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# ENGINEERING BEHAVIOR OF A WASTE CONVERSION END PRODUCT

by

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# [illegible] BEHAVIOR OF

# [illegible] CONVERSION END PRODUCT

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# FOREWORD

The author wishes to express gratitude toward Dr. A. M. Richardson for his guidance and encouragement during the investigation reported here and in the preparation of the thesis.

The material used in this investigation was donated by Westinghouse Research and Development division.

The help of Walt Marion was appreciated while conducting the laboratory tests.

Special thanks go to my wife, Gail, for her patience in typing this thesis.

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# PREFACE

There are three common methods of refuse disposal: incineration, sanitary landfill, and composting. This report is concerned with the engineering behavior of a high-rate composting end product.

The method used for composting the refuse was developed by Westinghouse Research and Development division. Normal composting methods may take up to six months to produce a useable product, whereas, the Westinghouse process is accomplished in six days.

With an increasing population density, refuse disposal is becoming more and more of a problem. The sanitary landfill requires large amounts of land and is sometimes objectionable to nearby property owners. The incinerator may also pose an air pollution problem.

The compost end product of the Westinghouse process has the appearance and smell of a rich humus soil. The material is, in fact, an organic material that makes a good fertilizer. However, the market for fertilizer does have a limit; and, since the quantity of refuse for disposal is ever increasing, another use will have to be available for the compost material. One of the potential uses of the compost material is the placement of the material in a controlled, compacted landfill. The reclaimed land will thus be made available for public use.

The laboratory investigation reported herein is concerned with the engineering behavior of the compost material in a compacted fill as mentioned above. From the information obtained in the laboratory, it is possible to recommend 1) the best method of placement, 2) the allowable bearing capacity, 3) the slope stability, and, 4) the expected settlement of the material placed in a landfill.

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# I. OBJECTIVE

This investigation represents a look at the engineering behavior of the end product of a waste conversion process. The behavior of this product is compared to typical engineering soil fill material regarding its usefulness in the following situations:

## A. Landfill

In a landfill, the relationship between compaction method, compaction moisture content, and density (quantity of solid material per unit volume), is investigated. The result is a recommendation of the most efficient method of compaction.

## B. Bearing Capacity

In a compacted fill beneath the foundation of a light structure, the relationship between compaction method, compaction moisture content, dry density and shear strength is determined. The effect of confining pressure (depth of cover) and shear strength is investigated along with the relationship between time and compression.

The above information will permit an analysis to be made of the behavior of structures founded on the material in question. From the analysis, a possible allowable

bearing capacity is recommended.

## C. Slope Stability

In a compacted fill on a slope, the information from the preceding section is used in further study of the mechanical stability of a slope terminating a landfill of the subject meterial. A maximum allowable slope angle is recommended.

## II. SCOPE OF THE TEST PROGRAM

In this investigation, the following aspects of behavior were investigated:

### A. Compaction Behavior

The relationship between compaction method, compaction moisture content (% of dry weight at 105°C), and quantity of solid material per unit volume was determined in tests simulating controlled field compaction.

### B. Compression Behavior

The relationship between applied load, compression, and time was determined in test simulating the material under load.

### C. Stress vs. Strain Behavior

The stress vs. strain and shear strength of confined and unconfined specimens of the material compacted at moisture contents in the range producing maximum dry density were determined.

A complete description of all tests will be found in Appendix A of this report.

# III. LABORATORY PROGRAM

## A. Material

The waste conversion end product material used in this investigation was obtained from a bag labelled "Naturizer Organic Compost, United Conversion Company, Inc., San Fernando, California," delivered to the laboratory by Mr. Harley Smith of Westinghouse Research.

The material was assumed to be biologically and chemically stable, and the moisture content of the material in the bag as received varied from 49% to 58% of dry weight (at 105°C).

## B. Specific Gravity

The specific gravity of a soil is the ratio of the weight in air of a given volume of soil particles to the weight in air of an equal volume of distilled water at a temperature of 4°C. The specific gravity of a soil is often used in relating the weight and volume of a soil. Unit weights are needed in nearly all pressure, settlement, and slope stability problems in soil engineering.

## C. Compaction Tests

Thirty-two compaction tests were conducted at

moisture contents (% of dry weight at 105°C) varying from 12% to 98%.

The Harvard Miniature Compaction Test was used as one indication of compaction behavior. In this test, the material is compacted into a mold 1 5/16 inch diameter, 2.82 inches high, having a volume of 1/454 cubic feet. The material is placed in five layers using 25 blows of a 40 pound release spring loaded compaction device to compact each layer. Following compaction, the total unit weight and moisture content are determined and the dry density determined.

It is usually assumed that this test indicates the compaction that can be obtained in the field by sheepsfoot roller compaction.

The Standard Proctor Compaction Test was used as another indication of compaction behavior. In this test, the material is compacted into a mold 4 inches in diameter, 4.6 inches high, having a volume of 1/30 cubic feet. The material is placed in three layers using 25 blows of a 5.5 pound hammer dropped 12 inches to compact each layer. After compaction, the total unit weight, moisture content, and dry density are determined. This test simulates the compaction that can be obtained in the field by flat wheeled or rubber tired rollers.

The Proctor test is the ASTM standard test for compaction performance.

The Static Compaction Test was used as still another indication of compaction behavior. In this test, the material is compacted into a rigid walled cylinder 0.964 inches in diameter and fitted with a piston at each end. The material is placed in the cylinder and quickly loaded to 100 psi, it is then compressed at 0.1 inches per minute to a resistance of 1,000 psi, 0.05 inches per minute to a resistance of 2,000 psi, and finally the 2,000 psi pressure is maintained for 1 minute. Following extraction, the volume, total unit weight, moisture content, and dry density are determined. This test is believed to simulate the field compaction obtained by a steel wheeled roller.

## D. Unconfined Compression Tests

Fourteen unconfined compression tests were performed on samples extruded from the Harvard Miniature mold. The moisture content varied from 55.2% to 74.7% of dry weight at 105°C giving dry densities from 47.4 pounds per cubic foot to 50.8 pounds per cubic foot. Seven tests were conducted in a controlled stress device and seven were conducted in a controlled strain device.

Eleven unconfined compression tests were performed on samples extruded from the static compaction cylinder. The dry density varied from 41.0 pounds per cubic foot at 60.3% moisture content to 59.7 pounds per cubic foot at

1.6% moisture content. These specimens were tested in a ontrolled strain device.

The unconfined compression test establishes the elationship between dry density and shear strength.

## E. Triaxial Compression Tests

Four triaxial or confined compression tests were erformed on samples compacted with the Harvard Miniature ompaction device. The samples were compacted at the oisture content producing the maximum dry density. Each ample was encased in a membrane and allowed to consolidate t a different chamber pressure before it was tested. The esults of these tests give some indication of the effect of epth of burial on the shear strength.

## F. Consolidation Tests

Three consolidation tests were performed on samples ompacted below, at, and above the maximum dry density obained in the Standard Proctor Compaction Test. Each sample s contained laterally in a brass ring and a porous stone s placed on the top and bottom of the specimen. The ample, ring, and stones are positioned in the loading frame nd a small load is applied. Shortly after the initial oad is applied, water is added (enough to submerge the ample) to see if the material will swell. At each load

increment, time and vertical deflection readings are recorded. The load is applied in increments equalling the total load on the sample. Thus, the total load is doubled with the addition of each load increment. Each load increment is maintained for 24 hours.

The information obtained from the consolidation tests will help determine the expected total compression (settlement) and the time rate of compression.

## G. California Bearing Ratio Tests

The California Bearing Ratio Test is part of the method of flexible pavement design developed by the California State Highway Department. The material was tested under saturated and optimum moisture conditions at selected densities. The result was a ratio of the shearing resistance of the material to that of a standard crushed stone.

A round piston of 3 square inch cross sectional area was pressed into the soil at the rate of 0.05 inches per minute. The resistance in pounds was measured at penetrations of 0.1 inches and 0.2 inches. The California Bearing Ratio (CBR) equals the material resistance pressure divided by the standard stone resistance pressure expressed as a per cent.

Three tests were conducted at a moisture content

of 80% (of dry weight at 105°C) and dry densities from 36.0 pounds per cubic foot to 44.1 pounds per cubic foot. Each sample was compacted in a 6 inch diameter mold of 0.0819 cubic foot volume using a 5.5 pound hammer with a twelve inch drop.

# IV. RESULTS OF TEST PROGRAM

## A. Specific Gravity

The specific gravity of the solid portion of the material determined by standard soil mechanics methods was 2.1.

## B. Compaction Behavior

The results of the compaction tests are shown in Appendix B. For the Harvard Miniature compaction tests, the optimum moisture content ranged from 58% to 62% of dry weight at 105°C. The maximum dry density obtained was 50.8 pounds per cubic foot at a moisture content of 60.9 percent. The dry density had a total range of 47.4 pounds per cubic foot to 50.8 pounds per cubic foot while the moisture content varied from 55.2 per cent to 60.9 per cent.

The optimum moisture content for the Standard Proctor compaction tests ranged from 65 per cent to 72 per cent of dry weight at 105°C, and the maximum dry density obtained was 45.4 pounds per cubic foot at a moisture content of 67.8 per cent. The total range of dry densities and moisture contents was 38.6 pounds per cubic foot to 45.4 pounds per cubic foot and 42.4 per cent to 97.6 per cent respectively.

The static compaction tests gave an optimum moisture range of 20 per cent to 24 per cent of dry weight at 105°C with the maximum dry density obtained of 59.7 pounds per cubic foot at a moisture content of 22 per cent. The overall range of dry density was from 41.0 pounds per cubic foot to 59.7 pounds per cubic foot, while the total range of moisture contents investigated varied from 12.1 percent to 60.3 per cent.

## C. Unconfined Compression Behavior

The results of the unconfined compression tests are shown in Appendix C. The stress controlled samples show a greater unconfined compressive strength than the strain controlled tests; however, the results from both series of tests are similar. The curve of water content, dry density, and unconfined compressive strength (Appendix C) shows that unconfined strength seems to be independent of water content or dry density in the range of optimum moisture. Since the strain controlled tests seem to give a better indication of behavior, an unconfined compressive strength of 30 pounds per square inch or 4400 pounds per square foot at 10 per cent strain is to be assigned to this material, at optimum moisture content and maximum dry density.

The strain controlled tests conducted on the static compaction samples show that the maximum unconfined

compressive strength again occurs in the range of optimum moisture. The assigned value of unconfined compressive strength is to be 200 pounds per square inch or 28,800 pounds per square foot at 10 per cent strain and optimum moisture content giving maximum dry density.

## D. Confined Compression Behavior

The results of the triaxial compression tests are shown in Appendix D. The appearance of the stress vs. strain curves for all four tests were similar to those of the unconfined test except that the former had larger values of stress resulting from the confining pressure.

The results of these tests were analyzed with the results of the unconfined compression tests applying the Mohr-Coulomb failure criteria as is customary in soil mechanics.

The apparent angle of internal friction, $\phi_a$, with an assumed failure at 10 per cent strain, is 24.8°. For an assumed failure at 10 per cent strain, the confining pressure and the compressive strength were 15, 30, and 60 pounds per square inch and 41.8, 50.2, 85.0 pounds per square inch respectively.

## E. Consolidation Behavior

The results of the consolidation tests are shown in Appendix E. For each of the three tests is plotted the void ratio (ratio of volume of voids to volume of dry solids) versus the log of the applied pressure in tons per square foot. These curves apply to saturated material and are similar to those of compressible soil material.

The compression index ($C_c$) or slope of the steepest straight line portion of the loading curve on this semi-log plot is seen to increase with increasing compaction moisture content.

There are six plots of the time versus compression behavior for the load increments. Figure a shows the time versus compression behavior to be expected if this behavior was a strict hydrodynamic phenomenon described mathematically as follows:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \qquad (1)$$

where: $u$ = water pressure in voids (excess over hydrostatic)

$t$ = time

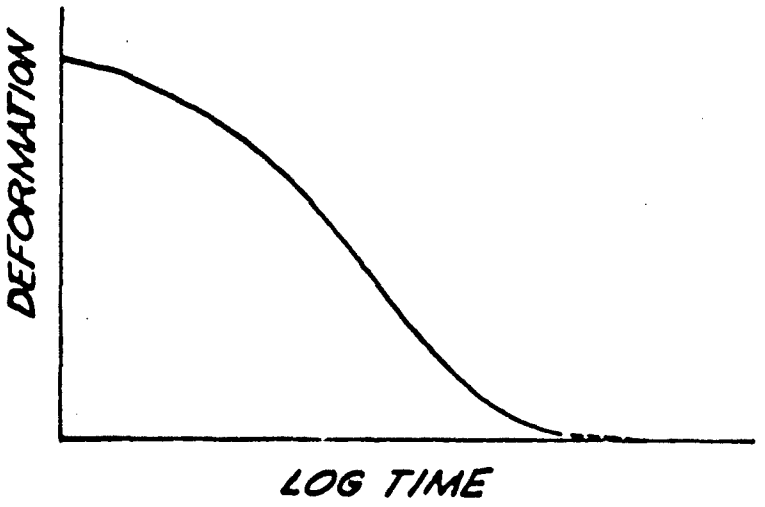
$z$ = distance from drainage boundary

$c_v$ = coefficient of consolidation (a constant)

This mathematical model has been used with fair success in analyzing inorganic soil behavior. The time-compression curves deviate so greatly from this model that

reasonable values of $c_v$ can not be obtained.

Figure a




F. California Bearing Ratio

The test results of the California Bearing Ratio (CBR) tests are shown in Appendix F. The test results indicated that the CBR value is dependant on the dry density at a constant moisture content. The dry density varied from 36.0 pounds per cubic foot to 44.1 pounds per cubic foot, and the CBR value went from 1.55 per cent to 3.86 per cent.

The samples were submerged for four days before testing with a surcharge of 115 pounds per square foot, afterwhich, they were removed for testing. All three samples had an objectionable odor when tested.

# V. DISCUSSION OF LABORATORY RESULTS

## A. General

The behavior patterns of ordinary engineering soils are not clearly understood. Soil systems consist of three phases: 1) inorganic, solid mineral grains; 2) water, either adsorbed on the surfaces of the grains or free in the void spaces and, 3) air (or other gases) existing as discontinuous bubbles or continuous air space through the soil system. The interrelationship of these three phases involves very complicated physical and chemical phenomena on which depend the observed macroscopic behavior.

The behavior of the compost material is additionally complex. The individual particles are highly organic. The particles themselves are easily deformed under the application of stresses. The size, shape, and nature of these particles will be altered by biological and chemical processes. The water in the system can be absorbed by the individual particles in addition to being adsorbed on their surfaces or free in the voids.

Thus, even the simplest engineering indices such as moisture content and specific gravity are of doubtful meaning and limited usefulness when applied to this material.

For these reasons, any attempt to describe the behavior of this material in ordinary soil mechanics

manners will be very crude at best. Probably the only good way to assess the engineering behavior of landfills and structures founded on landfills of this material is by means of rather long term large scale field tests.

The engineering behavior described herein is that of a rather small sample of the material as it existed in the laboratory at the time of testing and no study was made of the effects of biological or chemical action.

## B. Specific Gravity.

The specific gravity of 2.1 is much lower than usually encountered in a landfill material. The more common specific gravity would be 2.65 for granular soils, 2.7 for clays, and 2.6 for organic soils [1]*.

## C. Compaction Behavior

The dry density of 50 pounds per cubic foot is much lower than the usual soil fill material, which has been placed according to standard specifications. The range of soil dry densities is from 90 pounds per cubic foot for a highly plastic clay to 130 pounds per cubic foot for a well-graded sand with a small percentage of clay [5]. However,

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*Numbers in brackets designate references to be found in the bibliography.

a pumice may have a dry density of 69 pounds per cubic foot [2].

When a soil of low dry density is encountered, it is usually investigated for the particular property in question which is required for a successful design; these properties include strength, compressibility, permeability, etc.

The laboratory compacted dry density is considerably higher than the densities obtained in the ordinary sanitary landfill. The total density, not dry density, in a compacted sanitary landfill will vary from 17 pounds per cubic foot to 45 pounds per cubic foot depending upon quality of refuse, degree of compaction, and method of reporting. This range of densities is quoted in ASCE (1959). The total densities obtained in the laboratory compaction tests were from 55 pounds per cubic foot to 82 pounds per cubic foot at corresponding moisture contents of 42 to 62 per cent.

The dry density is a measure of the amount of solids material in a unit volume, thus it is a better indication of the compaction. In tests conducted in Seattle, Washington, and reported by Merz and Stone [3] on a sanitary landfill cell of a 9 foot lift of compacted refuse with no soil intermixed, the dry density was 12.8 pounds per cubic foot at a moisture content of 167 per cent. Therefore it is evident that the waste conversion end product, compacted to

a dry density of 50 pounds per cubic foot, would give a greater reduction in volume of refuse material.

## D. Strength and Stress vs. Strain Behavior

The unconfined compression samples were stronger than a typical stiff to very stiff clay. A typical stiff clay will have an unconfined compressive strength of 4000 pounds per square foot [2], whereas, the waste conversion end product gave an unconfined compressive strength of 4400 pounds per square foot. However, the deformation of the test material was very high (10 per cent strain), which leaves some doubt regarding the practical strength of the material.

The confined compression test samples were also stronger than a typical soil fill material, and the test material still had a large deformation. The angle of shearing resistance in the waste conversion end product was 24.8° which is larger than the 20° to 22° developed in a silt or silty sand, or the 14° to 20° in a remolded clay [5].

## E. Compression Behavior

The waste conversion end product had a virgin compression index ($C_c$) slightly higher than that of a Marine clay containing silt and glacial clay; and much

higher than the index of either a Boston blue clay or Morganza Louisiana clay [2]. It should be noted that the lower the compression index, the less that a material will consolidate under a load. One of the highest recorded compression indices is the 8.5 for a Newfoundland peat.

Thus this material is much more compressible than ordinary fill material or the soil upon which structures are usually founded.

# VI. ENGINEERING BEHAVIOR OF COMPOST LANDFILL

## A. Introduction

In ordinary civil engineering practice, the compost material would be rejected as a foundation material because of its highly organic nature. This study will be based on the following assumption: the behavior of the material as determined on the small sample is that of the field material and no subsequent biological or chemical action will alter this behavior.

This assumption is necessary to permit a preliminary analysis of the use of the compost material as landfill to be made. Future testing should be directed toward studies of variation of the engineering behavior with differing composition of the end product of the composting process. In addition, the variation of these properties with time as a result of chemical or biological alteration should be carefully studied.

Since this material is a rather unique material, analyses of its engineering behavior following the usual soil mechanics practices will be crude approximations at best. The behavior of the compost material in a landfill situation should be studied carefully in large-scale field tests.

## B. Advantages of Compost Material

The compost material has one great advantage: that will result in its increased utilization in situations each as described in this section. This is its availability. As more and more communities are caught with the enormity of the waste disposal problem as our population grows, they will turn to composting. While other uses of the compost material exist, the large output of such plants will more than satisfy these, and the material will be used in land reclamation and filling operations.

One additional advantage of this material as a fill material is its light weight. There are many situations in civil engineering practice which require a lightweight fill material. Backfill around buildings, behind retaining walls and fills on slopes often require lightweight material for stability purposes. Crushed slag is commonly used for lightweight fill and is quite expensive.

## C. Definition of Problem

In order to gain insight into the behavior of the compost material, a particular typical situation has been chosen for study. The particular problem chosen has been defined as follows:

1. The total height of the finished landfill is 20 feet. This thickness of fill has been chosen to

permit more direct comparison with the Seattle tests reported in Merz and Stone [3].

2. The material is placed at a rate of 3 feet per month. Thus, 6 to 7 months are required to place this fill.

3. The material has the properties in place as determined in the previously described laboratory tests.

4. The underlying soil material presents a good foundation condition.

The following aspects of this problem will be considered:

1. Method of placement.

2. Trafficability of the fill at all stages.

3. Settlement of the surface of the fill with time following placement.

4. Bearing capacity, settlement, and foundation treatment for structures on the completed fill.

5. Pavements for highways or airfield runways

6. General considerations for supporting utilities.

7. Slopes terminating the fill area.

## D. Method of Placement

In order to utilize as large a quantity of the compost product as possible and to obtain the most favorable engineering properties, the filling operation should be controlled so as to obtain the greatest in place dry density.

Best compaction in the laboratory was obtained with static compaction. This suggests that a large, heavy flat wheeled roller would produce the best field compaction. The high unconfined compressive strengths obtained in the static compaction tests indicate the feasibility and desirability of the use of such equipment. Field investigation alone will resolve this question. In lieu of such equipment, a sheepsfoot roller ballasted to capacity should be used.

The initial site preparation should consist of clearing and grubbing. All topsoil should be removed and the fill started on a firm undisturbed layer. Other initial preparations may be necessary in less suitable land reclamation areas.

The material should be placed in layers not exceeding 8 inches in compacted thickness. The compaction process should be conducted so that the compacting equipment makes four or more passes over any given point on the fill for each lift.

The moisture content should be controlled to fall within the optimum range as indicated by the compaction tests. As the filling operation proceeds, field determinations of inplace densities will yield data supplementing the laboratory tests in establishing field control of the filling operation and field moisture content.

The fill should be terminated with a three foot layer of cover material. This cover should be: 1) non-frost susceptible in areas where this is important; 2) strong enough to distribute imposed surface loads; 3) flexible to preclude cracking if fill settles differentially; and, 4) impermeable to prevent surface and rain waters from penetrating the fill.

One such possible cover layer would be compost material to which has been added sufficient clayey binder material to render the mixture impermeable and to supply the necessary load distributing ability.

In areas of heavy rainfall or other water conditions, provisions for drainage should be incorporated into the landfill. These would have to be designed for the particular situations encountered but could take the form of an initial layer of graded sand and occasional horizontal and possibly vertical graded sand drainage layers.

## E. Trafficability

The fill will be trafficable to the standard caterpillar tread tractor pulling a sheepsfoot roller. The fill will probably support a flat wheeled roller, but this can only be conclusively determined in field tests.

Dump trucks of 10 ton capacity will probably be supported by the fill. Analyses based on the unconfined

compressive strengths from the Harvard Miniature compaction samples indicate that this is a borderline situation. One could most probably drive an automobile over the compacted surface of the fill without becoming stuck.

Rainfall saturating the surface of the fill complicates this picture. Suspension of filling until the fill surface becomes sufficiently dry will most likely be necessary, unless some provision is incorporated to protect the exposed surface of the fill during placement.

## F. Settlement of Fill Surface

The deviation of the time-compression behavior observed in the laboratory from that of ordinary soil material makes any prediction of the settlement behavior of the fill surface very doubtful. However, various bounds can be drawn:

1. The ultimate surface settlement will not exceed 20 inches. This will occur at a time exceeding several human lifespans.

2. An upper boundary on the time settlement behavior can be obtained by linearly projecting laboratory behavior to the field situation. If the material behaved according to the ordinarily assumed hydrodynamic model, this projection should be made on a square of the thickness ratio basis (i.e., time in field = time in laboratory $\times \left(\frac{\text{Field thickness}}{\text{Laboratory thickness}}\right)^2$)

An analysis based on a simple extrapolation of laboratory behavior seems much more realistic for unsaturated

compacted compost material. Figures 1 and 2 in Appendix E present summaries of the load vs. compression and time vs. compression behavior of compacted compost material. The time-compression behavior was extrapolated according to the equation appearing in Figure 1 of Appendix E.

The results of such an analysis are presented in Figure 1. This figure indicates a settlement of 14 inches occurring 100 years after fill placement.

This time-settlement behavior agrees roughly with that observed by Merz and Stone [3] in Seattle. This settlement is large but not prohibitively large. Various procedures can be employed to minimize the effects of this settlement:

1. Allowing the fill to "season" for one year would allow about 1/3 of this settlement to occur harmlessly.

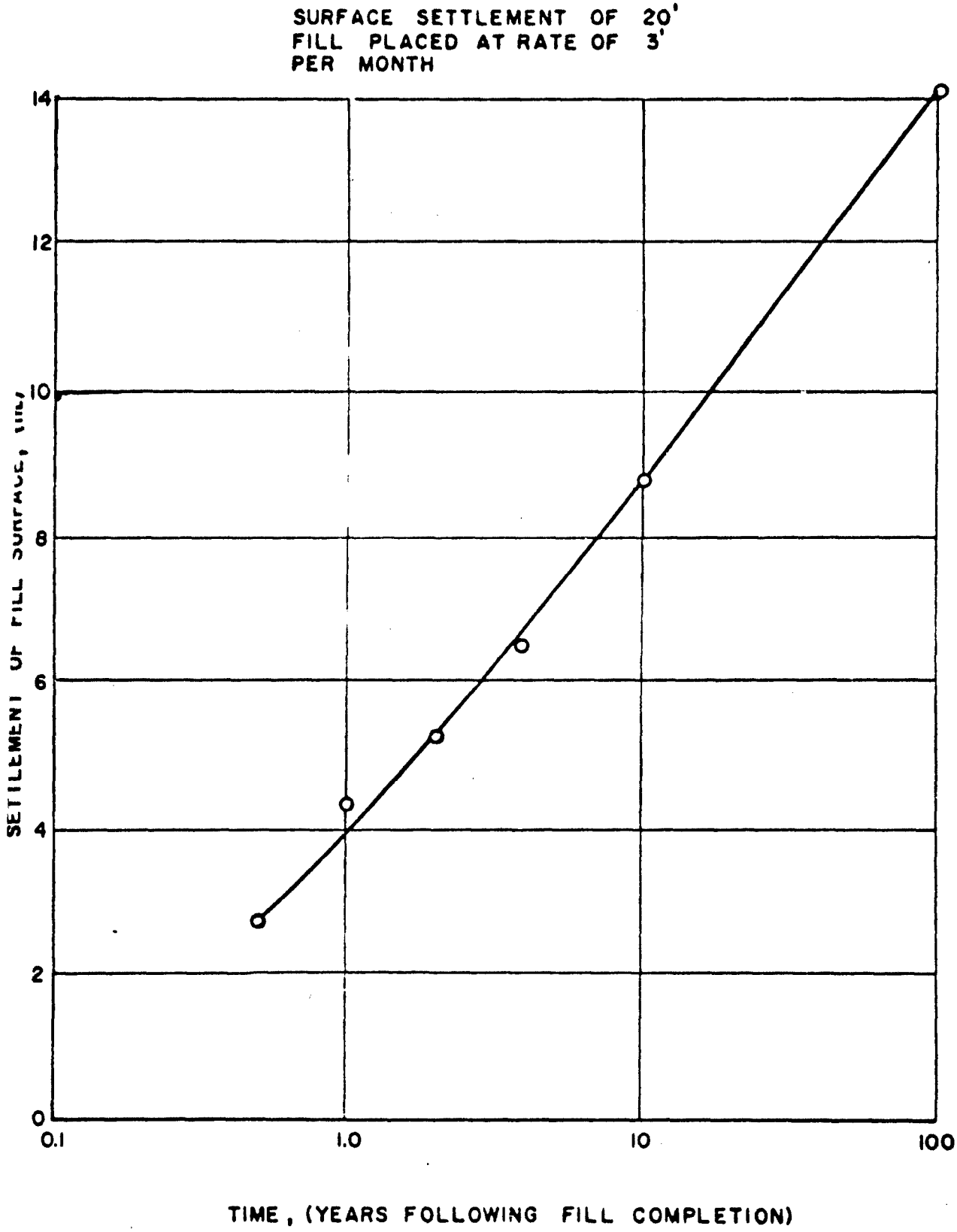
2. Utility connections can be designed to tolerate this settlement.

