SHEAR STRENGTH OF A COMPACTED SILTY CLAY

Thesis for the Degree of M. S.
MICHIGAN STATE UNIVERSITY

Anthony Frank Avellano

1959

This is to certify that the

thesis entitled

SHEAR STRENGTH OF A COMPACTED SILTY CLAY

presented by

ANTHONY FRANK AVELLANO

has been accepted towards fulfillment
of the requirements for

MASTER OF SCIENCE degree in CIVIL ENGINEERING

Major professor

Date May 13, 1959

O-169

SHEAR STRENGTH OF A COMPACTED SILTY CLAY

by

Anthony Frank Avellano

AN ABSTRACT

Submitted to the College of Engineering
Michigan State University of Agriculture and
Applied Science in partial fulfillment of
the requirements for the degree of

MASTER OF SCIENCE

Department of Civil Engineering

1959

Approved: _______________

ANTHONY FRANK AVELLANO ABSTRACT

This thesis is an investigation of the nature of shear failure of a compacted silty clay.

The investigation consists of the determination of the true angle of internal friction, an analysis of the interaction of the friction and cohesive components of shear strength, and a study of the deformation characteristics of the material.

Consolidated undrained (CU) triaxial tests with porewater pressure measurements were used to measure the shear strength. Triaxial creep tests were made to determine deformation characteristics.

It was found that there is a point of incipient failure at which the shearing stress equals the frictional resistance of the soil $\bar{\sigma}_n$ tan ϕe. This point occurs at the maximum positive porewater pressure. At lower shear stresses the measured pore water pressure agrees with that computed on the basis of elasticity. At higher stresses, the pore pressures are influenced by the displacement of particles along the failure plane. It was also concluded that cohesion becomes stressed at incipient failure. Excessive deformation occurs when cohesion is stressed.

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ACKNOWLEDGMENT

The author wishes to express his indebtedness and deep gratitude to Dr. T. H. Wu, Department of Civil Engineering, Michigan State University, without whose generous help and encouragement this thesis could not have been written.

The assistance of the National Science Foundation is also acknowledged for granting funds for this investigation.

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ERRATA SHEET

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1. P.1, line 2, Coulcomb should read Coulomb.
   Line 4 relatifs should read relatif.

2. P.2, par. 4, line 2 and par. 5 last line,
   Ruthledge should read Rutledge.

Chapter II

1. P. 7, line 1, modulus should read modui.
   Equation (11) N $(\epsilon_1' - \epsilon_3')$ should read N $(\epsilon_1 - \epsilon_3)$

   Equation (12) $\frac{3\epsilon_1' N}{2}$ should read $\frac{3\epsilon_1 N}{2}$

   Equation (13) $-\frac{N\epsilon_1'}{2}$ + $+\frac{N\epsilon_1'}{2}$ should read $-\frac{N\epsilon_1}{2}$ + $\frac{N\epsilon_1}{2}$

   respectively.

Chapter III

1. P. 9, line 1, studies should be studied.

2. P. 10, line 4, killograms should read kilograms.
   Last line 400 should be 300.

3. P. 11, par. 2, line 1, diped should be dipped.

4. P. 12, par. 2, line 1, thest should be these.
   Line 9 respresnt should be represent.

5. P. 13, par. 3, line 1, Tergaphi should read
   Terzaghi.

6. P. 15, par. 2, line 2, diped should read dipped.

Bibliography

1. No. 5, Ruthledge should be Rutledge, P. C.

I. DEVELOPMENT OF CURRENT KNOWLEDGE

The history of the shear strength theory of soils dates back over a century and a half to 1773 when Coulcomb wrote his essay "Essai sur une application des regles de Maximis et Minimis a' quelques problems de Statique, relatifs a' l' Architecture." In this paper, he expressed the classic equation

$$S = C + \sigma \tan \phi, \qquad [1]$$

in which C is the cohesion, and $\sigma \tan \phi$ a frictional resistance proportional to the normal pressure on the plane considered. Although the shear strength equation is simple in appearance, the determination of the parameters C and ϕ in a cohesive soil is a delicate and trying problem.

In 1937, after years of experimentaion, Hvorslev (1)* introduced the modified Coulcomb equation

$$S = Ce + (\sigma_n - u) \tan \phi e \qquad [2]$$

where S = shear strength

Ce = true cohesion

ϕe = angle of true friction

σ_n = total normal stress on the failure plane

u = porewater pressure

$(\sigma_n - u)$ = $\bar{\sigma}_n$, the effective normal stress on the failure plane

*Numbers in parentheses indicate reference listed in Bibliography.

Hvorslev also concluded that true cohesion was a function of the water content. These two criteria of true cohesion and true friction were profound advances in the understanding of the fundamental strength properties of soils.

Rendulic 1937 (2) made the first attempt to measure porewater pressures occurring in a triaxial test. He also showed that the void ratio depends upon the deviator and hydrostatic stresses in a test specimen.

Simultaneously, Hvorslev investigated rapid shearing in soils, and found that a negative pore pressure may be developed, apparently increasing the shearing resistance.(3)

As rapid advancements in the understanding of shear strength were made in Europe, work on triaxial apparatus and shear problems were brilliantly carried out in the United States by Jurgenson and A. Casagrande. Casagrande was influential in determining the effective pressures on specimens in the undrained triaxial test. (4)

Climaxing the Corps of Engineers' Soil Mechanics Fact Finding Survey on Shear Strength (1939-1947), Ruthledge prepared a review of the results obtained in the survey. Of paramount importance was his finding that the shear strength of a saturated soil depends only upon the water content at failure, being independent of the confining pressure σ_3, porewater pressure or the method of testing.(5)

In 1955, G. A. Leonards (6) approached the shear strength problem in a manner very similar to Ruthledge's.

He found that for a given set of initial conditions, the relationship between compressive strength and void ratio at failure is unique, regardless of the confining pressure, drainage, water content or method of testing.

In the Coulomb-Hvorslev equation $S = Ce + \bar{\sigma}_n \tan \phi e$, $\bar{\sigma}_n$ is calculated by $(\sigma_n - u)$. It was just a short period of time before a mathematical expression was derived to solve for $\bar{\sigma}_n$ and the pore pressure u, in terms of the applied stresses in an undrained triaxial test. In 1948, Skempton (7) (8) developed his λ theory for saturated normally loaded clays, based on the assumption that the soil is an elastic material. He also developed an expression relating the pore pressure to the deviator stress and λ . The term λ was introduced as a ratio of the expansibility of the soil to its compressibility. λ varies from $.5 \longrightarrow 0$, and is expressed by $\lambda = \dfrac{C_s}{C_c}$. The pore pressure U_o' is equal to

$$\frac{D}{1 + 2\lambda} \qquad [3]^*$$

where D is the deviator stress.

As determination of pore pressure is so vital in the analysis of effective stresses, Bishop and Henkel (1953)(9) made further studies of the problem. They found, as did Hvorslev, that a negative pore pressure will result in a preconsolidated clay specimen sheared in the consolidated

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*See derivation on pages 6 and 7.

undrained test. This negative pore pressure developed during shear, remained even after the load was removed, causing the soil to absorb water and subsequently fail. Bishop and Henkel explain the negative pore pressure as being caused by dilatancy (expansion of the soil when sheared due to particle movement along the failure plane).

The tendency to undergo volume change during shear develops an additional pore pressure U''_D . U''_D can be expressed using the λ theory and shear deformations by the equation

$$U''_D = - \frac{3\ PD}{1+2\ \lambda} \qquad [4]*$$

where $P = \dfrac{3\epsilon'_1\ N}{2\ Cc}$, N being a constant and ϵ'_1 the principle vertical strain. Knowing U'_D , the shear equation for saturated dilatant soils becomes

$$S = Ce + (\sigma_n - u) \tan \phi e = Ce + [\sigma_n - (U'_D + U''_D)] \tan \phi e. \qquad [5]$$

The triaxial test only measures U_D , but using the λ theory U'_D and U''_D may readily be found.

Finally the pore pressure in a partially saturated soil must be considered. J. W. Hilf has analyzed the pressure in air and water contained in the voids of a soil in the undrained test. (10) The simplified equation for porewater pressure in a partially saturated soil can be expressed as

$$U_D = \frac{C_c\ D + \left(\dfrac{Va}{Pa - u_c}\right) u_c}{C_c + 2C_s + \left(\dfrac{Va}{Pa - u_c}\right)} \qquad [6]$$

*See derivation on page 7.

where Pa = atmospheric pressure

Va = volume of air/unit volume of soil after application of D

u_c = Capillary pressure between grains, varies from 1/2 $Pa \longrightarrow 0$ [11]

Shear strength has thus far been regarded as the sum of Ce and $\bar{\sigma}_n \tan \phi e$. Retrogressing to 1948, A. W. Skempton (12) performed extensive field investigations on saturated impermeable clays using the $\phi e = 0$ analysis. This interpretation of S = C, may well suffice for clays having little or no drainage.

Skempton found that the $\phi e = 0$ method obtained good agreement between the computed and measured factors of safety, but the failure plane in the field did not agree with that calculated.

Opinions vary as to the interaction of the shear strength components. One hypothesis is that of P. W. Rowe (13). Rowe postulated that when a shear stress is applied to a soil, it is first resisted by the frictional component. The cohesive component is brought into action only after the stress exceeds the frictional part. Through circumstantial evidence, Rowe concluded that any shear stress applied to true cohesion results in creep or progressive deformation. In other words, equilibrium of a soil mass is attained only if the applied stress is resisted by the true friction.

II. THEORY

Incipient Failure

The purpose of this investigation is to study the shear strength characteristics and the behavior of the material under stress. One may expect the behavior of the porewater pressure and effective stress to undergo considerable change as a soil specimen is stressed to failure. From Rowe's hypothesis, it seems likely that at some point the shear stress equals the frictional component, and further increase in the stress mobilizes cohesion. This may be called the point of incipient failure.

The shear and normal stresses on the failure plane may be examined from a plot of $\bar{\tau}$ and $\bar{\sigma}_n$. See Figure 1. It is seen that the curve crosses the true ϕ line at point A. After point A is reached, cohesion is mobilized. Previously the shear is resisted entirely by the frictional component.

Porewater Pressure

If an elastic soil in a CU test is subjected to a deviator stress D and hydrostatic stress σ_3, the principle strains may be expressed as follows: See Figure 2.

$$\epsilon_1' = \frac{\Delta\sigma_1}{E_c} - 2\mu_s\frac{\Delta\sigma_3}{E_s} = \frac{D-U_D'}{E_c} + 2\mu_s\frac{U_D'}{E_s} \quad [7]$$

$$\epsilon_3' = -\frac{\mu_c\,\Delta\sigma_1}{E_c} + (1-\mu_s)\frac{\Delta\sigma_3}{E_s} = -\mu_c\left(\frac{D-U_D'}{E_c}\right) - (1-\mu_s)\frac{U_D'}{E_s}$$

where E_s and E_c are the modulus of expansion and compression, and μ_s and μ_c are Poissons' ratios for expansion and compression, respectively.

Also $\left(\frac{1-2\mu_s}{E_s}\right) = C_s$ and $\left(\frac{1-2\mu_c}{E_c}\right) = C_c$ [8]

where C_s and C_c are the expansibility and compressibility, respectively.

In the consolidated undrained test $\Delta V = 0$ for a saturated soil, so

$$\epsilon_1' = -2\,\epsilon_3' \qquad [9]$$

We may combine 7, 8, and 9 and solve for the pore pressure U_D'.

$$U_D' = \frac{D}{1+2\lambda} = \frac{D\,C_c}{C_c + 2\,C_s} \qquad [10]$$

Equation 10 is based on the assumption that the material is elastic, and that no volume change occurs when shear deformation is produced. If the soil tends to undergo volume change during shear, this volume change can be expressed by

$$3\,\Delta\epsilon = N\,\gamma = N\,(\epsilon_1' - \epsilon_3') \qquad [11]$$

where $\Delta\epsilon$ is the normal strain, γ the shear deformation, and N a constant. If equation 9 is substituted into equation 11, then

$$3\,\Delta\epsilon = N\gamma = \frac{3\epsilon_1'\,N}{2} \qquad [12]$$

The total principle strains become

$$\epsilon_1 = \epsilon_1' - \Delta\epsilon = \frac{D-U_D}{E_c} + \frac{2\mu_s U_D}{E_s} - \frac{N\epsilon_1'}{2} \qquad [13]$$

$$\epsilon_3 = \epsilon_3' - \Delta\epsilon = -\mu_c\left(\frac{D-U_D}{E_c}\right) - (1-\mu_s)\frac{U_D}{E_s} + \frac{N\epsilon_1'}{2}$$

Combining 13 with 8 and 9,

$$U_D = \frac{C_c\ (1-3P)\ D}{C_c + 2\ C_s} \quad \text{or} \quad \frac{D}{1+2\lambda} - \frac{3\ PD}{1+2\lambda} \qquad [14]$$

where $P = \dfrac{3\ \epsilon_1'\ N}{2\ C_c}$

Equation 14 is seen to consist of two parts. The first part represents the porewater pressure in an elastic material, and the second part the additional porewater pressure due to the tendency of the soil to undergo a volume change. Since the volume change is brought about by relative displacement of particles along the failure plane, it seems that the second part becomes important only at large strains near failure. Therefore, at low strains the pore pressure may be computed by the expression $U_D' = \dfrac{D}{1+2\lambda}$. As stress increases, soil particles are displaced along the failure plane and cause a decrease in pore pressure by the amount $\dfrac{-3\ P\ D}{1+2\lambda}$. Particle movement is initiated at the point of incipient failure. This point may possibly coincide with the maximum porewater pressure.

Creep

A further objective of the investigation is to study the deformation characteristics of the soil under slow loading. Rowe's hypothesis states that after true cohesion is mobilized, the soil undergoes excessive progressive deformation at constant stress. This phenomenon is called creep, and occurs after shear stress exceeds the frictional resistance.

III. METHOD OF INVESTIGATION

Soil Studied

The soil studies was Mississippi loess . Mississippi loess is a Pleistocene Aeolian deposit found along the east bank of the Mississippi River, extending the entire length of the state. This particular soil came from Vicksburg, Mississippi.

