RESIDUAL PORE PRESSURES
AND
SHEAR STRENGTH OF COMPACTED CLAY

By

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ABSTRACT

This thesis is an attempt to study the properties of compacted soil in terms of its structure and to isolate the effect of structure on one soil property - the drained shear strength.

Experiments were run to obtain relationships between the molding water content, dry density, residual pore pressure, and undrained shear strength. These relationships followed the well established general patterns.

To isolate the effect of structure on drained shear strength, compacted samples were saturated at constant volume by back pressuring while keeping the effective stress on the sample constant and equal to the initial residual pore pressure. The ϕ angle of the effective stress envelope obtained by running consolidated undrained tests with pore pressure measurements was determined to be 30.6^{o}; the cohesion intercept was essentially equal to zero.

It was observed that the saturation process used, tended to disturb the structure of samples compacted dry of optimum but not those compacted wet of optimum.

INTRODUCTION

The rapid development of the science of soil mechanics in the last few decades has led to greater use of soil for engineering structures. This in turn has created a demand for further improving our knowledge of the properties of soil. Research for understanding the engineering behavior of compacted clays received a strong impetus with the presentation of two papers by Dr. Lambe (1958) (1 and 2)* which attempt to explain the various known, but seemingly unconnected, facts about the properties of compacted clay in terms of its structure, that is "arrangement of particles and the electrical forces acting between them."

This thesis is concerned mainly with the explanation advanced for strength characteristics of compacted clay. It has been known for quite some time that undrained shear strength of soil compacted dry of optimum is higher than that of soil compacted wet of optimum and further that the former mobilizes most of its strength at low strains, whereas the latter attains strength gradually, but continues to do so even at large strains. Briefly Dr. Lambe explains this phenomenon as follows**: Soil compacted dry of optimum develops a flocculated structure characterized by random particle orientation. Soil compacted wet of optimum develops a dispersed structure that is with parallel particle arrangement. Associated with flocculated structure is greater water deficiency and consequently greater

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* Numbers refer to reference listing

** For details see Ref. 1 and 2

pore water tensions in comparison with dispersed structure. Larger pore water tension are synonymous with higher intergranular stresses and therefore lead to higher undrained strength. Pore pressures induced during shear reinforce this tendency towards higher undrained strength in soils compacted dry of optimum, since a greater portion of the applied stresses can be expected to be transferred into the soil skeleton when the skeleton is randomly oriented than when it has parallel orientation. Pore pressure induced can, therefore, be expected to be more positive in samples compacted wet of optimum than in those compacted dry of optimum.

One would expect according to this theory that when compacted samples were saturated while volume was kept constant, the general trends of strength characteristics would remain the same although differences would be reduced. Experimental evidence presented in reference 2 seems to support this theory.

The soil samples from which Dr. Lambe obtained most of his data differed, unfortunately, not just in structure, but also in other parameters such as density and water content which are also strength determining factors. In an effort to isolate the effect of structure, Seed and Chan (1959) (3 and 4) ran a series of tests in which they compacted samples at the same dry density, dry and wet of optimum, saturated them at constant volume by soaking them for six days in water and then sheared them in unconsolidated undrained tests. By this procedure they eliminated the two extraneous variables, dry density, and water content from effecting strength of the two samples. Their results seem to support the theory advanced by Professor Lambe. Seed and Chan further pointed out that statements such as, "The strength of soil compacted dry of optimum is greater than that of soil compacted wet of optimum" are meaningful only when accompanying the statement is a definition of strength, that is, the deformation criterion used. Their experimental data showed that the above statement is true only at low strains (5%) but that at high strains (20%) the strength was independent of the molding water content, possibly because the shearing action transformed initial structures to dispersed and at high strain samples compacted wet and dry of optimum had no difference in structure.

Since a considerably larger amount of information regarding stress condition in soil can be obtained from data from consolidated undrained triaxial tests with pore pressure measurements than from just unconsolidated undrained tests, both Professor Lambe and Professor Seed have presented such data. These data were obtained by compacting samples dry and wet of optimum, saturating them by soaking, and consolidating them to some arbitrary confining pressure. The results seem to indicate general agreement with the theory presented by Professor Lambe.

In these consolidated undrained tests, however, volume of sample changed during consolidation and the structure can be expected to have been affected. Professor Seed observing that the stress strain characteristics of these samples resemble those of unconsolidated undrained tests, concludes that the structure could not have changed appreciably. The argument is logically fallacious* but probably acceptable in engineering

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* Structure may be a sufficient condition for giving the specified stress strain characteristics but it may not be a necessary condition. The text following shows how two very different phenomena can explain the induced pore pressures- either being a sufficient condition but neither a necessary one.

circles. Anyhow the possibility that structure is changed cannot be ruled out, especially since other "scientific" theories are readily visible which also explain the data. For example, the data presented both by Professor Lambe and Professor Seed indicate that the pore pressures induced in samples compacted dry of optimum are more negative than those induced in samples compacted wet of optimum. This as explained earlier agrees admirably with the structure theory. An alternative explanation may go somewhat as follows. Samples compacted dry of optimum have pore tensions much larger than those compacted wet of optimum. For running consolidated undrained tests both samples are consolidated to the same confining pressure. If the initial residual pore pressure can, by analogy, be thought of as maximum past effective stress the sample compacted dry of optimum would behave somewhat like an "overconsolidated" sample and the sample compacted wet of optimum as a "normally consolidated" sample. On shearing the former is known to develop more negative pore pressure than the latter. The actual magnitude of the two initial residual pore pressure and the consolidation pressure are immaterial since the "over-consolidation" ratio of the former would always be larger than that of the latter and the relative magnitudes of the pore pressure in the same relationship as observed in the tests.

This thesis, then , was an attempt to obtain consolidated drained strength characteristics with relatively no change in the structure of the compacted samples. It was proposed that the residual pore pressures of the samples would be measured immediately after compaction and thus determine the effective stress on:

$$\bar{\sigma} = \sigma - u$$

$$\because \ \sigma = 0 \qquad \bar{\sigma} = -u = -(-u_{RES}) = u_{RES}$$

the sample. The sample would then be saturated by back pressuring, always keeping the effective stress, that is the difference between chamber and back pressure, equal to the initial residual pore pressures. The effect of structure on strength could therefore be isolated by testing the two samples, compacted, one dry, and the other wet of optimum at the same dry density; saturating them while keeping constant effective stress and therefore constant volume and initial structure.

OBJECT AND SCOPE

This thesis is the manifestation of an attempt to achieve three primary objectives. First to verify, once again, the established general relationships of dry density, residual pore pressure, and the unconfined shear strength of compacted clays to the molding water content of the clay. Second, to isolate, to a degree greater than that previously achieved, the effect of structure on the strength of compacted clays. And finally to obtain strength envelopes characterizing the clay after being saturated in its compacted state, that is, with initial volume and effective stress.

This study is restricted to one soil, to one type of compaction, and to only one specific amount of compaction energy. The only variable is the molding water content.

PROCEDURE

Soil Used

A mixture of 40% Hydrite UF (- 2 μ kaolinite) and 60% of Potters Flint (- No 200 quartz) was used. Specific gravity = 2.64.

Soil Preparation

Specified percentages of dry kaolinite and quartz were thoroughly mixed by hand for over an hour. The dry mix was prepared in large quantities. Random fluctuations in the percentage of the two component soils must necessarily be expected in any small portion taken from the mix. Desired amounts of water were added and mixed with small portion of dry mix about 48 hours before the soil was needed for testing. During these two days the soil was cured in sealed jars in the humid room. Soil was used only once and discarded after the test.

Compaction

Samples were compacted in the standard Harvard miniature mold (inside diameter = 1 5/16 inches, length = 2.816 inches) in three layers, each layer being tamped by 25 blows of a 40 lbs spring loaded tamper.

Residual Pore Pressures

Residual pore pressures were measured by utilizing a pressure transducer connected to a calibrated voltmeter. Samples were slid on to the top of a fine porous stone (air breaking pressure of about 60 psi) glued firmly on the pedestal of a triaxial cell. Great pains were taken to completely deair the water drain connecting the porous stone to the transducer. In measuring residual pore pressures, equilibrium was reached in about half an hour. The system could directly measure pore pressure as negative as 0.8 kg/cm^2 without any fear of cavitation of water in the connecting drain. A detailed description of this apparatus is given by Bourqué (1961) (5) with whom the author shared the equipment.

Unconfined Compression Tests

These tests were performed in a standard motor operated, strain controlled machine.

Saturation and Triaxial Tests

A schematic drawing of the apparatus used is given in Figure 1. The set up is essentially the same as that used in standard triaxial testing with the addition of a U tube which was attached to the pore pressure line during saturation and substituted by the pore pressure device during the shear test. The U tube was to serve a dual purpose: to enable the observation of the amount and the time relation of the seepage of water into the sample. Back pressure and the chamber pressure were increased simultaneously usually by increments of 1 kg/cm^2 keeping the differences between them equal to the initial residual pore pressure. The mercury in the U tube was always kept balanced by adjusting the left branch of the U tube. Knowing the quantity of water seeping in, the degree of saturation could be determined by computation. Pore pressures response was measured when it was felt that the sample was saturated. B factor of 0.95 was considered adequate.

The sample was sheared at the rate of approximately 0.65% per hour for the first 1% of strain and thereafter at the rate of about 1.4% per hour. The back pressure was kept constant and the chamber pressure varied to keep constant volume. Eight filter strips and two rubber membranes with silicon grease between them clothed the sample. Deaired water was used in the chamber and in the U tube.

RESULTS

In order to be able to measure the residual pore pressures of two samples of the same density one compacted dry and the other wet of optimum, it was necessary to experiment with different mixtures of kaolinite and quartz to obtain a mix with suitable measurable range of residual pore pressures. Graph 1 presents the data obtained for three mixes.

For the soil selected unconfined compression tests were run to obtain a relationship between undrained strength and the molding water content. Graph 2 indicates the obtained results along with residual pore pressure measurements. Strength for these samples was defined as maximum deviator stress or stress at 10% strain, whichever occurred first. Graph 3 shows plots of deviator stress vs strain for some of the samples tested in unconfined compression.

Data obtained from triaxial tests has been summarized in graphs 5 to 13. Graph 4 identifies the five samples tested, on the dry density v/s molding water content curve.

The equation of the effective stress envelope best fitting the experimental data was determined to be:

$$\tau = 0.06^{*} + \bar{\sigma} \tan 30.6^{\circ} \qquad (kg/cm^2)$$

The maximum deviation of experimental points from the line is only 0.02 kg/cm^2.