3. Using flexible paving cross sections for roadways, etc.

## G. Foundations on Fill

Following completion of the filling operation, a reclaimed area exists. Future use of this reclaimed area requires construction of various types of structures and their service facilities (utility lines, roadways, parking lots, etc.). This section studies the structural foundation

FIGURE 1


SURFACE SETTLEMENT OF 20' FILL PLACED AT RATE OF 3' PER MONTH
14
12
10
8
6
4
2
0
0.1
1.0
10
100
TIME, (YEARS FOLLOWING FILL COMPLETION)

treatment necessary to safely and economically support a building on the completed landfill.

It is assumed that the completed landfill has the idealized cross section shown in Figure 2. Three possible foundation designs for structures are shown in Figure 3. These represent the three basic foundation treatments. Variations and combinations of these are, of course, possible.

The first, illustrated schematically in Figure 3a, is to found each wall, column, or other structural element on its individual "spread" footing. The second involves carrying the load through the fill into the underlying material. Drilled in and belled out concrete caissons are shown in Figure 3b. However, the nature of the underlying subsoil will dictate whether steel or reinforced concrete piles, drilled in caissons without a bell, or belled caissons as shown will be the most economical. The third foundation type illustrated involves one large continuous foundation element for the entire building.

1. Spread Footings

Spread footings must satisfy two main design criteria: 1) they must have an adequate factor of safety against a complete punching type shear failure; and, 2) they must not experience settlements during the life of the structure of sufficient magnitude to cause a loss

FIGURE 2

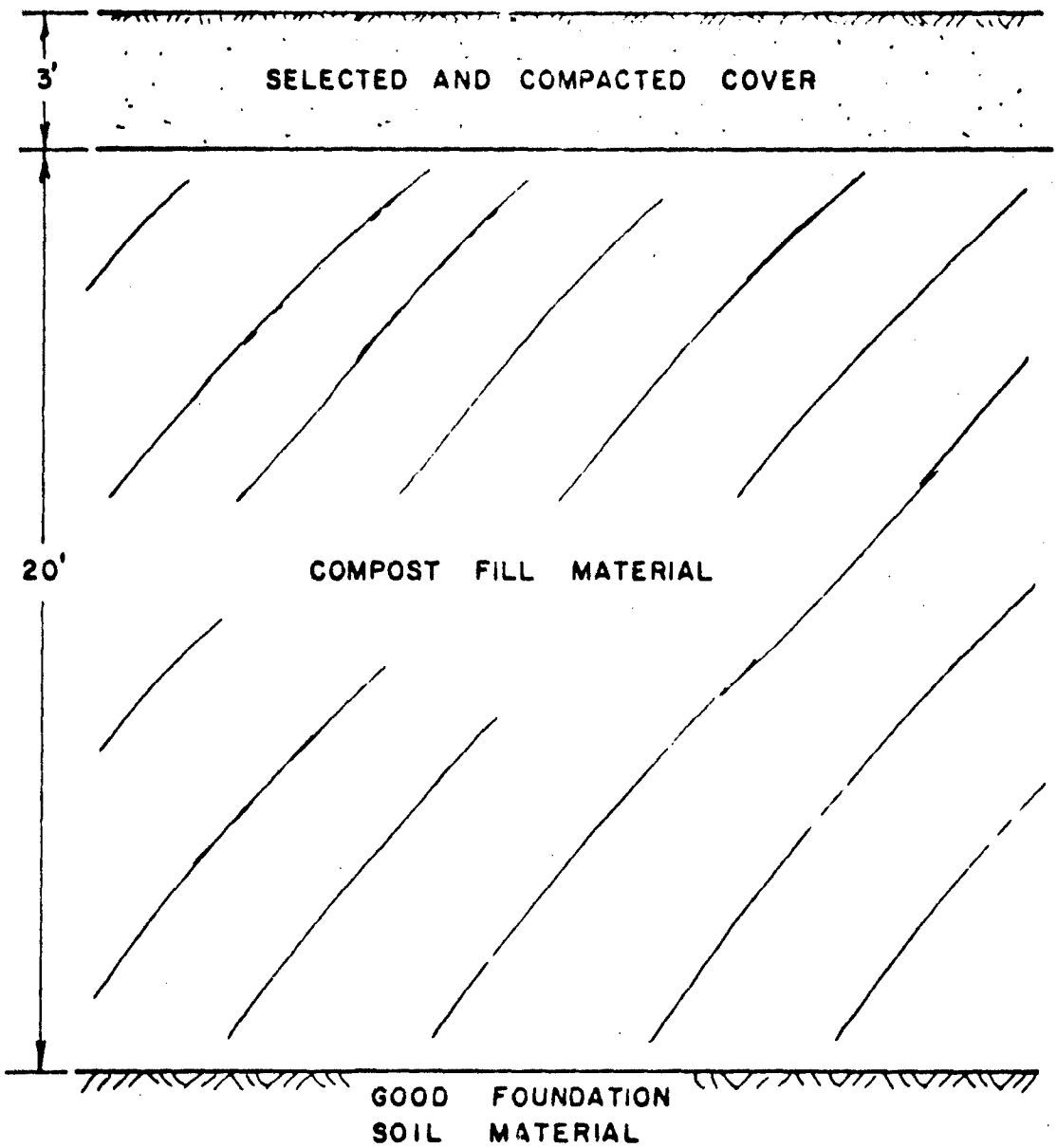
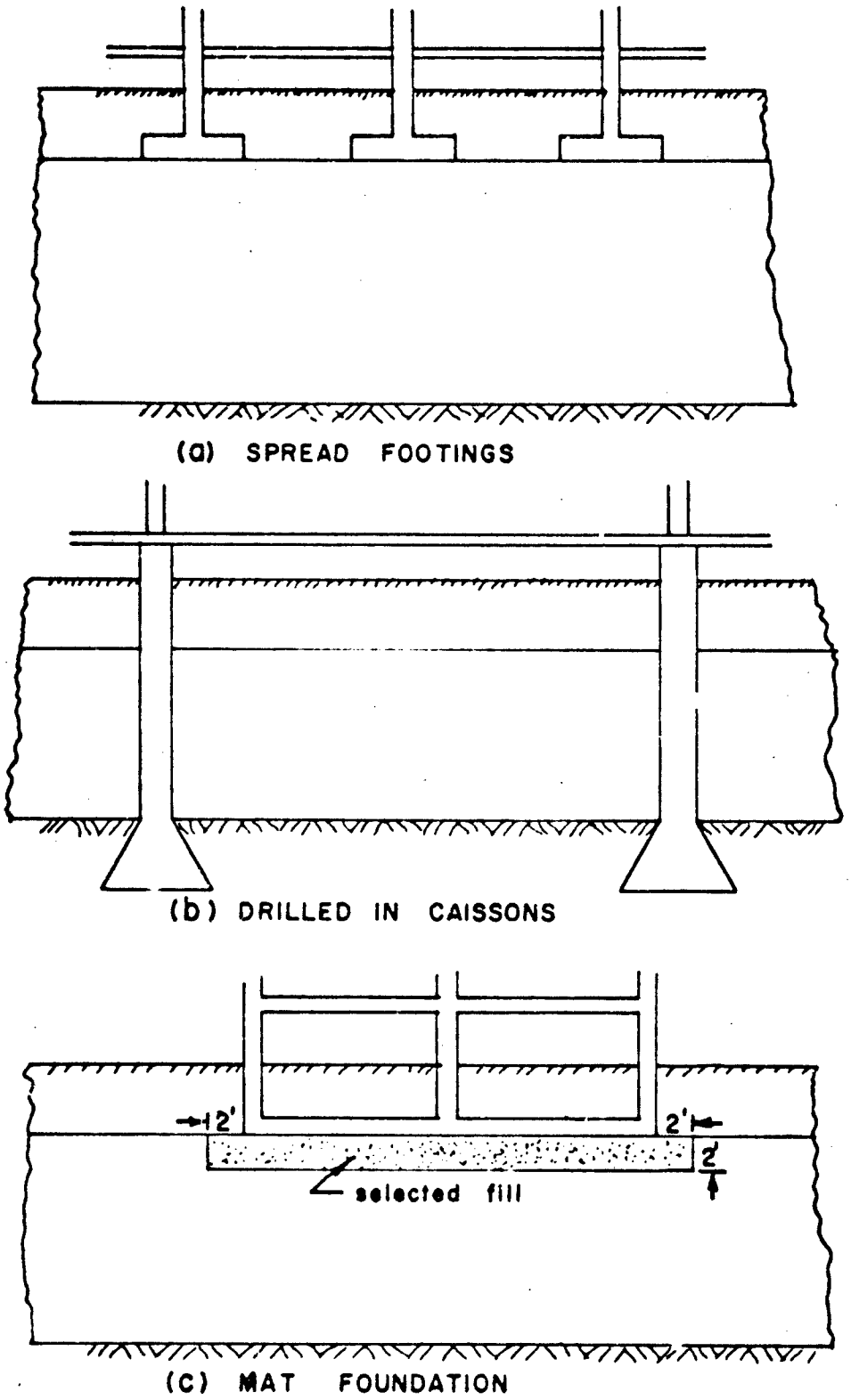

3'
SELECTED AND COMPACTED COVER
20'
COMPOST FILL MATERIAL
GOOD FOUNDATION
SOIL MATERIAL

FIGURE 3


(a) SPREAD FOOTINGS
(b) DRILLED IN CAISSONS
2'
2'
2'
selected fill
(c) MAT FOUNDATION

usability of the structure. This last criteria is rather d to define. Certain existing structures are known to e experienced large settlements and are still fulfilling ir design function. A balance of the cost of larger or ferent foundation types versus the cost of repair and ustment of settlement damage is required.

Current foundation engineering practice in the U.S.A. ally requires a factor of safety of three against a plete shear failure and limits the differential settlement ween adjacent foundation elements to 0.75 inch maximum. assumption is often made that if the settlement of any foundation element is one inch maximum, the last uirement will be satisfied.

Bearing Capacity. The bearing capacity or shear failure blem is a variation of the Prantl punching problem. In l mechanics practice Terzaghi's solution to this problem usually employed [5].

For a square footing, Terzaghi's bearing capacity ation reduces to:

$$q = 1.3cN_c + \gamma_a D_f N_q + 0.4\gamma_b B N_\gamma \qquad (2)$$

where:

$q$ = ultimate bearing capacity
$\gamma_a$ = unit weight of material above base of foundation
$\gamma_b$ = unit weight of material beneath base of foundation
$B$ = width of footing
$D_f$ = depth to base of footing
$N_c, N_q, N_\gamma$ = bearing capacity factors

Using the results of the triaxial compression tests at 10 per cent axial strain shown in Appendix D, $\phi$ can be taken as 15° and C as 1,440 pounds per square foot. Assuming the following:

$N_c$, $N_q$, $N_\gamma$ = 9, 3, and 1 respectively

$D_f$ = 3 feet

$\gamma_a$ = 120 pounds per square foot

$\gamma_b$ = 50 pounds per square foot

$$q = 1.3 \times 1440 \times 9 + 120 \times 3 \times 3 + B \times .4 \times 50 \times 1$$

Therefore the ultimate bearing capacity is:

$$q = (16850 + 1080 + 20B) \text{ pounds per square foot}$$

Thus for a factor of safety of 3:

$$q(\text{allowable}) = (5620 + 360 + 7B) \text{ psf.}$$

Table 1

| B in ft. | q(allowable) in psf. |
|---|---|
| 3 | 6000 |
| 5 | 6015 |
| 7.5 | 6032 |
| 10 | 6050 |

Thus an allowable bearing capacity of 3 tons per square foot would be safe against a shear failure. This value compares favorably with that of a more conventional fill material. However, criteria for settlement has not been satisfied.

b. Settlement. In order to investigate the settlement behavior of a spread footing on the surface of the compost fill material, a particular case will be assumed as follows:

School building with columns spaced 15 feet on centers E-W and 20 feet on centers N-S

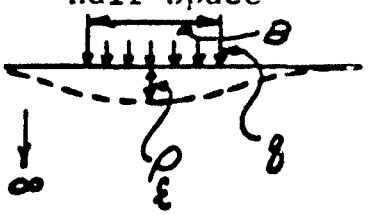
| | | |
|---|---|---|
| Loading: | slab load | = 75 psf |
| | roof | = 20 psf |
| | snow | = 40 psf |
| | live load | = 100 psf |
| | Total | = 235 psf |

A typical column load = 15 x 20 x 240 = 72,000 lbs or 36 tons

A footing 36/3 = 12 square feet or 3 feet by 4 feet will satisfy the bearing capacity criteria established in the preceding paragraph.

One boundary to the settlement problem can be obtained by the elastic theory. The solution of the problem of deflection of the surface of the surface of an elastic half space due to an applied-uniformly distributed surface loading as shown below.

Figure 4
Elastic
Half Space




$$\rho_\ell = \frac{0.8qB(1-\nu^2)}{E}$$

$\rho_\ell$ = Deflection at center of footing
q = Average contact pressure
E = Young's modulus - assumed as 57.6 k/ft$^2$
v = Poisson's ratio - assumed as 0.5
B = Footing width

The value of Young's modulus was assumed after a careful study of the unconfined compression stress versus strain behavior presented in Appendix C. Figure 5 presents a plot of footing settlement versus width of footing for a column load of 72 kips (one story building), 150 kips (two story building),

and the previous assumptions. Thus, if the elastic settlement is limited to 1 inch, a 10 foot square footing would be needed for the one-story school building. This would ordinarily be an economically unacceptable solution to this problem.

The elastic settlement is one deformation occurring at the time of initial loading. If the structure loading is predominantly dead load, its construction can be controlled so that the elastic settlement is "built out" during erection.

Additional settlement will occur with time after the building is completed. This settlement cannot be "built out" and, if excessive, will give rise to structural damage. An idea of this component of settlement can be obtained by looking at the time versus settlement behavior of an individual footing. The maximum economical footing size for a one-story school building of the type considered is six feet square. Assuming an average operational load of:

Slab = 75 psf
Roof = 20 psf
Bldg. dead load = 15 psf
Avg. live load = 1/2 floor + 1/2 snow = 70 psf

Total = 180 psf

Gives a column loading of 15 x 20 x 180 = 48 kips and a contact pressure of 48/6 x 6 = 1.33 ksf. Using the plots of one day strain versus log applied load and per cent of one day strain versus log time in Appendix E, the following settlements were computed.

# FIGURE 5

## ELASTIC SETTLEMENT VS FOOTING SIZE

ONE STORY BUILDING —— P = 72,000 lbs.
TWO STORY BUILDING —— P = 150,000 lbs.

SQUARE FOOTING — B x B

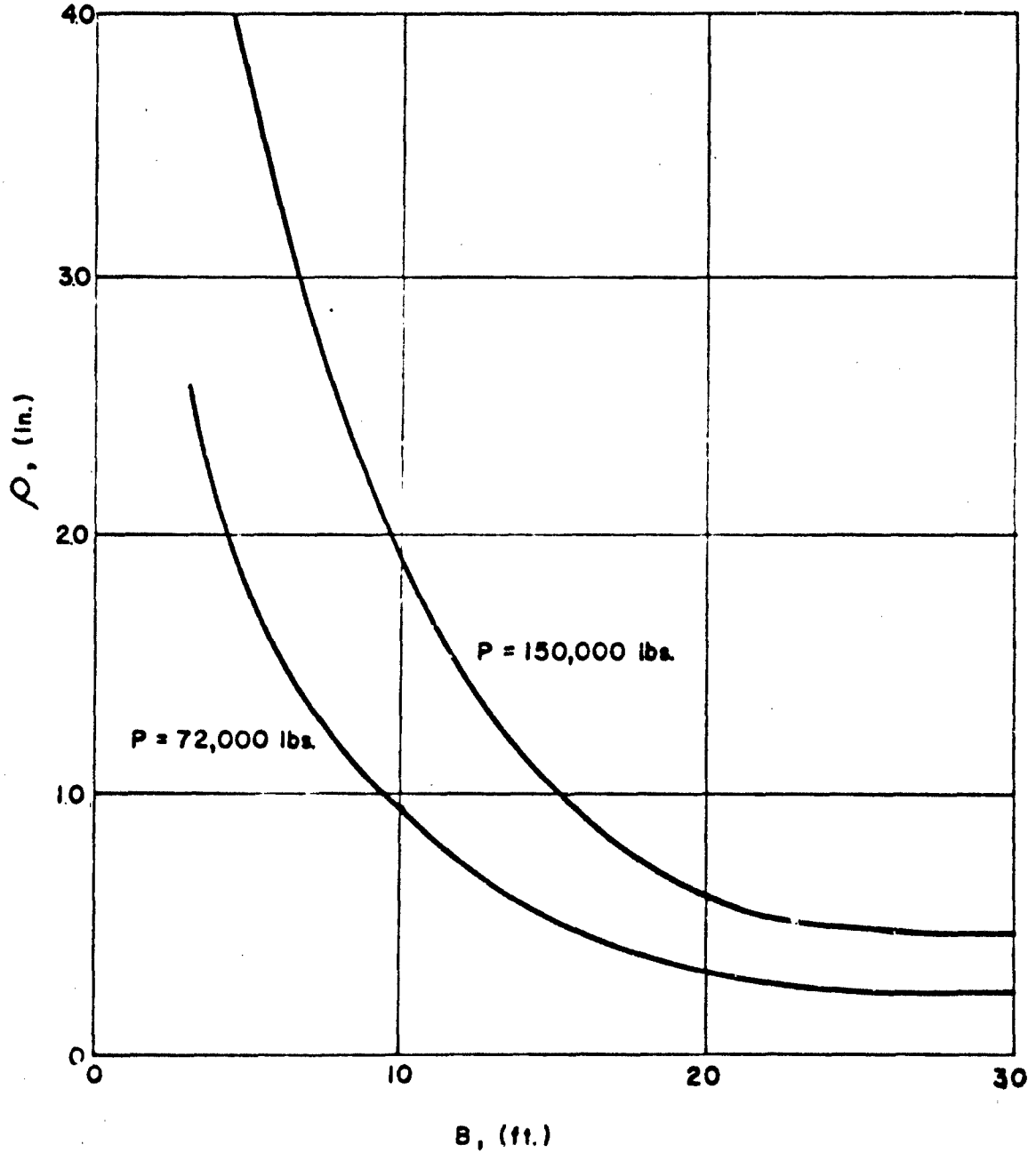


Table 2

Settlement of footing of a 1 story school (6'x6' footing)

| Time after Completion | Settlement (inches) |
|---|---|
| 6 months | 8.0 |
| 1 year | 8.5 |
| 5 years | 9.1 |

The settlement occuring in the first six months of building occupancy is many times that considered tolerable and the building would experience differential settlements rendering it inoperative during this period. Thus spread footings would not be an acceptable solution to this problem.

2. Drilled in Caissons

Assuming good underlying material affording a reasonable supporting value within six feet of the base of the landfill, drilled in caissons would probably be an economical solution to this foundation problem. Two problems would have to be considered in such a foundation design.

a. Negative Skin Friction or Down Drag. The caisson would have to be formed inside a cased hole since the consolidation or settlement of the fill would otherwise transfer a sizeable portion of the fill weight onto the

isson. An augered hole 2 or 3 inches larger than the isson diameter should be cased with a steel lining tending to within 2 or 3 feet of the base of the fill. e caisson should be formed in a cardboard lining within is casing thus allowing an inch wide void annulus to round the caisson.

Utilities. The utilities and other services have to designed with flexible connections at the building since e surrounding fill will settle several inches with respect the building.

If the underlying foundation material is poor, pre-st concrete or steel piles should be considered. Here ain the problem of negative skin friction or down drag l. play an important role.

Mat Foundation

An alternate foundation type for a one story build-g would be a compensating mat or raft foundation. Such foundation would consist of a rigid concrete mat rein-rced in two directions having the dimensions of the build-g and designed to carry the total building load. This ft should have its base 3 feet beneath the surface of the pping layer at the level of the top of the fill. However, ie upper two feet of the compost material should be removed id replaced with selected compacted fill with lateral

dimensions 2 foot in each direction beyond the perimeter of the mat as shown in Figure 3c.

Such a mat could carry a loading of dead load plus average live load of 220 pounds per square foot over the building area with no increase in load on the compost material. This is computed as follows:

| | |
|---|---|
| 3 foot cover material at 120 pcf | = 360 psf |
| 2 foot compost at 50 pcf | = 100 psf |
| | 460 psf |
| Replacement of 2 foot fill at 120 pcf | = 240 psf |
| Total available loading | 220 psf |

Thus, such a foundation can be designed to safely carry a one-story building such as the school building previously described or a light two-story building.

A building on a compensated raft foundation will have the advantage of settling at the same rate as the surrounding fill surface. If the raft is correctly designed structurally, slight differential settlements will cause only a possible slight tipping of the structure with no cracking or interference with normal structure function.

However, the building location should be so chosen as to insure a reasonably uniform depth of fill beneath the base of the mat.

## II. Pavement Cross Sections for Highways and Airport Runways

The expected surface settlement of the fill surface ›uld seem to preclude the placement of rigid concrete pave- ːnts on the landfill.

The soaked CBR value of 3 per cent obtained in the ːst at maximum dry density is the lower limit of all the ːsign procedures for flexible pavements for all purposes. ›wever, this value is extreme since the fill material will ː protected from soaking. Using this CBR value of 3 per ːnt, one can arrive at reasonable design cross sections ›r flexible pavement for parking lots, roadways, and air- ›rt aprons, taxiways, and runways for any class vehicle or .rcraft.

The design of a modern, multilane expressway over ıch a fill is possible. Such conditions would not be as ːvere as encountered in the Jersey Meadow section of the ːw Jersey Turnpike. However, such a situation should be ’oided if at all possible since high maintenance costs ıd rather unpleasant and somewhat dangerous surface ıdulations will result from differential settlements.

For similar reasons, portions of modern jet air- ’aft runways passing over such a landfill should be ːsigned as founded on the underlying material. However, .rport sites are usually in poor foundation areas. The

main runway at Logan Airport in Boston has settled an amount in excess of 18 inches since construction in 1947.

Ordinary asphalt roadways and parking lots can be built on a landfill of this material as can secondary airport runways, taxiways, and aprons. The cross section of an airport runway for aircraft up to and including loadings equivalent to DC3 type aircraft is shown in Figure 6. Channelized traffic and touchdown areas would need additional treatment. Maintenance costs for such pavements would probably be slightly higher than ordinarily anticipated.

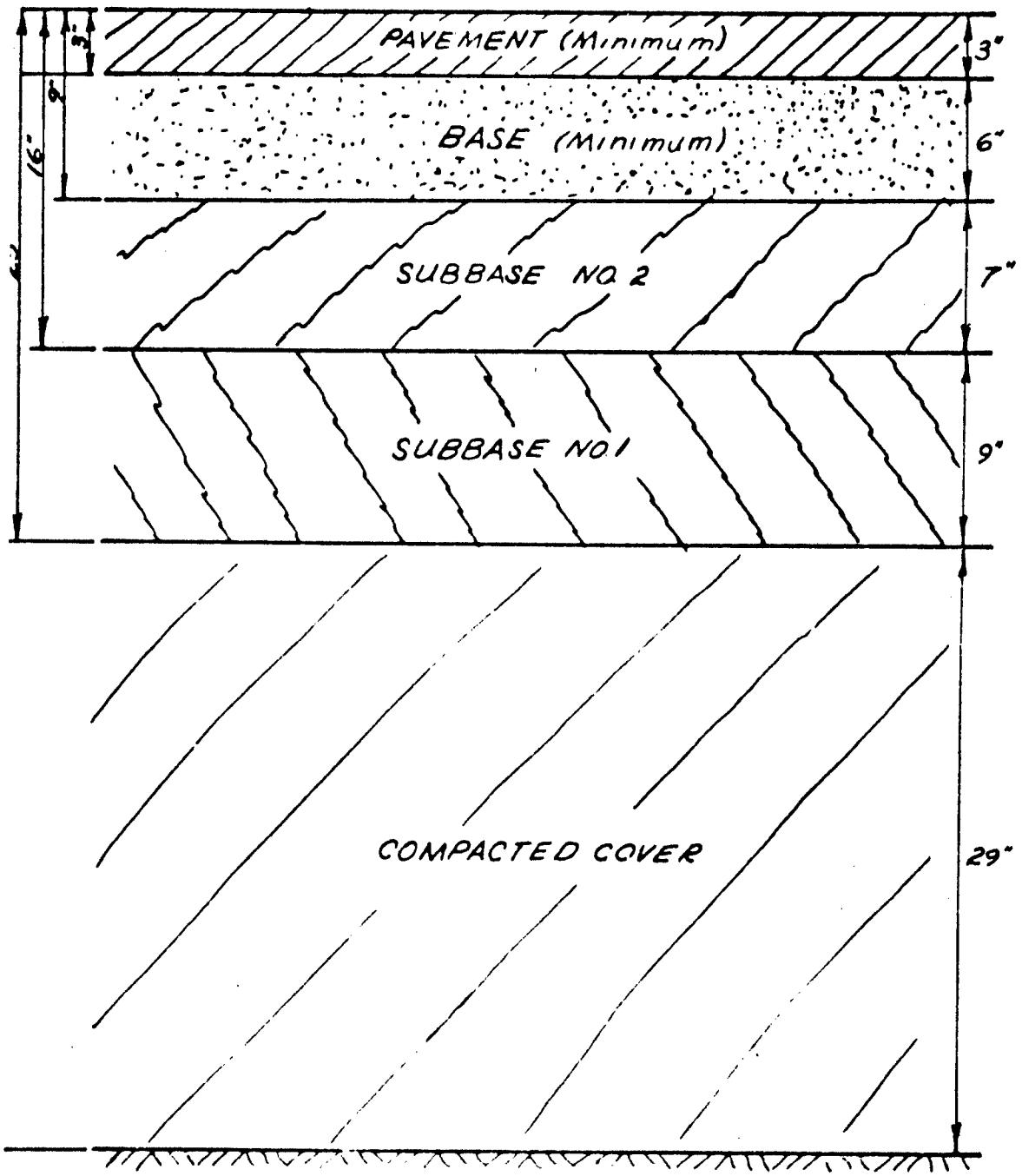
## I. Stability of Slopes

Slopes terminating landfills of this material will be stable if protected with a compacted cover against erosion and drying. Such a slope would be stable at 1 1/2 horizontal to 1 vertical to a height of 100 feet.

