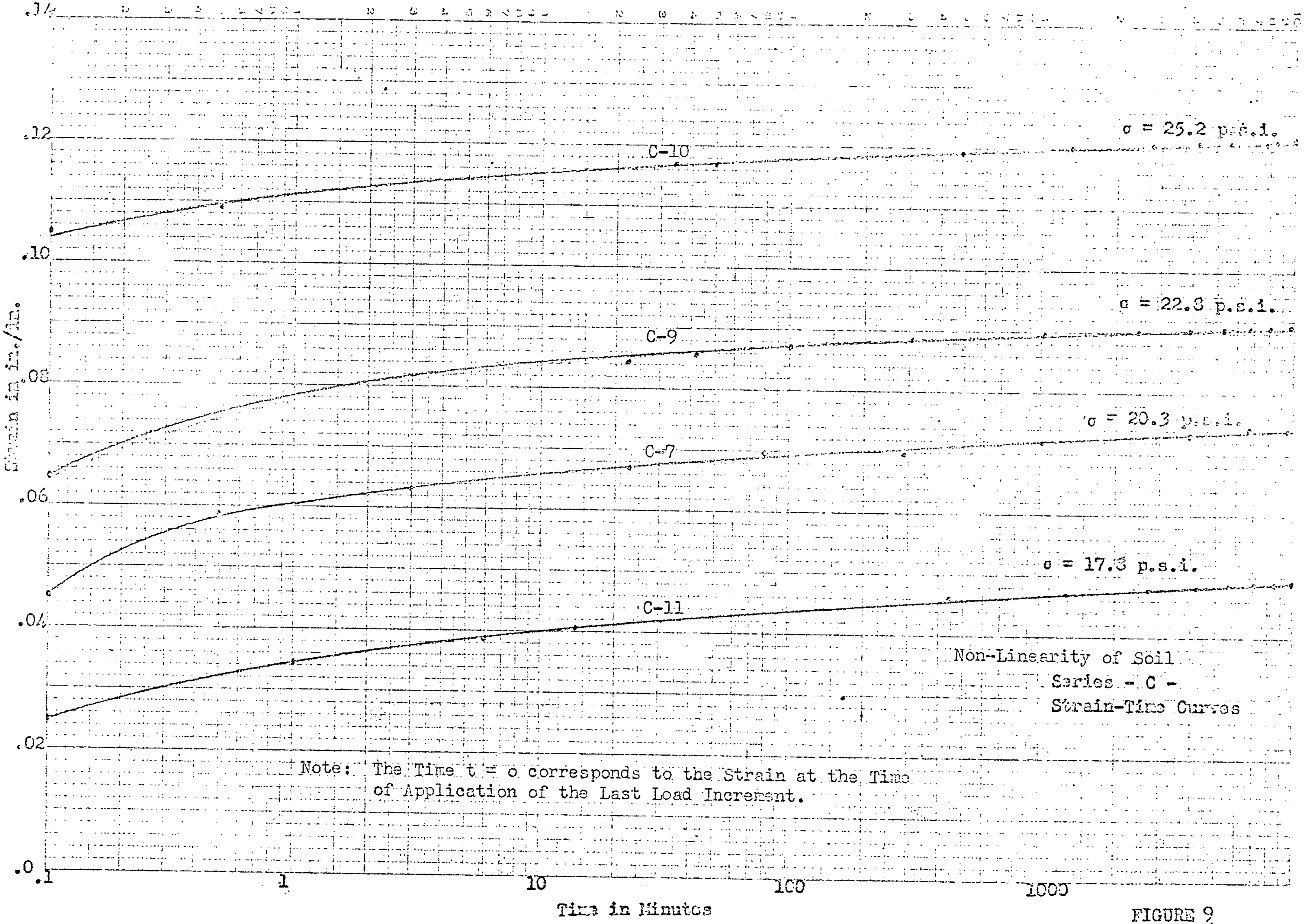

σ = 25.2 p.s.i.
C-10
σ = 22.8 p.s.i.
C-9
σ = 20.3 p.s.i.
C-7
σ = 17.8 p.s.i.
C-11
Non-Linearity of Soil
Series - C -
Strain-Time Curves
Note: The Time t = o corresponds to the Strain at the Time of Application of the Last Load Increment.
Strain in in./in.
.14
.12
.10
.08
.06
.04
.02
.0
.1
1
10
100
1000
Time in Minutes