Graph 14 presents the back pressure which was used in relation to the initial degree of saturation. The degree of saturation achieved,

* SEE FOOTNOTE NEXT PAGE

as determined from the final water content, labels the various points. The theoretical curves derived from Henry's and Boyle's law following the relationship: (6)

$$\Delta U = p_o \left(\frac{1 - S_o (1 - H)}{1 - S_p (1 - H)} \right) - 1$$

ΔU = back pressure
p_o = initial pressure of air in sample
S_o = initial degree of saturation
S_p = desired degree of saturation
H = Henry's coefficient 0.02 at room temperature

are also shown on the graph. The final degree of saturation can easily be expected to have an error of $\pm$ 1%. The correlation with the theoretical curves, then, seems to be reasonably high.

Considerable amount of data was taken of the water movement in the U tube as samples were being saturated. However, they were not thought important enough to be presented in graphical form. As would be expected, with each increment of chamber and back pressure, the water seeped in quickly at first but with time its movement dropped to neglible quantities. Larger amounts of water entered the soil during the initial increments than in the subsequent ones. Total time for saturation varied from two to four days mainly because of reasons other than those inherent in the saturation process. The total quantity of water observed to seep into the sample compared well (about 10% higher) with that computed from initial and final water contents, the discrepancy resulting mainly from water leakage through the adapter (see Figure 1) during setting up of the sample.

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* See discussion of results

TRIAXIAL AND SATURATION APPARATUS

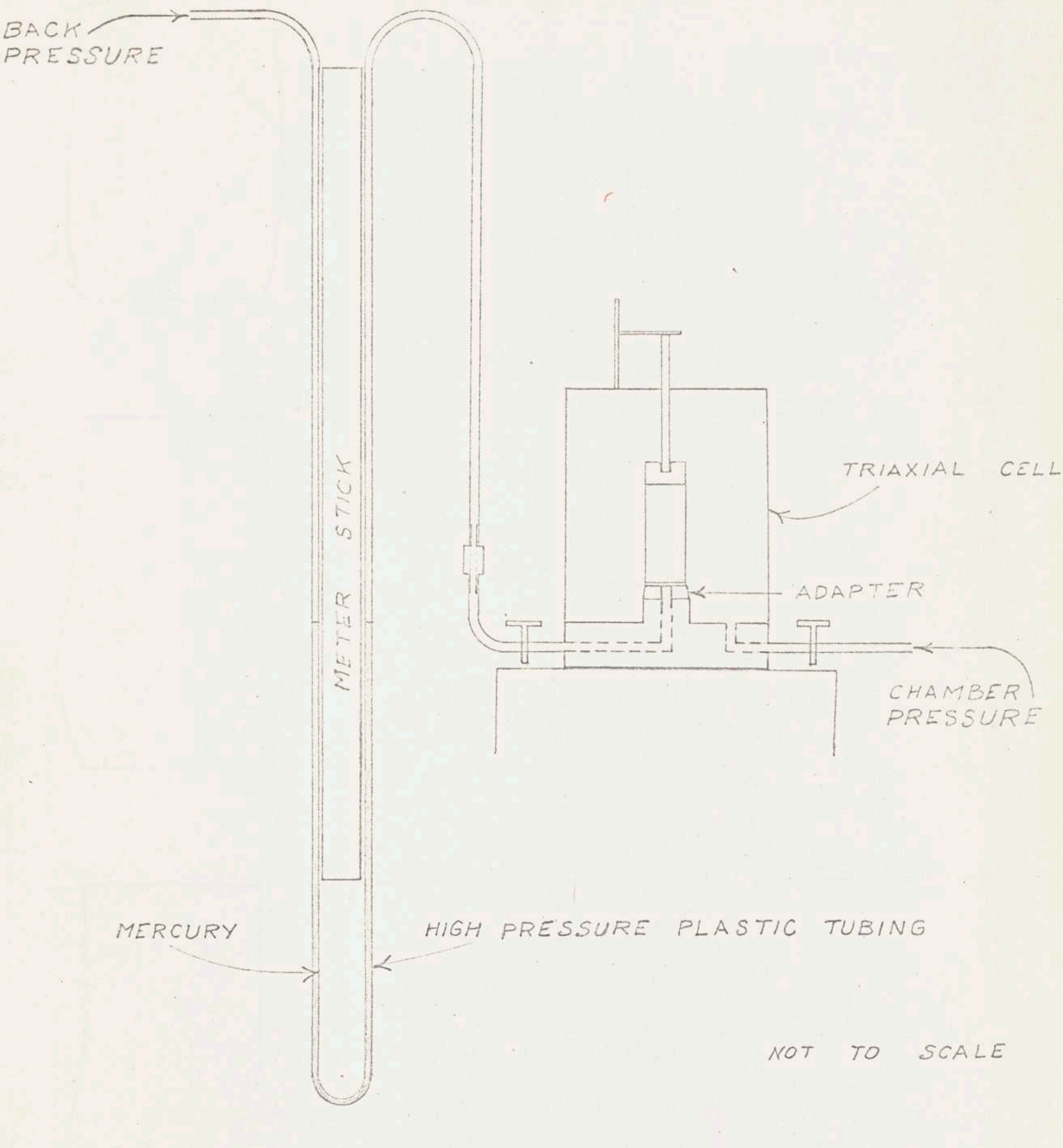


fig 1.