In its natural state, loess is a calcareous clayey silt containing a variety of fresh water and land shells. Loess is light buff in color and rather fine in texture to the touch. The index properties of the loess are given in Table 1 and Figure 3.

Preparation of Soil Specimens

The soil was received in a disturbed condition with most of the natural water content retained. It was put into an air tight metal container and stored until ready for use. The natural water content remained substantially the same during storage.

It is very important to produce quality specimens for triaxial testing. This criterion demands a uniform distribution of soil particles, moisture, and void ratio. These properties were obtained by a tedious process of hand grinding the soil in a commercial meat grinder and then mixing the material in a 12 quart mechanical mixer for 10--15 minutes at a moisture content of about 25%. The soil was thoroughly mixed until a homogeneous substance was obtained. Approximately five killograms of soil were ground and mixed to make one batch of specimens. The preparation procedure resulted in a very satisfactory soil mix.

To obtain a constant void ratio throughout the length of a specimen, it is essential that the compactive effort be uniformly distributed. A CBR mould (6" diameter and 8" high) was used to contain the soil.

A fine copper screen was placed on the bottom of the mould to facilitate drainage during compaction and give the soil a smooth surface. Approximately 4-1/2 inches of soil were placed in the mould in four increments, each layer being kneaded with a rubber tamper. A second copper screen was placed on top of the soil cake.

A 2" high aluminum compacting piston was used having a diameter 1/8" smaller than the CBR mould, thus eliminating the friction between the wall and piston.

The mould was then statically compacted in a 60,000 lb. capacity Tinus-Olson testing machine. The rate of loading was applied at approximately 2% of the total load per minute, and held at the desired value for ten minutes by the automatic load holder. Removal of the stress was instantaneous. Compaction pressure of 400, 1000, and 2000 psi were used.

After a 24 hr. period in a 100% humidity moisture room, the soil was extracted from the mould yielding a cake roughly 6" x 3-1/8". The soil cake was cut into six specimens with a coping saw. Steel plates and a "C" clamp were used to restrict movement while cutting. This procedure was laborious, but proved satisfactory in obtaining uniform undisturbed specimens.

Each specimen was diped in wax, placed in a sealed bottle, then stored in the moisture room for a period of 10 days. This was done to reduce the effects of thixotropy.

Seed and Chan (14) showed that Mississippi loess increases in strength with prolonged storage time. The greater part of the thixotropic strength increase was found to occur during the first 10 days of storage. The time interval between the testing of the first and the last specimen of a batch was about five to six days. Thus, a period of 10 to 15 days of storage was incorporated to minimize the effect of thixotropy.

All test specimens were 2.8" high and 1.40" in diameter. They were trimmed on a hand operated lathe to reduce disturbance. The uniformity obtained was very good.

Experimental Program

The experimental program consists of the determination of the true friction, study of the porewater pressure behavior, measurement of the compressibility and expansibility, and the measurement of creep.

To study the shear strength, the value of the true angle of internal friction ϕe must be known. The true angle of friction may be found by producing two soil specimens with equal void ratios, but different effective normal stresses at failure. Cohesion depends upon the void ratio of a partially saturated soil. If two specimens have equal void ratios at failure, their true cohesion values must be the same. If these two specimens have equal cohesion, the difference in strength must be attributed to the difference in the frictional component, $\bar{\sigma}_n$ tan ϕe.

If the effective principle stresses of thest two specimens at failure are plotted, with $\bar{\tau}$ as the ordinate and $\bar{\sigma}$ as the abscissa, the Mohr's envelope will intersect the ordinate at Ce with a slope of ϕe. Figure 4 shows the graphical construction used to compute the value of the true angle of internal friction. Figure 4a shows two void ratio vs $\bar{\sigma}_{3f}$ curves obtained for one soil. The corresponding effective stress envelopes are plotted in Figure 4b. Points 1 and 2 respresnt equal void ratios at failure, and the Mohr's circles for these conditions are labeled 1 and 2 in Figure 4b. The common tangent then makes an angle ϕe with the horizontal and intercepts the y-axis at a distance Ce above the origin.

Consolidated undrained (CU) tests were performed to obtain the necessary envelopes. Each sample was placed in the triaxial unit and consolidated under a hydrostatic pressure σ_3 until the volume change had ceased. After consolidation, a deviator stress was applied at an approximate rate of 4% of the maximum deviator. Pore pressure measurements being taken simultaneously.

The bulk of the triaxial tests and all of the pore pressure measurements were performed on the three bay triaxial unit shown in Figure 5, designed by Dr. T. H. Wu. This unit is similar to that developed at Harvard with certain modifications that greatly facilitate pore pressure measurements and over-all ease of operation. The unit was designed to provide an axial load of 240 Kg. and a σ_3 of 4.22 Kg. The right cell in Figure 5 is equipped with a harness that allows the application of a constant axial load. The machine has one instrument panel from which saturation, drainage, and pore pressure measurements can be controlled. Each cell functions independently of the other two.

Porewater measurements are made by balancing the capillary tube A in the schematic diagram Figure 6.

In 1936, Tergaphi (15) showed that the inclination of the failure plane with respect to the minor principle axis was included at an angle of $\alpha = 45^\circ + \frac{\phi e}{2}$. Specimens tested in the unconfined compression test provide a convenient means of measuring this angle. This method of determining ϕe is not very accurate, but if the angle is measured as it first appears, a fairly constant value of the true angle of friction is obtained. In this study, a large number of tests were made (approximately 20) and the mean value of ϕe constitutes a reliable estimate. Unconfined compression tests were performed at 2% strain per minute with a standard controlled strain apparatus.

Creep tests were made to study the deformation of the soil. They are very analogous to the CU tests. The specimens were consolidated under a σ_3 pressure until drainage had stopped. Then allowing no drainage, load increments equal to a fraction of the maximum deviator stress were applied. This increment was maintained until all deformation had terminated, then another increment was added. The procedure was continued until the specimen failed. Creep tests were done only on samples compacted to 1000 and 300 psi.

It was imperative to obtain values of C_c and C_s to evaluate the porewater pressures for a partially saturated clay. They may be determined from the consolidation curve, to an arithmetic scale of e vs σ.

$$C = \frac{\Delta V}{V \Delta \sigma} = \frac{\Delta e}{(1+e_o)\Delta \sigma} \qquad [15]$$

Consolidation tests were carried out on 300 and 1000 psi specimens. The first load increment was 125 grams, each increment thereafter being double the previous load.

Void Ratio Determination

A critical factor in the determination of the true angle of internal friction is the void ratio at failure. Extreme care was taken in measuring this void ratio as it is highly sensative to the least error in weight or volume measurements.

After failure each specimen was immediately weighed in air; it was then diped in wax at 124°C and again weighed in air. Thereafter the sample was suspended from a thin thread and weighed in water. Knowing the specific gravity of the wax and soil, the moisture content, and using Archimedes principle, the void ratio can be found. Because of the accuracy needed to evaluate the various components of the void ratio, extensive practice in perfecting the technique was necessary.

IV. RESULTS

True Phi

From the consolidated undrained tests, void ratio at failure vs $\bar{\sigma}_3$ at failure curves and Mohr's effective stress envelopes were determined. Figures 7, 8, 9, and 10 contain the results for soils compacted to 300, 1000, and 2000 psi. The graphical procedure explained in Chapter III was used to compute the true friction. A value of 34^o was obtained from the 1000--300 psi specimens and a value of 31^o from the 2000--1000 psi specimens. See Figure 11. As the number of tests for the 1000 and 300 psi groups is much greater than that for the 2000 psi series, it is felt that the value of 34^o is more reliable. The unconfined compression tests resulted in a ϕe of 39^o. The value of 34^o, being derived from a more accurate method, was used in all ensuing calculations.

Incipient Failure

The results of the CU tests performed are given in Tables 2, 3, and 4. Each table is a summary of data gathered from specimens of one given batch. The data presented does not include all the tests performed, but is representative of the results obtained.

The deviator stress-strain and porewater pressure-strain curves are shown in Figures 12, 13, and 14. Plots of $\bar{\tau}$ vs $\bar{\sigma}_n$ are shown in Figures 15, 16, and 17. Also shown in Tables 2, 3, and 4 are the measured normal and shear stresses on the failure plane at the point where the shear stress is equal to the frictional component of the shear strength. These stresses are found to be in excellent agreement with the calculated stresses at the point of maximum porewater pressure. In other words, the maximum pore pressure occurs at the point of incipient failure.

Unconfined Compression

The 20 unconfined compression tests gave an approximate value of 39° for ϕe. The stress strains curves for a 300 psi and 1000 psi specimen are illustrated in Figures 18 and 19.

Creep Tests

The results of creep tests are shown in Figures 20 and 21. In these figures, the deviator stress is plotted against the strains that occurred under the particular increment of loading.

In essence Rowe's therom is associated with the creep deformation in clay. Upon examining the creep-deviator curves 20 and 21, it is evident that at a particular deviator stress a break in the curve takes place. After this break the strain increases at a much faster rate, terminating in failure of the specimen.

When the 300 and 1000 psi creep tests are compared to CU tests of similar samples, the sudden increase in the rate of deformation occurs at the same stresses as the maximum positive pore pressure (Tables 5 and 6). This finding verifies Rowe's hypothesis and further substantiates the fact that the behavior of the soil at stresses above the point of incipient failure is different from that at lower stresses.

Consolidation

The void ratio vs. pressure curves from the consolidation tests are drawn to an arithmetic scale in Figures 22 and 23. For the 300 psi soil C_c = .0019 and C_s = .0023. C_c and C_s were also determined for the 1000 psi compacted loess and found to be .00393 and .00181, respectively.

By the use of equation (6) the values of U_D for u_c of 1/2 of Pa and 0 were computed and are given in Tables 7 and 8, together with the measured porewater pressure at the point of incipient failure. The agreement between computed and measured values is within an allowable range.

V. CONCLUSIONS

1. The value of ϕe for this soil was found to be 34°.
2. Incipient failure takes place when the shear resistance is equal to $\bar{\sigma}_n$ tan ϕe.
3. Maximum positive porewater pressure occurs at the point of incipient failure. The pore pressure is seen to be positive at low strains, then decreases as failure progresses. Up to incipient failure the pore pressure agrees well with that predicated by the elastic theory. Subsequently the displacement of particles along the failure plane greatly affects the porewater pressure.
4. Equilibrium of a soil can not be maintained without large deformations after cohesion has been mobilized. Cohesion is initiated at incipient failure.

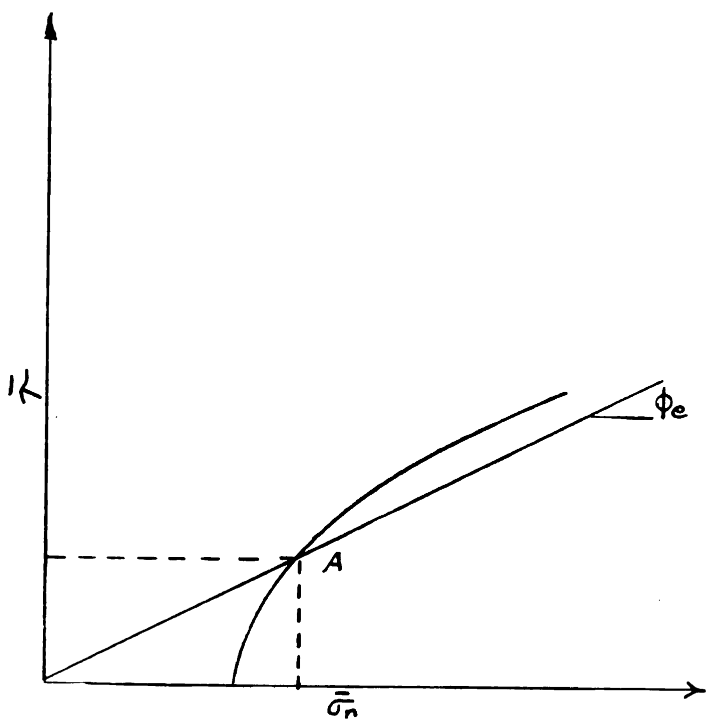
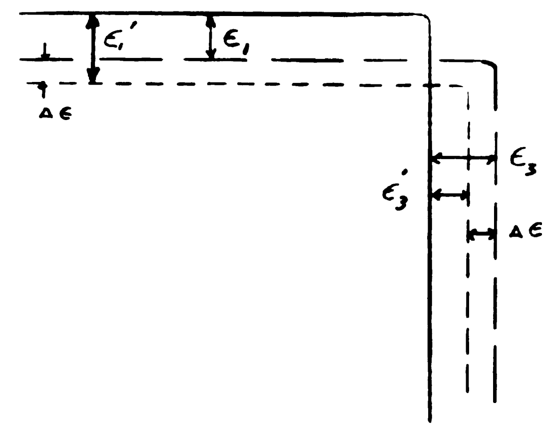