FIGURE 9

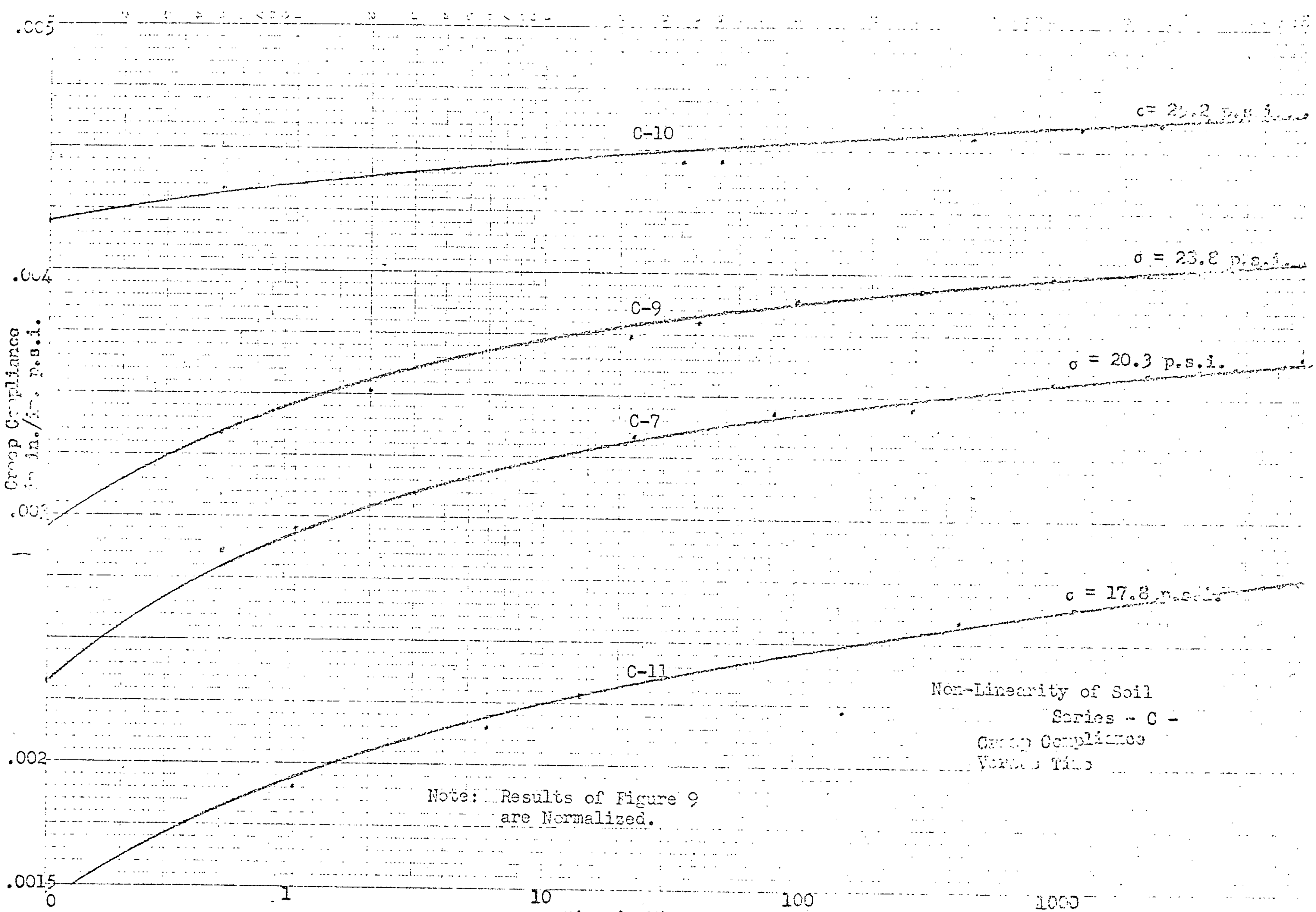