SHAPE AND WATER CONTENT AT FAILURE

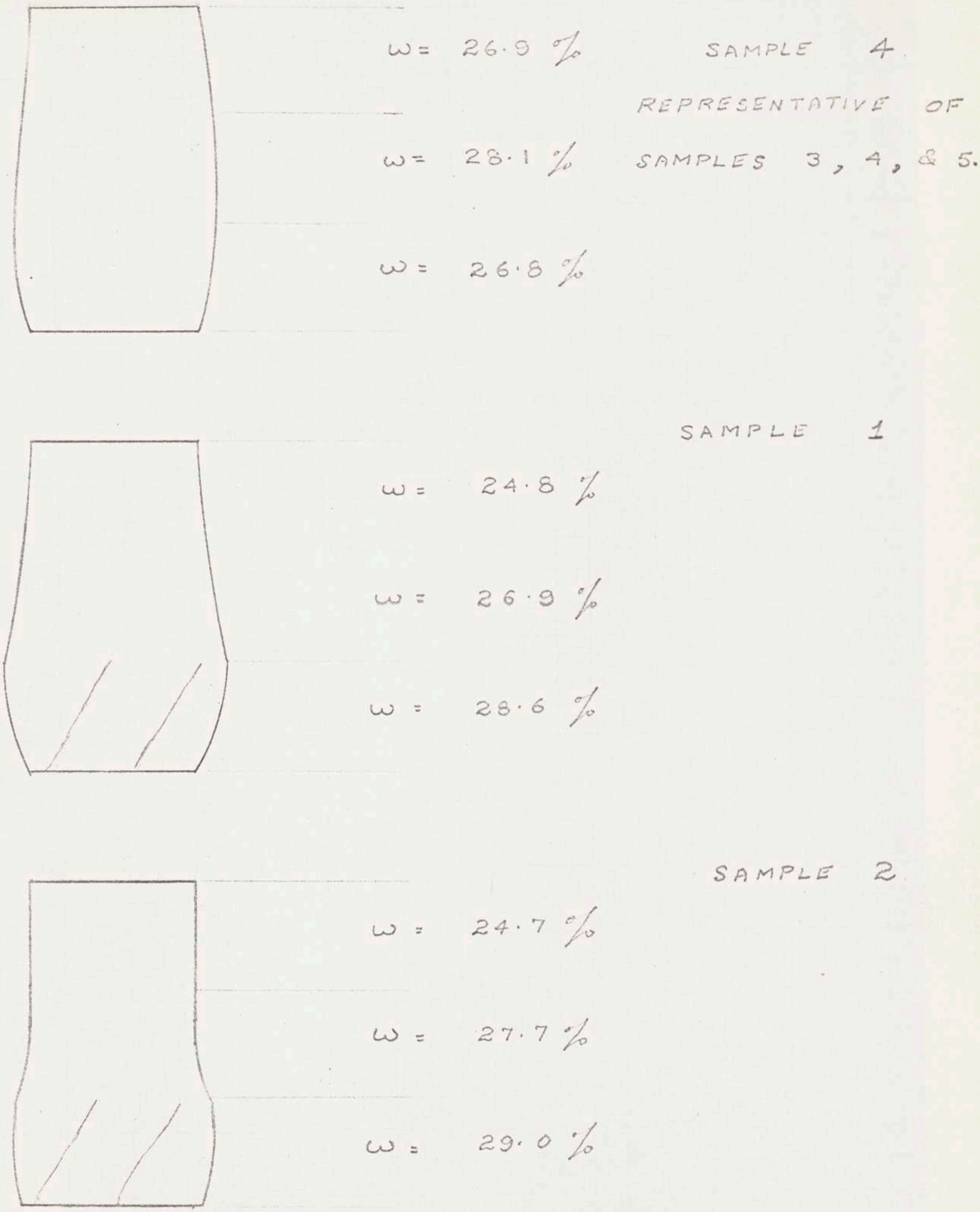
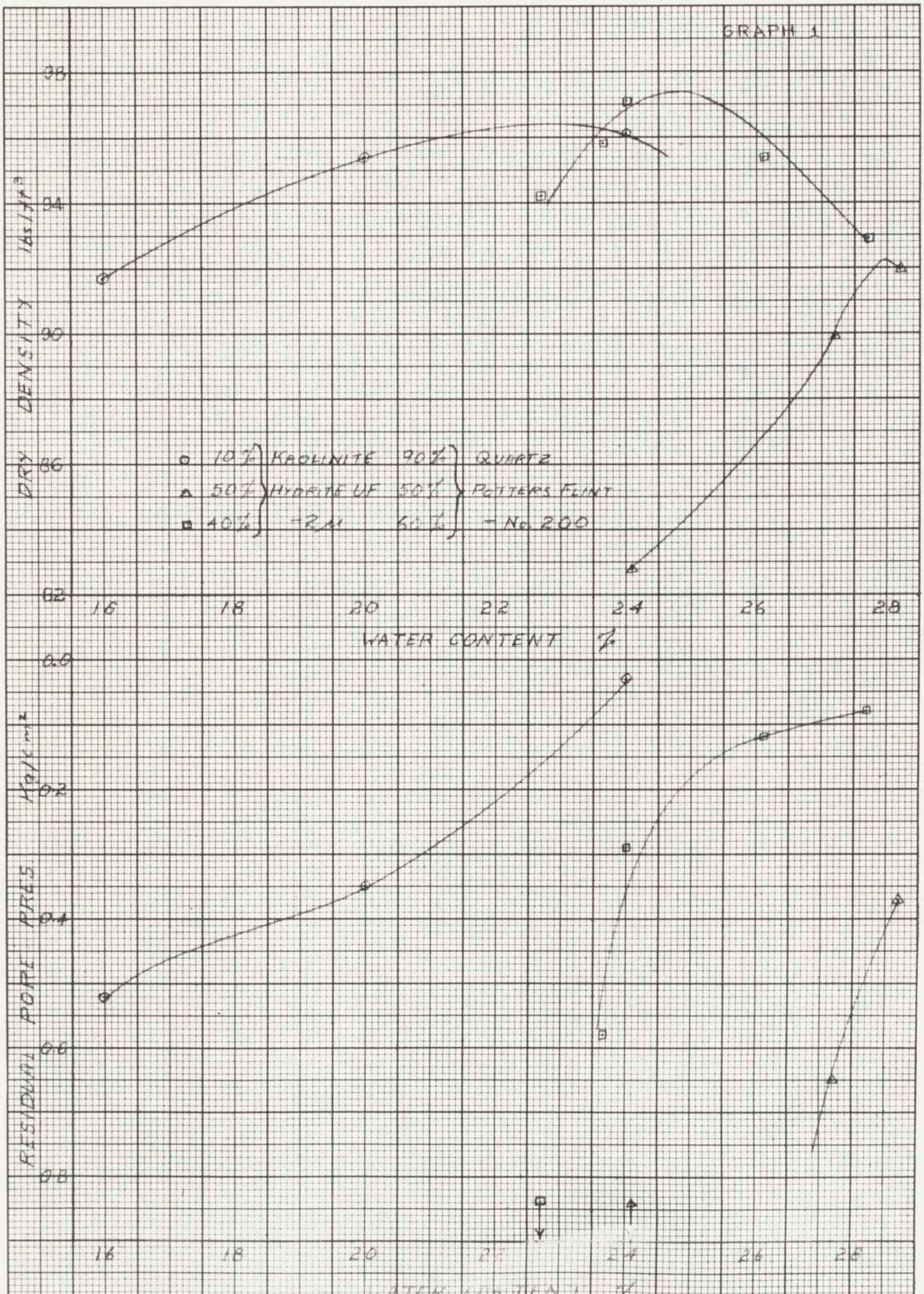
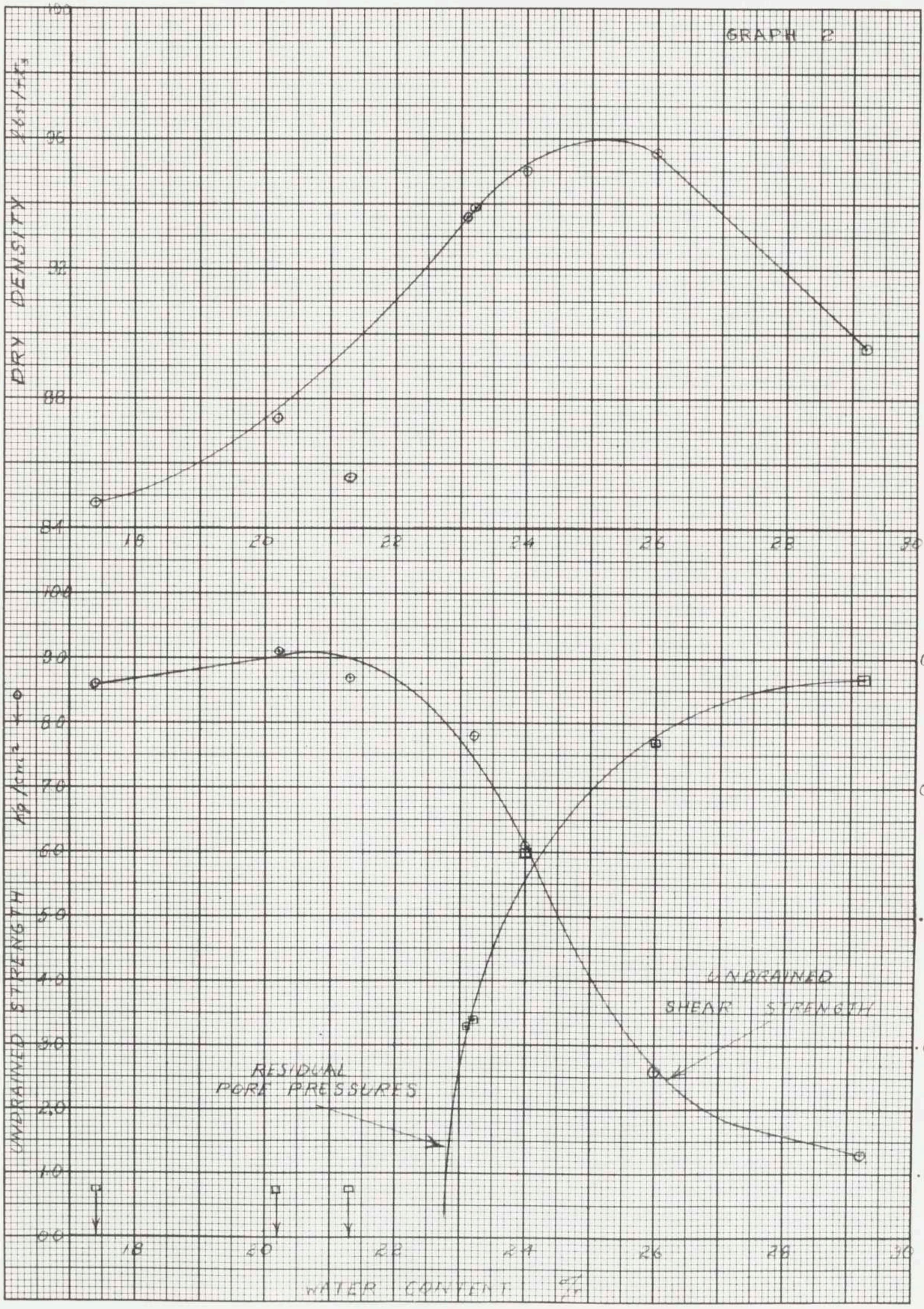
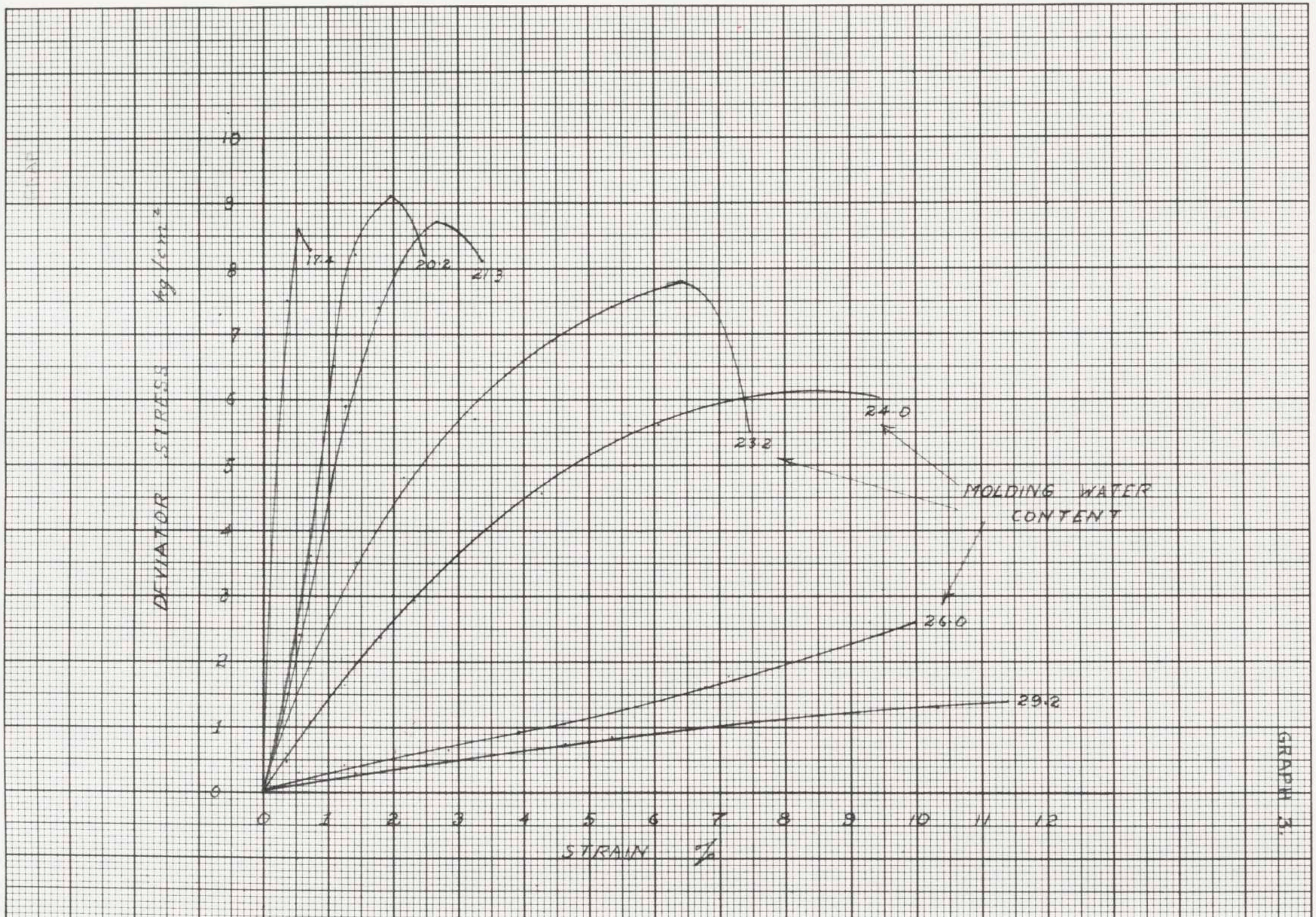