FIG. #1 $\bar{\tau}$ vs $\bar{\sigma}_n$ ON FAILURE PLANE



ϵ' = PRINCIPLE STRAIN
ϵ_1 = TOTAL STRAIN
$\Delta\epsilon$ = NORMAL STRAIN DUE TO SHEAR DEFORMATION

FIG # 2 ELEMENT UNDER STRAIN

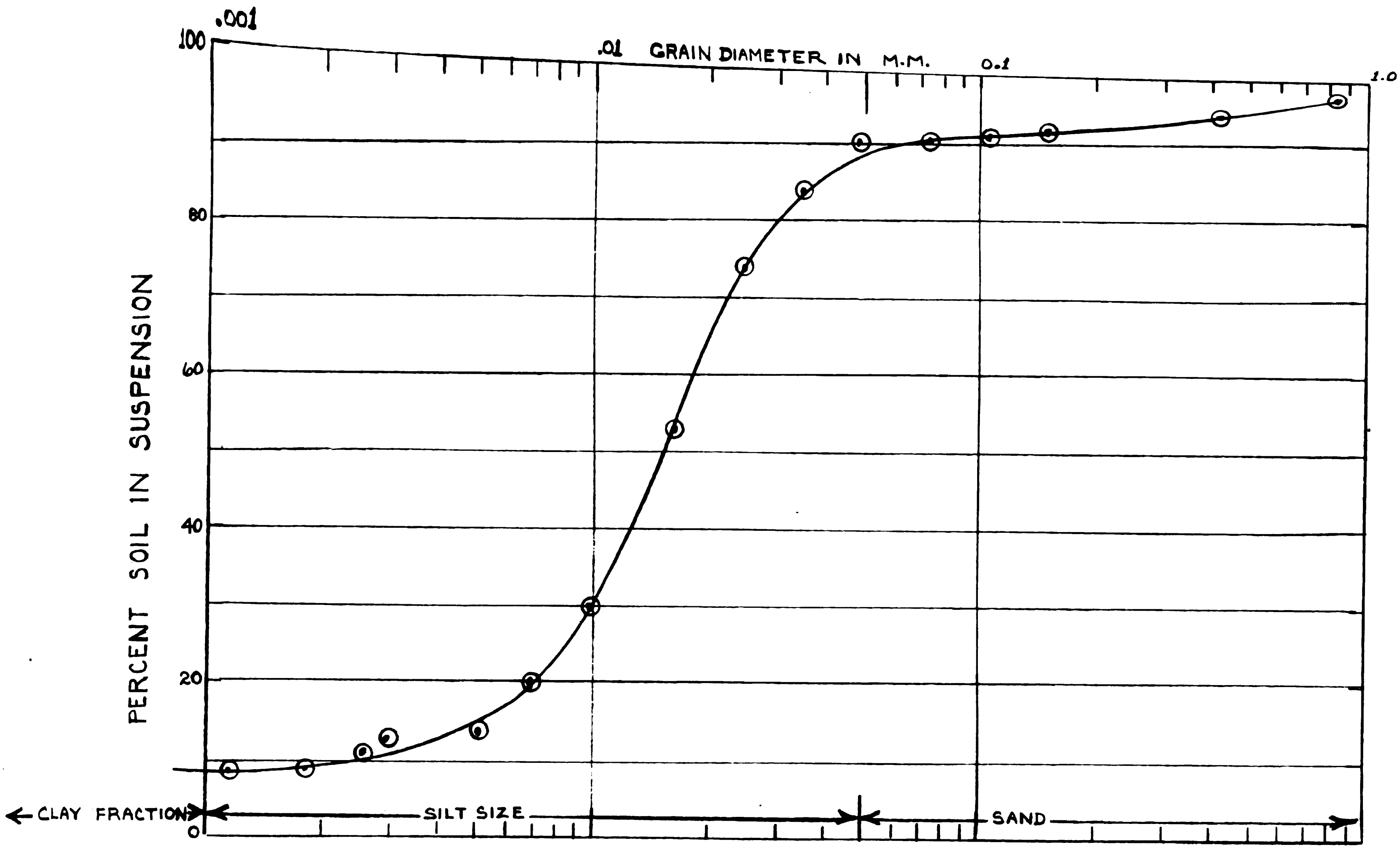


FIG.# 3 GRAIN SIZE CURVE
MISSISSIPPI LOESS

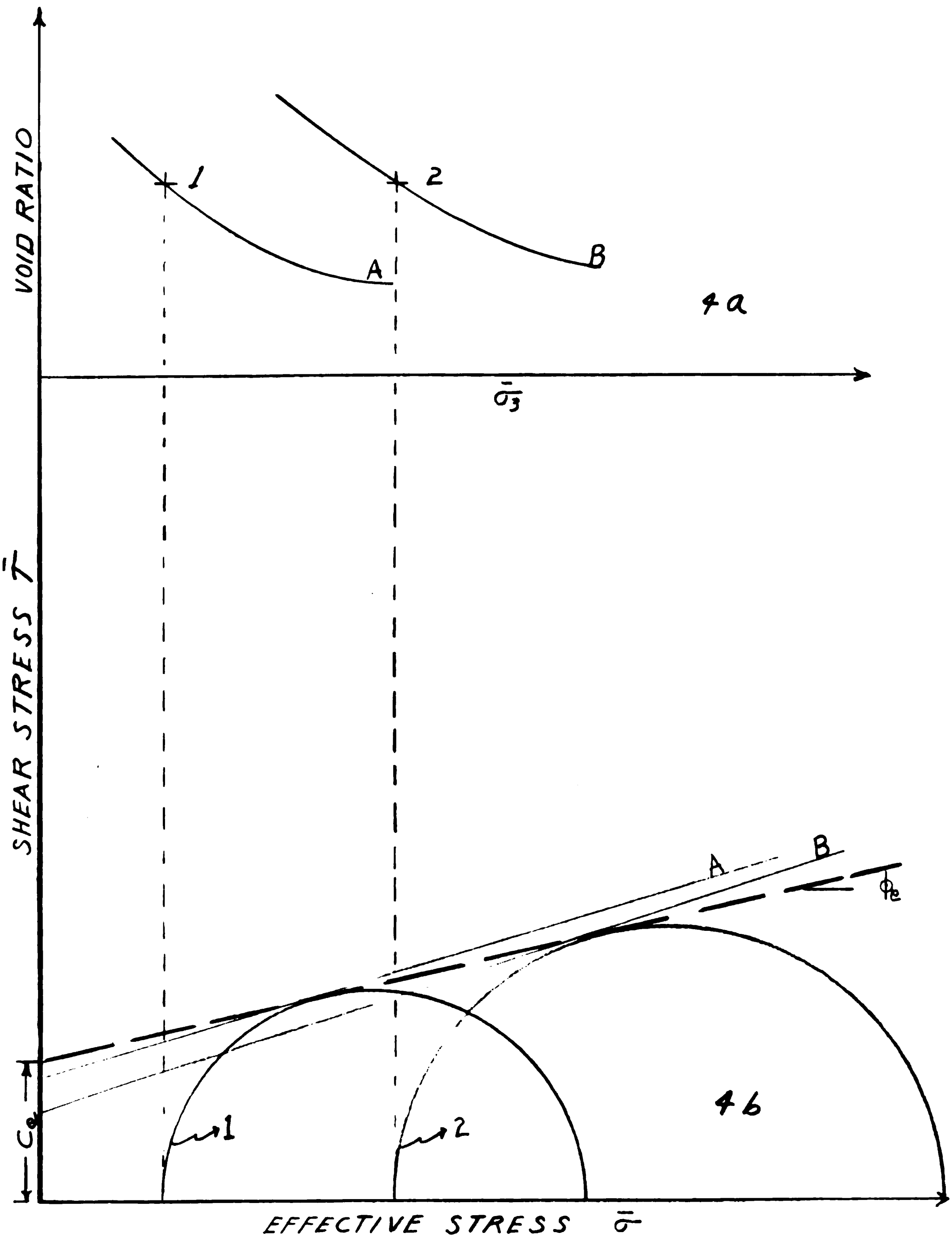


FIG. #4 GRAPHICAL SOLUTION FOR ϕ_e

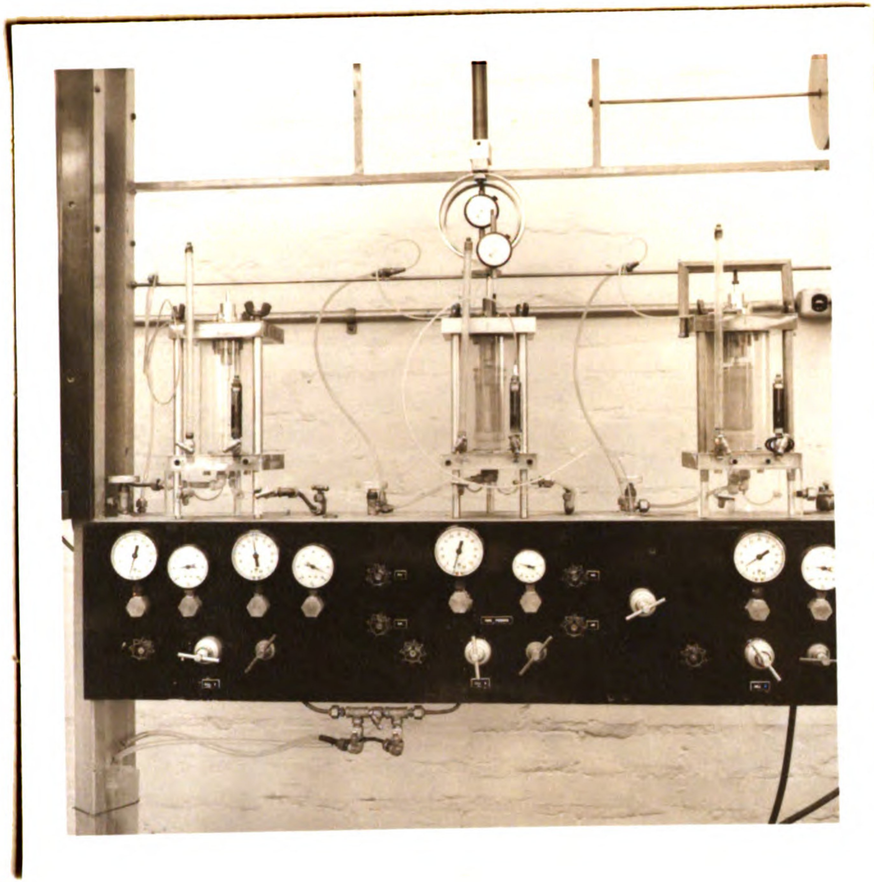


FIG.#5 TRIAXIAL UNIT FOR CU TEST

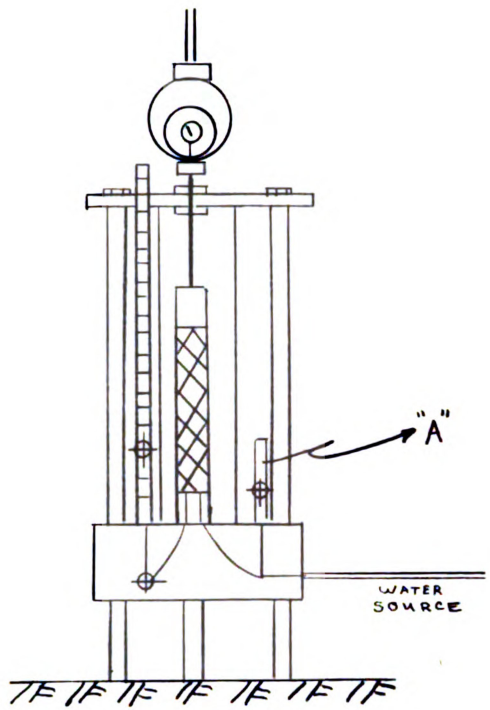
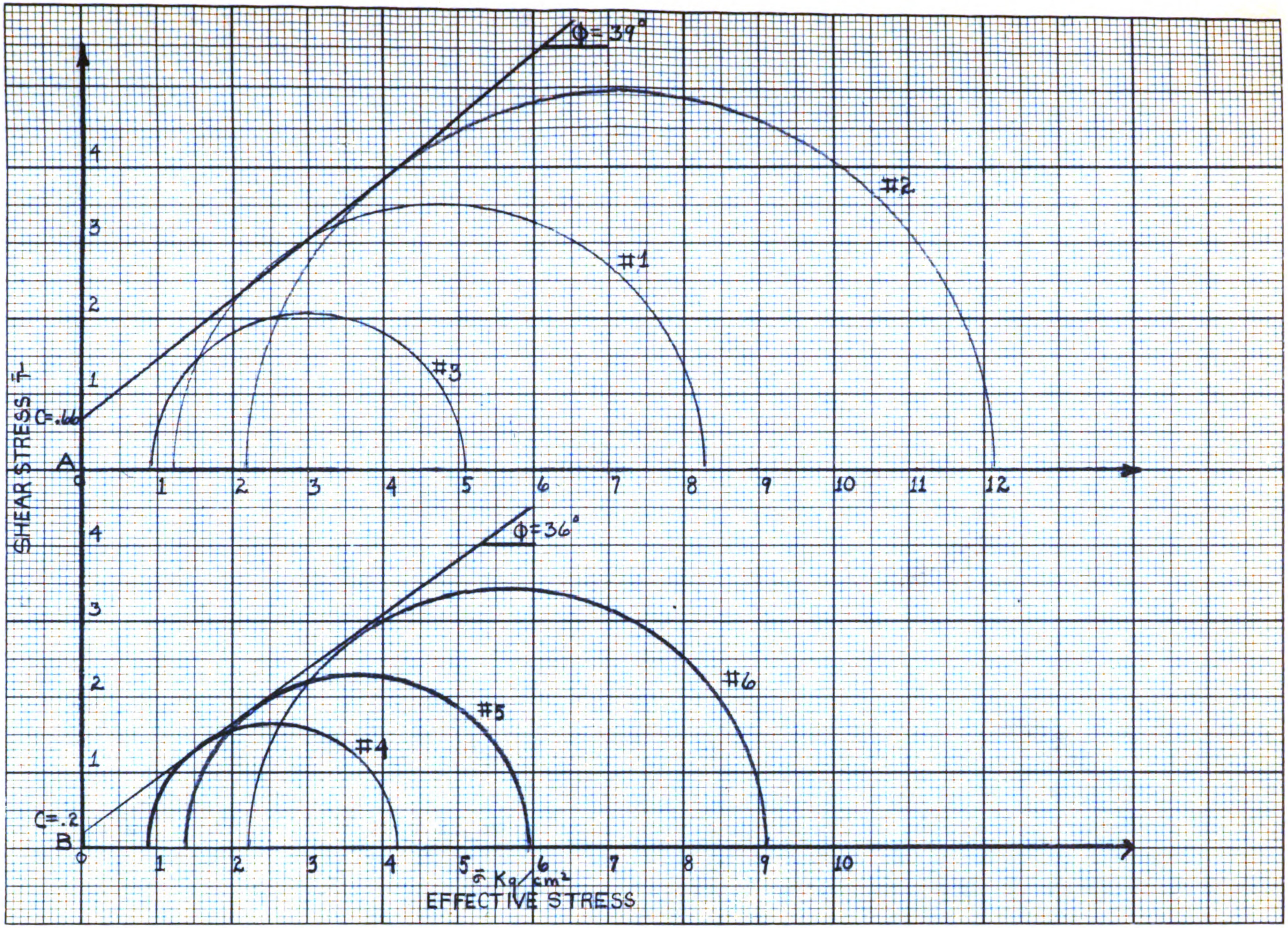