If excavation in the fill is necessary, vertical side walls can be obtained without bracing to depths of 50 feet.

If slopes of the material are either permitted to dry or become completely saturated they will slough off and come to rest at approximately a 24° slope.

# FIGURE 6


PAVEMENT (Minimum)
3"
BASE (Minimum)
6"
SUBBASE NO 2
7"
SUBBASE NO 1
9"
COMPACTED COVER
29"
COMPACTED COMPOST FILL

# VII. SUMMARY AND CONCLUSIONS

## A. Introduction

This section contains conclusions based on:

1. An attempt to investigate the behavior of the compost material and to analyze engineering situations based on this observed behavior utilizing relatively standard soil mechanics techniques.

2. Intuition gained from working with the compost material in the laboratory and from experience with soil mechanics problems.

Thus the following conclusions can only be taken as indications of expected behavior and as guides to utilization of the compost material. These conclusions are subject to either proof or adjustment following observations and experience with actual field situations.

## B. Conclusions

### 1. Fill Material

The compost product can be successfully utilized as a fill material. As such it is much superior to ordinary sanitary landfill.

### 2. Strength

While not the best landfill material, the compost is reasonably strong and has two distinct advantages: 1) its availability, and 2) its light weight.

3. Placement

Landfills of this material should be placed in 8 inch layers compacted to maximum dry density in the range of optimum moisture content. A fully ballasted sheepsfoot roller will probably be the best compaction equipment to use. Filling should be carefully controlled and continual checks should be made of the dry densities actually obtained in the field.

4. Cover

Landfills should be capped with a suitable covering.

5. Slopes

Slopes terminating the landfill will be stable at 1 1/2 horizontal on 1 vertical if properly protected.

6. Settlement

The major disadvantage of this material as landfill, neglecting its organic nature, is its compressibility. The surface of a completed landfill will ultimately settle from 3 per cent to 8 per cent of the total thickness. The capacity of the fill to support applied loads is limited by the settlement of the loaded area.

## 7. Foundation Types

Suggested foundation types for structures on compacted compost fill are as follows: 1) secondary structures (garages, etc.) on spread footings with prepared selected fill replacement and slab on grade; 2) one story and light two story buildings on compensating mat or raft foundations; 3) larger buildings (important machinery, etc.) on caissons or piles through fill.

## 8. Pavements

Roadways and runways for medium weight aircraft can be constructed on the fill. However, inportant highways and main airport runways would best be founded on the underlying material.

## 9. Trafficability

As the fill is constructed, it will probably be trafficable to all but the heaviest trucks and construction equipment.

# VIII. RECOMMENDATIONS FOR FUTURE STUDY

## A. Laboratory Study

### Chemical Analysis

Investigations of the nature, magnitude, and rate
: the biological and chemical processes acting to
ıcompose the material, to alter its engineering properties,
: to produce other undesirable or objectionable effects.

### Composition

Investigation by simple index tests of the
ıriability of engineering behavior with variations in
ɔmposition of the compost material. The Harvard Miniature
ɔmpaction and unconfined compression tests probably
ɔuld be reasonable indices of engineering behavior.

### Biological Degradation

Investigation by the same simple index tests of
ıe effect of chemical or biological degradation of the
ıterial upon its engineering behavior.

### Permeability

Investigation of the hydraulic behavior (permeability)
f the material would be useful. It is felt that first

uses of the material as landfill should occur in situations where the material was placed above permanent ground water level and protected from flow or infiltration of water. However, future use may dictate the utilization of the material in situations where water will be flowing through the material.

## B. Field Tests

It is felt that the next logical step in studying the engineering behavior of the compost material would be in a rather large scale field situation. Probably the most direct approach would be to construct a landfill of rather limited horizontal extent and a total height of about 20 feet with sections placed with different types of compaction equipment.

This landfill should be carefully controlled with moisture content and density measurements being continually made. The trafficability of the fill at all stages to various types of vehicles could be studied.

The landfill should be carefully instrumented with settlement measuring devices. Studies of settlement versus time should be carried on continuously after completion of the fill.

Various sizes and shapes of loaded areas instrumented to record deflections should be built and loaded on the

npleted surface of the fill. Thus the load carrying pacity of the fill can be evaluated.

One or two bored caissons with load cells at their ses could be installed through the fill to evaluate the gnitude of the load that such members will receive rough negative skin friction or down drag as the fill nsolidates.

The fill should be instrumented at various pths and horizontal locations with thermistors to record y variations in temperature throughout the fill dicating nonuniform, unusual, or unexpectedly severe ological activity. Cores can be taken from various cations if such activity seems to be important. Gas llection installations could also be provided.

Various smaller scale field tests could be made circumstances do not permit a test program of the scope tlined above. For example, load carrying ability and ttlement characteristics could be studied on material mpacted in a 10x10x10 foot test pit excavation or re desirably in a 10 foot diameter x 10 foot high steel nk sunk into the ground with greased inside walls. Field ological and chemical degradation could be studied in terial compacted in a 3 foot diameter hole bored in the ound and instrumented with gas collection and temperature cording devices at various levels.

# APPENDIX A

## Description of Laboratory Tests

1. Compaction Tests
   a. Proctor Test
   b. Harvard Miniature Test
   c. Static Test
2. Unconfined Compression Test
3. Triaxial Compression Test (Confined)
4. Consolidation Test
5. California Bearing Ratio Test

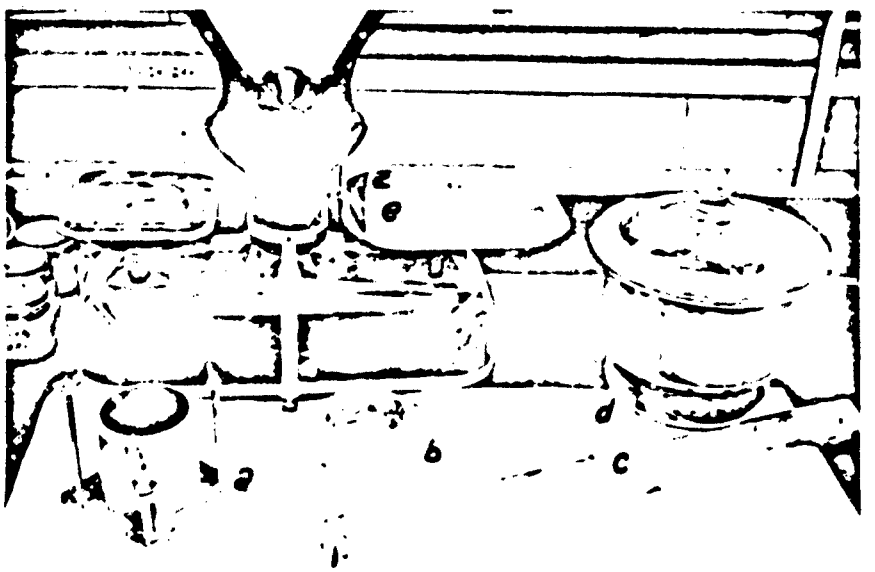
## COMPACTION TESTS

The purpose of the laboratory compaction test is to etermine the relationship between water content and dry ensity in a compacted material. The water content that gives ne maximum dry density is the optimum water content.

### Proctor Compaction Test

The equipment used in this test is shown in the igure below.

Figure 1-COM



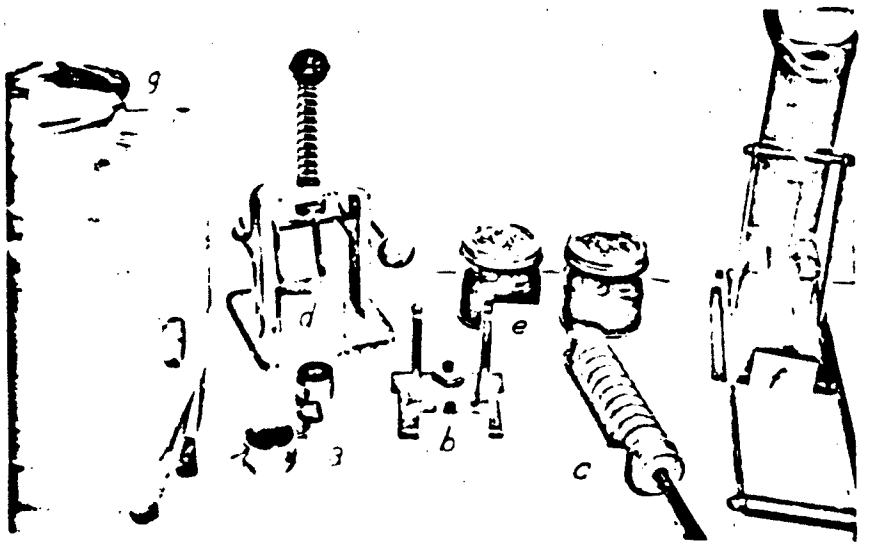

a. Mold and stand; b. collar; c. compaction hammer; d. tempered soil; and e. scale

This test was conducted in accordance with the pecifications of ASTM D698-58T, using Method A to prepare ne sample.

## Harvard Miniature Compaction Test

The equipment used in this test is shown in the figure below.

Figure 2-COM




a. Mold and collar; b. mold stand; c. tamper; d. collar remover; e. tempered soil samples; f. sample extractor; and g. scale

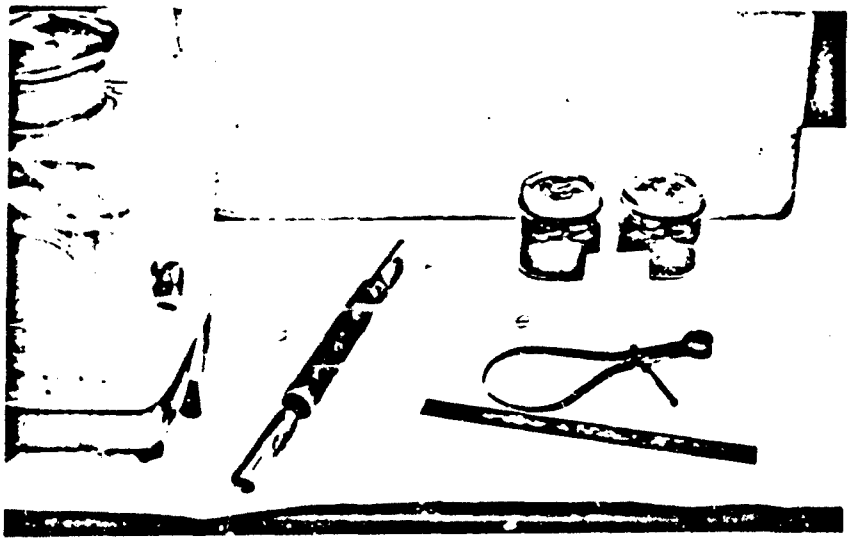
This test uses a mold which is 1 5/16 inches in diameter and 2.816 inches long; which yields a volume of 1/454 cubic feet. (The weight of the sample in grams is numerically equal to the density in pounds per cubic foot.) The mold and collar are clamped into the mold stand and the soil compacted in five layers with 25 "blows" per layer. The tamper is pushed down just until the spring begins to compress, (constitutes one blow); where the spring is set to begin compression at 40lbs.. After compacting, the collar is removed and the sample is trimmed

off at the top of the mold. The weight of the mold and sample less the weight of the mold gives the weight of the sample. The sample is then extracted from the mold for further testing, i. e., water content, unconfined compression test, etc.. This test was conducted in accordance with Soil Testing for Engineers, T. William Lambe, John Wiley & Sons Inc., 1951.

## Static Compaction Test

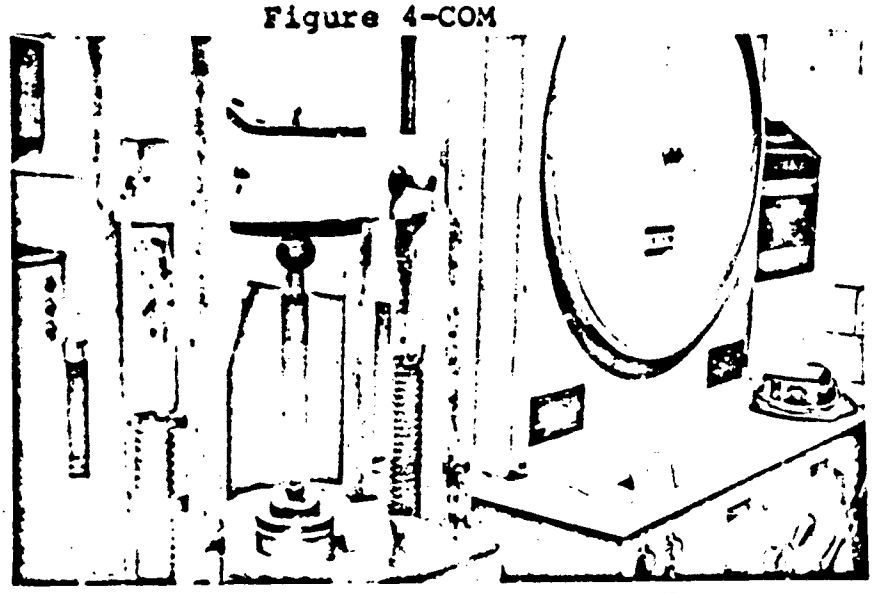
The equipment used in this test is shown in the figures below.

Figure 3-COM



a. Rigid walled cylinder; b. one of two pistons; c. tempered soil; d. scale; and e. calipers and scale.

Figure 4-COM



a. Sample in cylinder, positioned for compaction;
b. testing machine

The material is loosely placed in the cylinder and the device is positioned in the testing machine. The sample is compressed at a relatively high rate until 100psi. resistance is reached, compressed at the rate of 0.1 inch per minute until 1000 psi. is reached, 0.05 inch per minute until 2000 psi. is reached, and the 2000 psi. is maintained for one minute.

After compaction, the sample is extracted from the cylinder, measured, and weighed. The sample can then be used for further testing, ie., water content or unconfined compression test.

This test was conducted in accordance with Soil Testing for Engineers, T. William Lambe, John Wiley & Sons Inc., 1951.

## UNCONFINED COMPRESSION TEST

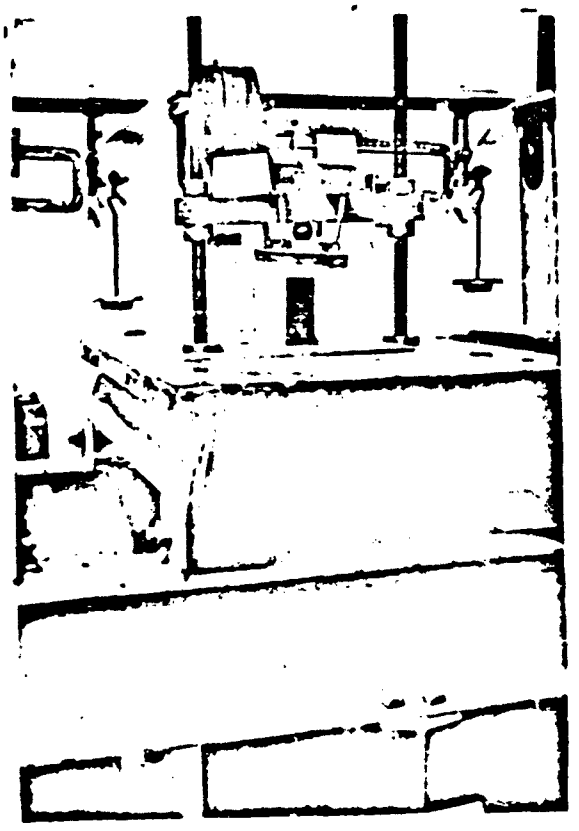
The unconfined compressive strength of a material is a measure of its consistency. (Consistency denotes the degree of firmness of the soil, and is indicated by such terms as soft, firm, and hard.) This test is the simplest and quickest laboratory method commonly used to measure the shear strength of a cohesive soil.

This test was conducted in accordance with ASTM D2166-63T; however the unconfined compressive strength was taken at the maximum or at 10% strain whichever came first instead of the maximum or 20% strain, and the minimum diameter of specimen, (1.3 in.), was not acheived in the static compaction samples.

### Stress Controlled

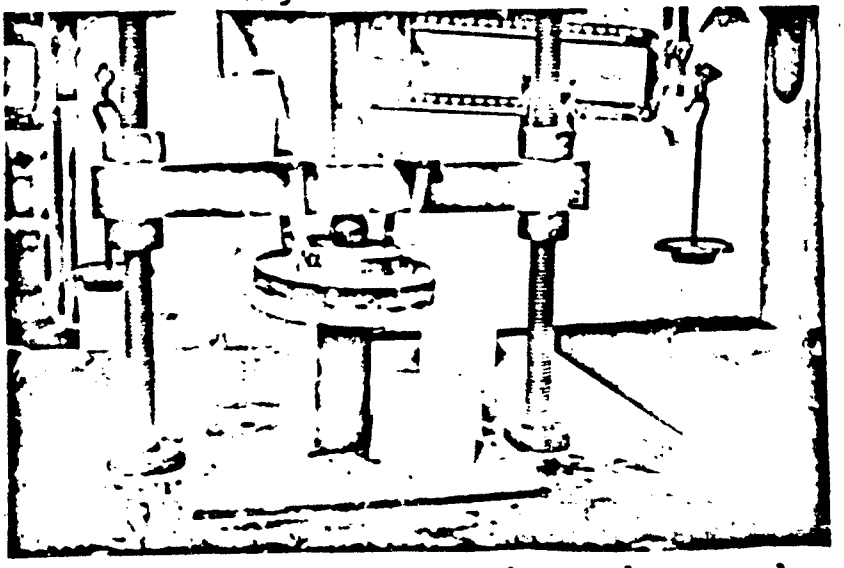
The equipment used in this test is shown in the figures on the following page.

Figure 1-UC



a. Sample; b. loading head; c. scale; and d. loading wheel

Figure 2-UC

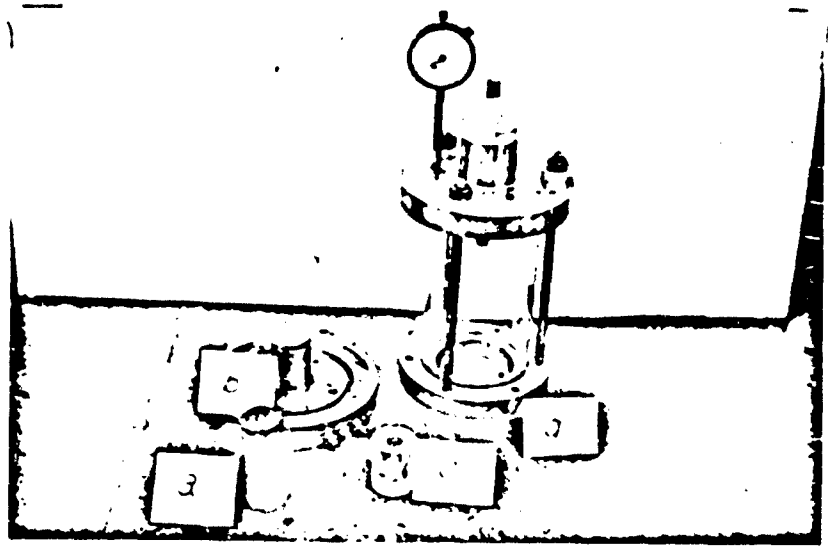


a. Sample; b. loading head; and c. scale

Strain Controlled

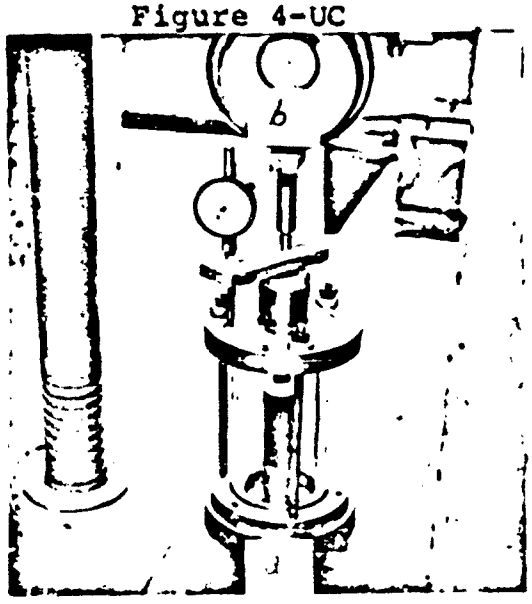
The equipment used in this test is shown in the figures below.

Figure 3-UC



a. Sample; b. base of loading device; c. loading block; and d. top of loading device with loading piston and deflection dial

Figure 4-UC



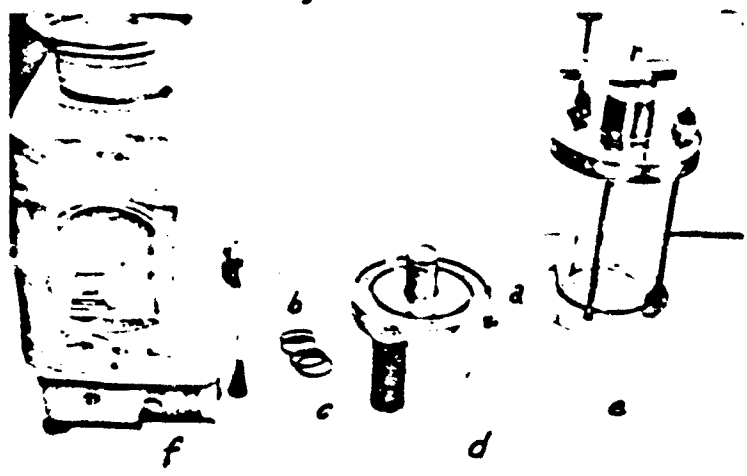
a. Specimen positioned in loading device; b. proving ring used to measure the applied load

## TRIAXIAL COMPRESSION TEST

The triaxial compression test will give the shearing stresses in a cylindrical soil specimen resulting from varying the magnitude of the vertical and horizontal principal stresses. This test gives some indication of how a material will act under a vertical load and a confining lateral pressure.

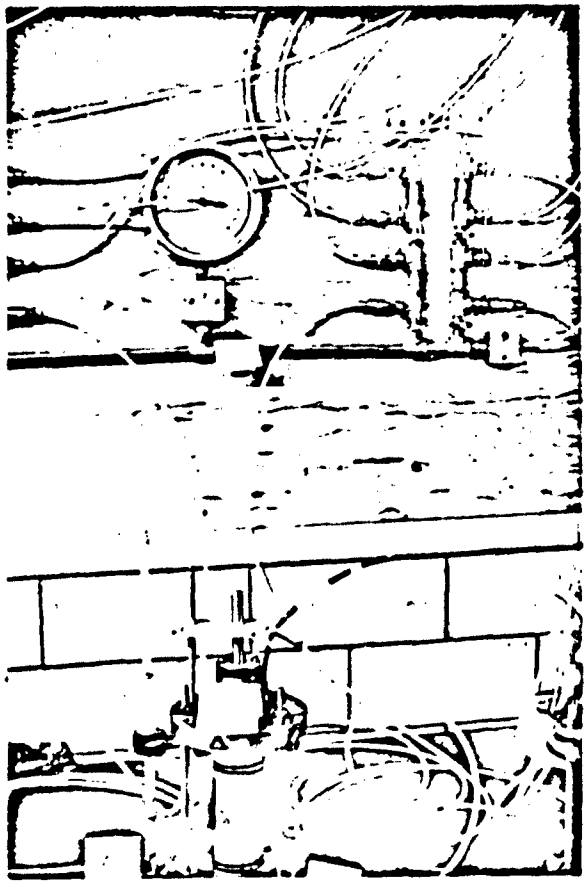
The equipment used in this test is shown in the figures below.