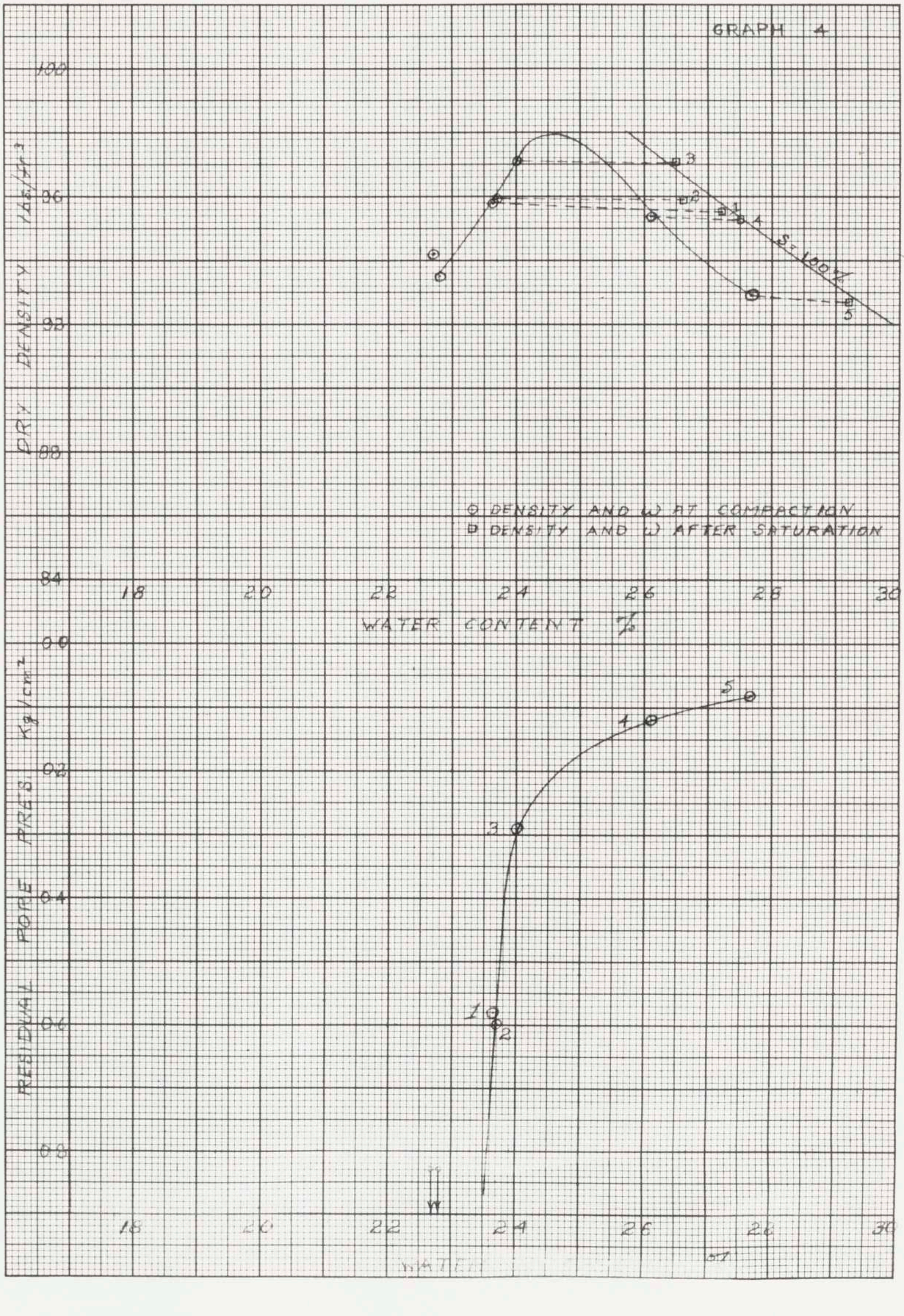
fig 2

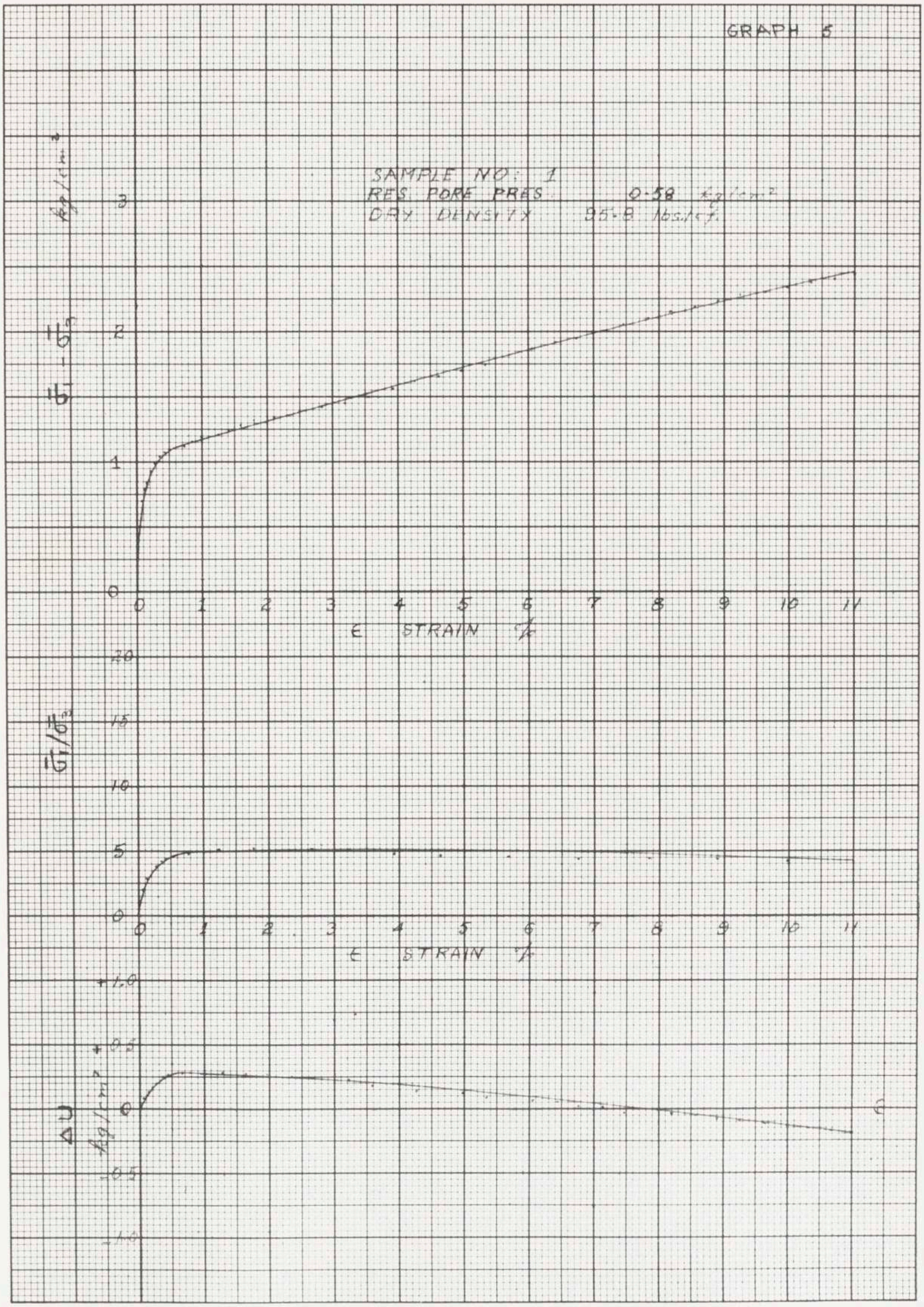

GRAPH 1
DRY DENSITY lbs/ft³
98
94
90
86
82
10% KAOLINITE 90% QUARTZ
50% HYDRITE UF 50% POTTERS FLINT
40% -2μ 60% - No. 200
16
18
20
22
24
26
28
WATER CONTENT %
RESIDUAL PORE PRES Kg/cm²
0.0
0.2
0.4
0.6
0.8

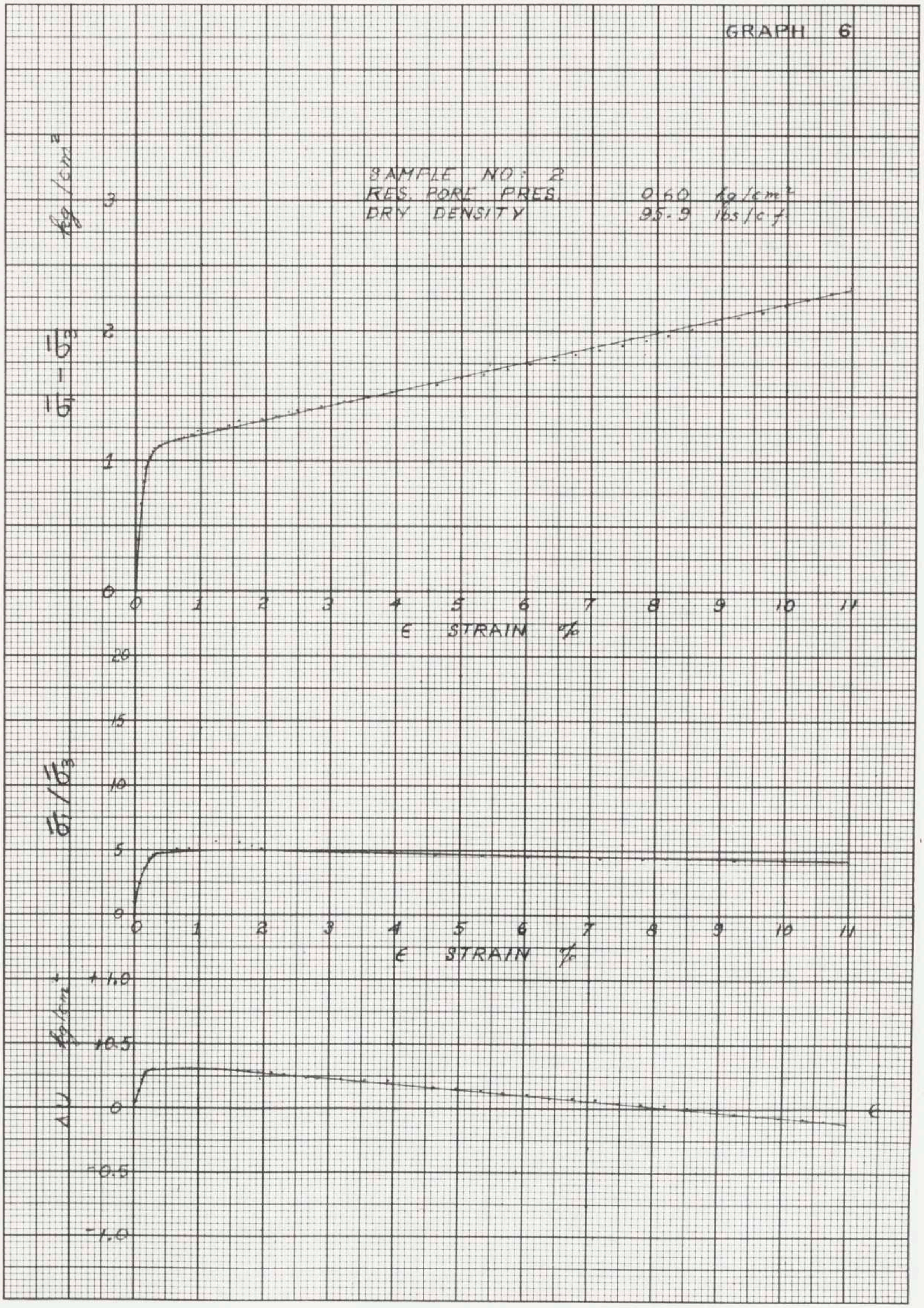

GRAPH 2
DRY DENSITY lbs/cu ft
UNDRAINED STRENGTH Kg/cm²
RESIDUAL PORE PRES Kg/cm²
RESIDUAL PORE PRESSURES
UNDRAINED SHEAR STRENGTH
WATER CONTENT %