FIG.# 6 SCHEMATIC DIAGRAM TRIAXIAL CELL



FIGURES 7+8 MOHR'S ENVELOPES FOR MISS. LOESS - A = 2000 PSI, B = 300 PSI

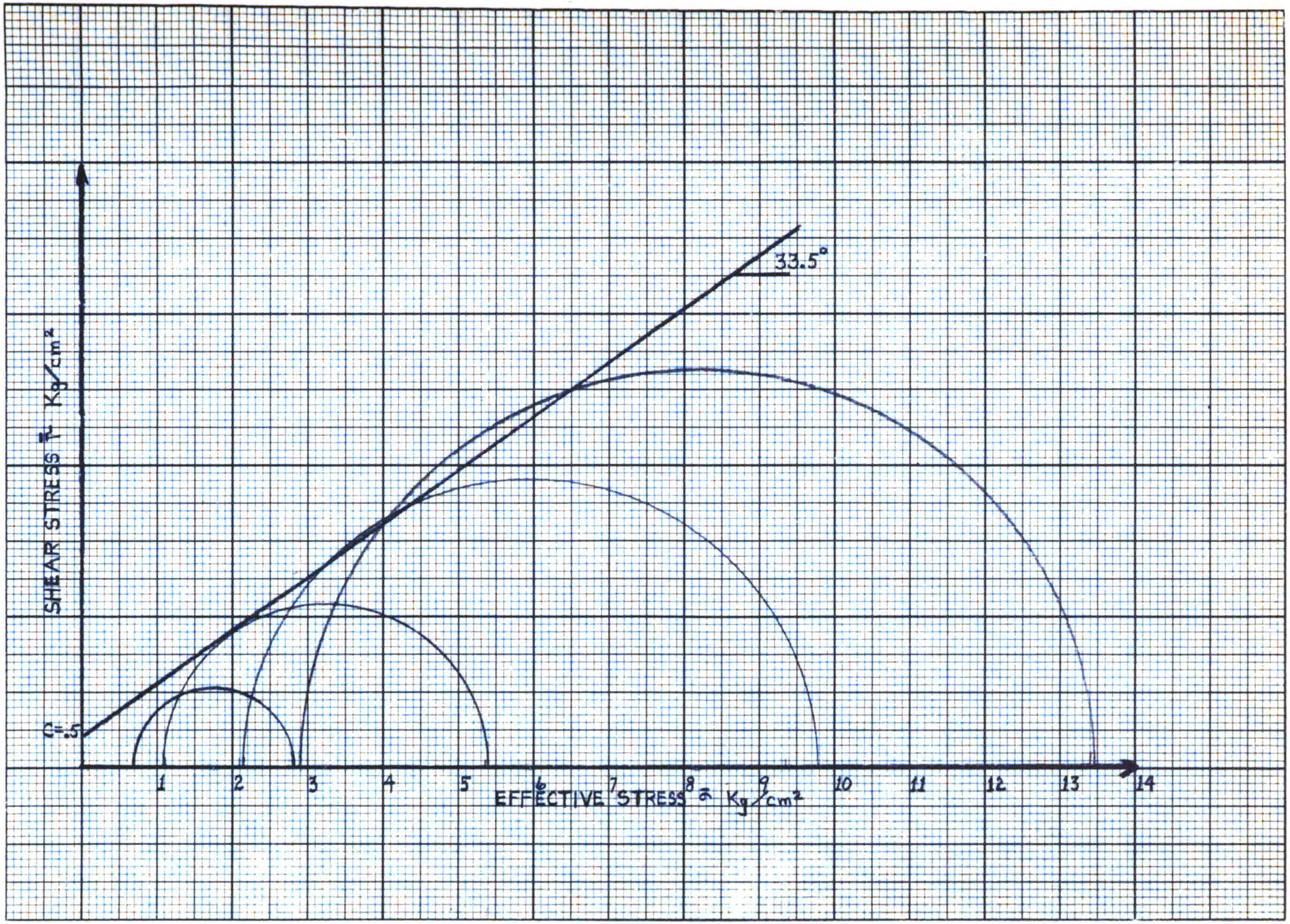


FIG. # 9 MOHR'S ENVELOPE FOR MISS. LOESS 1000 PSI

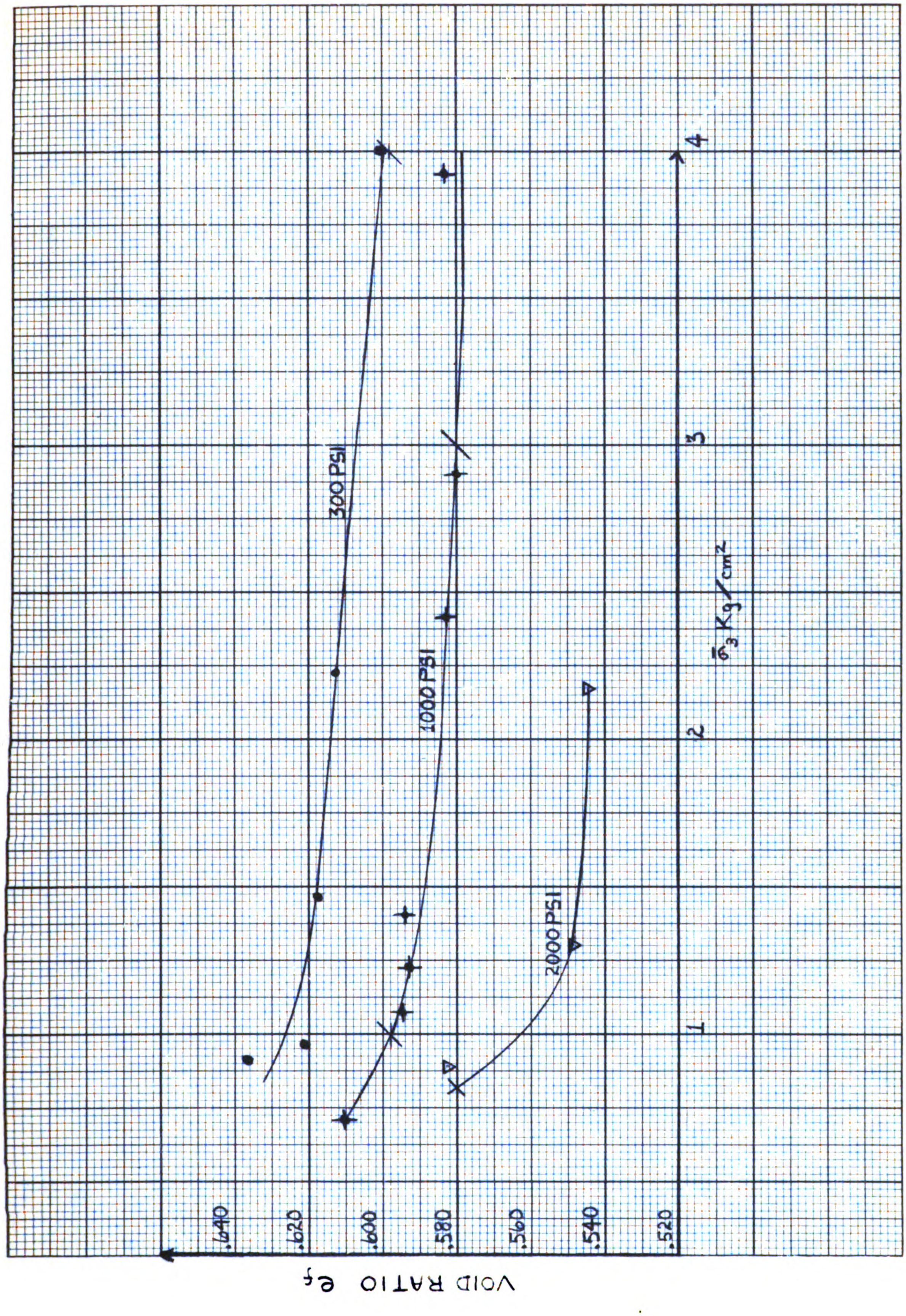


FIG.# 10 VOID RATIO e_f vs $\bar{\sigma}_{3f}$

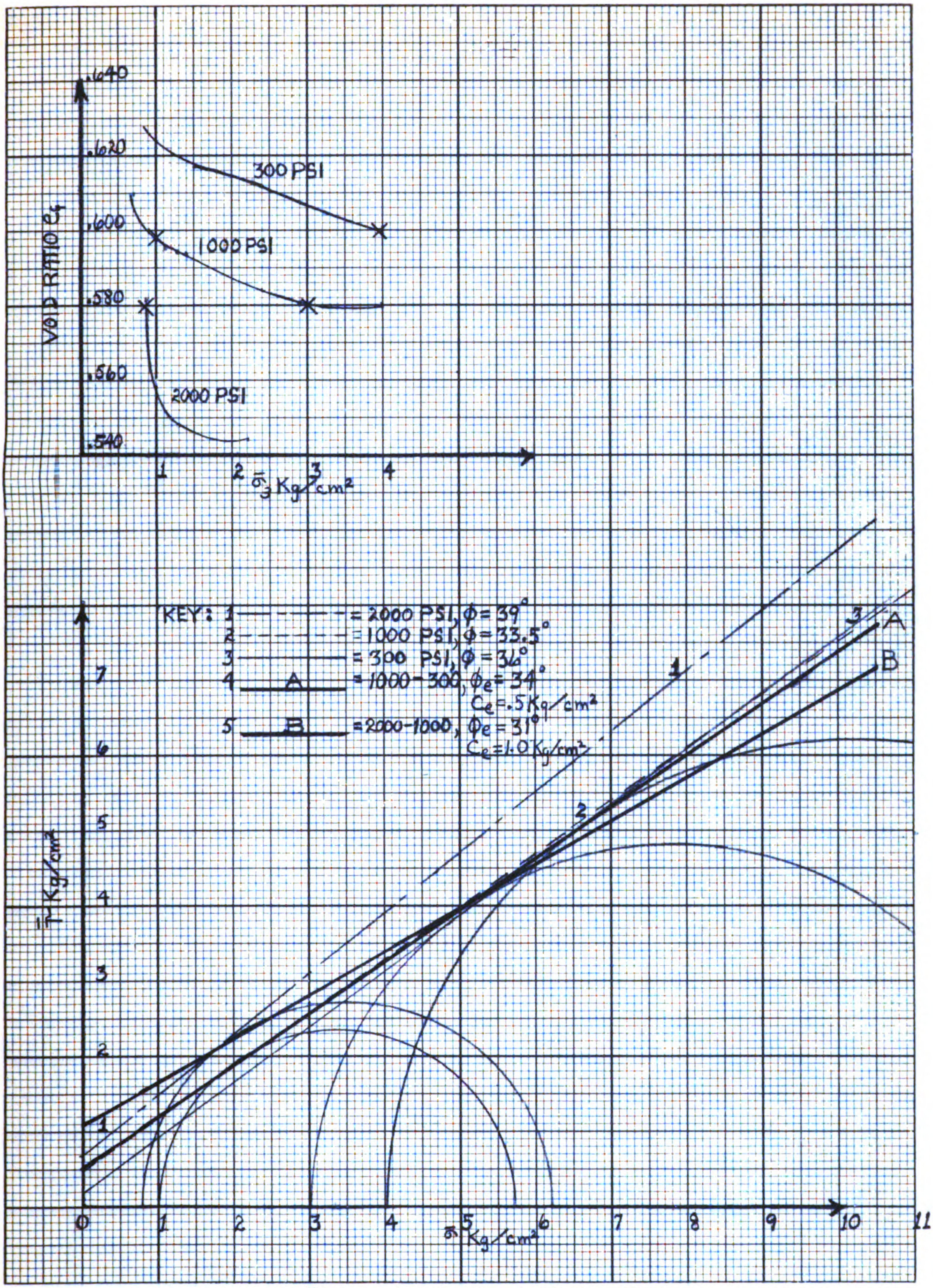