Figure 1-TRI



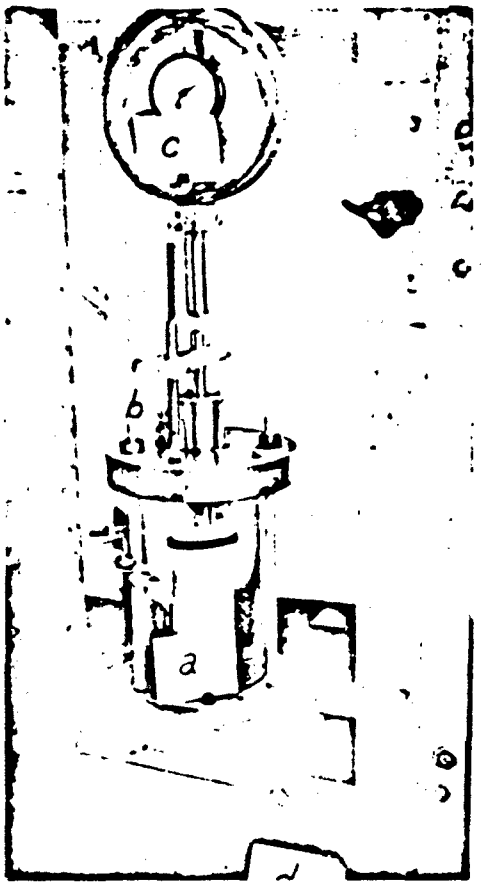

a. Testing cell, bottom and top; b. "O" rings; c. specimen; d. membrane; e. top bearing block; f. scale

Figure 2-TRI



a. Chamber pressure connection; b. measurement of drained water; c. assembled test cell with specimen in membrane; d. chamber pressure gauge

Figure 3-TRI




a. Assembled cell and specimen; b. chamber pressure connection; c. load proving ring; d. loading frame

This test was conducted as a consolidated-undrained triaxial compression test. The compacted specimen was encased in a rubber membrane and mounted in the test cell. The sample was allowed to consolidate for 24 hours at each pressure. Consolidation was accomplished by applying a pressure to the chamber surrounding the sample. During consolidation, a burette was used to measure the amount of water that was forced out of the sample.

After consolidation and just before shearing, the valve was closed to eliminate drainage during further testing, thus an undrained test.

The test cell was placed in the loading machine, afterwhich a strain controlled load was applied to the laterally confined sample. The loading continued until it became a maximum or the sample had been strained to at least 10 per cent total strain. After loading, the test cell was dismantled and a water content determination made of the tested specimen.

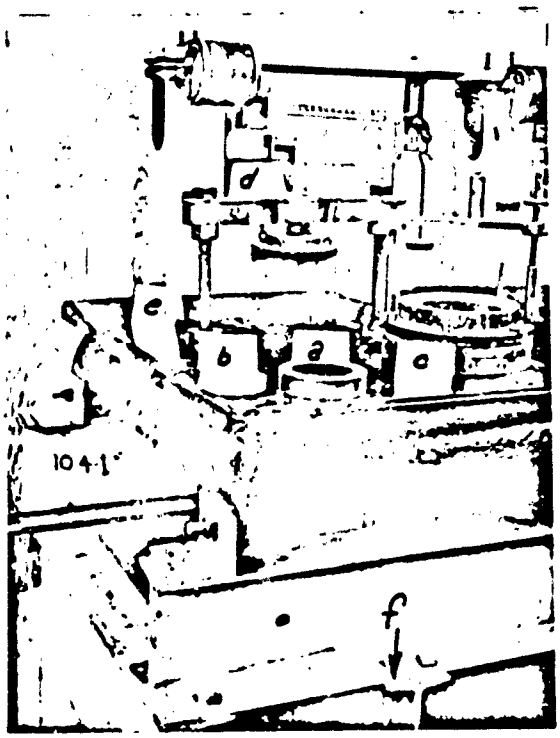
The test was conducted in accordance with Soil Testing for Engineers, T. William Lambe, John Wiley and Sons Inc., 1951.

## CONSOLIDATION TEST

The consolidation test is used to obtain data which is used in predicting the rate and amount of settlement of structures founded on the material. The most important soil property furnished by a consolidation test is the compression index, $C_c$, which indicates the compressibility of the specimen. This test is a one-dimensional compression test.

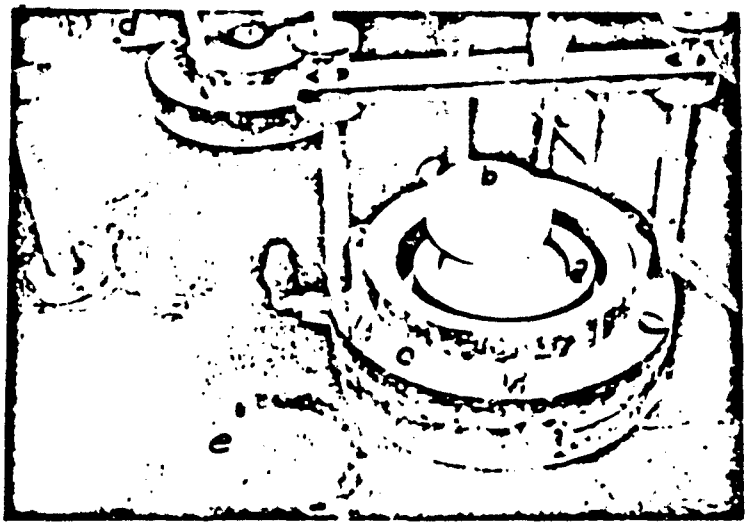
The equipment used in this test is shown in the figures below.

Figure 1-CON



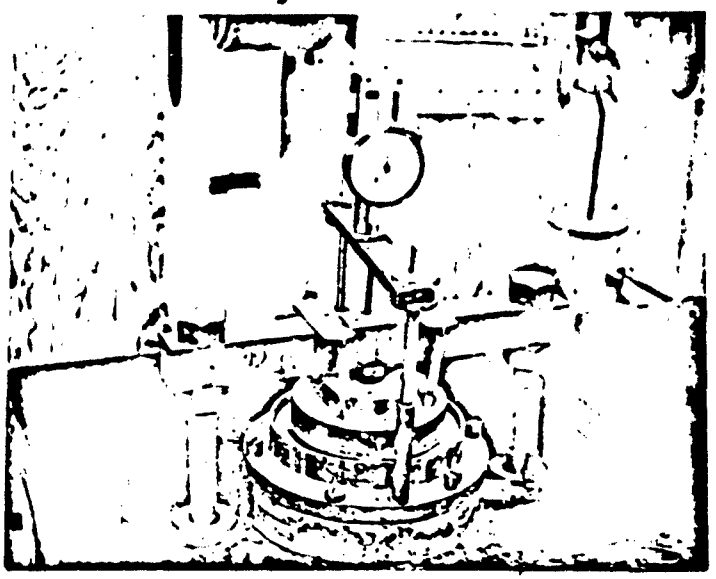

a. Confining ring; b. top porous stone; c. base unit with bottom porous stone; d. loading head; e. scale; f. loading wheel

Figure 2-CON



a. Confining ring; b. top porous stone; c. base unit; d. loading head; e. scale

Figure 3-CON



Apparatus completely assembled and ready to be loaded.

The material was compacted in the ring to a predetermined density, and the ring and soil were placed on the bottom porous stone in the base unit. The assembled base unit was centered beneath the loading bar. The scale

as calibrated will give 1/8 ton per square foot on the sample if the scale is balanced at 10.1 pounds. Each loading was set on the scale and balanced on the minute with deflections being recorded at 0, 0.25, 0.50, 1.0, 2, 4, 8, 15, 30, etc. minutes for each 24 hour load increment. The applied loads were as follows: 0, 0.125, 0.25, 0.50, 1.0, 2, 4, and 8 tons per square foot.

Between the one and two minute readings, under the initial load increment, (0.125 $T/Ft^2$), water was added to the base unit until the entire ring was submerged. The water prevented the sample from drying out.

After the eight ton load increment, the load was reduced to two tons per square foot and the deflections recorded at the end of a few hours. The reduced loading procedure gave an indication of how the material rebounded.

The base unit was dismantled and the ring containing the soil was dried off, weighed, and the ring and soil were placed in the oven for a water content determination.

From the initial data and time-deflection data, a curve of void ratio vs. log pressure was plotted; this curve yielded the compression index, $C_c$.

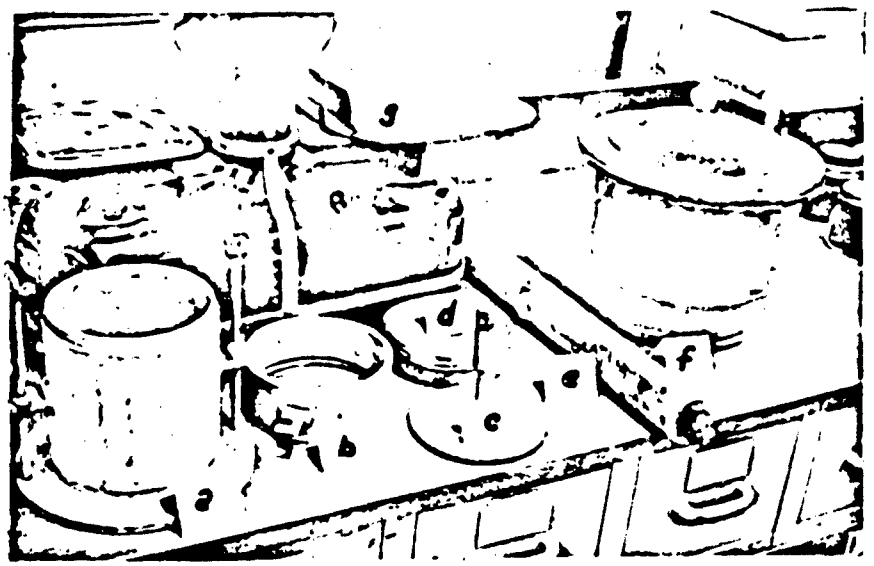
This test was conducted in accordance with Soil Testing for Engineers, T. William Lambe, John Wiley and Sons Inc., 1951.

# CALIFORNIA BEARING RATIO TEST

The California Bearing Ratio test is part of the method of flexible pavement design; and it measures the ratio of the shearing resistance of a soil to that of a standard crushed stone.

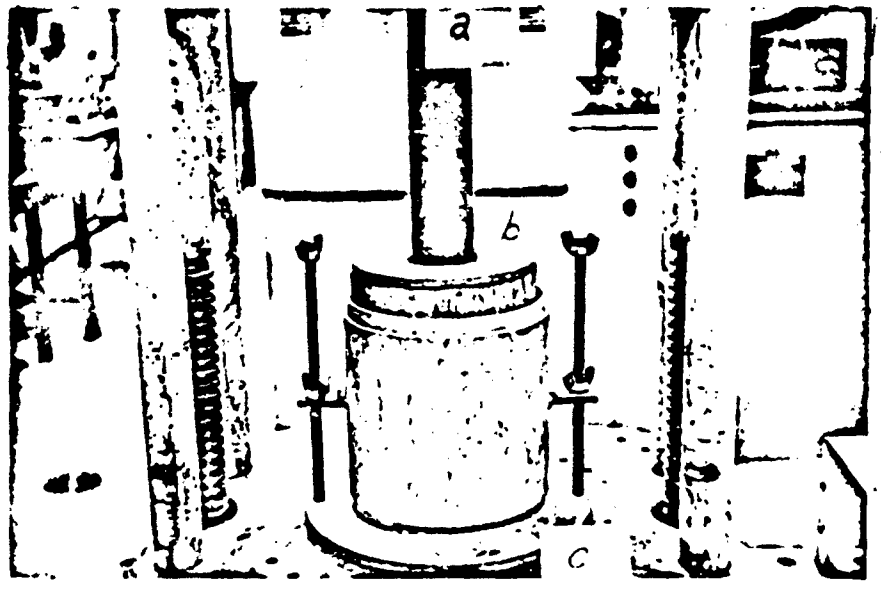
The equipment used in this test is shown in the figures below.

Figure 1-CBR




a. Mold and stand; b. collar; c. top plate;
d. surcharge weights; e. compaction hammer;
f. tempered soil; and, g. scale

Figure 2-CBR



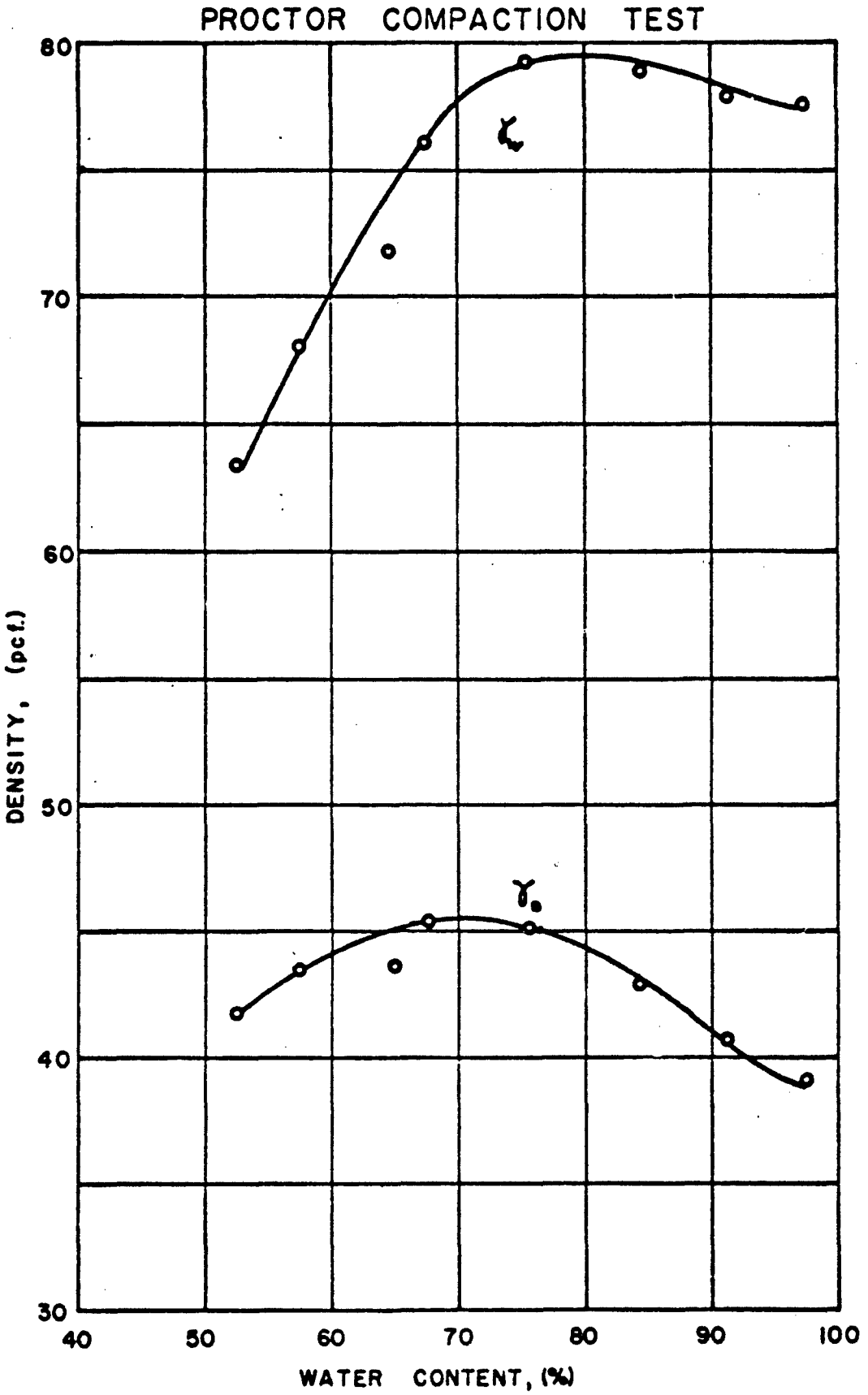

a. Penetrating piston; b. sample with surcharge; and, c. loading machine

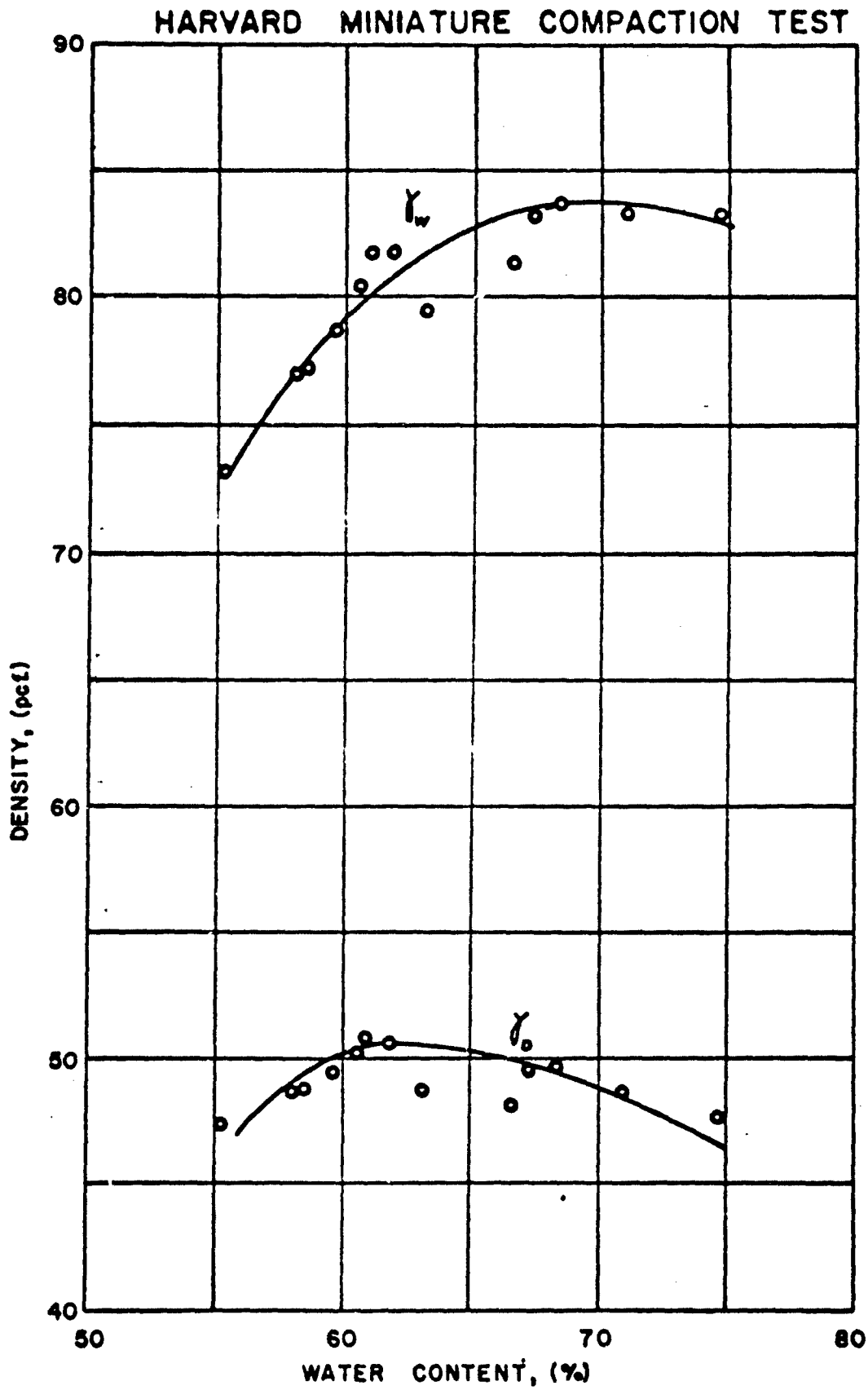
This test was condusted in accordance with ASTM D1883-61T; using ASTM .698, Method B to prepare the sample; and allowing the sample to soak for 120 hours instead of 96.

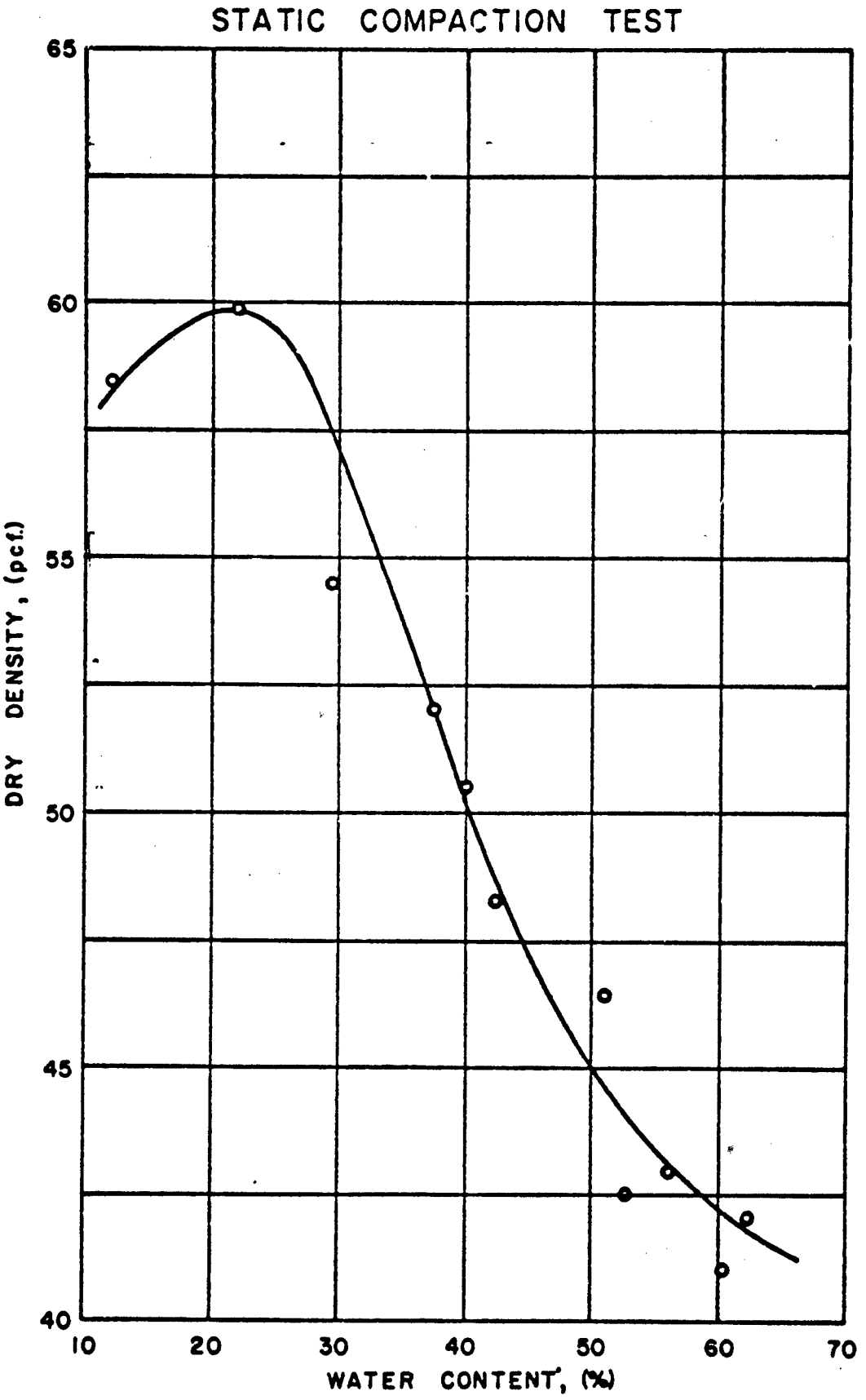
# APPENDIX B

## Results of Compaction Tests

1. Proctor Compaction
2. Harvard Miniature Compaction
3. Static Compaction


PROCTOR COMPACTION TEST
80
70
60
50
40
30
DENSITY, (pcf.)
$\gamma_t$
$\gamma_d$
40
50
60
70
80
90
100
WATER CONTENT, (%)


HARVARD MINIATURE COMPACTION TEST
90
80
70
60
50
40
DENSITY, (pcf)
$\gamma_w$
$\gamma_o$
50
60
70
80
WATER CONTENT, (%)


STATIC COMPACTION TEST
DRY DENSITY, (pcf.)
65
60
55
50
45
40
10
20
30
40
50
60
70
WATER CONTENT, (%)

# APPENDIX C

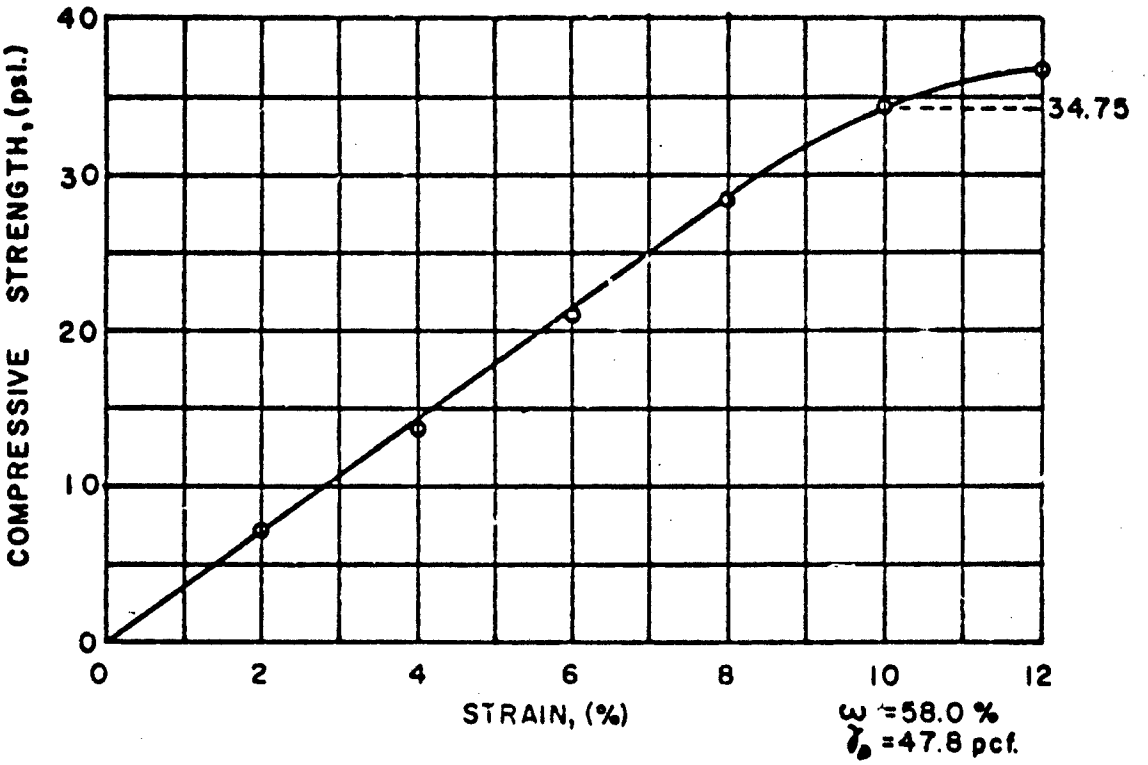
## Results of Unconfined Compression Tests