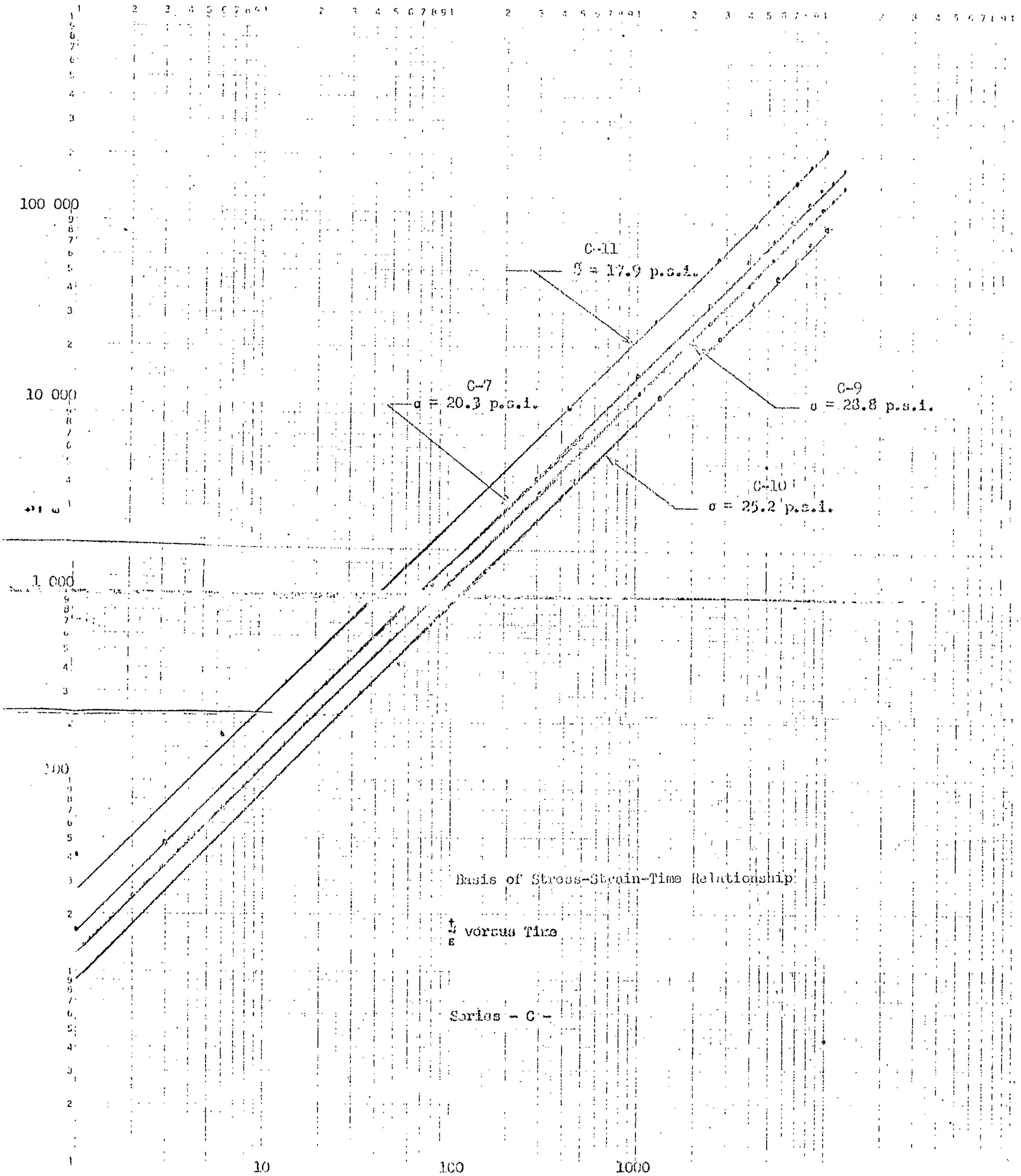