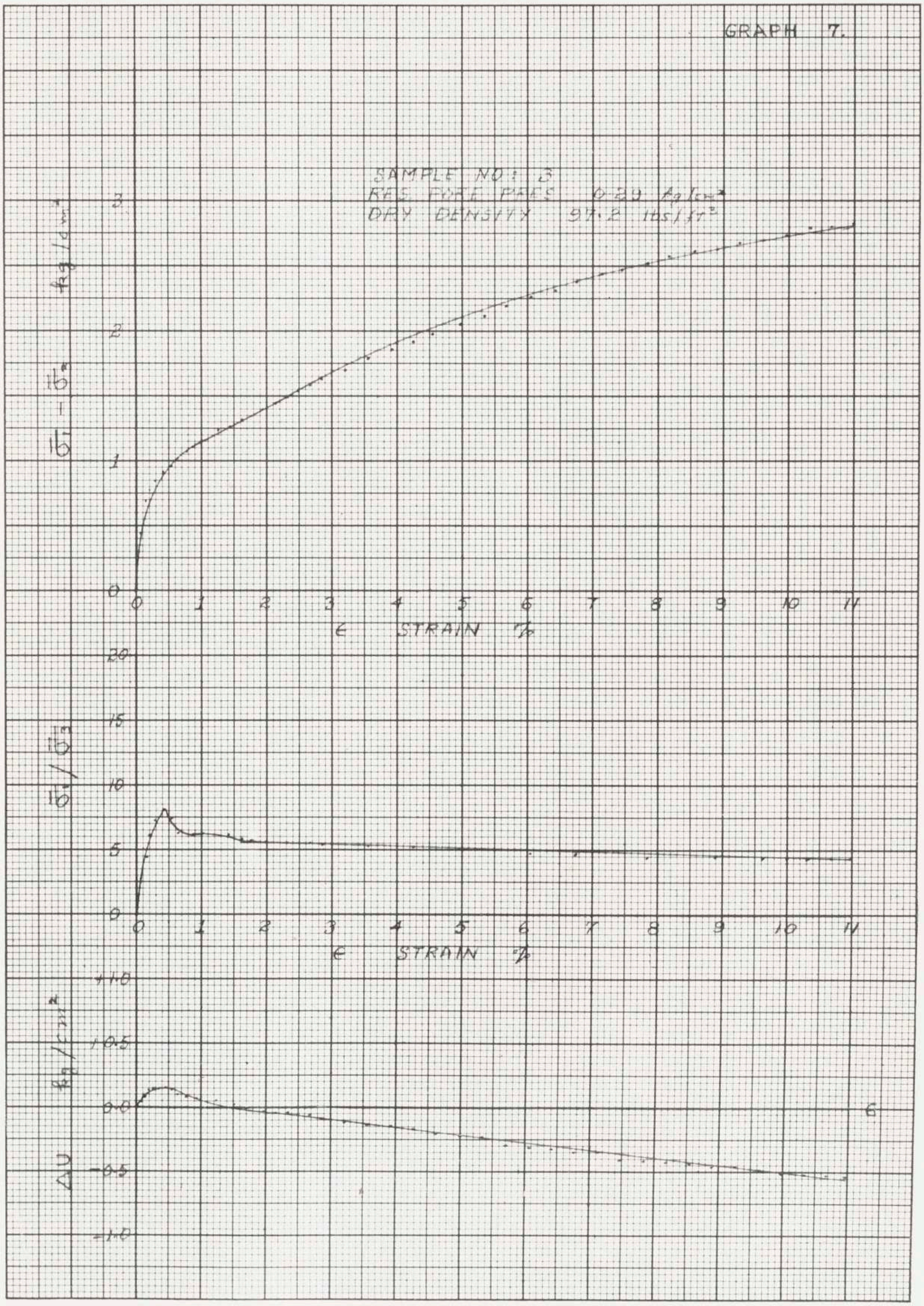

DEVIATOR STRESS kg/cm²
STRAIN %
0
1
2
3
4
5
6
7
8
9
10
11
12
17.4
20.2
21.3
23.2
24.0
26.0
29.2
MOLDING WATER CONTENT

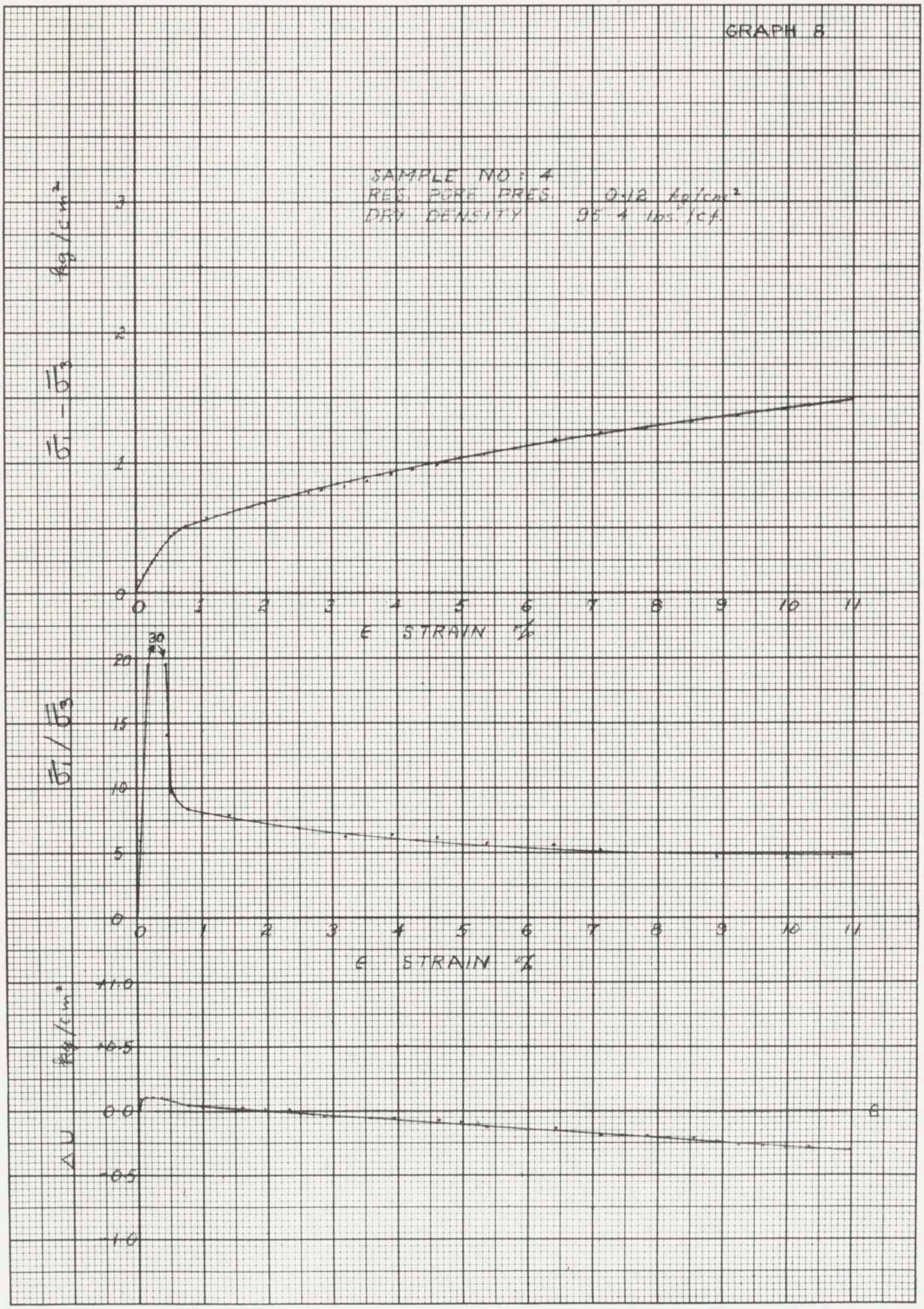
GRAPH 3.