FIG.# 11 TRUE PHI DETERMINATION

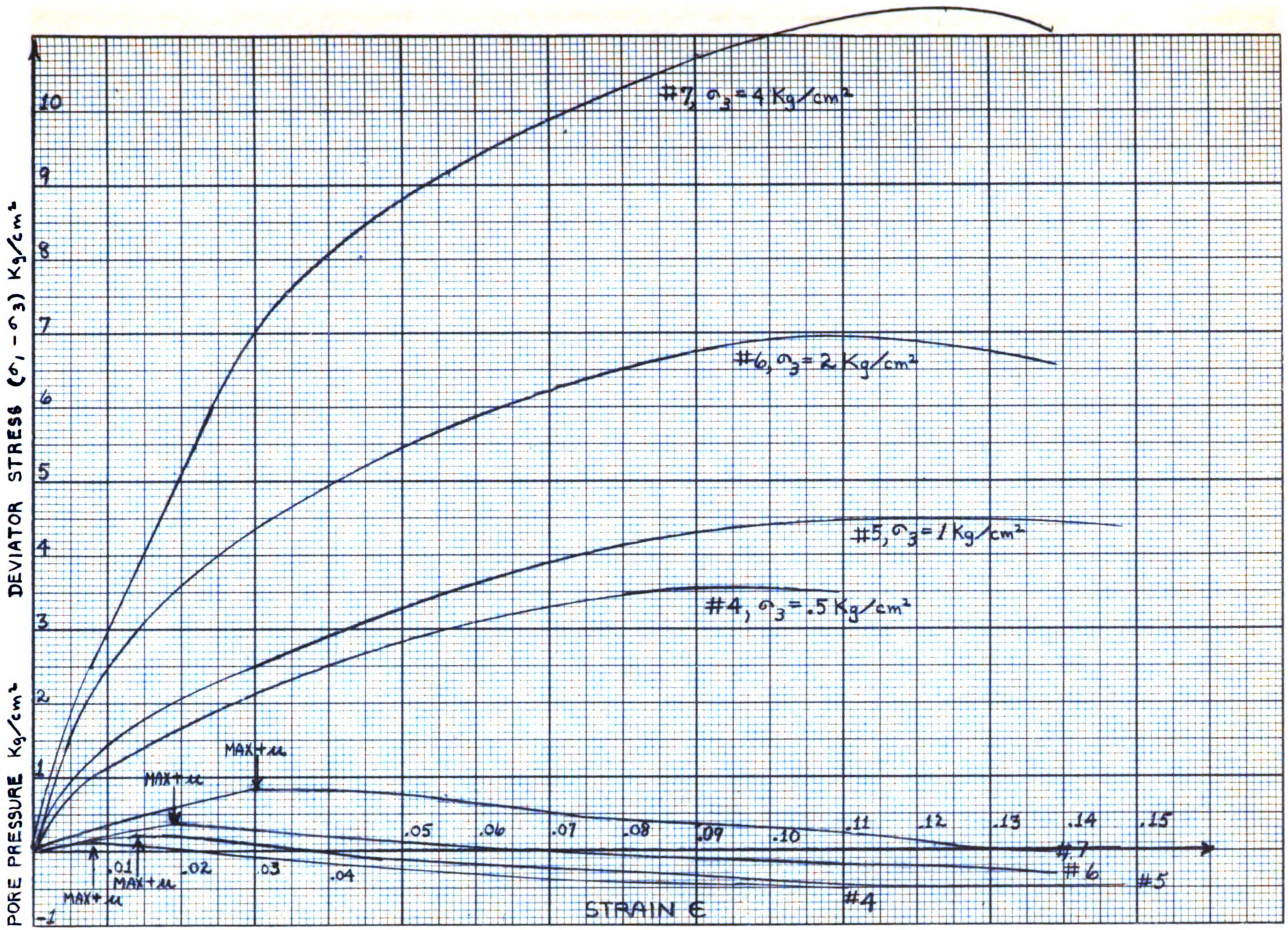


FIG. # 12 STRESS-STRAIN AND PORE PRESSURE-STRAIN MISS. LOESS 300 PSI

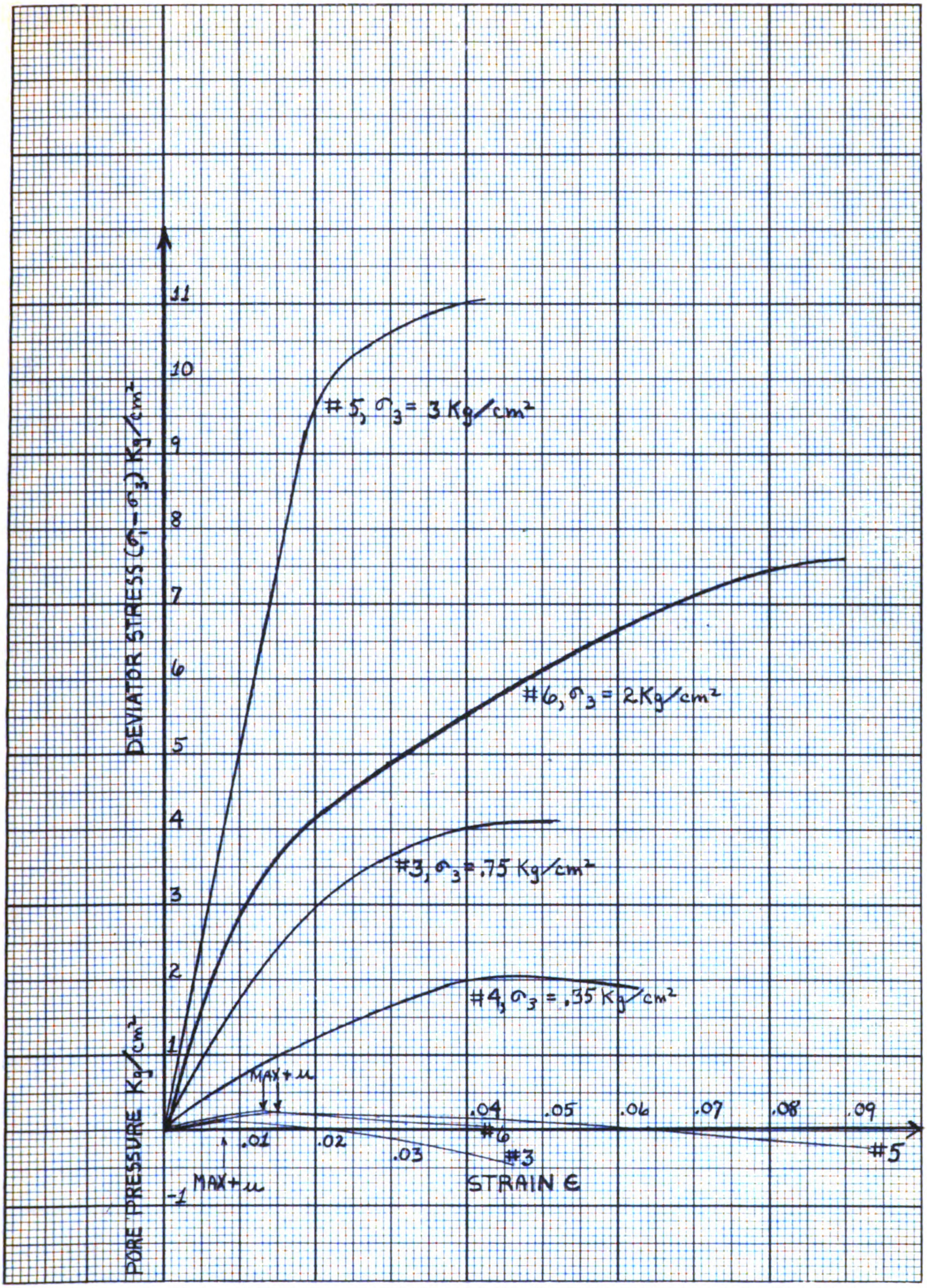


FIG. # 13 STRESS-STRAIN AND PORE PRESSURE-STRAIN
MISS. LOESS 1000 PSI

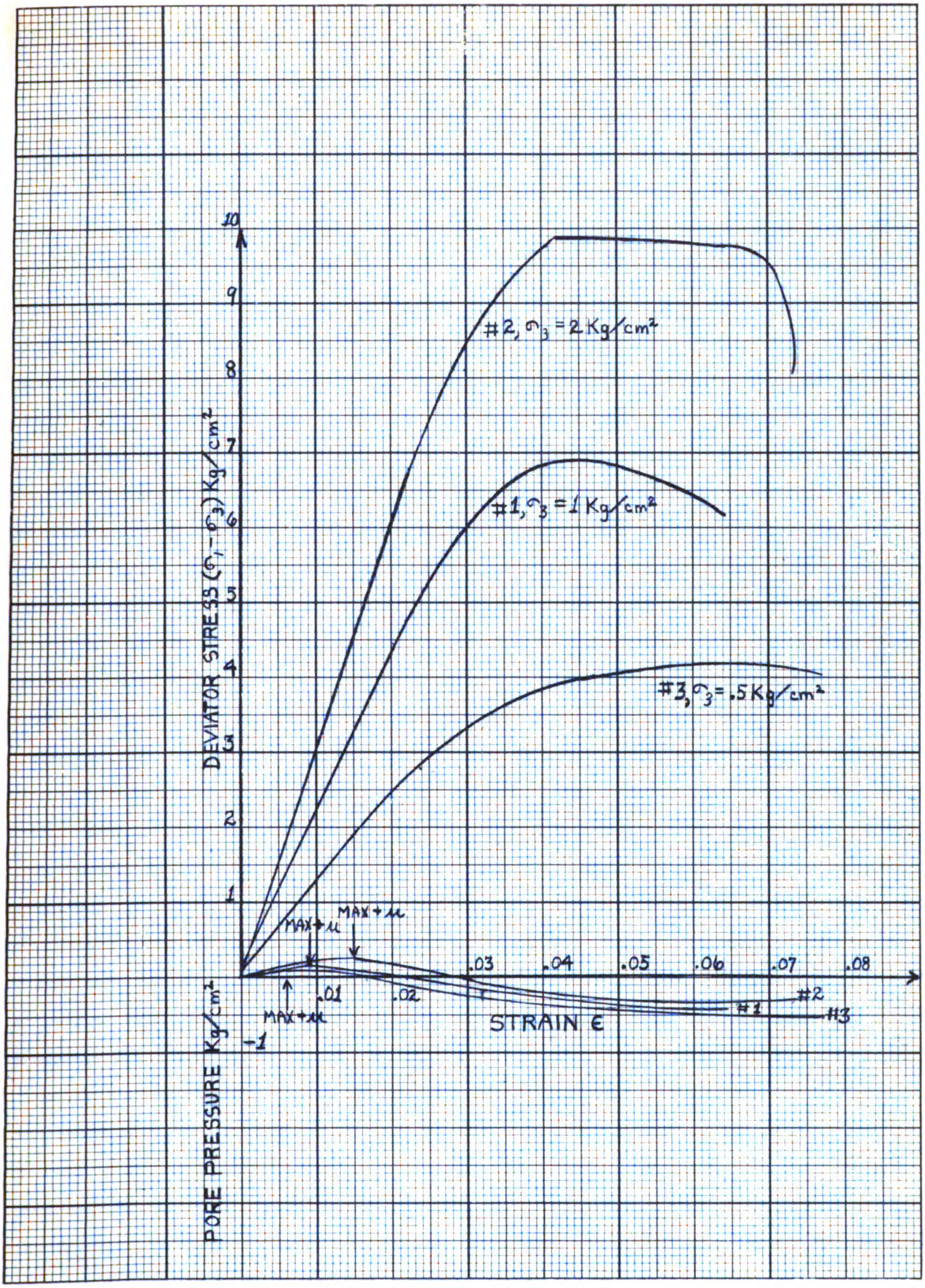


FIG.# 14 STRESS-STRAIN AND PORE PRESSURE-STRAIN