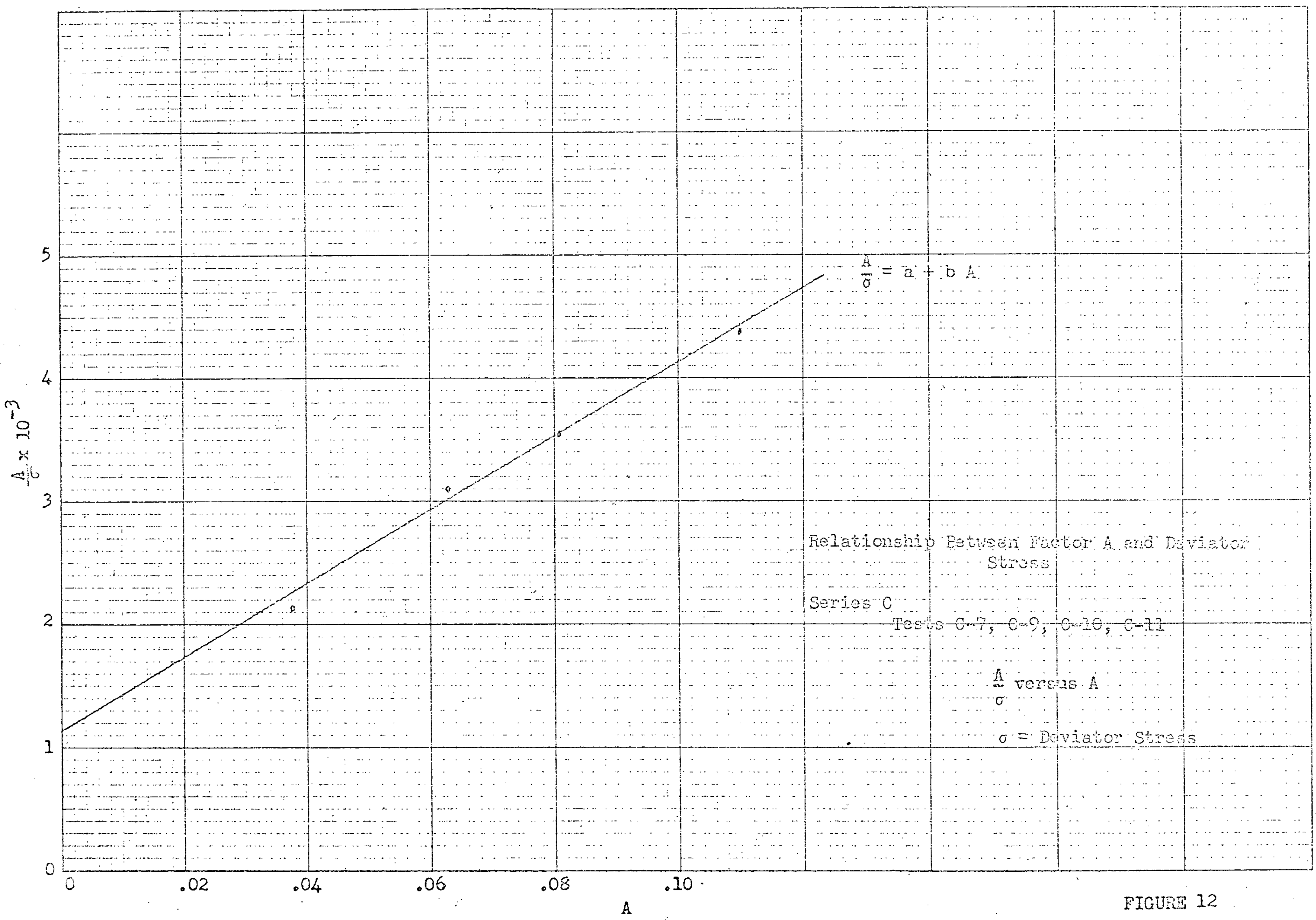