1. Harvard Miniature Samples

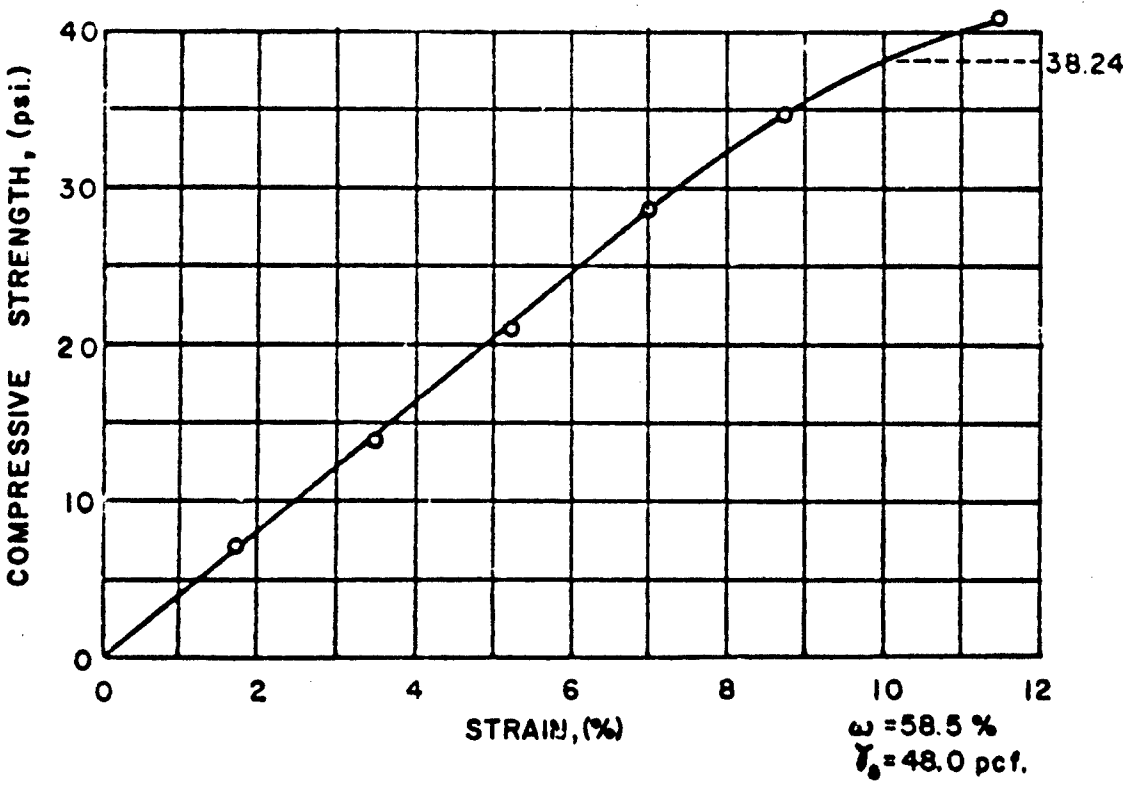
   a. Stress Controlled (7)
   b. Strain Controlled (7)

2. Static Compaction Samples - Strain Controlled (10)

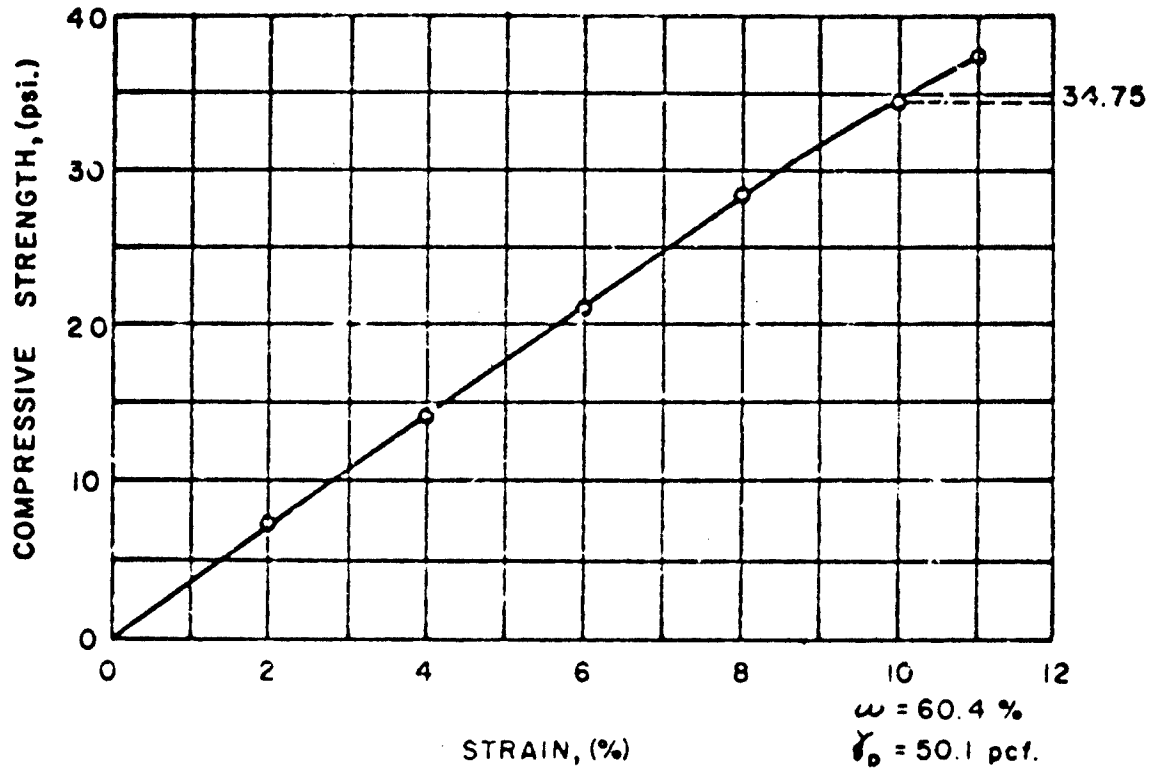
TEST 1


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
34.75
ω =58.0 %
γd =47.8 pcf.

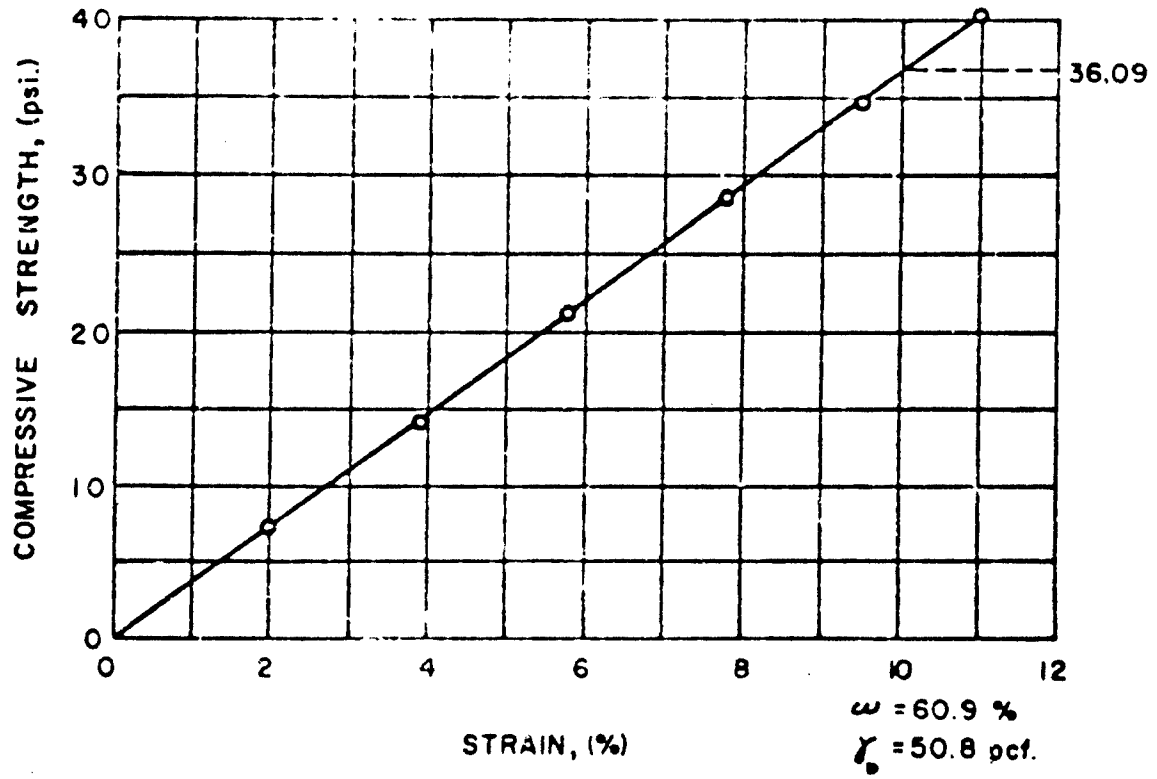
TEST 2


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
38.24
ω =58.5 %
γd =48.0 pcf.

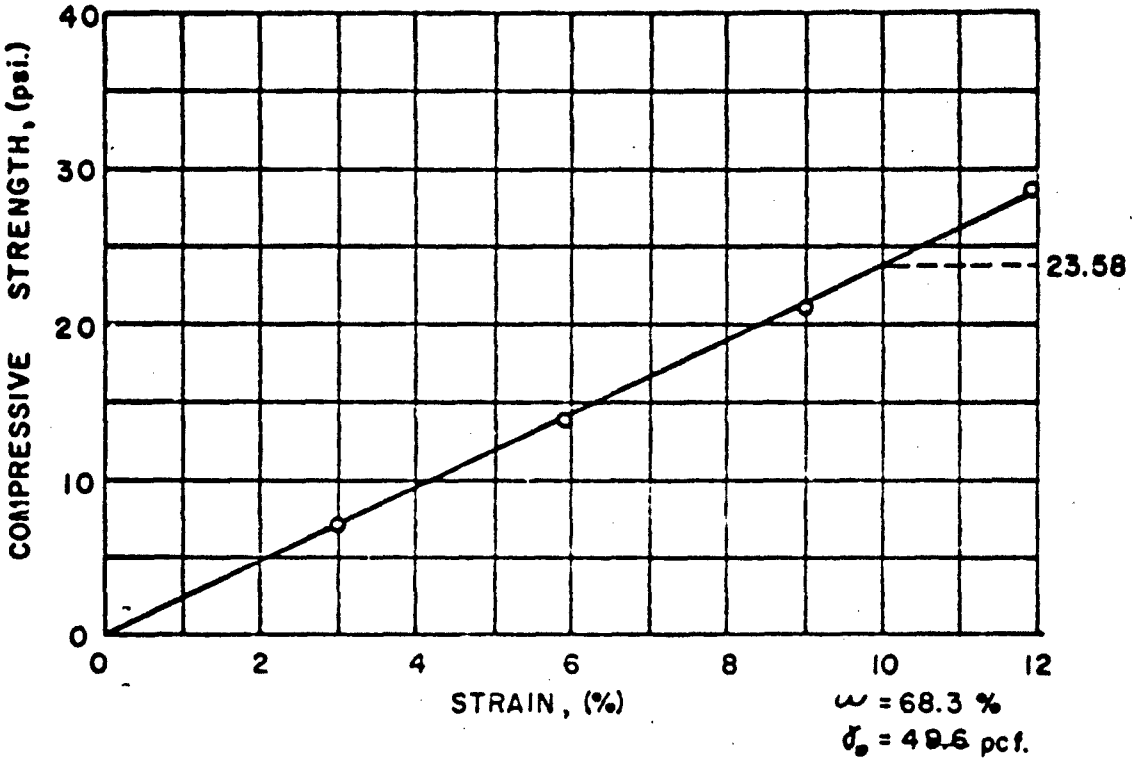
TEST 3


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
34.75
STRAIN, (%)
ω = 60.4 %
γ_D = 50.1 pcf.

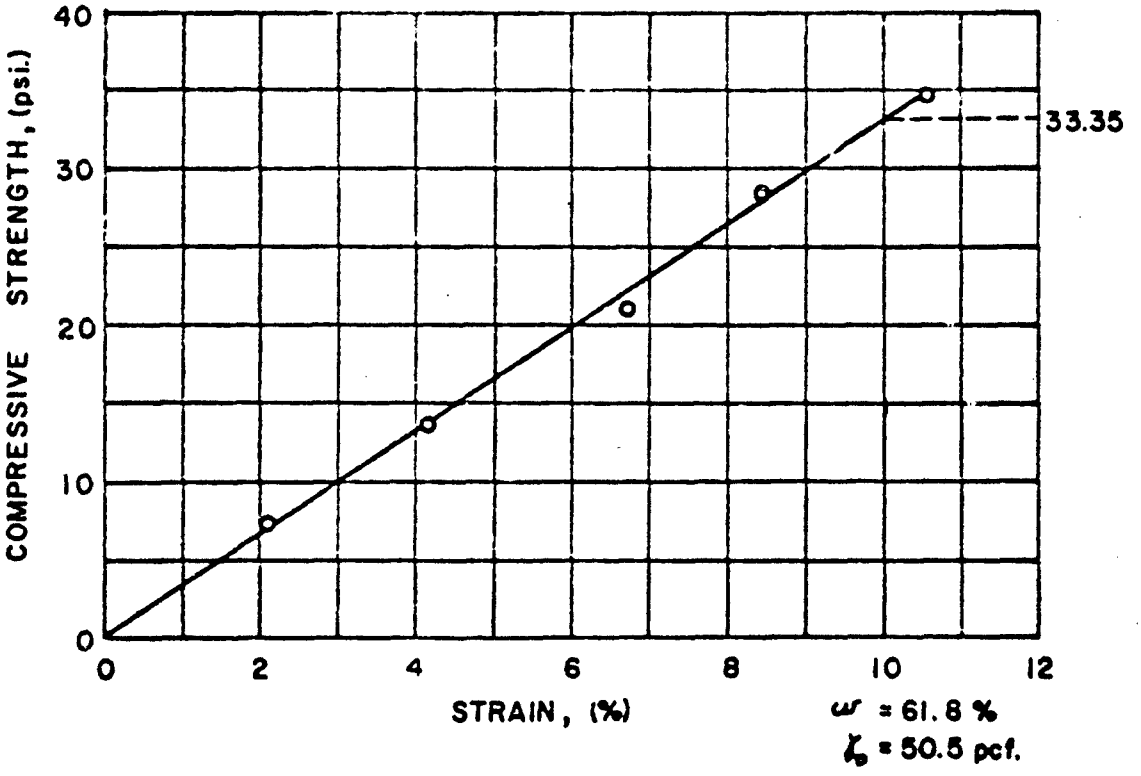
TEST 4


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
36.09
STRAIN, (%)
ω = 60.9 %
γ_D = 50.8 pcf.

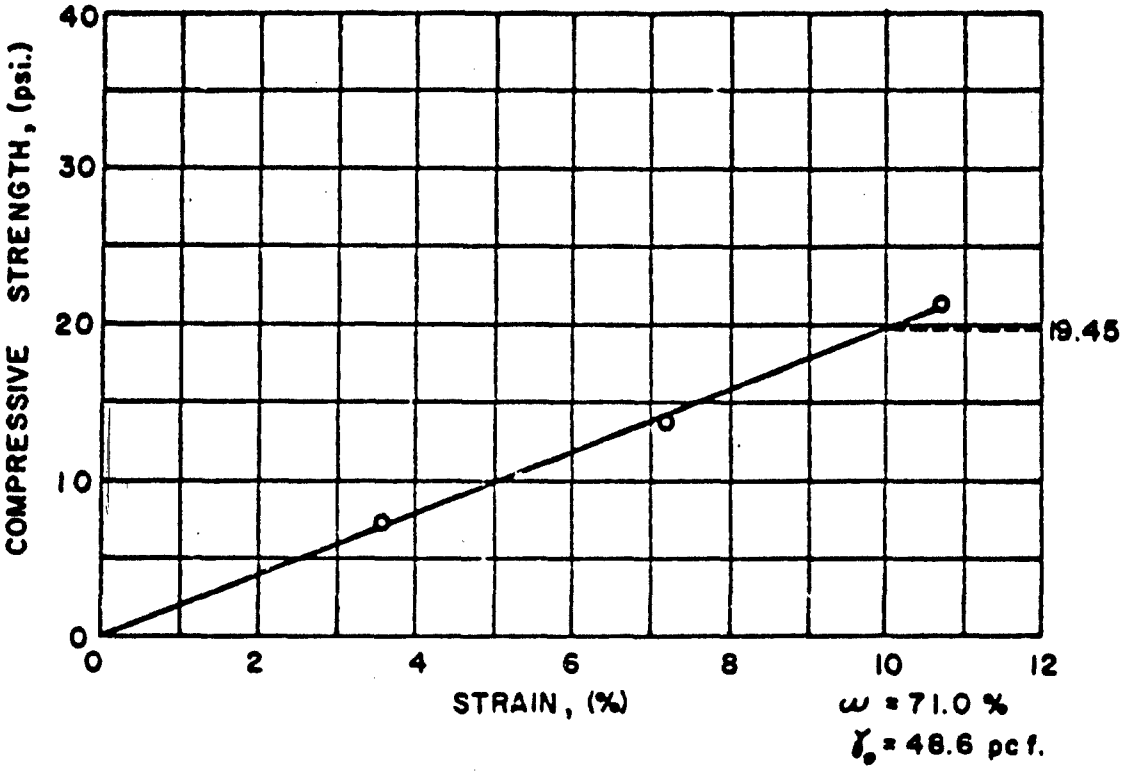
TEST 5


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
23.58
ω = 68.3 %
γd = 49.6 pcf.

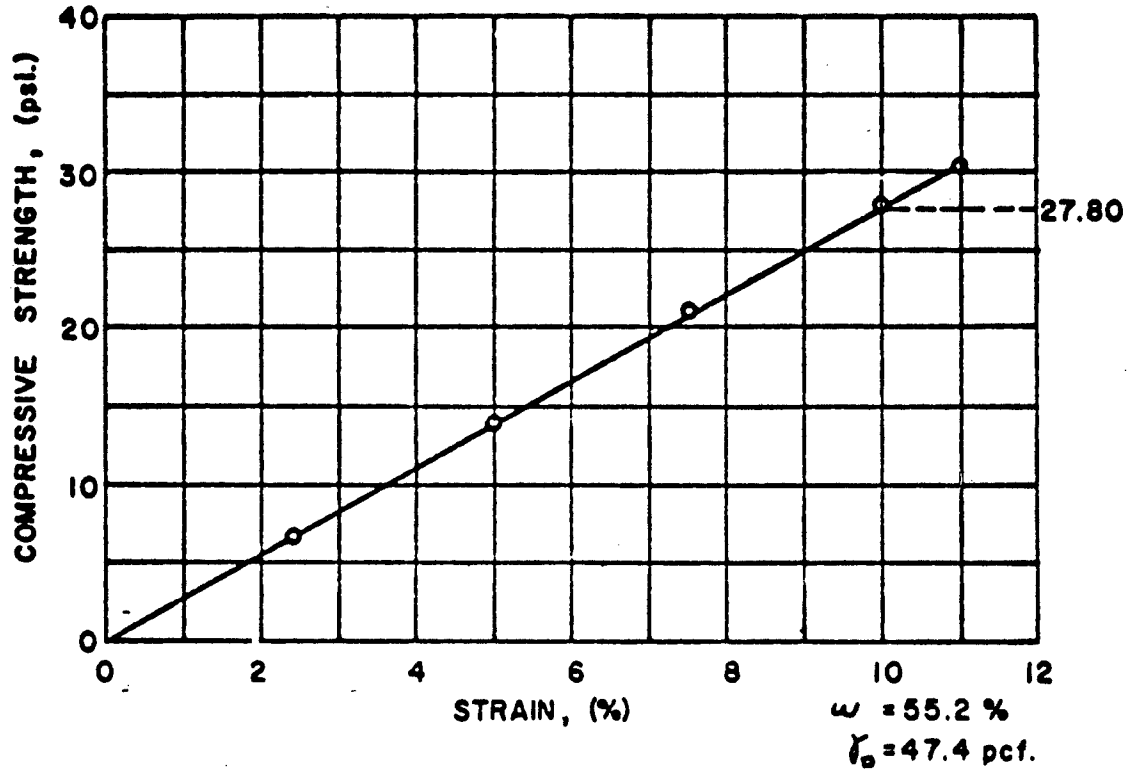
TEST 6


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
33.35
ω = 61.8 %
γd = 50.5 pcf.

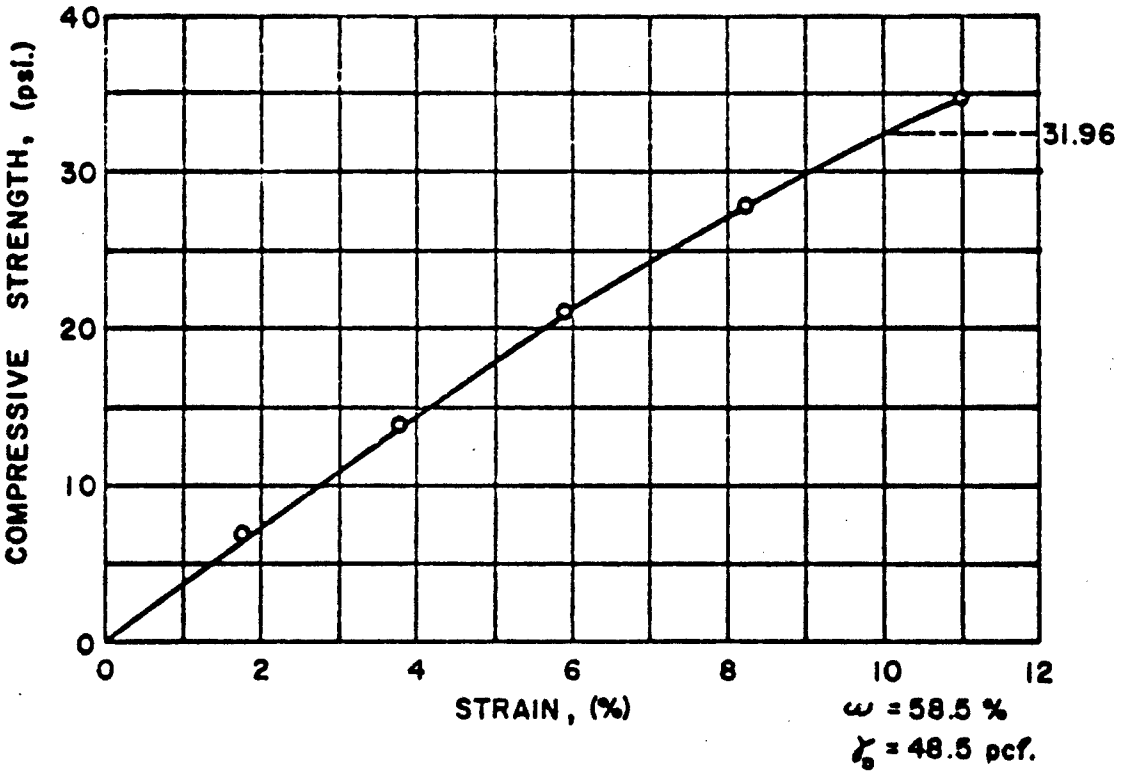
TEST 7


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
19.45
ω = 71.0 %
γ_d = 48.6 pcf.

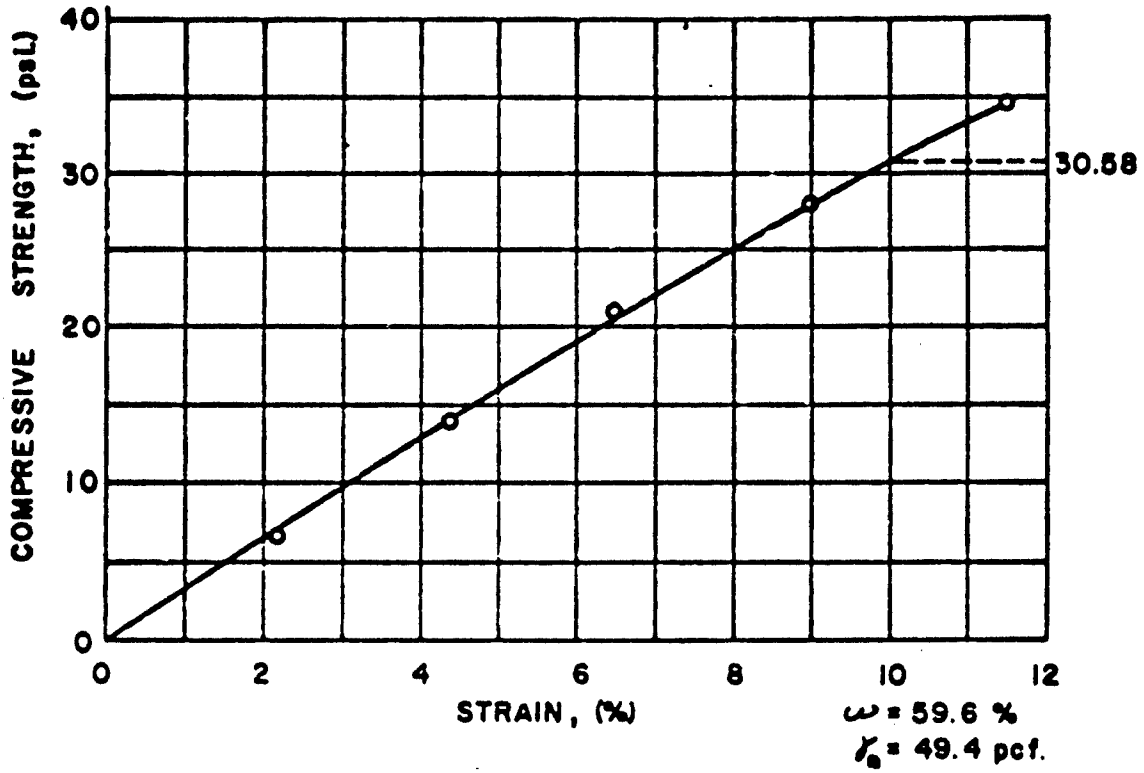
TEST 1


COMPRESSIVE STRENGTH, (psi.)
0
10
20
30
40
0
2
4
6
8
10
12
STRAIN, (%)
27.80
ω = 55.2 %
γd = 47.4 pcf.