MISS. LOESS 2000 PSI

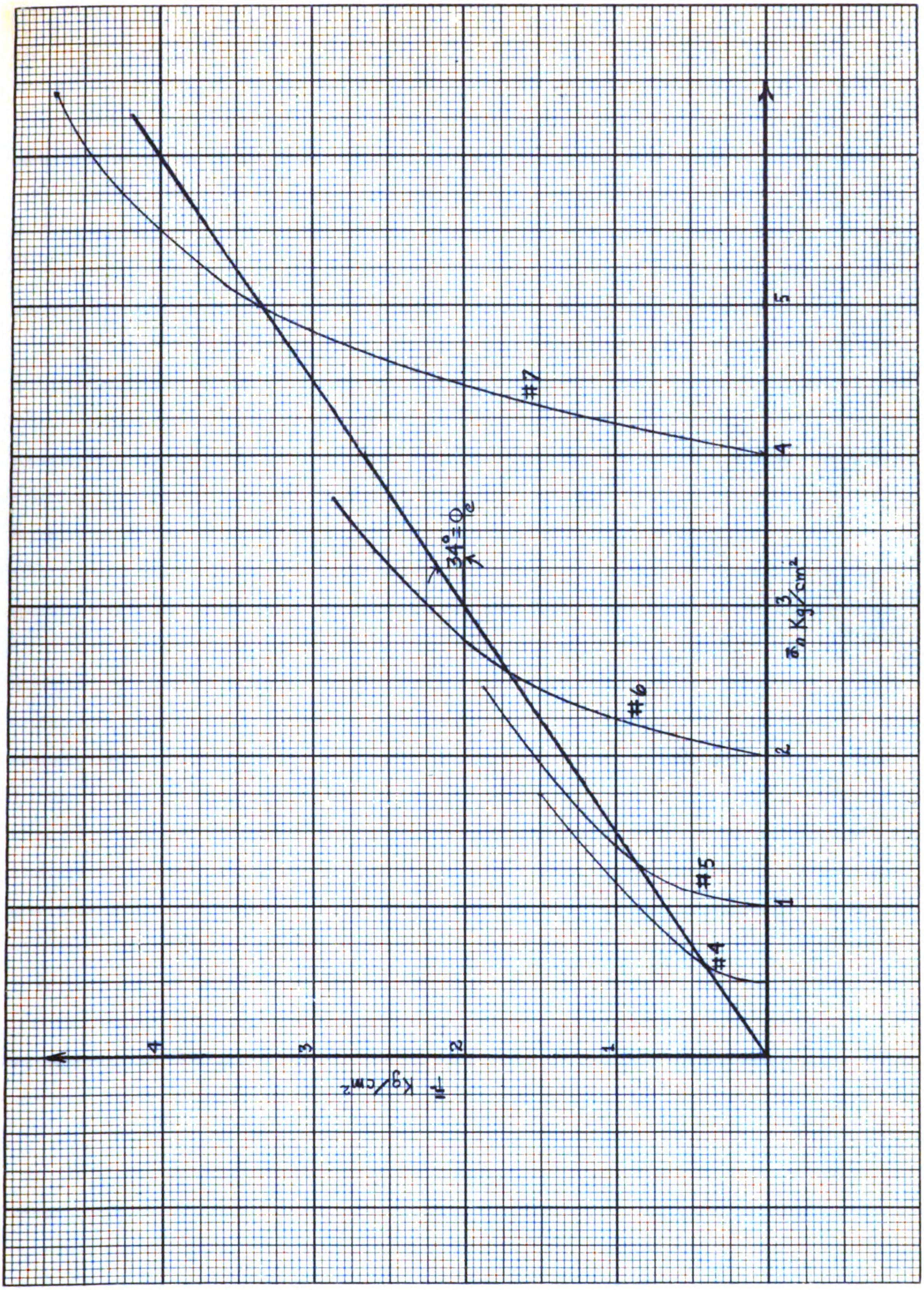


FIG. 15 $\bar{\tau}$ vs $\bar{\sigma}_n$ MISS. LOESS 300 PSI

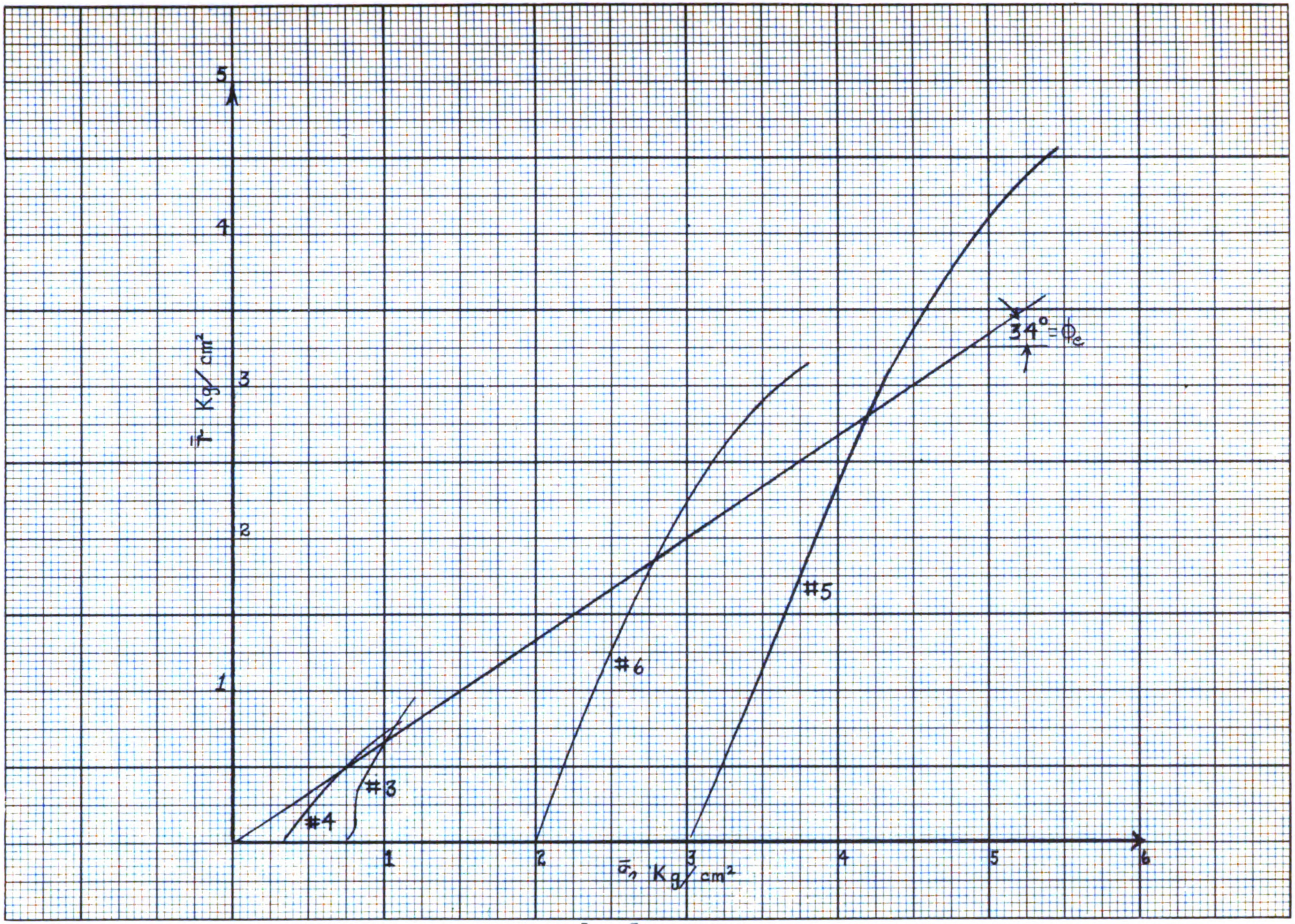


FIG.# 16 $\bar{\tau}$ vs $\bar{\sigma}_n$ MISS. LOESS 1000 PSI

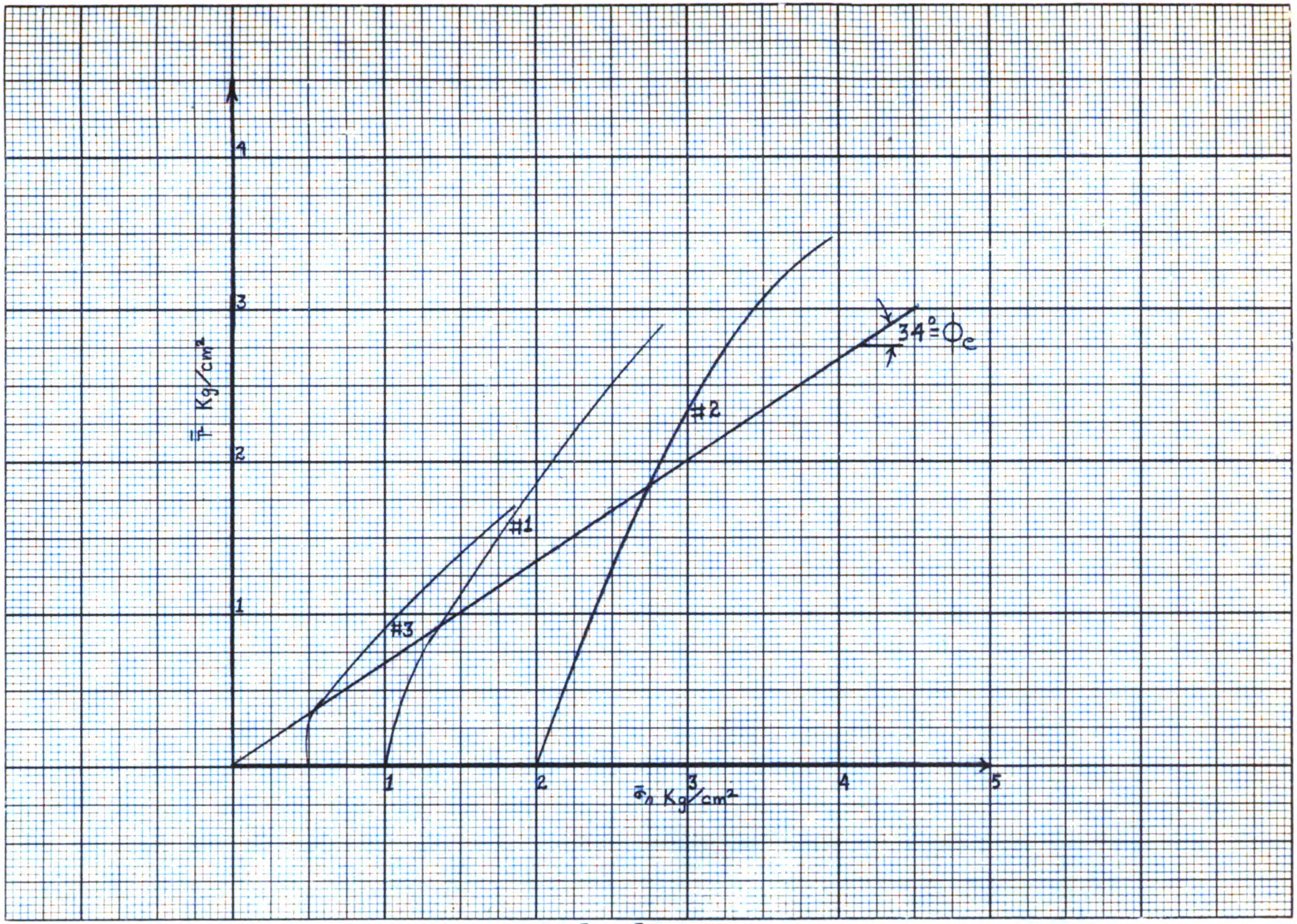
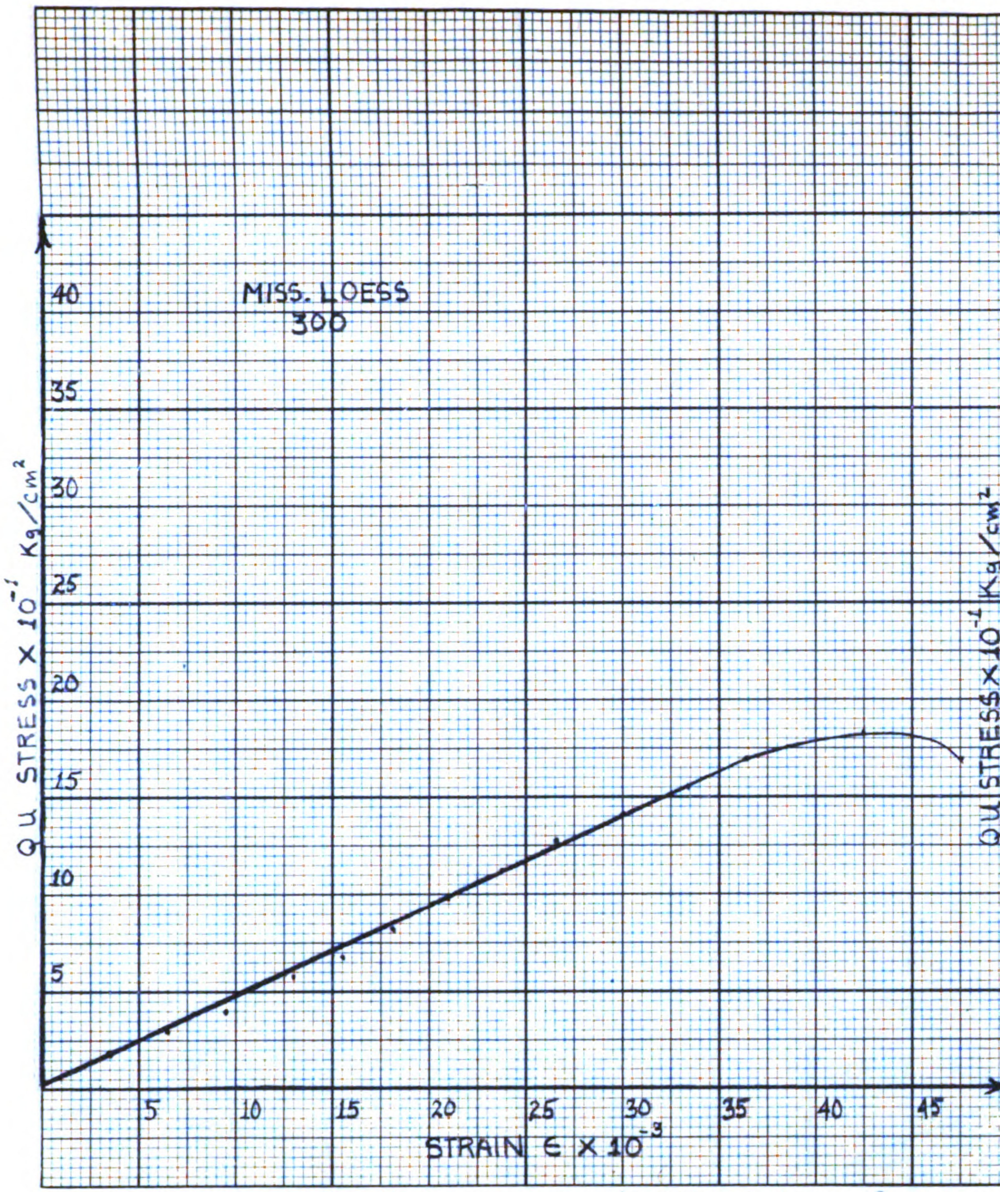