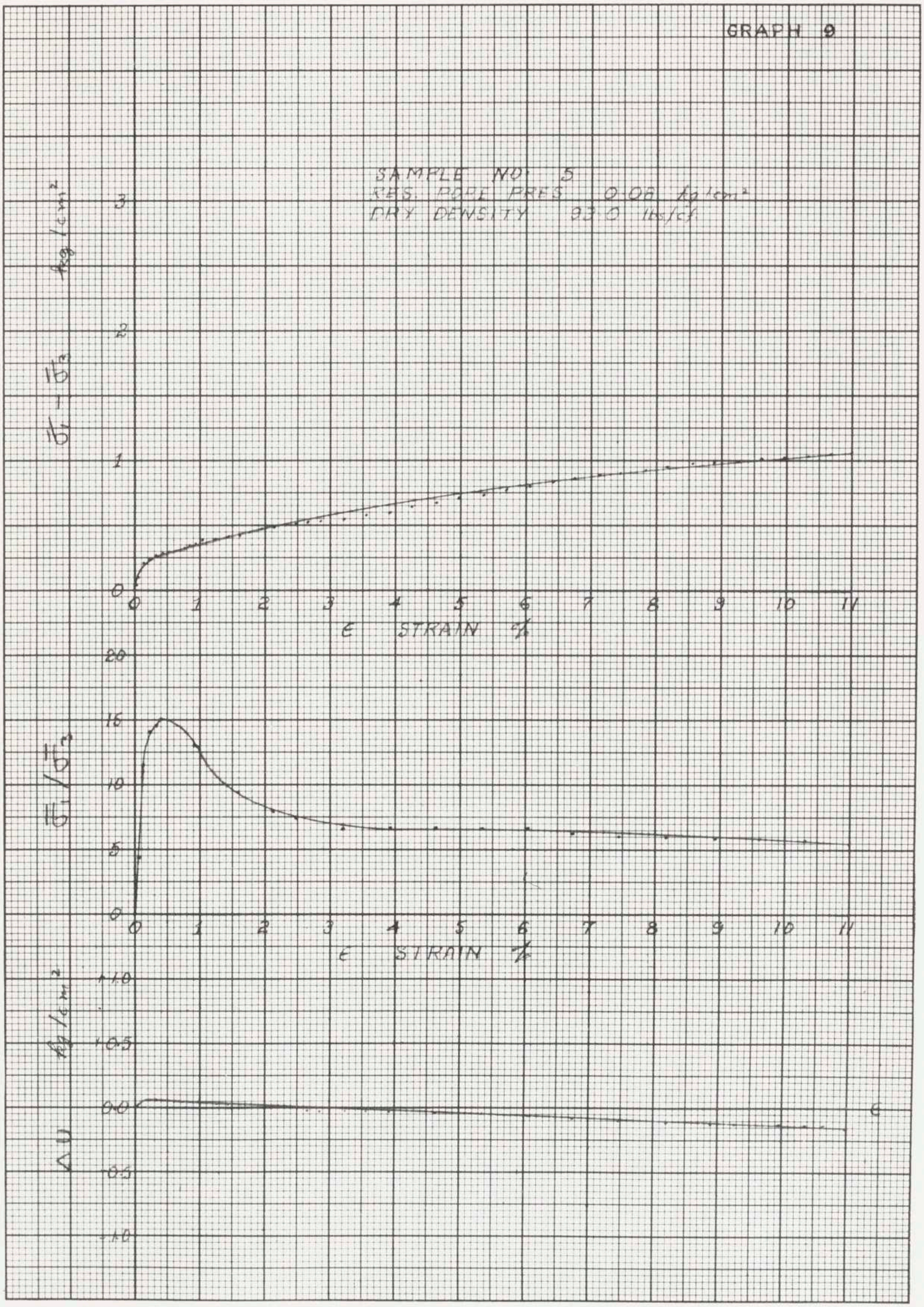

GRAPH 4
100
96
92
88
84
DRY DENSITY lbs/ft³
S=100%
⊙ DENSITY AND ω AT COMPACTION
⊡ DENSITY AND ω AFTER SATURATION
18
20
22
24
26
28
30
WATER CONTENT %
0.0
0.2
0.4
0.6
0.8
RESIDUAL PORE PRES. Kg/cm²
1
2
3
4
5


GRAPH 5
SAMPLE NO: 1
RES PORE PRES 0·58 kg/cm²
DRY DENSITY 95·8 lbs/cf.
$\bar{\sigma}_1 - \bar{\sigma}_3$ kg/cm²
ε STRAIN %
$\bar{\sigma}_1/\bar{\sigma}_3$
ε STRAIN %
ΔU kg/cm²

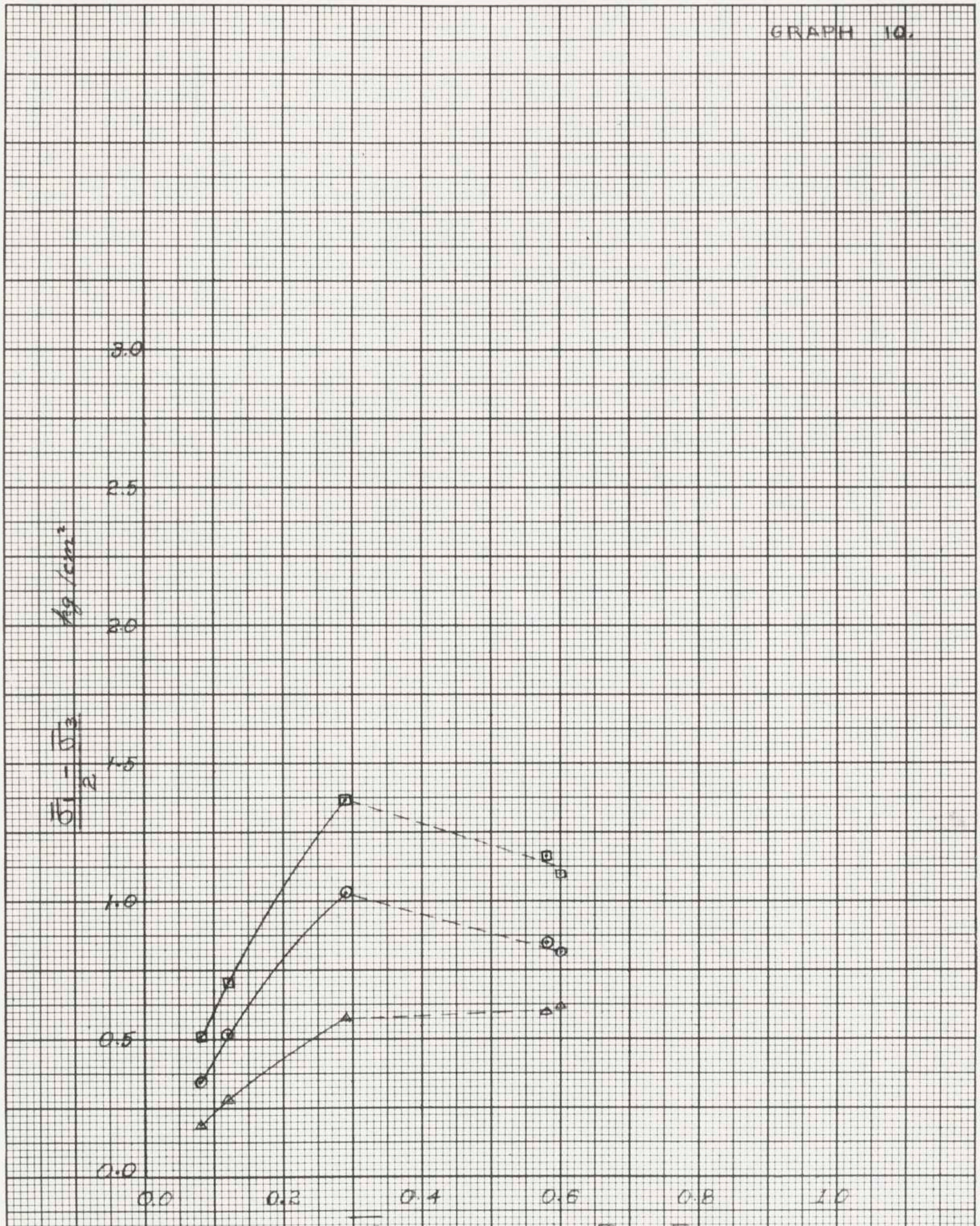

GRAPH 6
SAMPLE NO: 2
RES. PORE PRES. 0.60 kg/cm²
DRY DENSITY 95.9 lbs/c.f.
$\sigma_1 - \sigma_3$ kg/cm²
0
1
2
3
0 1 2 3 4 5 6 7 8 9 10 11
ε STRAIN %
σ_1 / σ_3
0
5
10
15
20
0 1 2 3 4 5 6 7 8 9 10 11
ε STRAIN %
Δu kg/cm²
+1.0
+0.5
0
-0.5
-1.0
ε

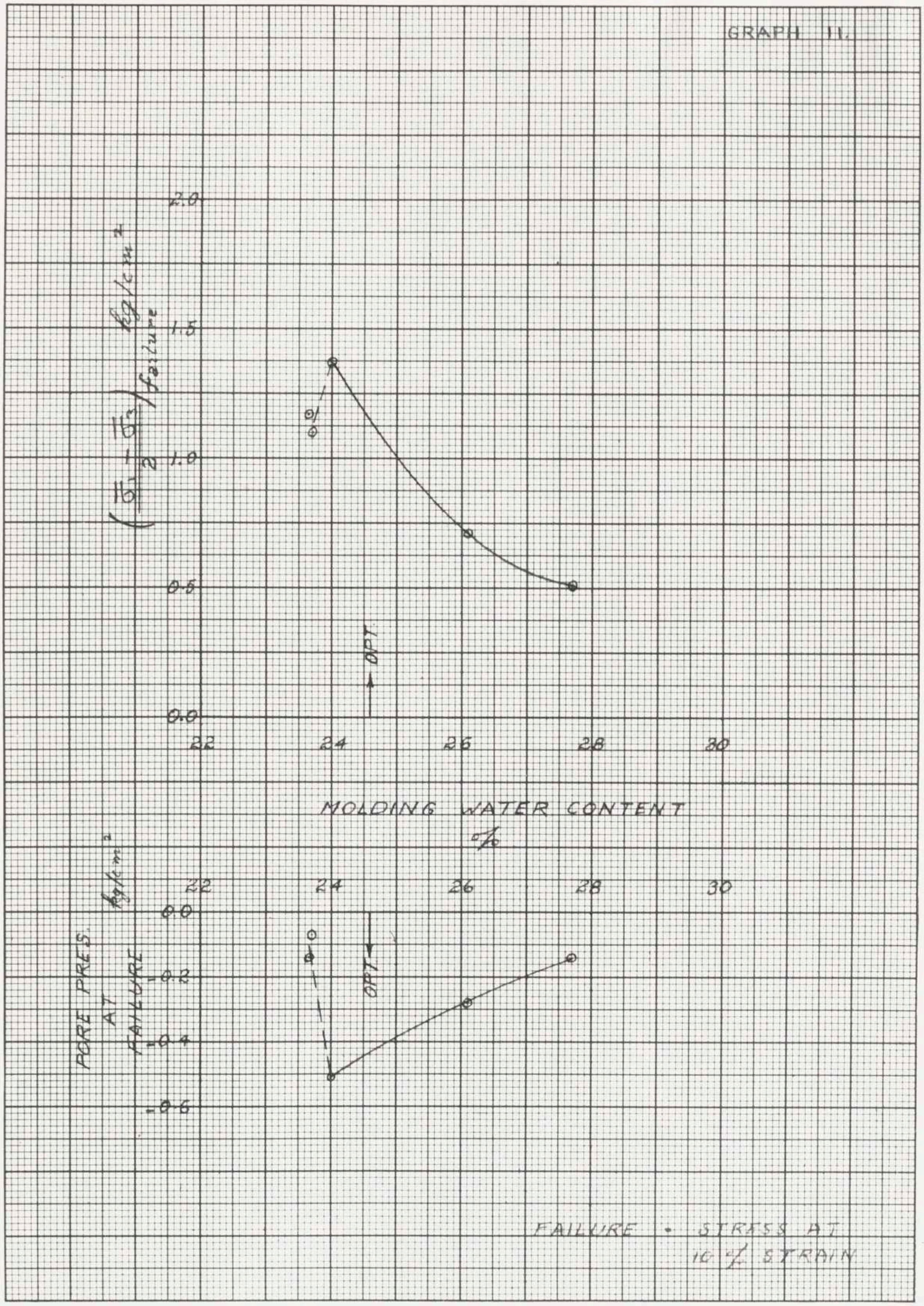

GRAPH 7.
SAMPLE NO: 3
RES. PORE PRES. 0.29 kg/cm²
DRY DENSITY 97.2 lbs/ft³
σ1 − σ3 kg/cm²
0
1
2
3
0 1 2 3 4 5 6 7 8 9 10 11
ε STRAIN %
σ1 / σ3
0
5
10
15
20
0 1 2 3 4 5 6 7 8 9 10 11
ε STRAIN %
Δu kg/cm²
+1.0
+0.5
0.0
−0.5
−1.0
ε