TEST 2

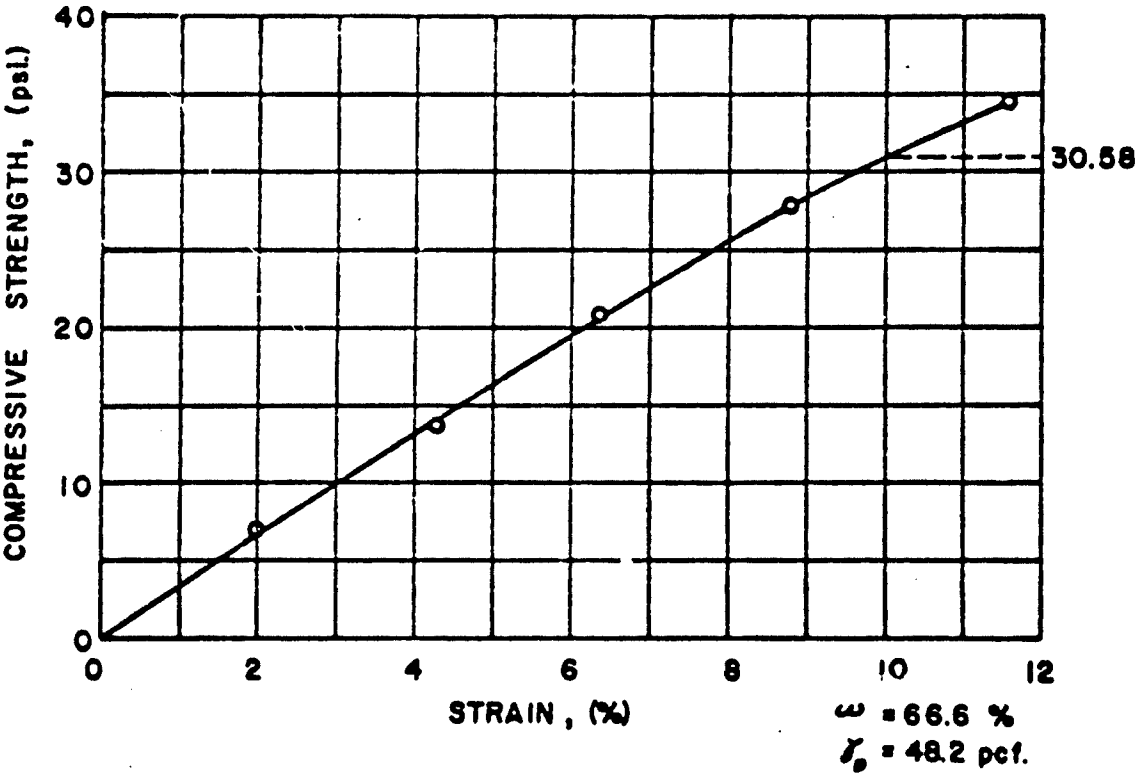

COMPRESSIVE STRENGTH, (psi.)
0
10
20
30
40
0
2
4
6
8
10
12
STRAIN, (%)
31.96
ω = 58.5 %
γd = 48.5 pcf.

TEST 3



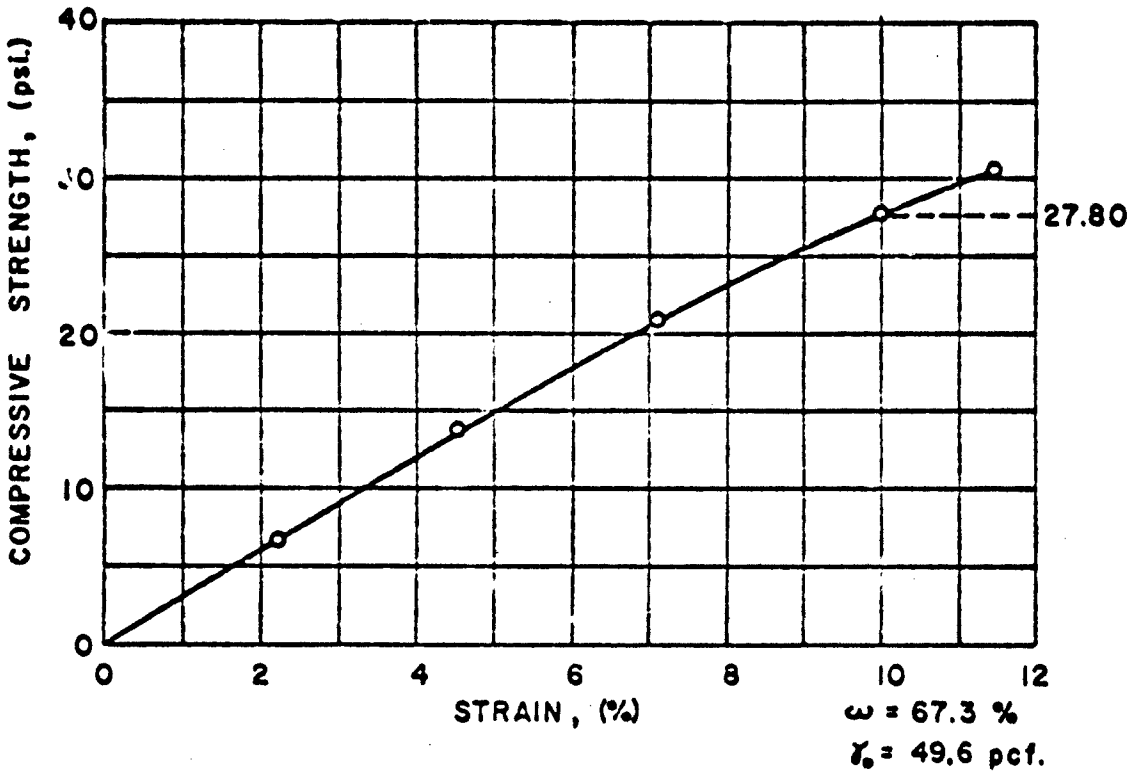

$\omega$ = 59.6 %
$\gamma_d$ = 49.4 pcf.

TEST 4

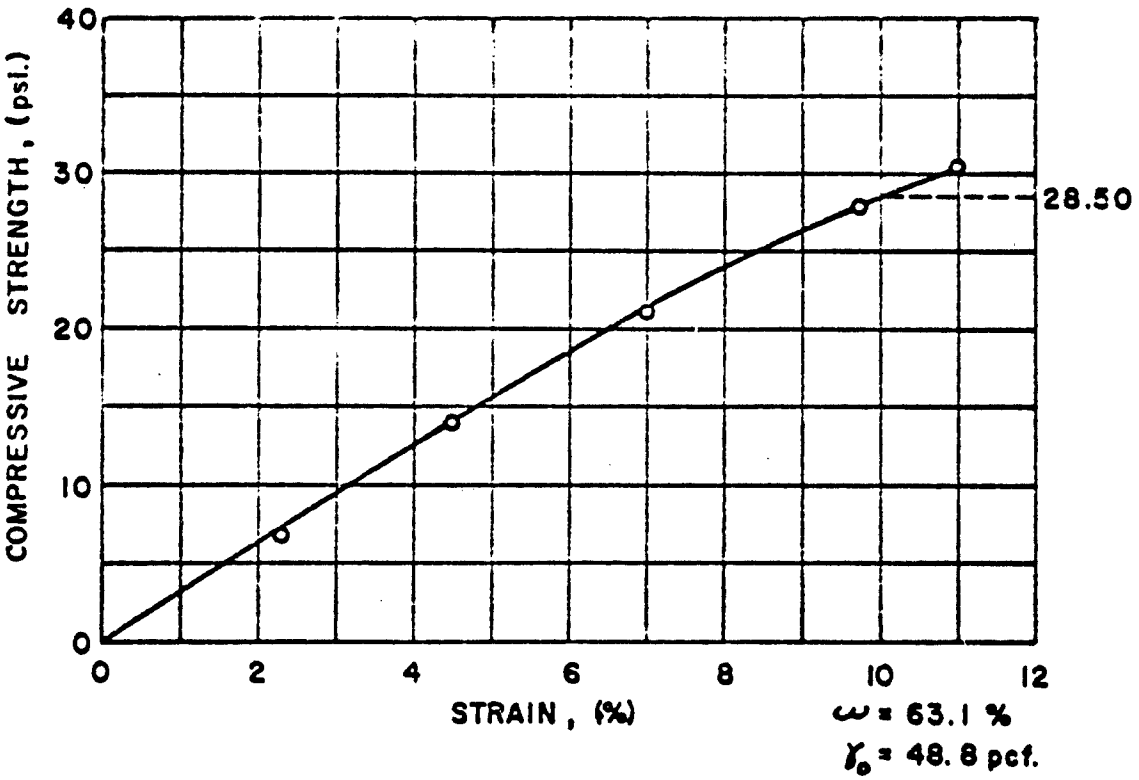



$\omega$ = 66.6 %
$\gamma_d$ = 48.2 pcf.

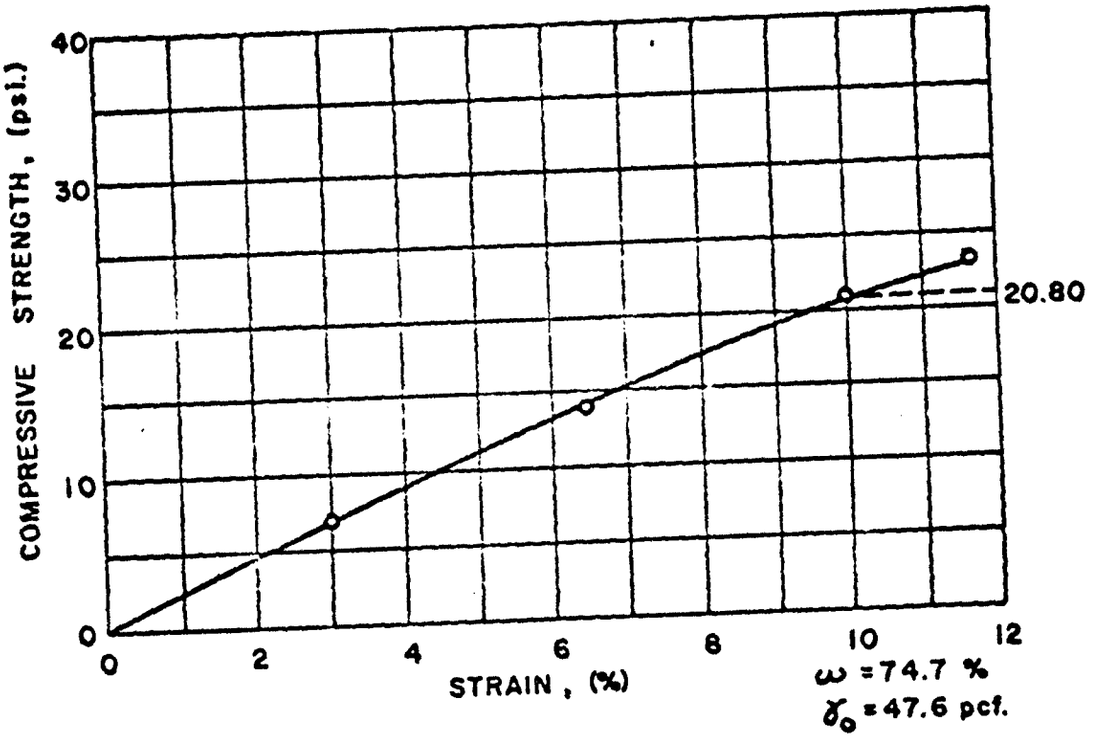
TEST 5


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
27.80
ω = 67.3 %
γ₀ = 49.6 pcf.

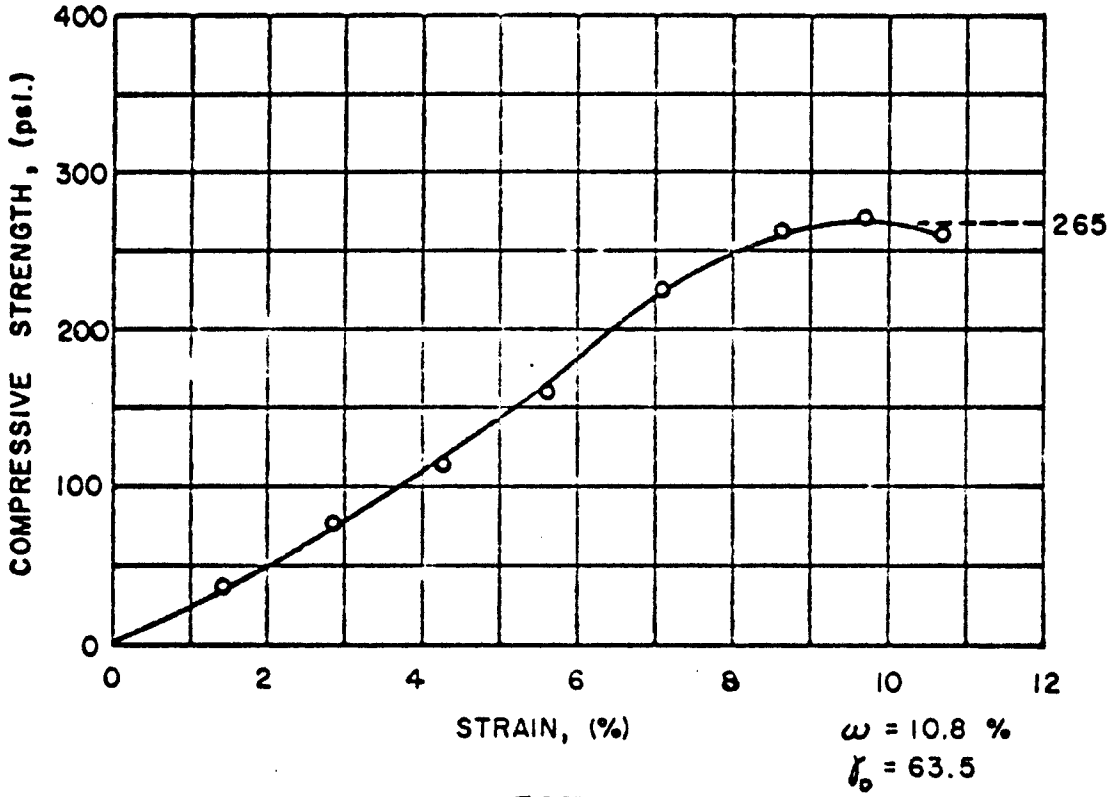
TEST 6


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
28.50
ω = 63.1 %
γ₀ = 48.8 pcf.

TEST 7


COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
20.80
ω = 74.7 %
γ₀ = 47.6 pcf.