FIG. #17 $\bar{\tau}$ vs $\bar{\sigma}_n$, MISS. LOESS 2000 PSI



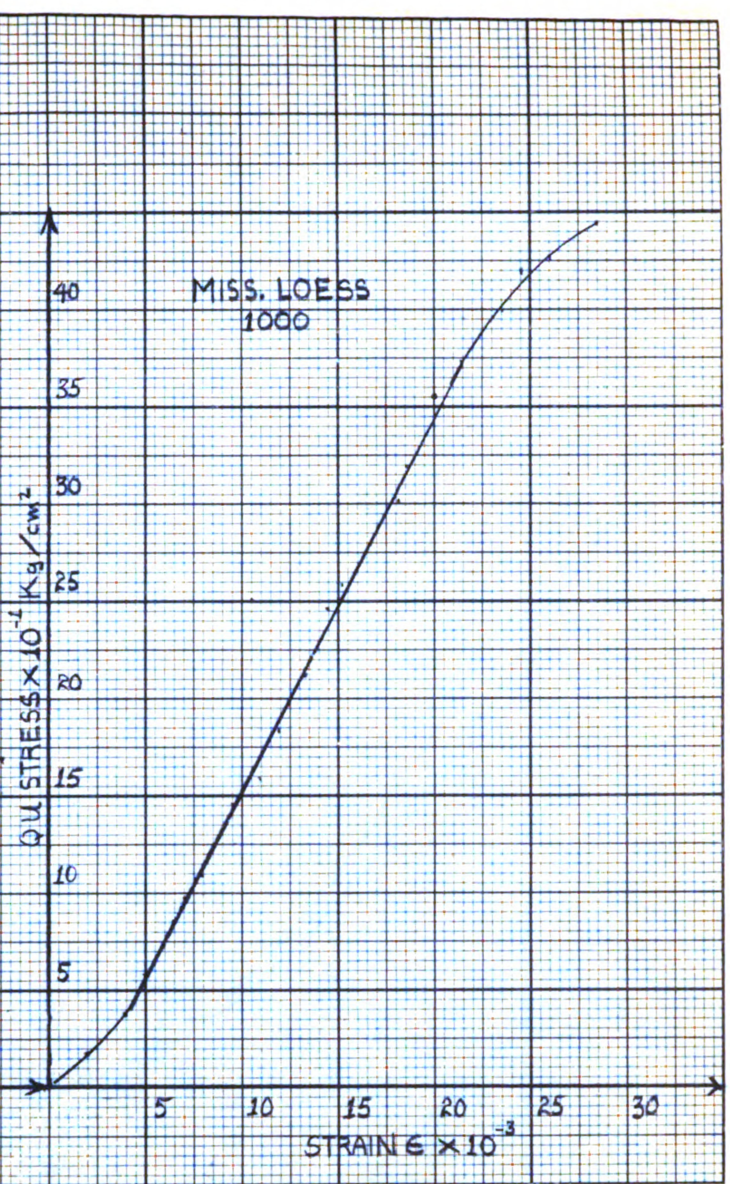


FIG. # 18 UNCONFINED COMPRESSION TEST FIG. # 19

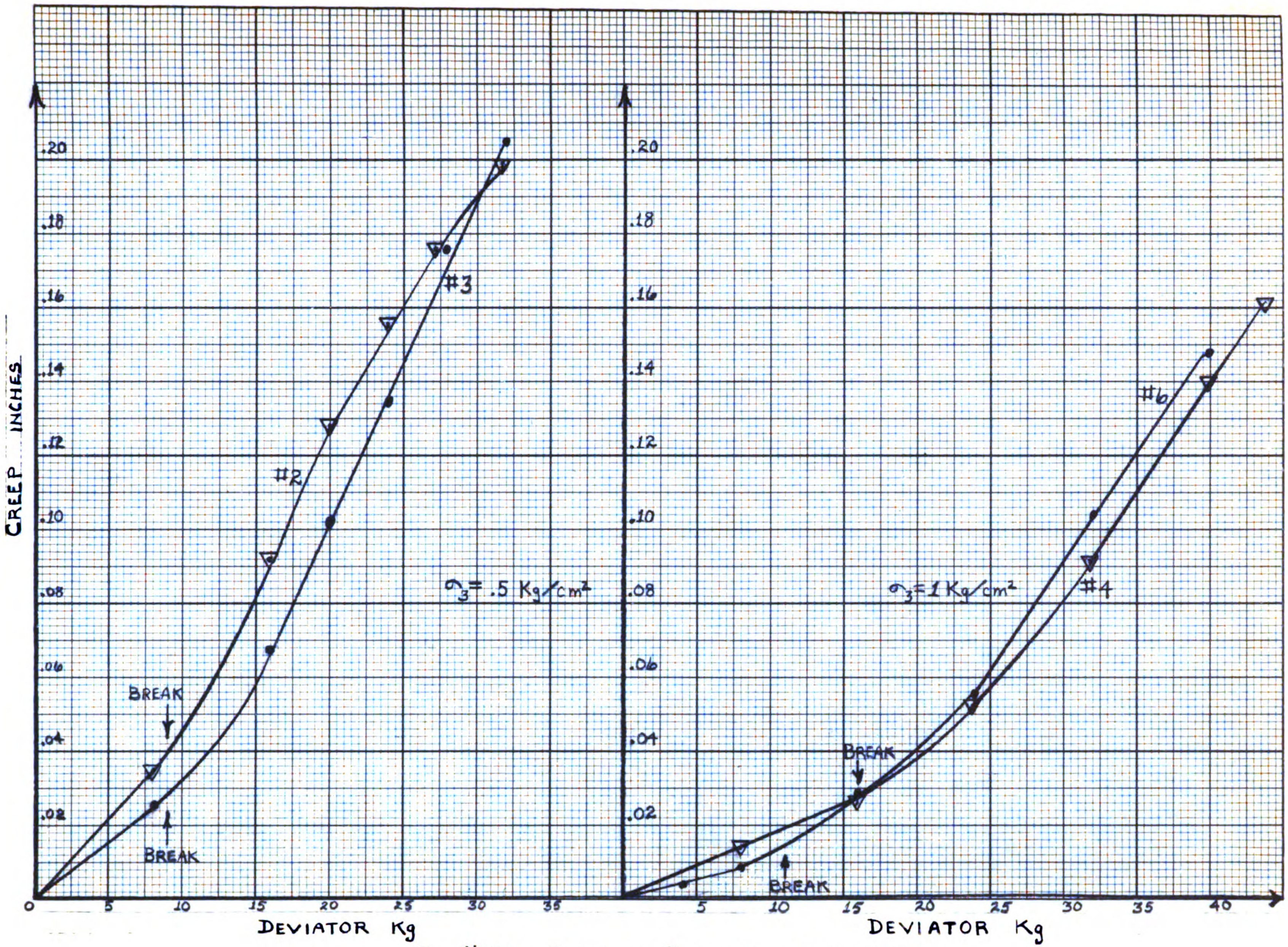


FIG. #20 CREEP CURVE 300 PSI

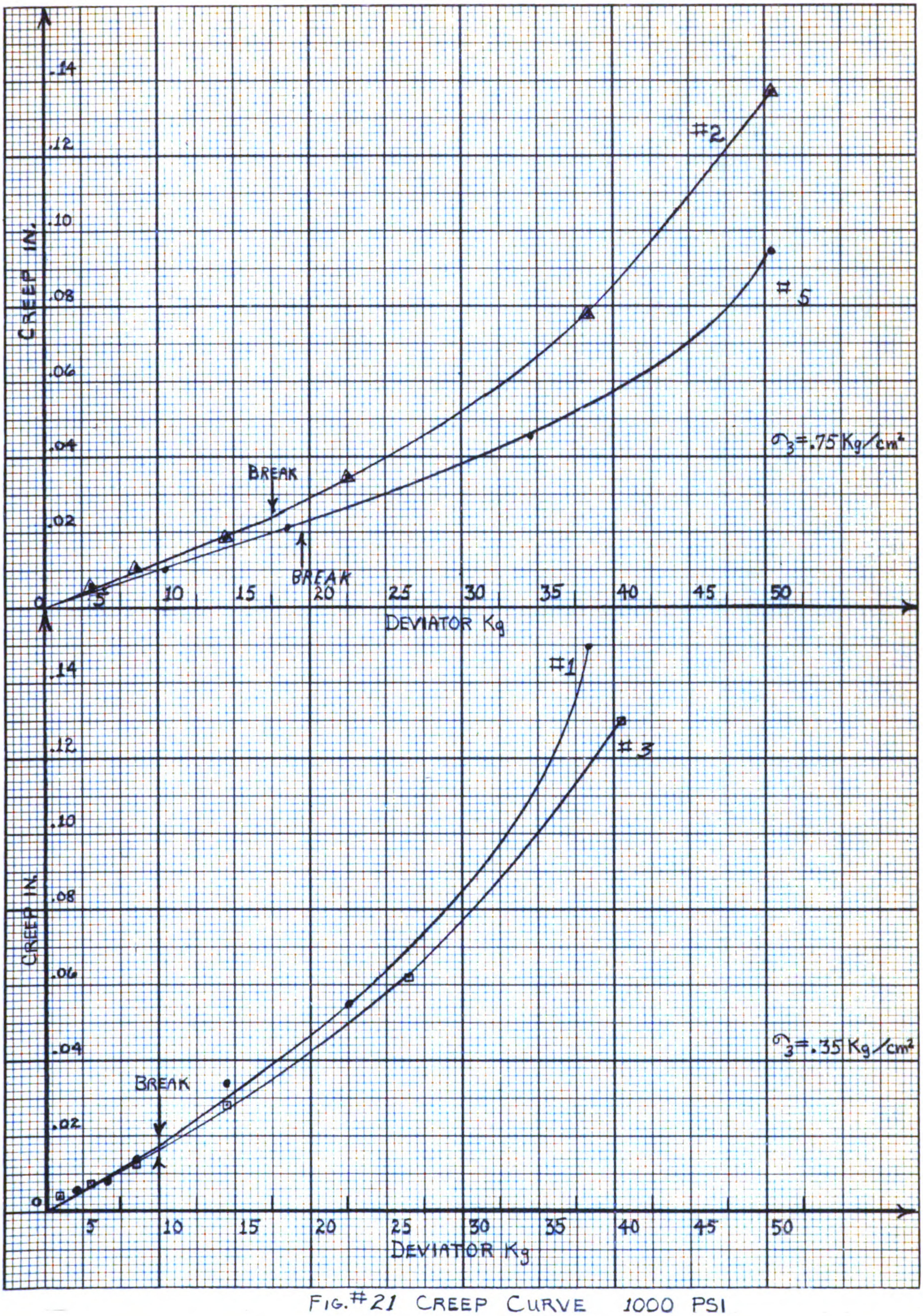


FIG. #21 CREEP CURVE 1000 PSI

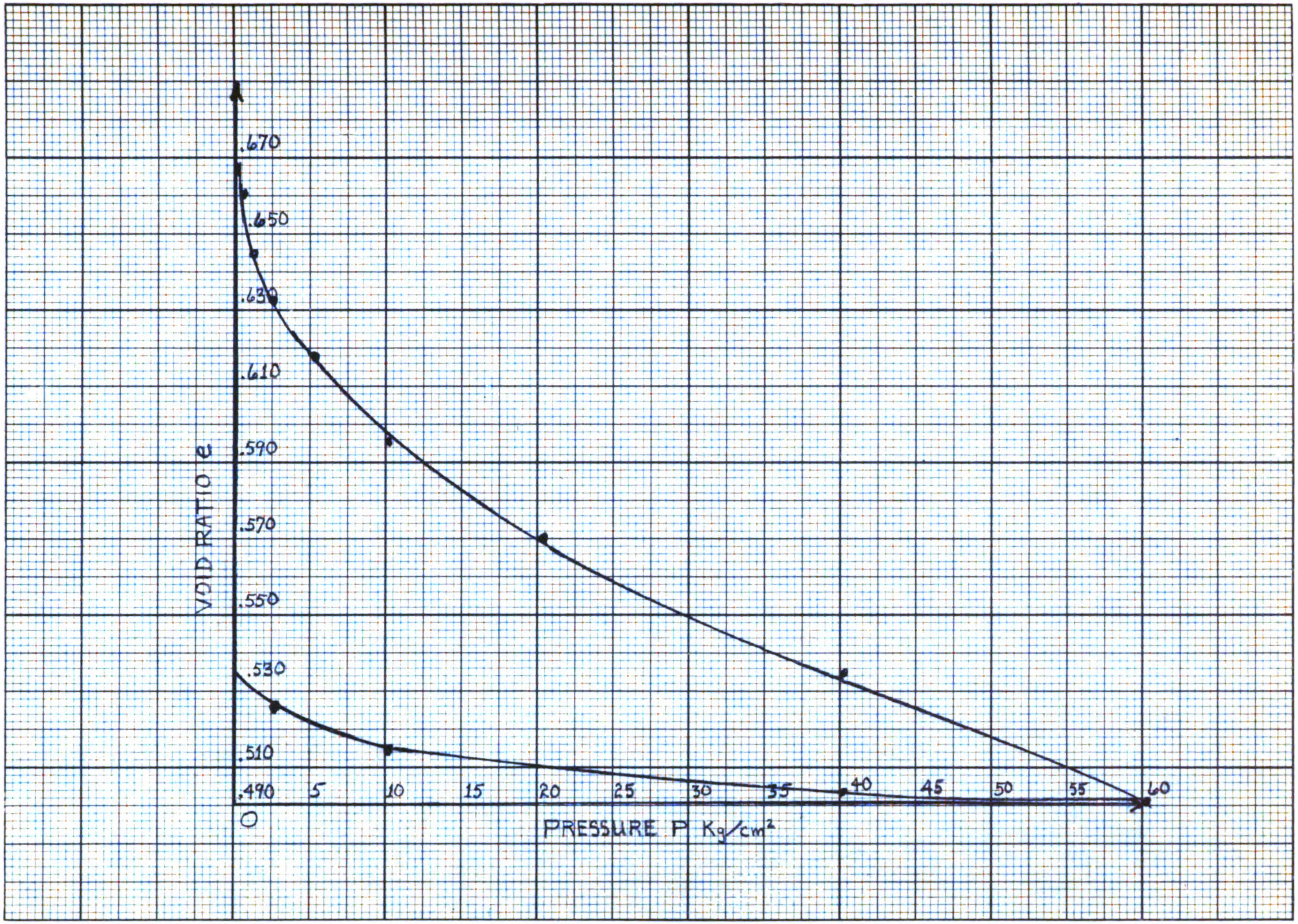
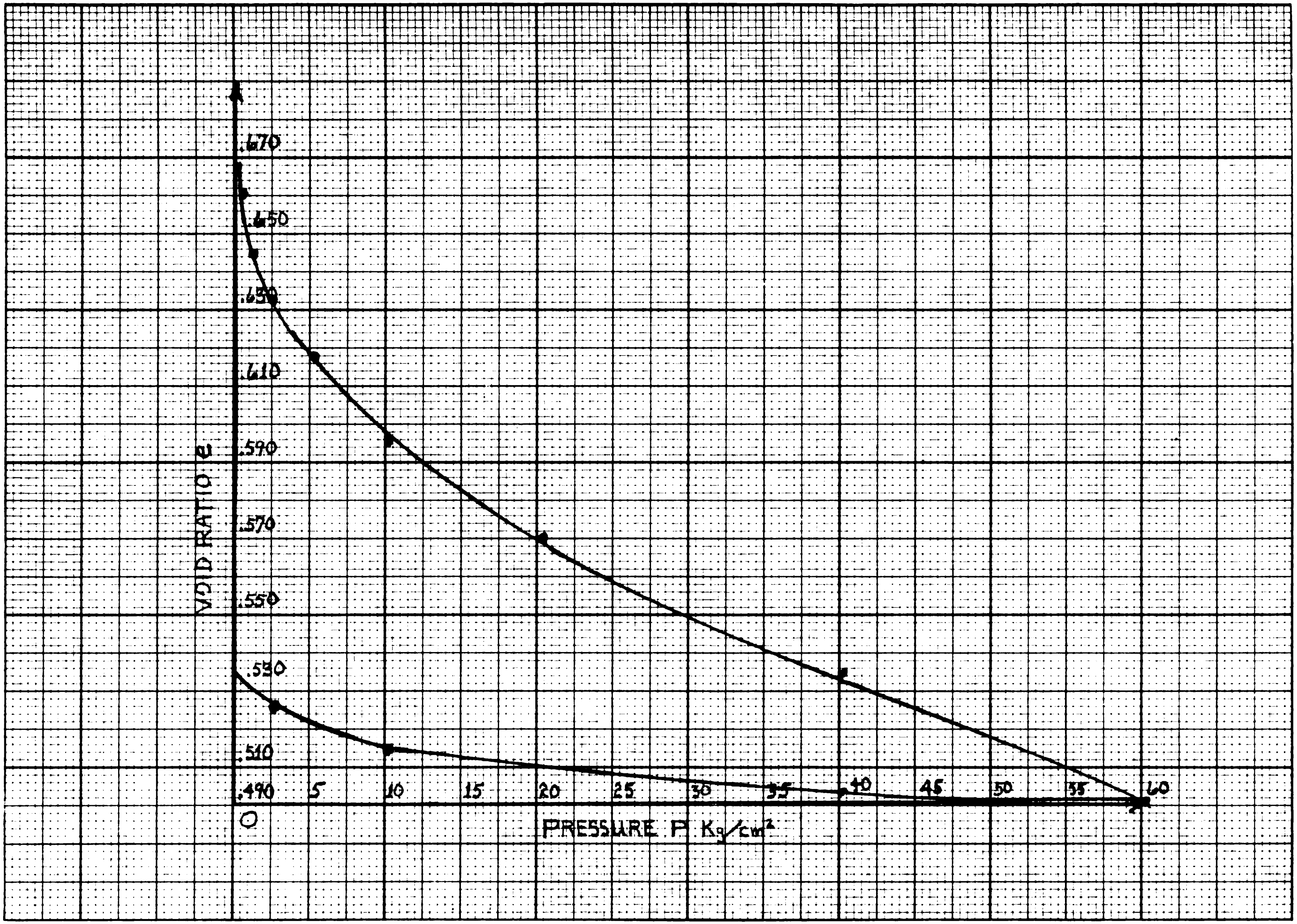