.005
.004
.003
.002
.0015
Creep Compliance in in./in. p.s.i.
C-10
σ = 25.2 p.s.i.
C-9
σ = 23.8 p.s.i.
C-7
σ = 20.3 p.s.i.
C-11
σ = 17.8 p.s.i.
Non-Linearity of Soil
Series - C -
Creep Compliance
Versus Time
Note: Results of Figure 9
are Normalized.
0
1
10
100
1000
Time in Minutes

FIGURE 10.



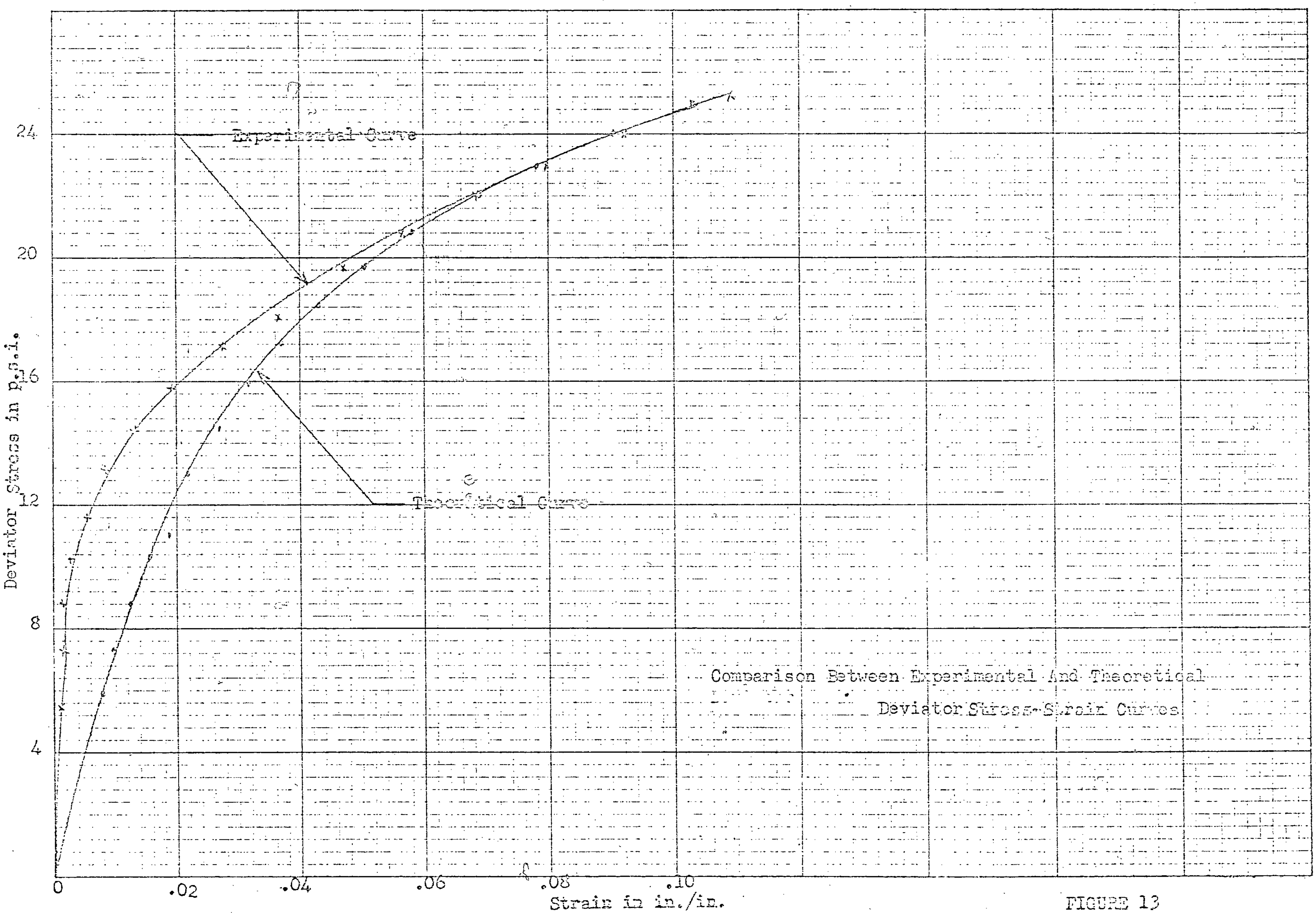
Time in Minutes

FIGURE 11



$\frac{A}{\sigma}$ = a + b A
$\frac{A}{\sigma} \times 10^{-3}$
0
1
2
3
4
5
0
.02
.04
.06
.08
.10
A
Relationship Between Factor A and Deviator Stress
Series C
Tests C-7, C-9, C-10, C-11
$\frac{A}{\sigma}$ versus A
σ = Deviator Stress