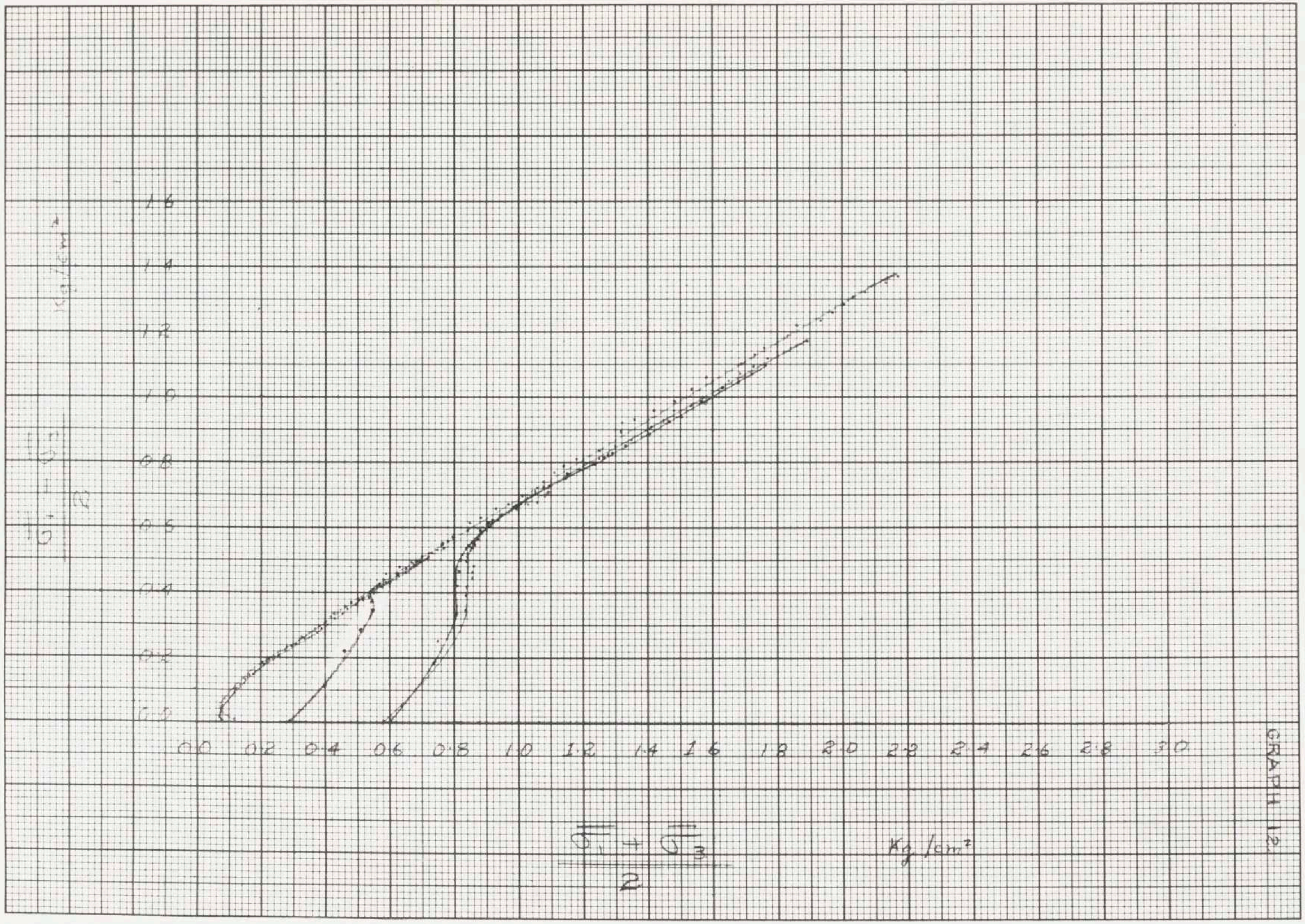

GRAPH 8
SAMPLE NO: 4
RES. PORE PRES. 0.12 kg/cm²
DRY DENSITY 95.4 lbs./cf.
σ1 − σ3 kg/cm²
ε STRAIN %
σ1 / σ3
30
ε STRAIN %
ΔU kg/cm²



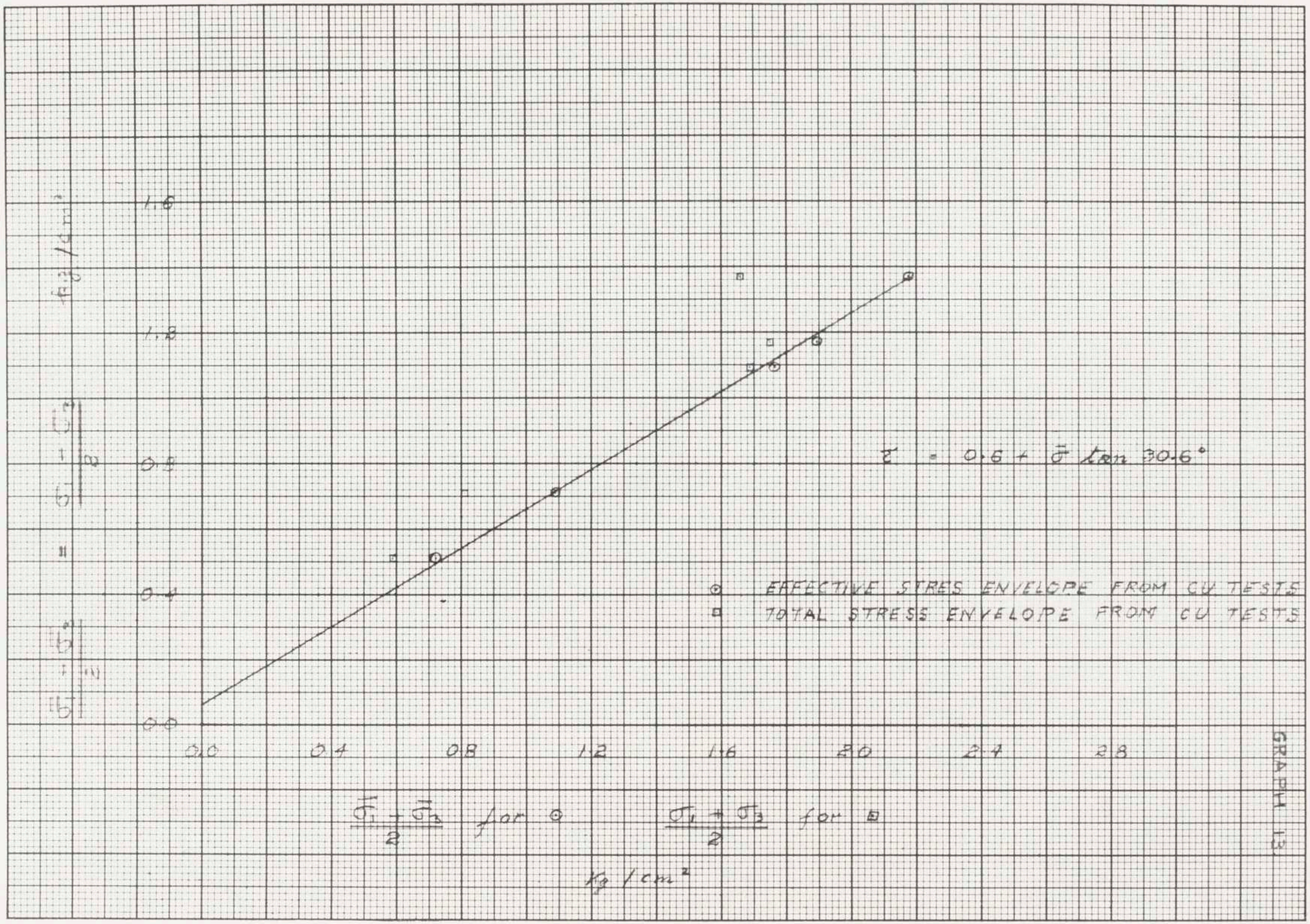
GRAPH 9
SAMPLE NO. 5
RES. PORE PRES. 0.08 kg/cm²
DRY DENSITY 93.0 lbs/cf
σ1 - σ3 kg/cm²
0
1
2
3
0 1 2 3 4 5 6 7 8 9 10 11
ε STRAIN %
σ1 / σ3
0
5
10
15
20
0 1 2 3 4 5 6 7 8 9 10 11
ε STRAIN %
ΔU kg/cm²
+1.0
+0.5
0.0
-0.5
-1.0


GRAPH 10.
3.0
2.5
2.0
1.5
1.0
0.5
0.0
kg/cm²
$\frac{\bar{\sigma}_1 - \bar{\sigma}_3}{2}$
0.0
0.2
0.4
0.6
0.8
1.0
$\bar{\sigma}_c$ = RESIDUAL PORE PRES
kg/cm²
at 1 % STRAIN
at 5 % STRAIN
at 10 % STRAIN