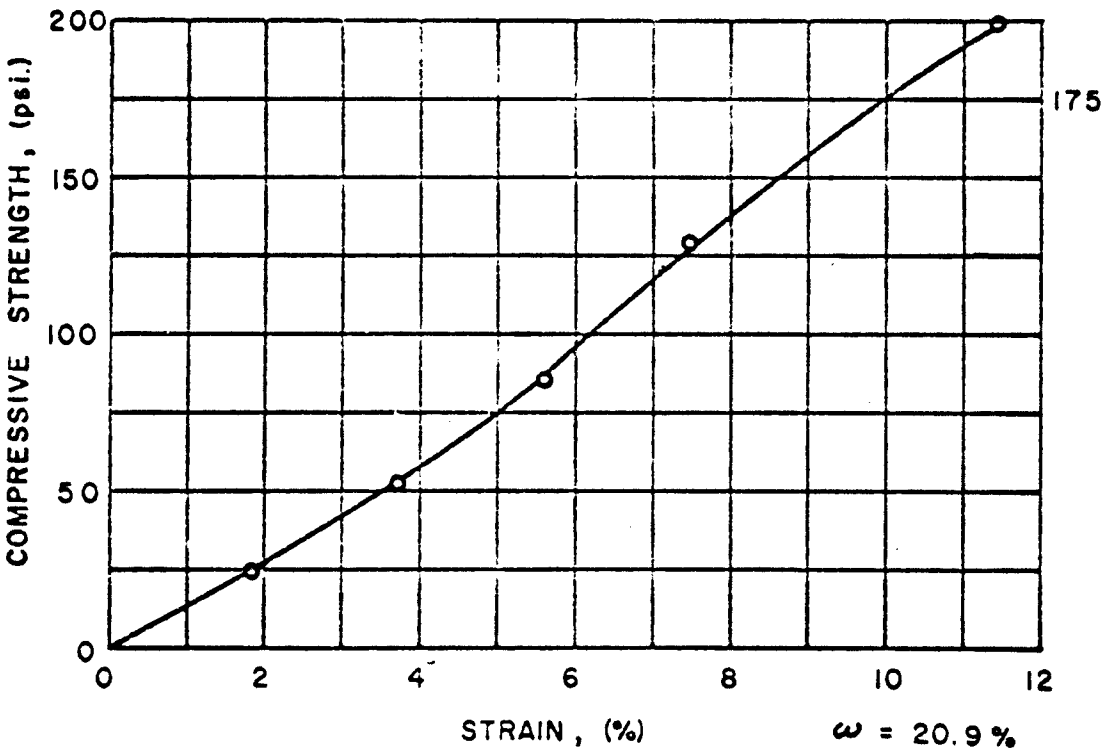
TEST 1




$\omega$ = 10.8 %

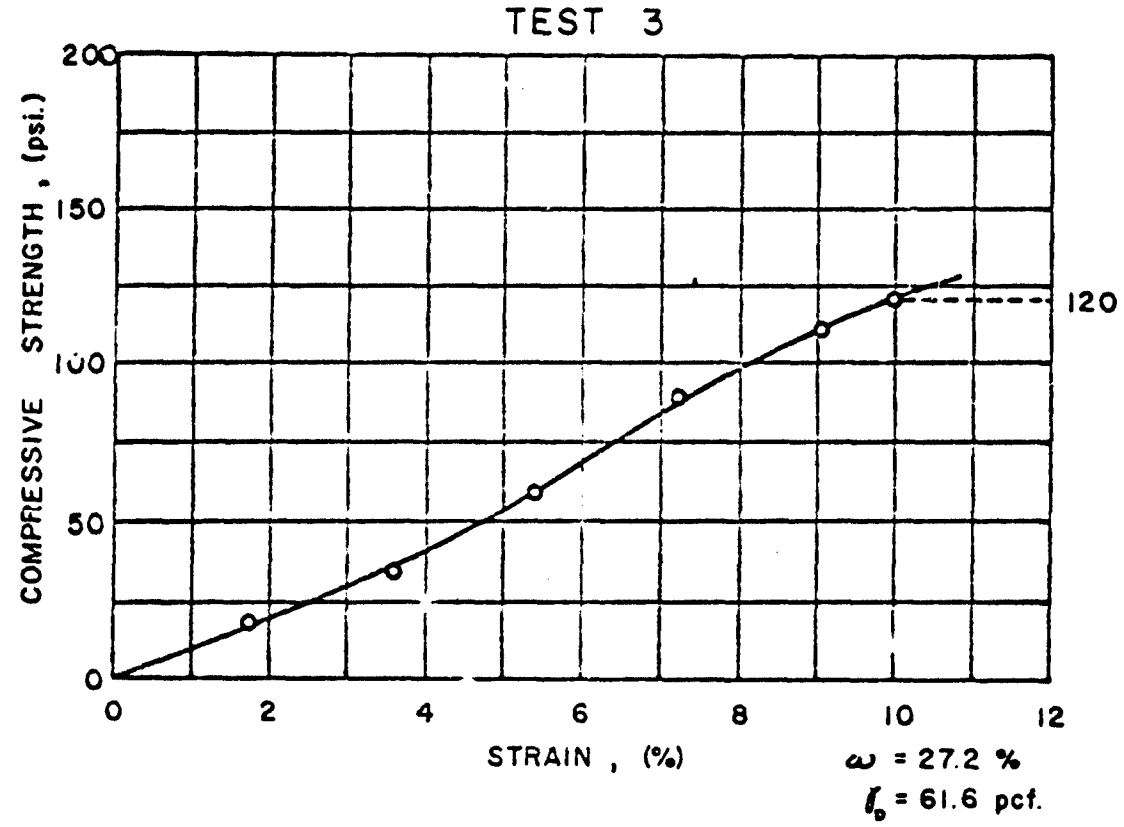
$\gamma_o$ = 63.5

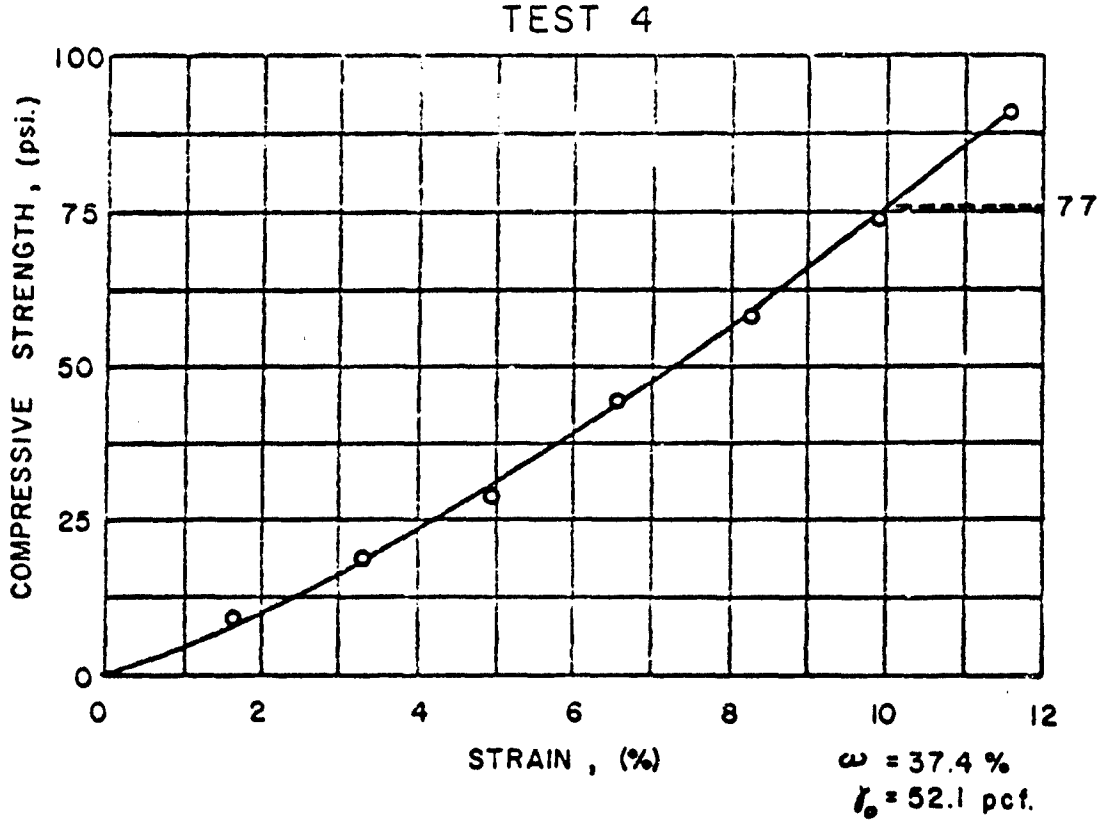
TEST 2

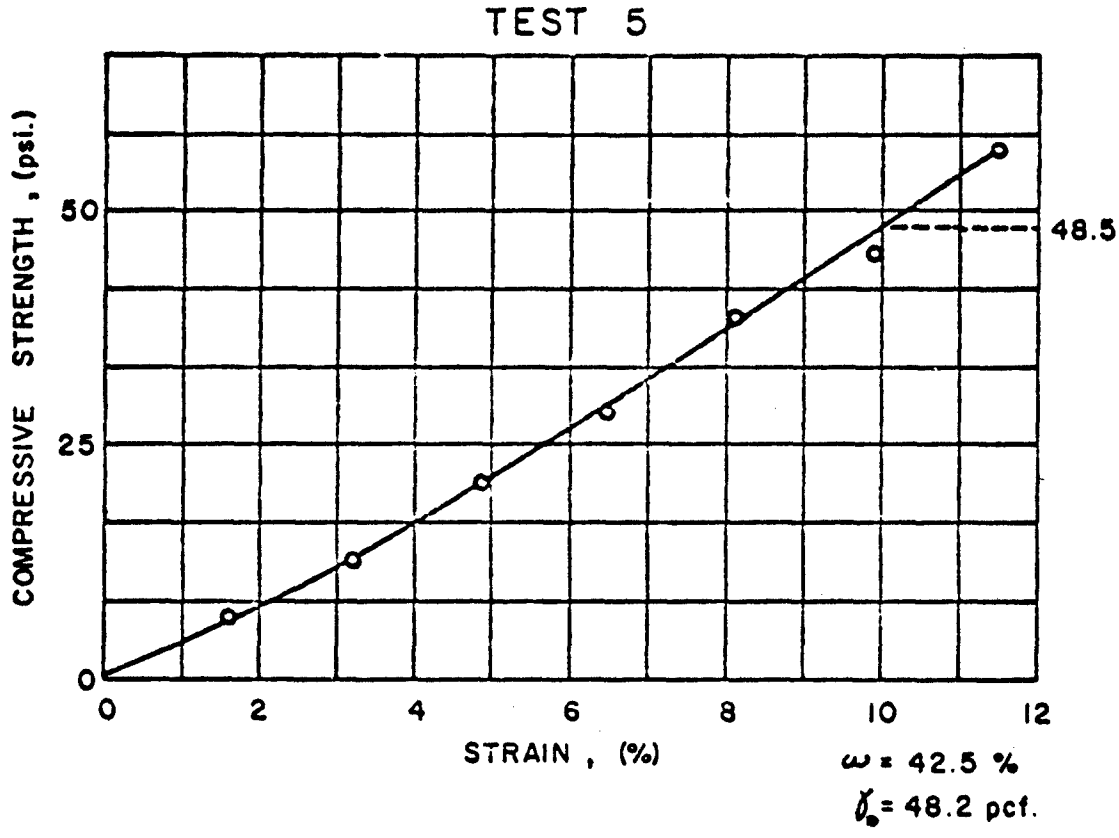


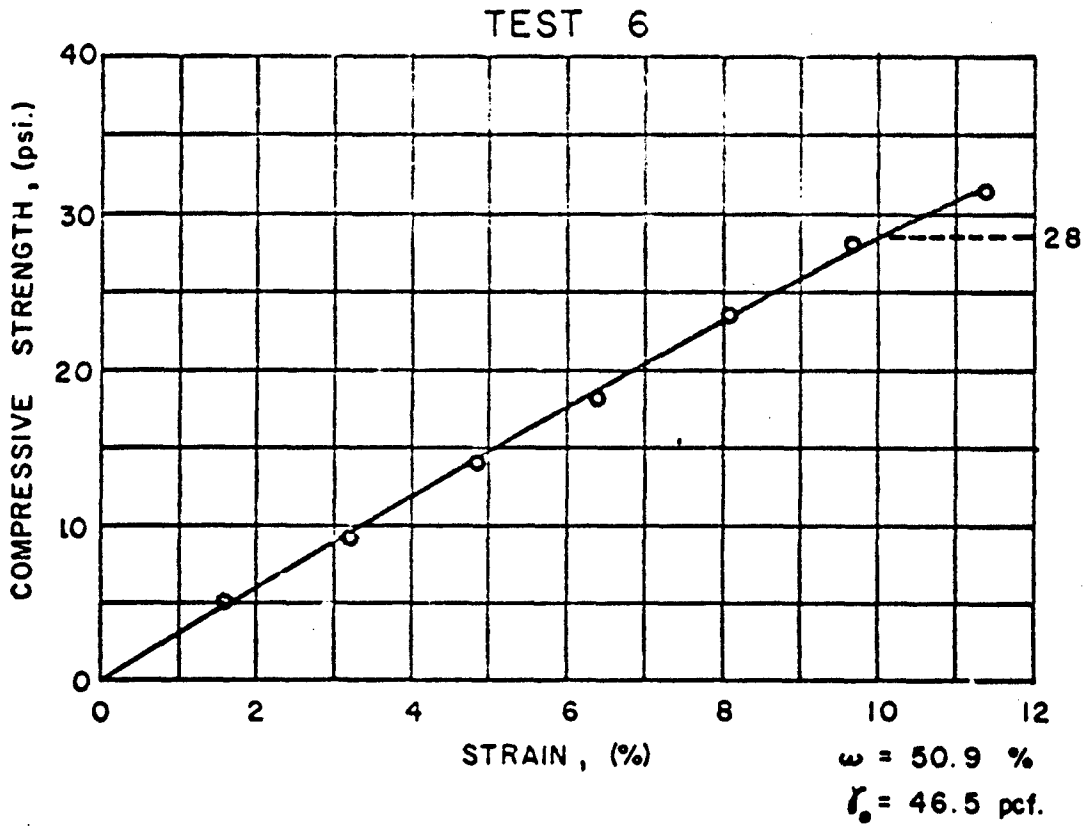

$\omega$ = 20.9 %

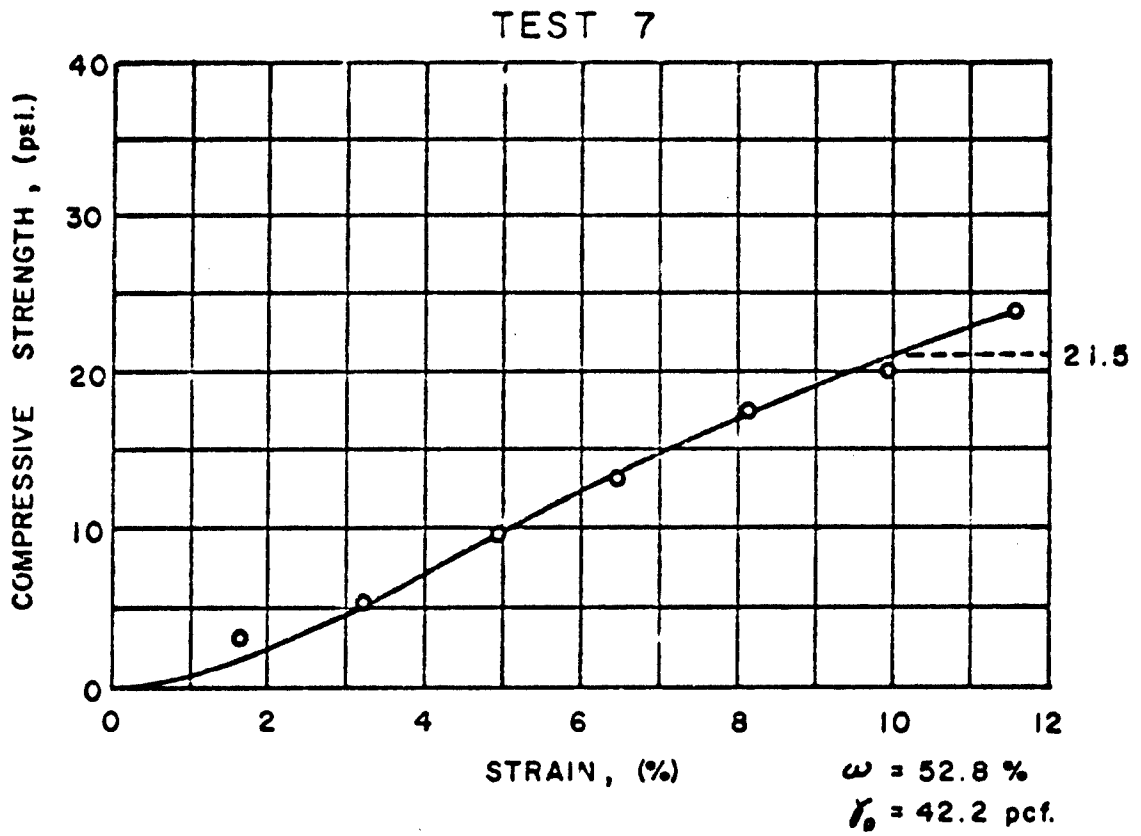
$\gamma_o$ = 63.7 pcf.

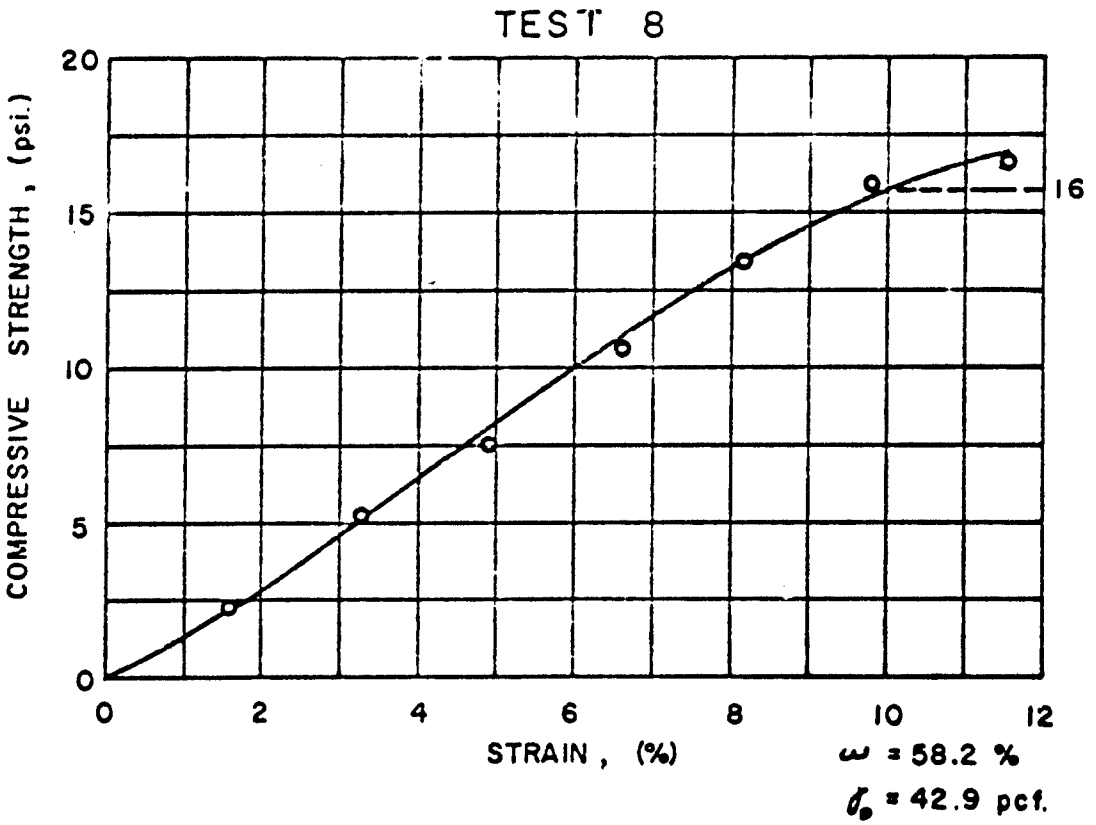

TEST 3
COMPRESSIVE STRENGTH, (psi.)
200
150
100
50
0
0
2
4
6
8
10
12
120
STRAIN, (%)
ω = 27.2 %
γd = 61.6 pcf.


TEST 4
COMPRESSIVE STRENGTH, (psi.)
100
75
50
25
0
0
2
4
6
8
10
12
77
STRAIN, (%)
ω = 37.4 %
γd = 52.1 pcf.

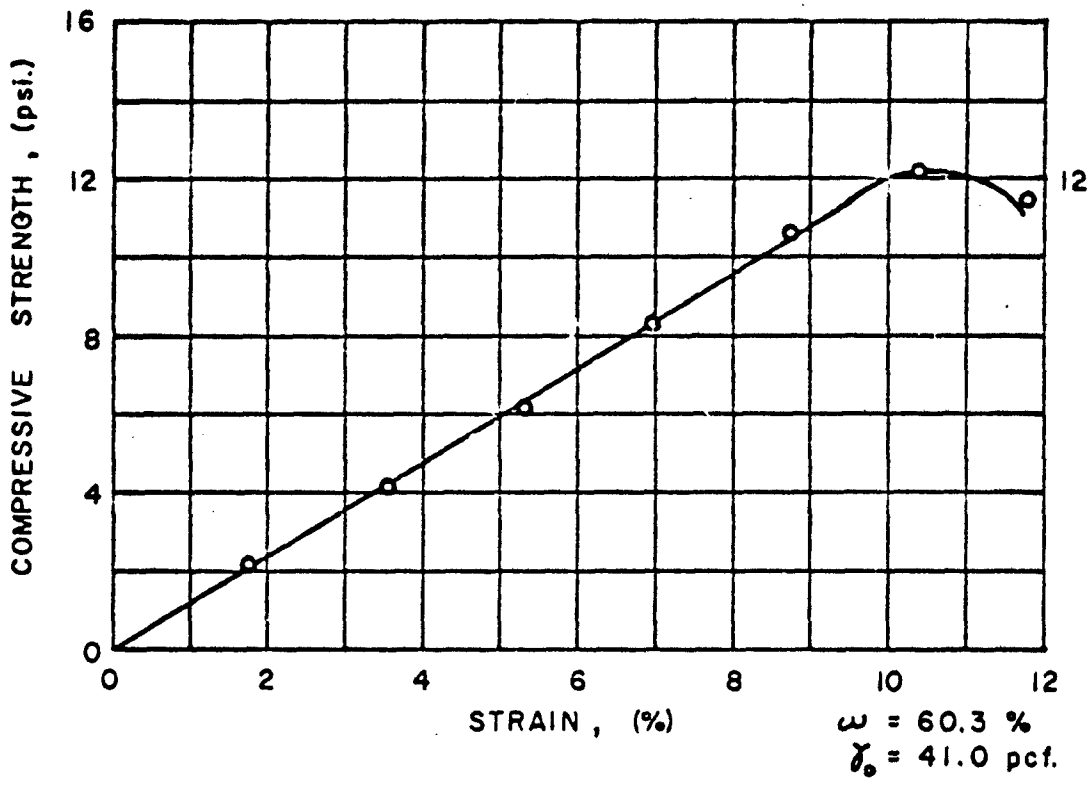

TEST 5
COMPRESSIVE STRENGTH , (psi.)
0
25
50
0
2
4
6
8
10
12
STRAIN , (%)
48.5
ω = 42.5 %
γ₀ = 48.2 pcf.


TEST 6
COMPRESSIVE STRENGTH , (psi.)
0
10
20
30
40
0
2
4
6
8
10
12
STRAIN , (%)
28
ω = 50.9 %
γ₀ = 46.5 pcf.


TEST 7
COMPRESSIVE STRENGTH, (psi.)
40
30
20
10
0
0
2
4
6
8
10
12
STRAIN, (%)
21.5
ω = 52.8 %
γ₀ = 42.2 pcf.

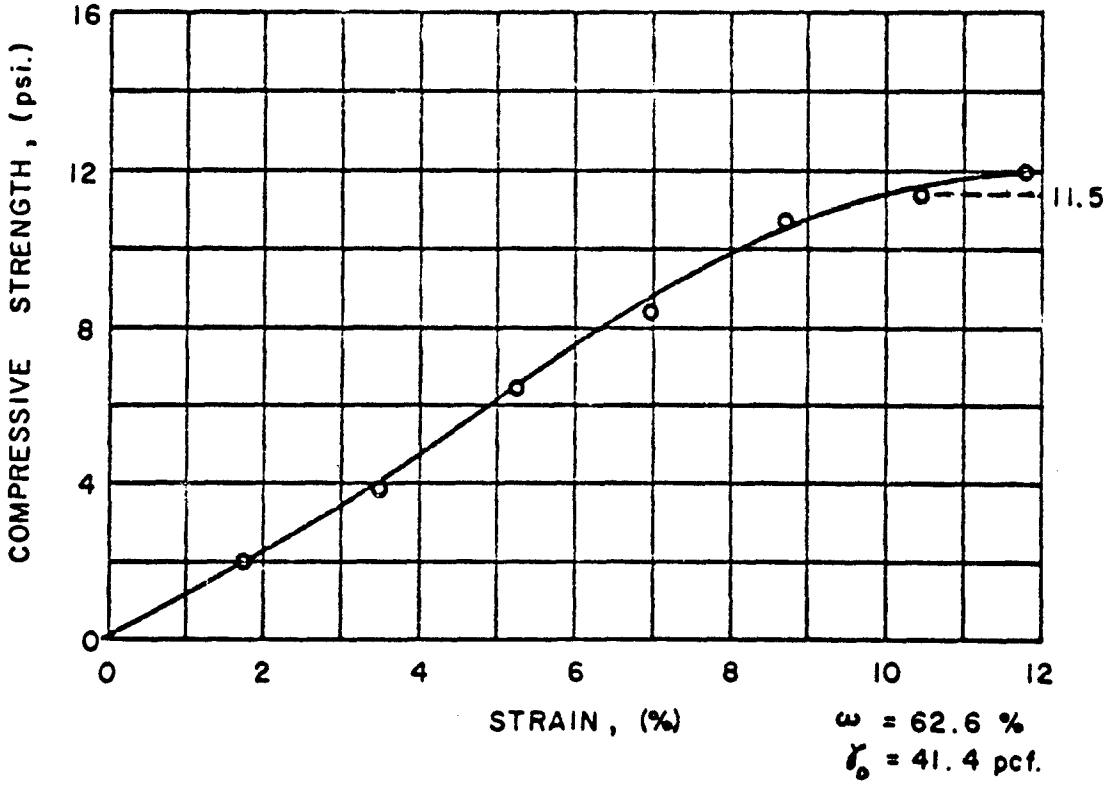

TEST 8
COMPRESSIVE STRENGTH, (psi.)
20
15
10
5
0
0
2
4
6
8
10
12
STRAIN, (%)
16
ω = 58.2 %
γ₀ = 42.9 pcf.

TEST 9




$\omega$ = 60.3 %
$\gamma_o$ = 41.0 pcf.

TEST 10




$\omega$ = 62.6 %
$\gamma_o$ = 41.4 pcf.

# APPENDIX D

## Results of Triaxial Tests

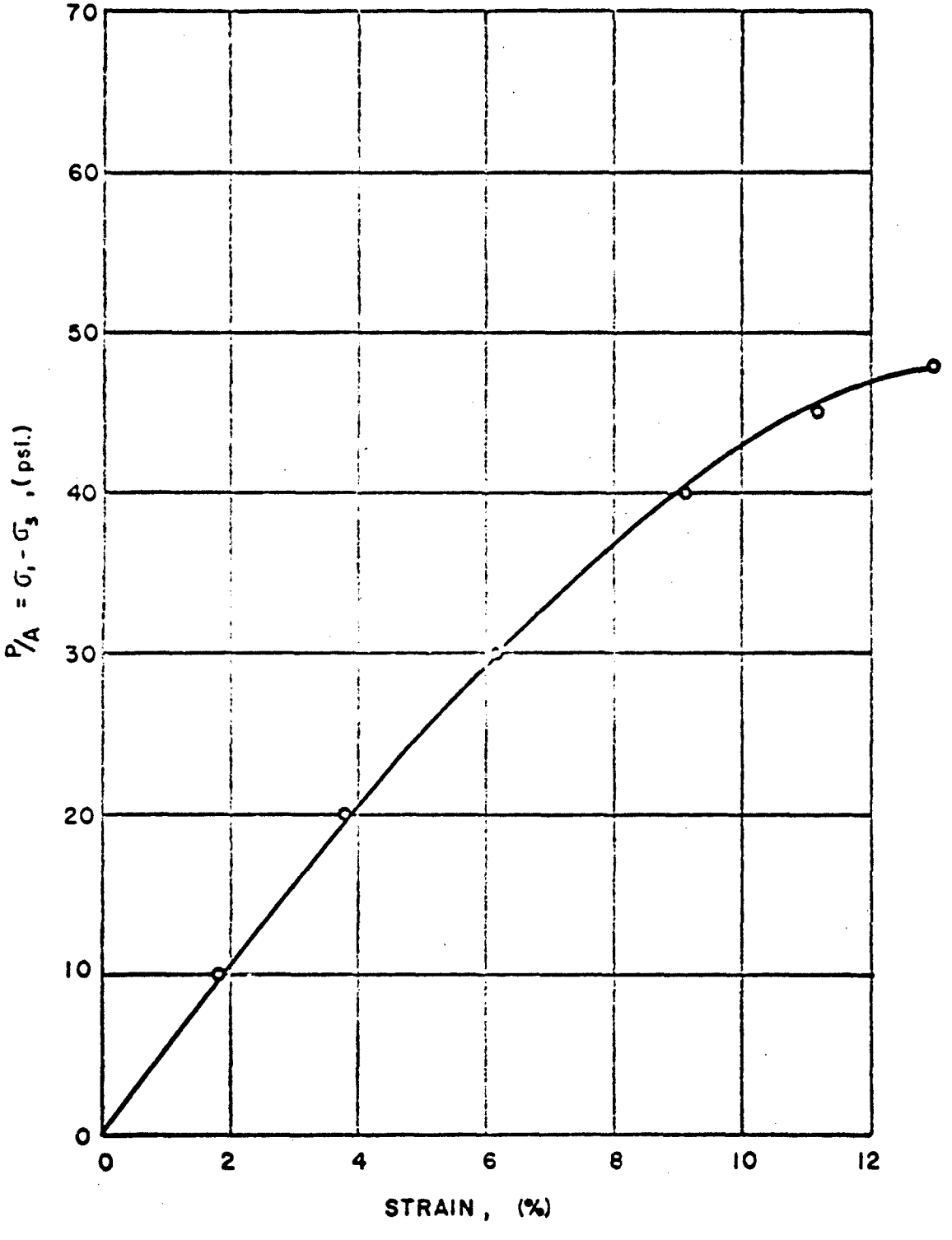
1. Stress versus Strain (4)
2. Mohr's Circles (4)

# TEST I

## DEVIATOR STRESS VS STRAIN

CONSOLIDATED – UNDRAINED $\omega_o = 49\%$

$\sigma_3 = 15$ psi. $\gamma_o = 34.7$ pcf.



# TEST 2

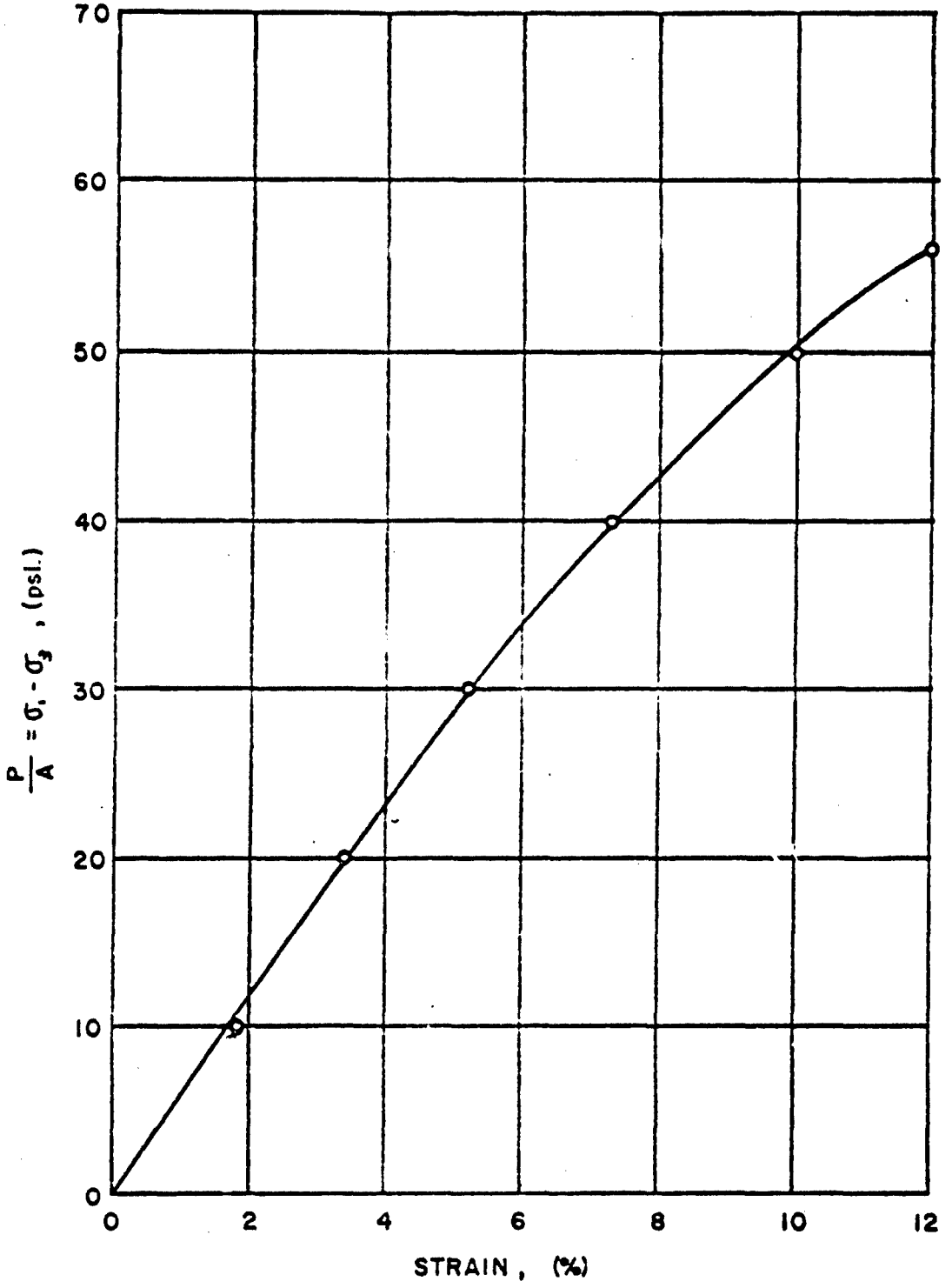
## DEVIATOR STRESS VS STRAIN

CONSOLIDATED – UNDRAINED

$\omega_o$ = 48 %

$\sigma_3$ = 30 psi.

$\gamma_o$ = 35.6 pcf.