FIGURE 12



Comparison Between Experimental And Theoretical Deviator Stress-Strain Curves

FIGURE 13

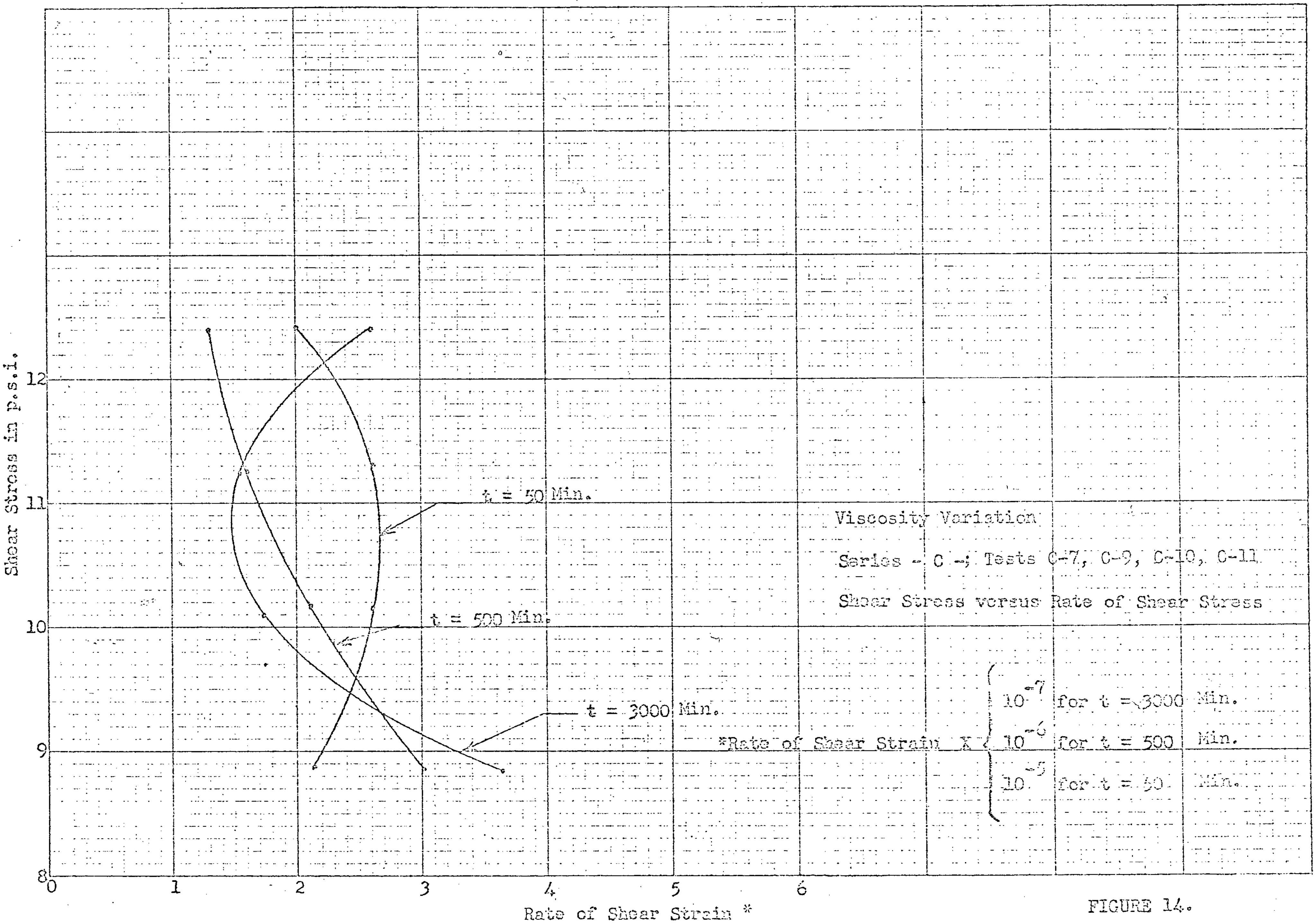

Shear Stress in p.s.i.
8
9
10
11
12
0
1
2
3
4
5
6
Rate of Shear Strain *
t = 50 Min.
t = 500 Min.
t = 3000 Min.
Viscosity Variation
Series - C -; Tests C-7, C-9, C-10, C-11
Shear Stress versus Rate of Shear Stress
*Rate of Shear Strain X
10^-7 for t = 3000 Min.
10^-6 for t = 500 Min.
10^-5 for t = 50 Min.