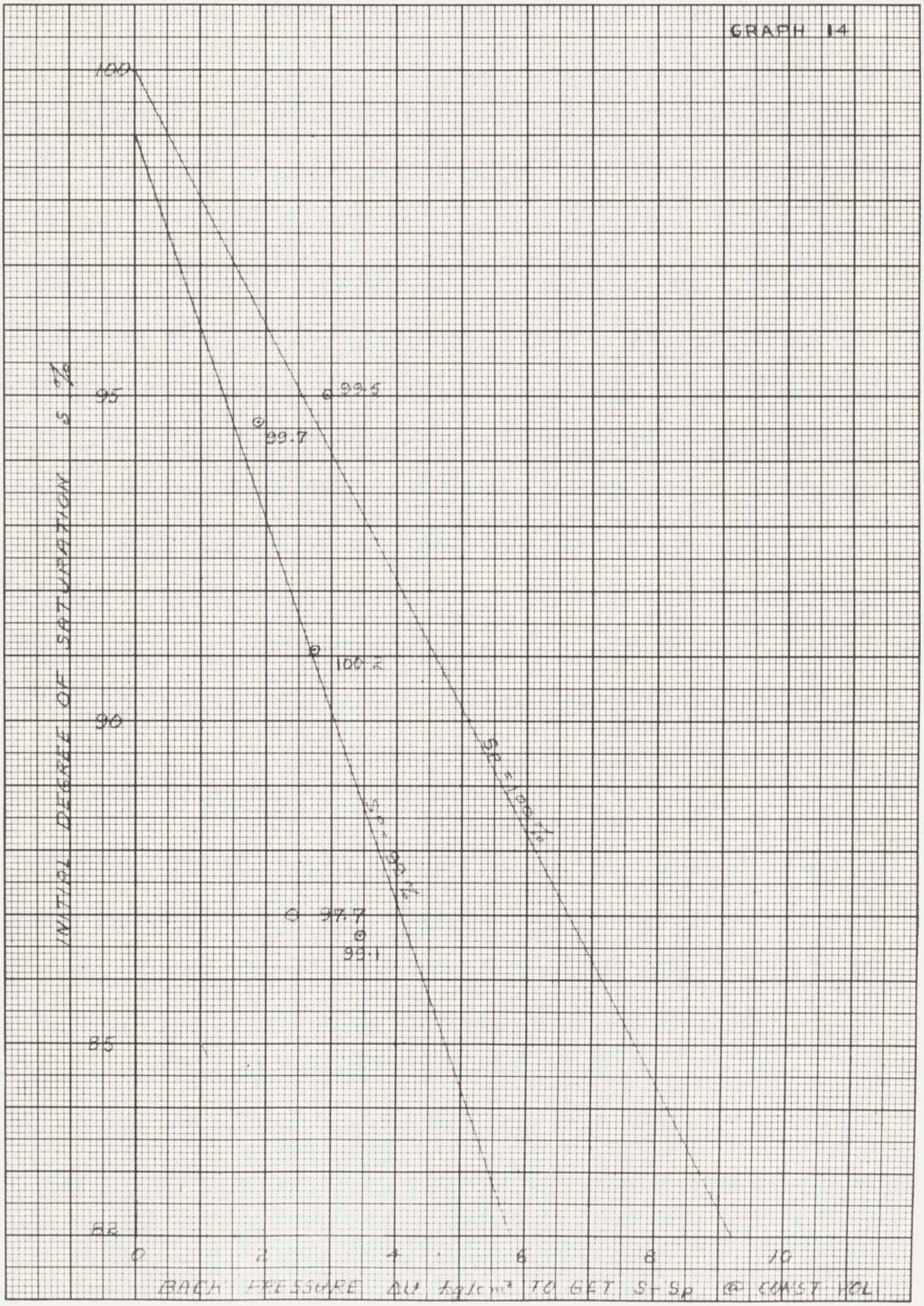

GRAPH 11.
2.0
1.5
1.0
0.5
0.0
22
24
26
28
30
OPT.
MOLDING WATER CONTENT
%
PORE PRES.
AT
FAILURE
kg/cm²
0.0
-0.2
-0.4
-0.6
FAILURE - STRESS AT
10 % STRAIN


1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0.0 0.2 0.4 0.6 0.8 1.0 1.2 1.4 1.6 1.8 2.0 2.2 2.4 2.6 2.8 3.0
$\frac{\sigma_1 - \sigma_3}{2}$ Kg/cm²
$\frac{\sigma_1 + \sigma_3}{2}$ Kg/cm²

GRAPH 12.



1.6
1.2
0.8
0.4
0.0
0.0
0.4
0.8
1.2
1.6
2.0
2.4
2.8
τ = 0.6 + σ̄ tan 30.6°
EFFECTIVE STRES ENVELOPE FROM CU TESTS
TOTAL STRESS ENVELOPE FROM CU TESTS
(σ̄1 + σ̄3)/2 for ⊙
(σ1 + σ3)/2 for ⊡
Kg / cm²
GRAPH 13


GRAPH 14
100
95
90
85
82
99.5
99.7
100
97.7
99.1
Sp = 100 %
Sp = 99 %
0
2
4
6
8
10
INITIAL DEGREE OF SATURATION S %
BACK PRESSURE ΔU kg/cm² TO GET S = Sp @ CONST VOL

DISCUSSION OF RESULTS

Data presented in Graphs 1 to 3 are quite typical and have been discussed at length elsewhere (Ref. 2 and 3). To avoid redundancy it will not be discussed here.

From the results of triaxial tests one finds samples 3, 4 and 5 portraying the characteristics one would expect in view of the structure theory of Dr. Lambe which was discussed briefly in the Introduction. Samples 3, 4 and 5 have progressively increasing molding water content (graph 4): progressively decreasing initial pore water tensions (graph 4); progressively decreasing undrained shear strengths (graph 10); and progressively less negative induced pore pressures at failure (graph 11). Samples 1 and 2 compacted drier than 3, however, do not fit the pattern. The shear stress they mobilized at any strain greater than 1% is less than that mobilized by sample 3 and their pore pressures at failure resemble that of sample 5.

After considerable amount of deliberation, the author finds the explanation advanced below most plausible. Observing the shape of sample 1 at failure (see Figure 2) it was initially thought that possibly the plane between the lower two layers of compaction might have been a plane of weakness resulting from the relative dryness of the soil at the time of compaction. During subsequent compaction of samples dry of optimum, great care was taken to insure bond between layers by grooving the surface of each layer with a knife before placing the next layer.

By pure coincidence*, another sample (sample 2) with essentially the same molding water content, residual pore pressure, and dry density got compacted. It was tested to check sample 1; the results were almost identical. (Compare graphs 5 and 6) The initial idea regarding the plane of weakness lost much of its aura of validity since considerable amount of care had been taken in compacting sample 2.

Attention was focused at the distribution of water content in samples 1 and 2 after failure. In contrast to those of 3, 4 and 5 (see Figure 2) variation is indeed large. Another look at the shape at failure seemed to suggest that a bulging type of failure was occurring in the lower one third of the sample whereas the top two thirds was essentially unaffected.

This behavior might possibly be due to the fact that during saturation seepage forces resulting from the seepage of water from the bottom to the top of the sample may have transformed the structure of the bottom third of the sample to a dispersed structure. The seepage force would be more important as a structure changing element in samples 1 and 2 than in samples 3, 4 and 5 because of two reasons. First the seepage force itself would be larger in samples 1 and 2 because of larger initial residual pore pressures and second because structure initially was flocculated and was susceptible to change into the dispersed type. (The structures in sample 3, 4 and 5 were relatively dis-

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* It was indeed a coincidence since the molding water content could not be controlled with much precision. Six samples were compacted in an attempt to get water content slightly less than that of sample 2 so as to enable testing of a sample with residual pore pressure less than 0.6 kg/cm^2. All six attempts were unsuccessful. It was unfortunate that the residual pore pressure curve was very steep in this range.

persed to begin with).

During the shear test, then, the top two thirds of the sample having a much more rigid structure, acted essentially as a load transmitter and the bottom third controlled the strength characteristics. At least a partial check of this hypothesis could be obtained by measuring the pore pressure on both ends of the sample at failure and seeing if, indeed, they differ markedly.*

It is somewhat unfortunate that a procedure which theoretically seemed very reasonable, in implementation seems to have lost some of its advantages. However, this disturbing effect could possibly be reduced as explained under "Recommendations".

Because of the unreliability of the results of samples 1 and 2, it would not be particularly meaningful to compare them with sample 4 which is essentially at the same dry density as samples 1 and 2 and, as such, ought to have had structure as the only variable.

Graphs 12 and 13 show the effective stress paths and the consolidated drained curve for the clay. Points for a consolidated undrained curve are also shown though they do not seem to indicate any identifiable pattern. This point was also noted by Huning (1957) (7) although he consolidated his samples under pressures selected rather arbitrarily.

The cohesion of 0.06 kg/cm^2 would undoubtedly vanish if a correction for filter strips which was overlooked by the author would be made to the data.

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* Soils compacted considerably dry of optimum can have meta-stable structures that collapse upon saturation. This would agree with the behavior of samples 1 and 2 although their molding water content was not too dry of optimum.

RECOMMENDATIONS

On Apparatus

1. The effect of seepage forces could be reduced considerably if the sample was backpressured not just through the bottom but also through the top.
2. The U tube did not serve any very important function in the saturation process. It turned out to be rather cumbersome since the mercury had to be balanced ever so often. It might, therefore, be eliminated and proximity to saturation tested by measuring the B factor.
3. The adapter would be more manageable if it was permanently glued on to the pedestal. (Figure 1)

On Further Study

1. A repetition of this study after the implementation of the above mentioned modifications.
2. It would be most interesting to see how the consolidated drained strength characteristics of soil samples compacted and then consolidated to arbitrary consolidation pressures as done by Seed and Chan (3) and Huning (7) compare with consolidated drained characteristics as obtained in this study.*
3. Bourque (5) by the application of air pressures has made indirect measurements of pore pressures considerably below one atmosphere. Using this technique data obtained in this study could be considerably extended into the dry of optimum range.

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* An attempt to perform such a test was made by the author but due to a laboratory accident the sample was utterly disturbed. The test could not be repeated due to lack of time.

REFERENCES

1. Lambe, T.W. "The Structures of Compacted Clay", Journal, Soil Mechanics Foundation Division. (SM and FD) ASCE Vol. 84, No. SM 2, May 1958.

2. Lambe, T.W. "The Engineering Behavior of Compacted Clay", Journal (Sm and FD) ASCE Vol. 84, No. SM 2, May 1958.

3. Seed, H.B. and Chan, C.K., "Structures and Strength Characteristics of Compacted Clays", Journal, (SM and FD) ASCE Vol. 85, No. SM 5, October 1959.

4. Seed, H.B. and Chan, C.K., "Undrained Strength of Compacted Clays after Soaking", Journal, (SM and FD) ASCE Vol. 85, No. SM 6, December 1959.

5. Bourque, Paul, "Residual Pore Pressure Measurements of Soils", S.M. Thesis, MIT 1961.

6. Lowe, J. and Johnson, T.C., "Use of Back Pressure in Increase Degree of Saturation of Triaxial Test Specimens", Research Conference of Shear Strength of Cohesive Soils, Boulder, Colorado, 1960.

7. Huning, R.C., "The Effect of Structure on the Strength of Compacted Soil", S.M. Thesis, MIT 1957.