FIG* 22 CONSOLIDATION CURVE, MISS. LOESS 300 PSI



FIG# 22 CONSOLIDATION CURVE, MISS. LOESS 300 PSI

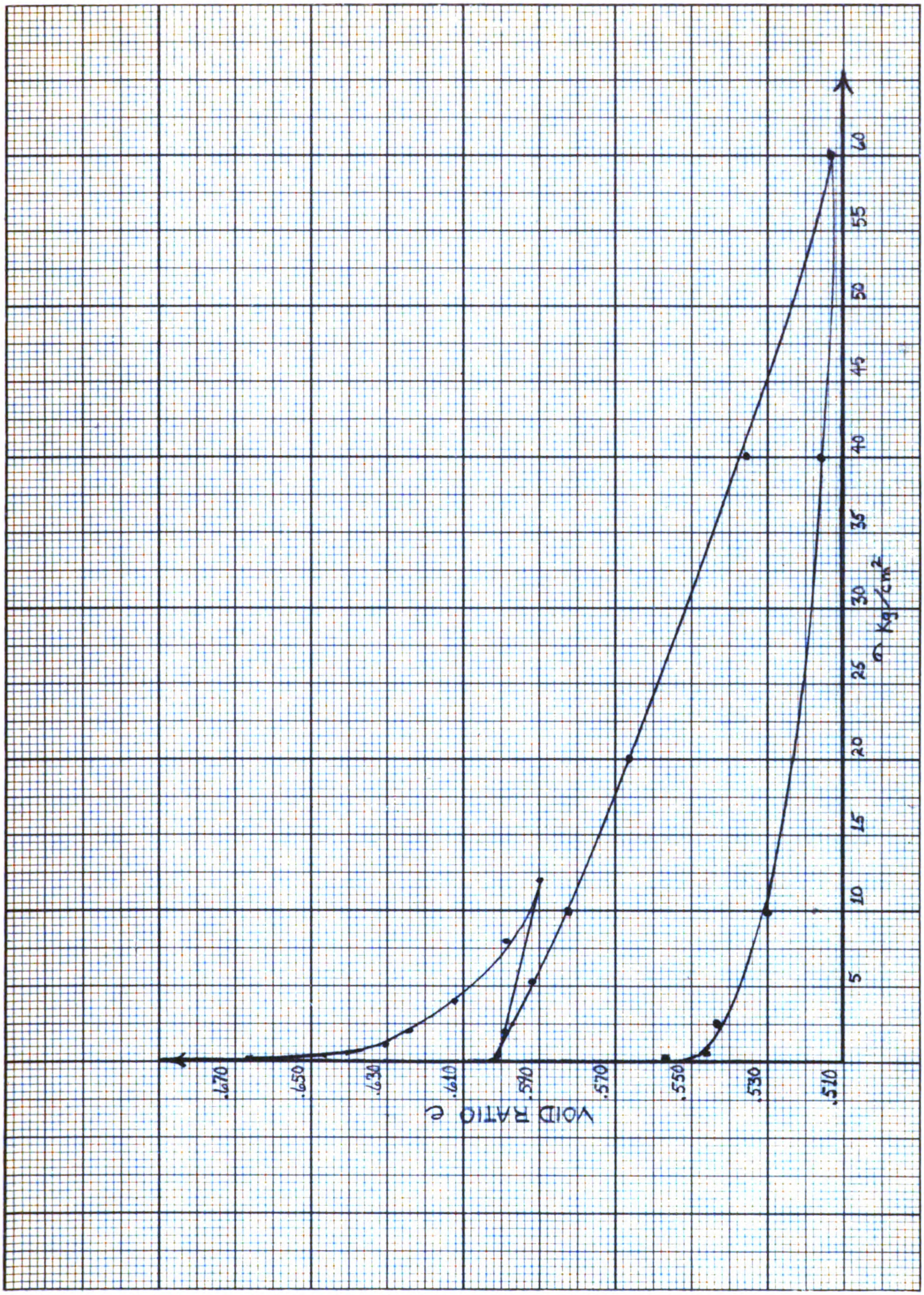


FIG. #23 CONSOLIDATION TEST MISS. LOESS 1000 PSI

SOIL CHARACTERISTIC	VALUE
PLASTIC LIMIT	23%
LIQUID LIMIT	29%
PLASTICITY INDEX	6%
ACTIVITY	.66%
SPECIFIC GRAVITY	2.72
D_{60}	.0180 mm
D_{10}	.0025 mm
$\frac{D_{60}}{D_{10}}$	7.2

TABLE #1 SOIL INDEX PROPERTIES

CONSOLIDATED UNDRAINED TEST Miss. LOESS 300 PSI	SPECIMEN #4	SPECIMEN #5	SPECIMEN #6	SPECIMEN #7
σ_3, Kg/cm²	.5	1	2	4
INITIAL e	———	———	———	.62
FINAL e	.621	.618	.613	.573
INITIAL W %	23.2	22.4	22.2	22.1
FINAL W %	22	21.7	21.2	21
FINAL SAT. %	96.7	96	96	98.5
γ_d, g/cc	1.68	1.68	1.68	1.72
γ_w, g/cc	2.05	2.05	2.05	2.07
MAX $(\sigma_1 - \sigma_3)$, Kg/cm²	3.61	4.53	6.91	11.38
$\bar{\sigma}_1$ AT FAILURE, Kg/cm²	4.56	5.98	9.12	15.31
$\bar{\sigma}_3$ AT FAILURE, Kg/cm²	.948	1.45	2.21	3.93
DAYS STORAGE	19	14	13	13
ϵ AT MAX $(\sigma_1 - \sigma_3)$	.100	.128	.119	.125
ϵ AT MAX $+u$	.0071	.0142	.0189	.0304
ϵ AT $u = 0$	.023	.037	.071	.130
MAX $+u$, Kg/cm²	.105	.21	.343	.84
MAX $-u$, Kg/cm²	.446	.448	.206	.052
D AT MAX $+u$, Kg	10	17.4	37	73
CALCULATED: $\bar{\tau}$ AT MAX $+u$, Kg/cm²	.414	.705	1.49	2.83
CALCULATED: $\bar{\sigma}_n$ AT MAX $+u$, Kg/cm²	.61	1.16	2.45	4.77
$\bar{\tau}$ AT ϕ_c LINE, Kg/cm²	.41	.84	1.76	3.35
$\bar{\sigma}_n$ AT ϕ_c LINE, Kg/cm²	.61	1.26	2.62	5.0

TABLE #2. MISS. LOESS 300 PSI C U TEST

CONSOLIDATED UNDRAINED TEST MISS. LOESS 1000 PSI	SPECIMEN #4	SPECIMEN #3	SPECIMEN #6	SPECIMEN #5
σ_3, Kg/cm²	.35	.75	2	3
INITIAL e	.600	.590	—	.589
FINAL e	.611	.595	.575	.580
INITIAL W %	20	20.3	20	20.2
FINAL W %	22.3	21.5	20.6	20.3
FINAL SAT. %	99	98.5	98	97
ρ_d, g/cc	1.68	1.71	1.72	1.71
ρ_w, g/cc	2.06	2.07	2.08	2.07
MAX $(\sigma_1-\sigma_3)$, Kg/cm²	2.09	4.18	7.61	11.1
$\bar{\sigma}_1$ AT FAILURE, Kg/cm²	2.71	4.86	9.74	13.19
$\bar{\sigma}_3$ AT FAILURE, Kg/cm²	.71	1.04	2.13	2.89
DAYS STORAGE	12	12	18	15
ϵ AT MAX $(\sigma_1-\sigma_3)$	.05	.046	.092	.0413
ϵ AT MAX + u	.0063	.0075	.0157	.0132
ϵ AT $u=0$	.007	.017	.063	—
MAX + u, Kg/cm²	.07	.14	.245	.28
MAX − u, Kg/cm²	.42	.412	.224	—
D AT MAX + u, Kg	4	12	38	70
CALCULATED: $\bar{\tau}$ AT MAX + u, Kg/cm²	.3	.535	1.54	2.87
CALCULATED: $\bar{\sigma}_n$ AT MAX + u, Kg/cm²	.55	.90	2.56	4.24
$\bar{\tau}$ AT ϕ_e LINE, Kg/cm²	.4	.655	1.90	2.87
$\bar{\sigma}_n$ AT ϕ_e LINE, Kg/cm²	.65	.97	2.82	4.24

TABLE #3 MISS. LOESS 1000 PSI CU TEST

CONSOLIDATED UNDRAINED TEST MISS. LOESS 2000 PSI	SPECIMEN #3	SPECIMEN #1	SPECIMEN #2
σ_3, Kg/cm²	.5	1	2
INITIAL e	—	—	—
FINAL e	.582	.549	.545
INITIAL W %	19.2	18.3	18.4
FINAL W %	20.1	19.2	19.2
FINAL SAT. %	98	94.4	94.5
γ_d, g/cc	1.73	1.75	1.77
γ_w, g/cc	2.09	2.09	2.10
MAX $(\sigma_1 - \sigma_3)$, Kg/cm²	4.15	6.99	9.9
$\bar{\sigma}_1$ AT FAILURE, Kg/cm²	5.08	8.28	12.06
$\bar{\sigma}_3$ AT FAILURE, Kg/cm²	.931	1.29	2.16
DAYS STORAGE	19	14	15
ϵ AT MAX $(\sigma_1 - \sigma_3)$	.06	.0445	.0406
ϵ AT MAX $+u$	.006	.009	.014
ϵ AT $u = 0$	.017	.021	.029
MAX $+u$, Kg/cm²	.119	.161	.245
MAX $-u$, Kg/cm²	.504	.364	.154
D AT MAX $+u$, Kg	8.5	21.5	44
$\bar{\tau}$ AT MAX $+u$, Kg/cm²	.35	.87	1.98
$\bar{\sigma}_n$ AT MAX $+u$, Kg/cm²	.56	1.29	2.81
$\bar{\tau}$ AT ϕ_e LINE, Kg/cm²	.35	.87	1.86
$\bar{\sigma}_n$ AT ϕ_e LINE, Kg/cm²	.56	1.29	2.75

TABLE #4 MISS. LOESS 2000 PSI C U TEST

CREEP TEST 300 PSI	SPECIMEN #2	SPECIMEN #3	SPECIMEN #4	SPECIMEN #6
σ_3, Kg/cm²	.5	.5	1	1
INITIAL W %	20.5	20.5	20.8	20.9
FINAL W %	20.6	20.6	20.7	20.8
INITIAL e	.605	.605	.608	.610
SATURATION, %	92	94	93	92
MAX + U IN CU TEST, Kg/cm²	.105	.105	.21	.21
D AT ϕ_e LINE FIG.#15, Kg	10	10	20	20
D AT BREAK IN CREEP CURVE Kg	9	9	16	12

TABLE # 5 MISS. LOESS 300 PSI

CREEP TEST 1000 PSI	SPECIMEN #1	SPECIMEN #3	SPECIMEN #2	SPECIMEN # 5
σ_3, Kg/cm²	.35	.35	.75	.75
INITIAL W %	18.9	19.0	18.9	19.0
FINAL W %	19.4	20.6	19.6	19.9
INITIAL e	.576	.573	.567	.565
SATURATION, %	92	95	94	95
MAX AT U IN CU TEST, Kg/cm²	.07	.07	.14	.14
D AT ϕ_e LINE FIG.# 16, Kg	10	10	16	16
D AT BREAK IN CREEP CURVE Kg	7.5	7.5	15	17

TABLE # 6 MISS. LOESS 1000 PSI

MISS. LOESS 300 PSI	SPECIMEN # 4	SPECIMEN # 5	SPECIMEN # 6	SPECIMEN # 7
σ_3, Kg/cm^2	·5	1.0	2.0	4.0
VOID RATIO	·621	·618	·613	·573
SAT. %	96.7	96	96	98.5
CALCULATED $\mu_c = P_a/2$, U_D = Kg/cm^2	.47	·515	·615	1.07
CALCULATED $\mu_c = 0$, U_D = Kg/cm^2	·099	·15	·327	1.11
MEASURED U_D, Kg/cm^2	1.05	·21	·343	·84

TABLE #7 CALCULATED AND MEASURED VALUES OF PORE PRESSURE U_D AT MAX + U

MISS LOESS 1000 PSI	SPECIMEN #4	SPECIMEN #3	SPECIMEN #6	SPECIMEN #5
σ_3, Kg/cm^2	·35	·75	2.0	3.0
VOID RATIO	·611	·595	·575	·580
SAT. %	99.0	98.5	98.0	97.0
CALCULATED $\mu_c = P_a/2$, U_D = Kg/cm^2	·45	·56	·976	1.30
CALCULATED $\mu_c = 0$, U_0 = Kg/cm^2	·102	·35	·963	·93
MEASURED U_0, Kg/cm^2	·07	·14	·245	·28

TABLE #8 CALCULATED AND MEASURED VALUES OF PORE PRESSURE U_0 AT MAX + U

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