FIGURE 14.

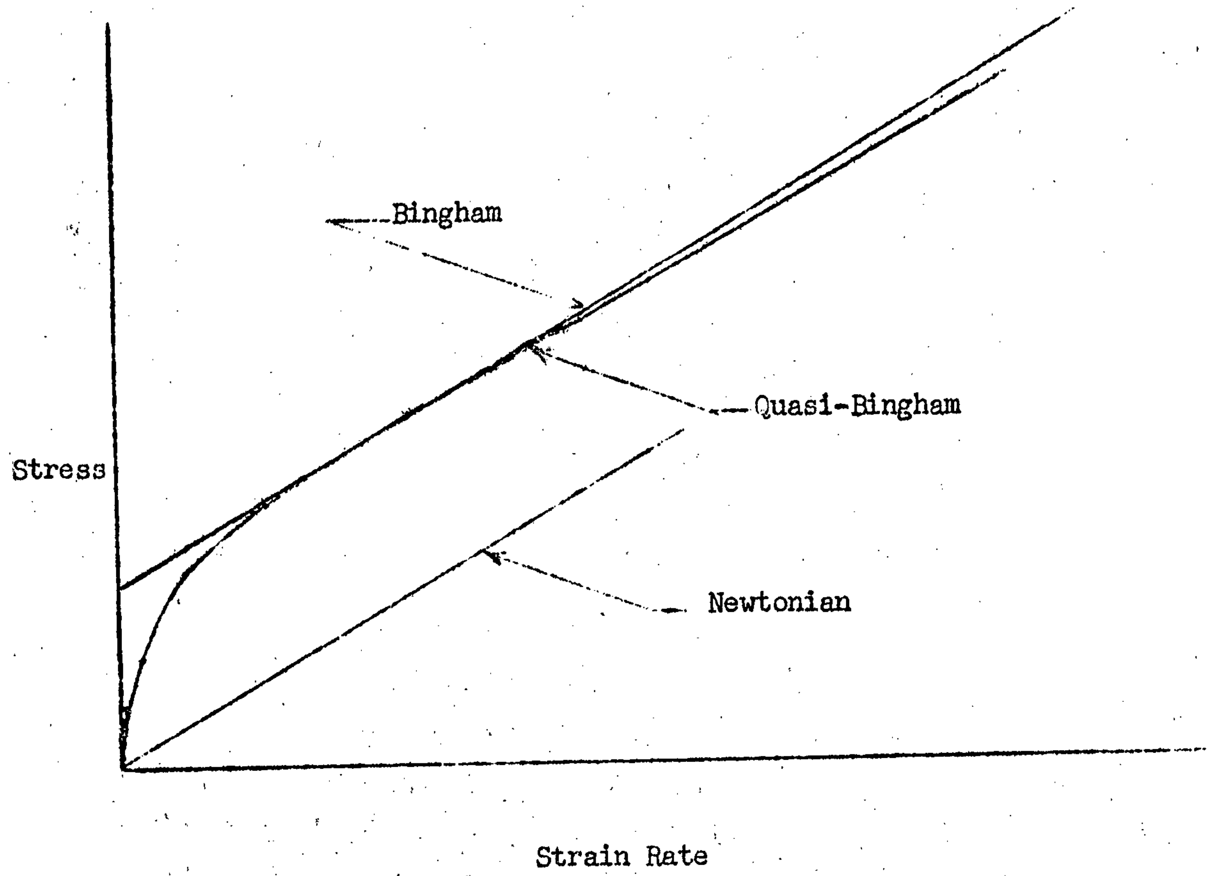


FIGURE 15

Bingham, Quasi-Bingham and Newtonian Materials

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