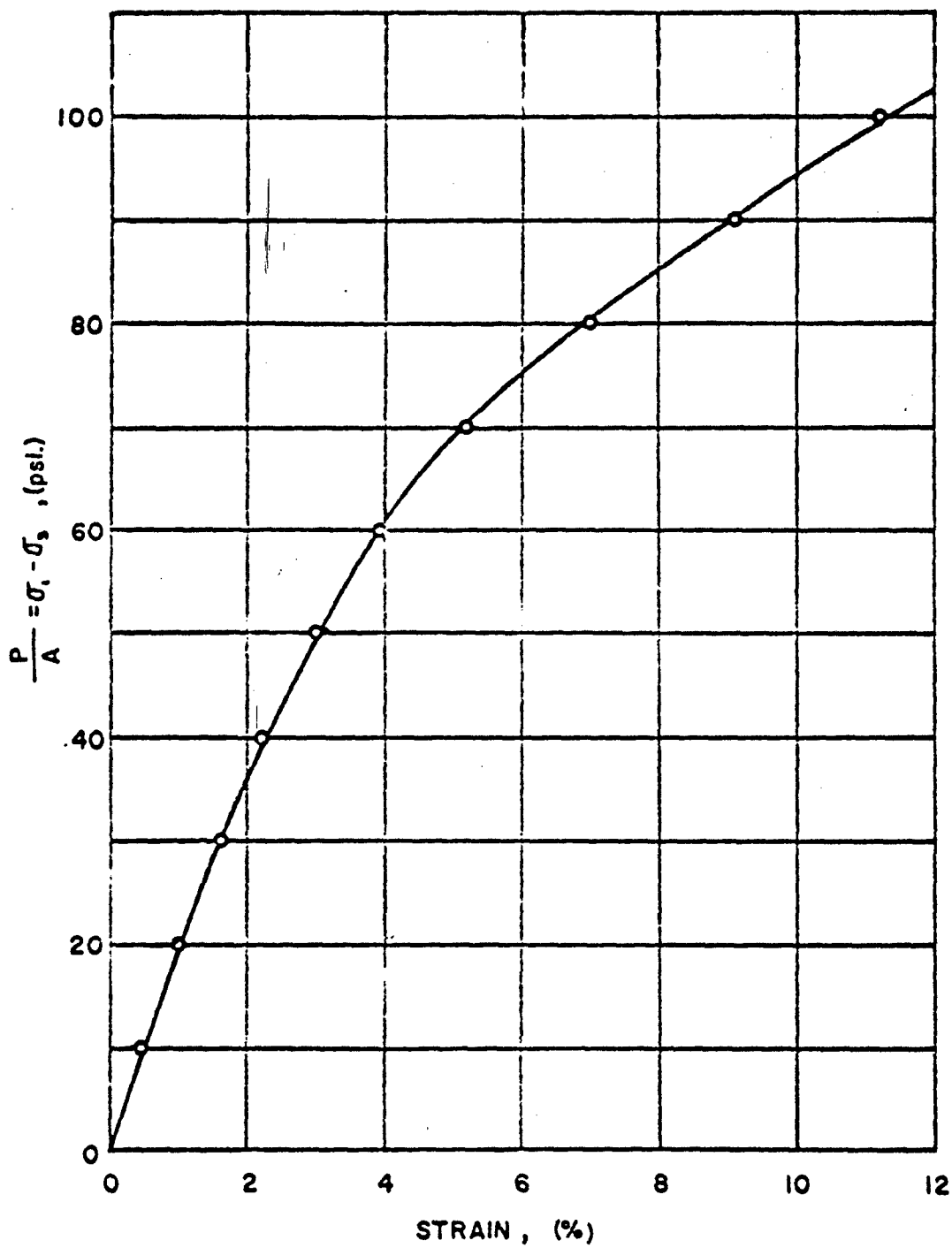


# TEST 3

DEVIATOR STRESS VS STRAIN

CONSOLIDATED — UNDRAINED $\omega_o$ = 46 %

$\sigma_3$ = 60 psi. $\gamma_o$ = 35.2 pcf.



# TEST 4

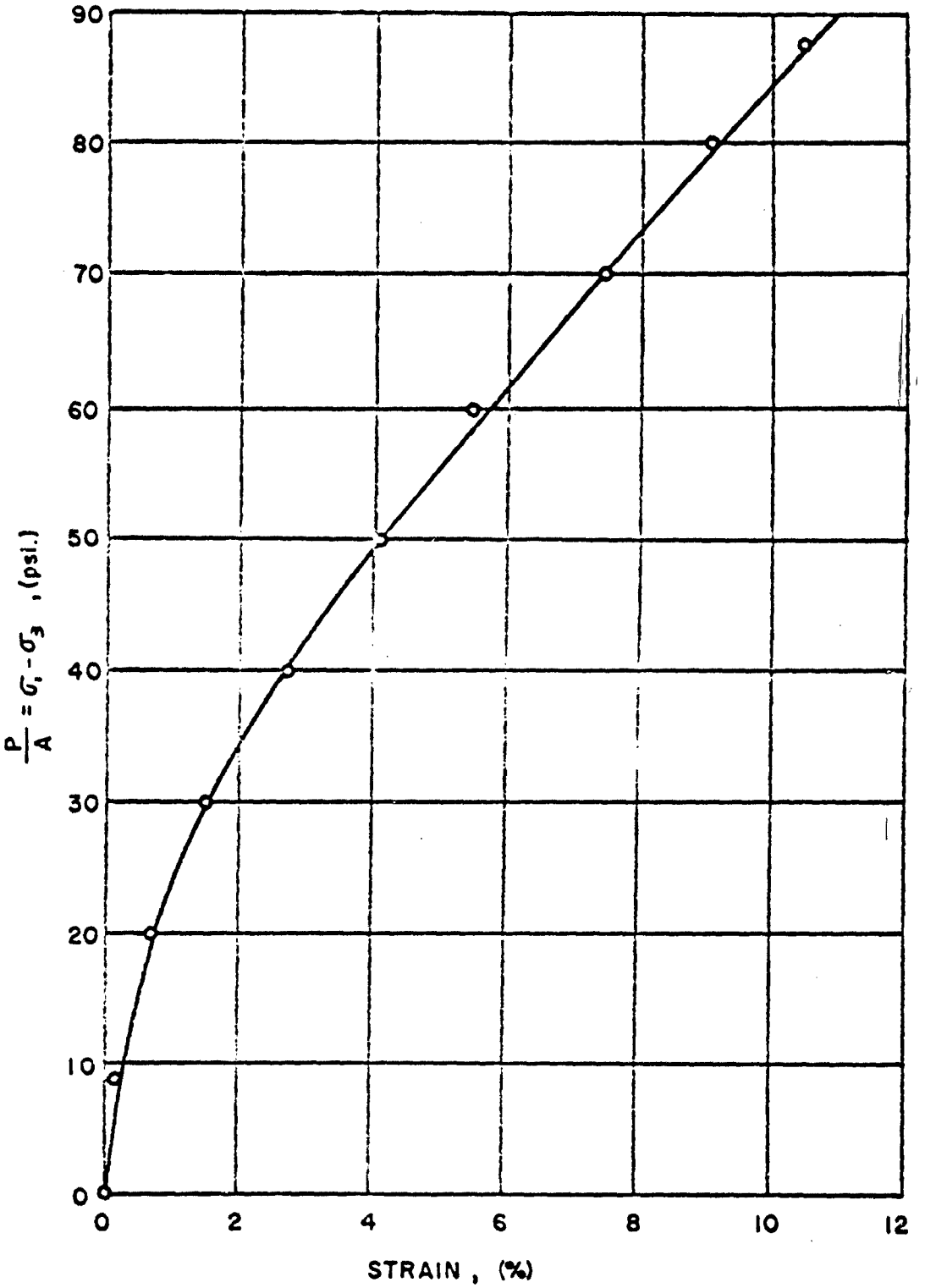
## DEVIATOR STRESS VS STRAIN

CONSOLIDATED — UNDRAINED

$\sigma_3$ = 60 psi.

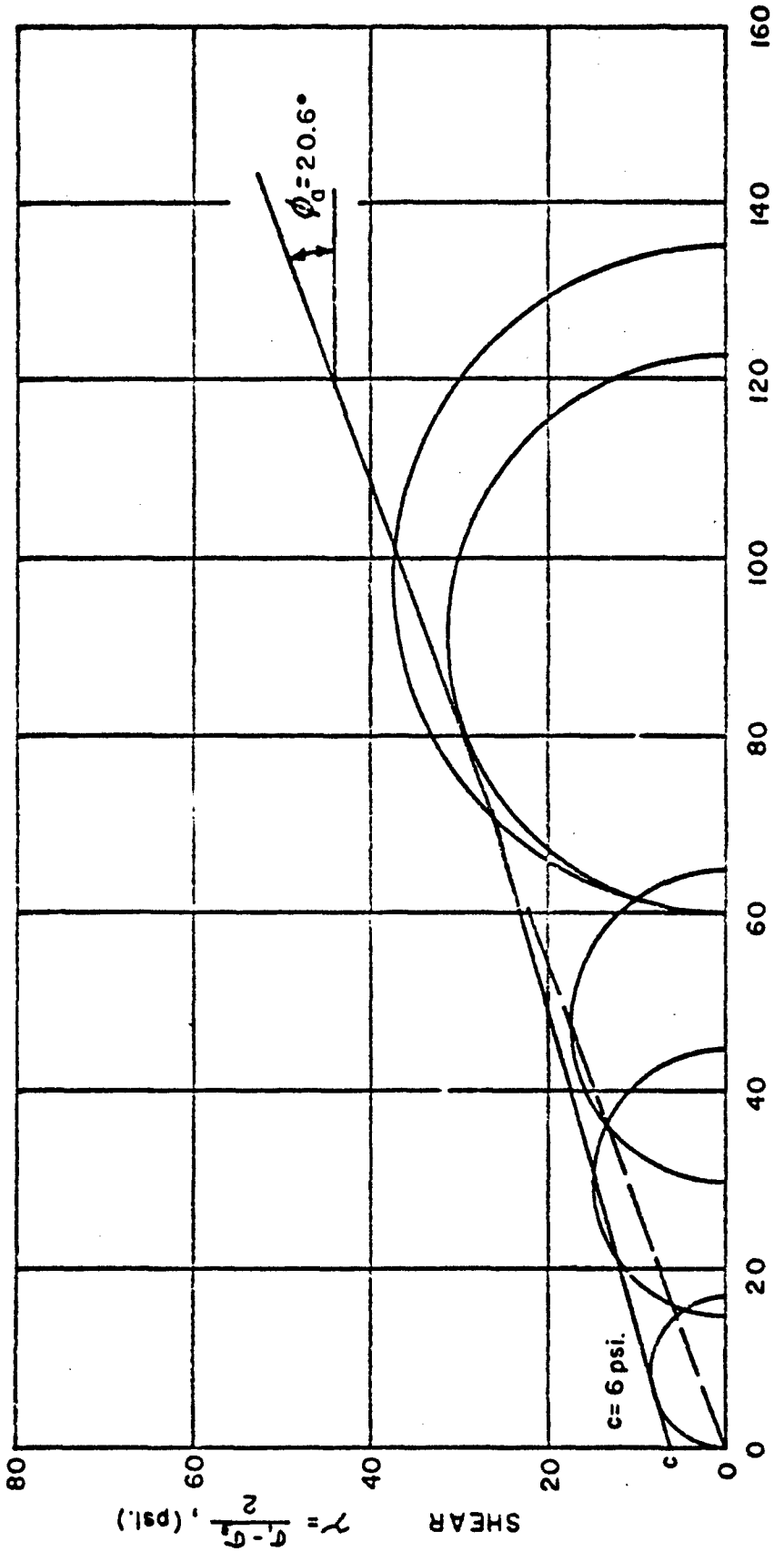
$\omega_o$ = 57 %

$\gamma_o$ = 36 pcf.



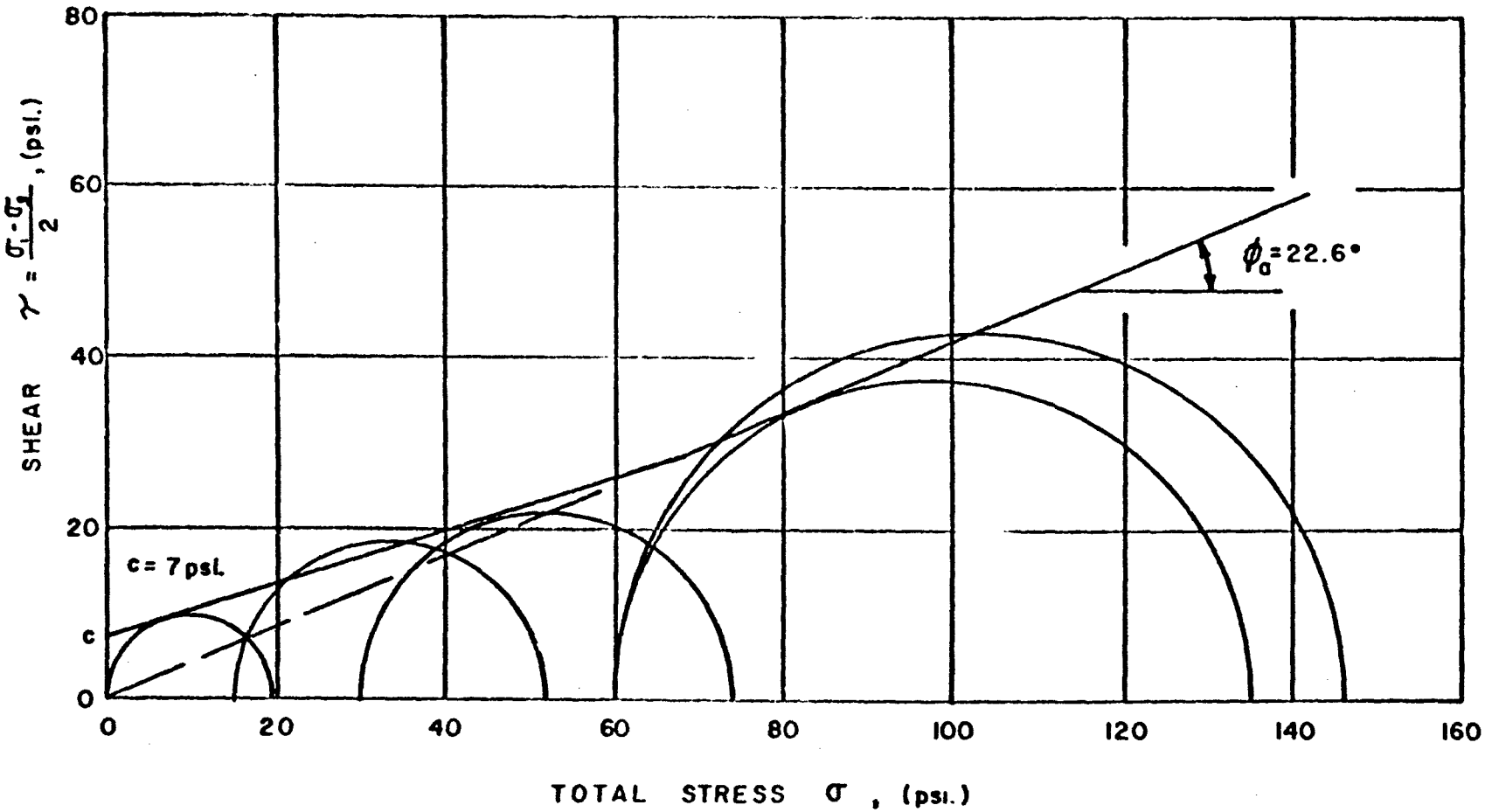
# MOHR STRESS CIRCLES

(ASSUMING FAILURE AT 6% STRAIN)



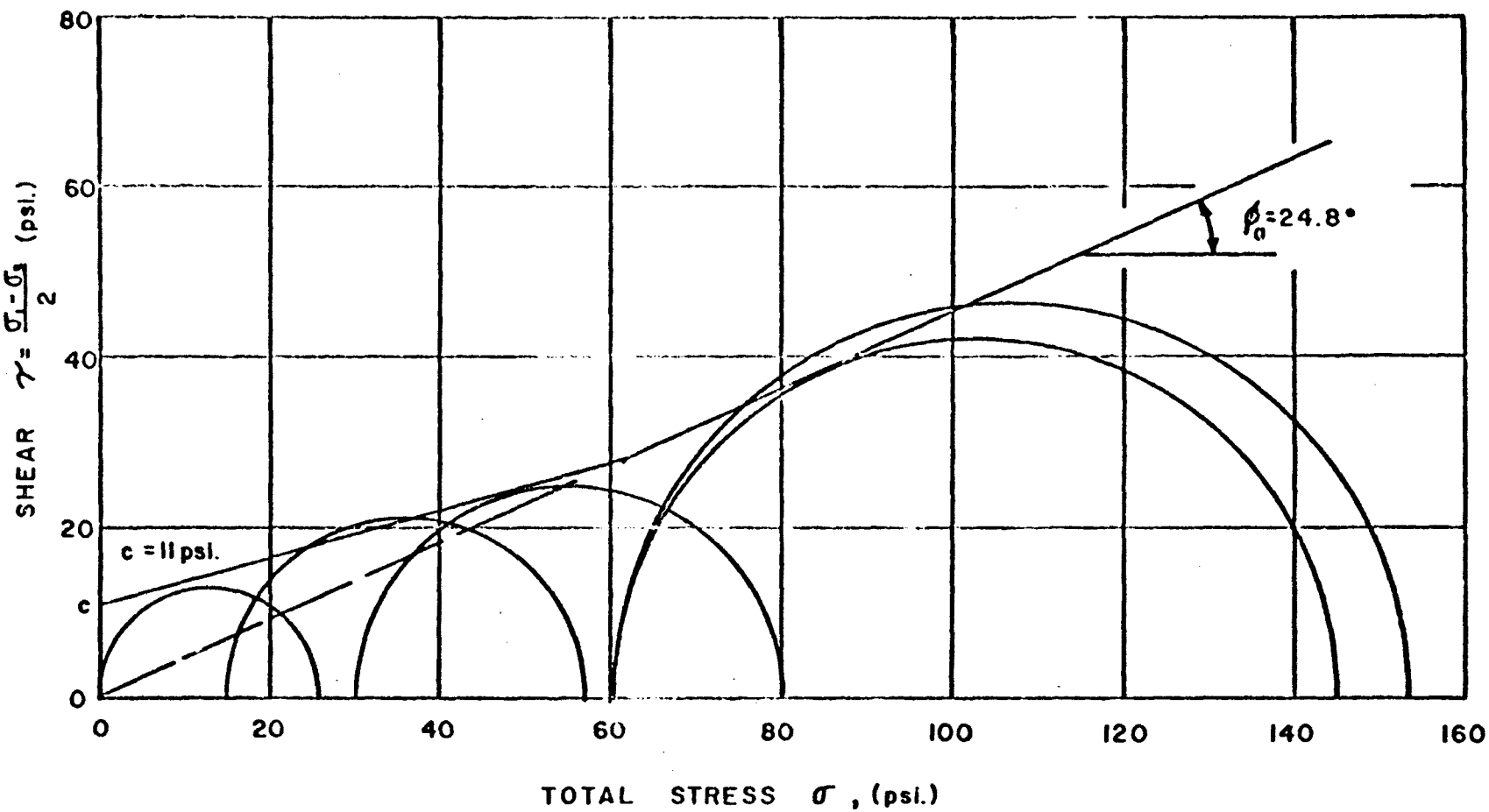
# MOHR STRESS CIRCLES

(ASSUMING FAILURE AT 8% STRAIN)



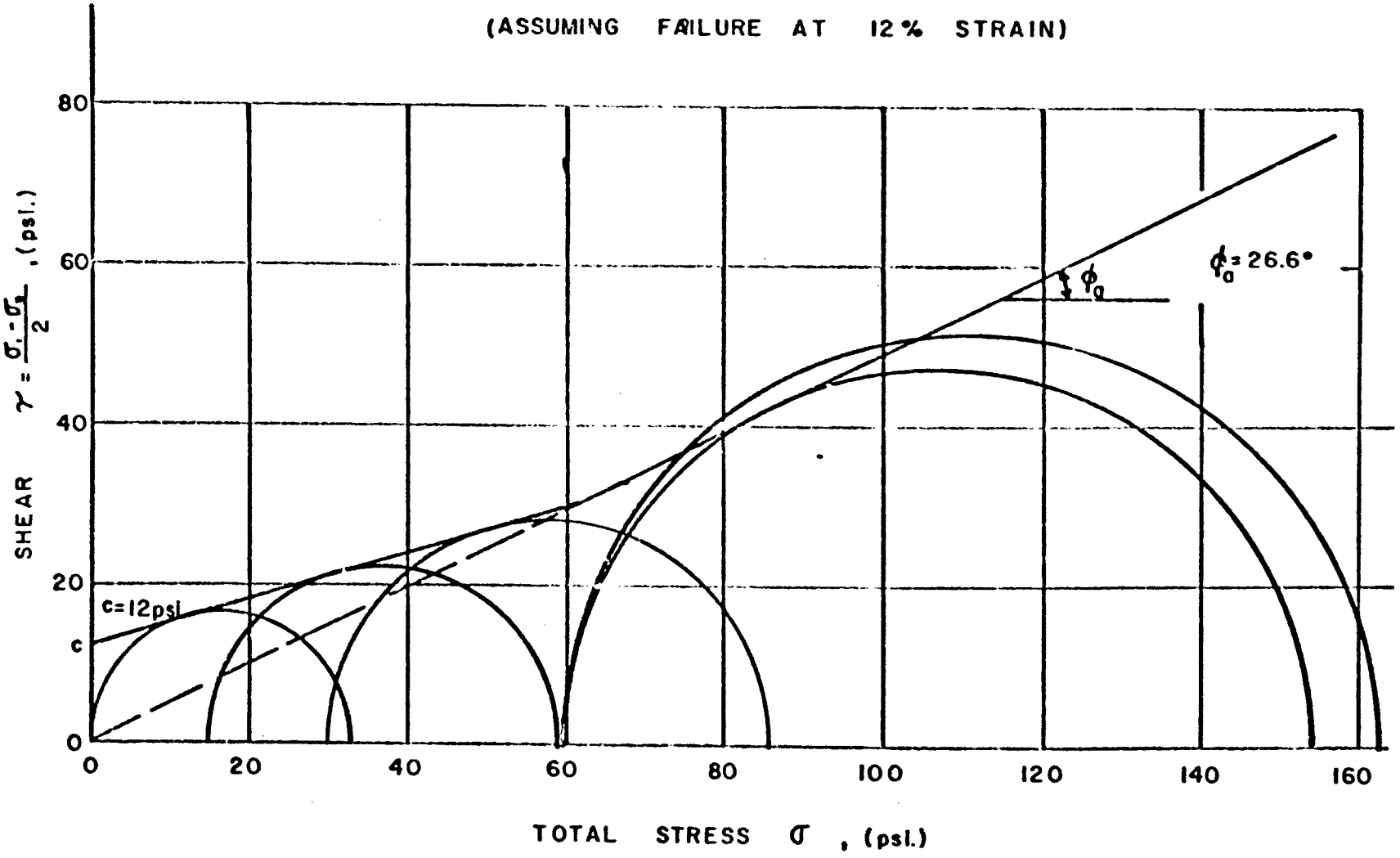
# MOHR STRESS CIRCLES

(ASSUMING FAILURE AT 10% STRAIN)



# MOHR STRESS CIRCLES

(ASSUMING FAILURE AT 12% STRAIN)




T6

# APPENDIX E

## Results of Consolidation Tests

1. Test Data (3)
2. Deformation versus Log Time (6)
3. Typical Curve, Void Ratio versus Pressure (1)
4. Void Ratio versus Log Pressure (3)
5. Summary of Time-Compression Behavior (Figure 1)
6. Summary of Load-Compression Behavior (Figure 2)

# CONSOLIDATION TEST, 1

$\omega_o = 61.0\%$ $\quad\quad$ $2H_o = 0.278$ in

| APPLIED PRESSURE ($T/FT^2$) | SCALE LOAD (lbs) | FINAL DIAL (in) | DIAL CHANGE (in) | 2H FROM DIAL CHANGE (in) | VOID HEIGHT $2H-2H_o$ (in) | VOID RATIO $e$ | $t_{90}$ (sec) | $c_v$ ($10^{-5}$ $in^2/sec$) | $a_v$ ($10^{-5}$ $ft^2/lb$) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | 0.0000 | | 0.8450 | 0.567 | 2.04 | | | |
| | | | 0.0269 | | | | 31.98 | 46.3 | 236 |
| $\frac{1}{8}$ | 10.2 | 0.0269 | | 0.8180 | 0.540 | 1.94 | | | |
| | | | 0.0141 | | | | 7.78 | 180 | 78.9 |
| $\frac{1}{4}$ | 20.4 | 0.0410 | | 0.8040 | 0.526 | 1.88 | | | |
| | | | 0.0258 | | | | 18.15 | 73.3 | 39.4 |
| $\frac{1}{2}$ | 40.8 | 0.0668 | | 0.7780 | 0.500 | 1.80 | | | |
| | | | 0.0242 | | | | 67.42 | 18.48 | 19.7 |
| 1 | 81.6 | 0.0910 | | 0.7540 | 0.476 | 1.71 | | | |
| | | | 0.0452 | | | | 117.6 | 9.66 | 9.85 |
| 2 | 163.2 | 0.1362 | | 0.7090 | 0.431 | 1.55 | | | |
| | | | 0.0579 | | | | 264.6 | 3.70 | 4.92 |
| 4 | 326.4 | 0.1941 | | 0.6510 | 0.373 | 1.34 | | | |
| | | | 0.0552 | | | | 161.38 | 5.11 | 2.46 |
| 8 | 652.8 | 0.2693 | | 0.5960 | 0.318 | 1.14 | | | |
| | | | -0.0118 | | | | | | |
| 2 | 163.2 | 0.2575 | | 0.6080 | 0.330 | 1.19 | | | |
| | | | -0.0455 | | | | | | |
| $\frac{1}{16}$ | 5 | 0.2120 | | 0.6530 | 0.375 | 1.35 | | | |

# CONSOLIDATION TEST, 2

$\omega_o = 74.3\%$ $\quad\quad$ $2H_o = 0.282$ in.

| APPLIED PRESSURE ($T/FT^2$) | SCALE LOAD (lbs) | FINAL DIAL (in) | DIAL CHANGE (in) | 2H FROM DIAL CHANGE (in) | VOID HEIGHT $2H - 2H_o$ (in) | VOID RATIO $e$ | $t_{90}$ (sec) | $C_v$ ($10^{-5}$ $in^2/sec$) | $a_v$ ($10^{-5}$ $FT^2/LB$) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | 0.0000 | | 0.8450 | 0.563 | 2.00 | | | |
| | | | 0.0380 | | | | 101 | 143 | 3 |
| $\frac{1}{8}$ | 10.2 | 0.0380 | | 0.8070 | 0.525 | 1.86 | | | |
| | | | 0.0092 | | | | 696 | 20 | 96 |
| $\frac{1}{4}$ | 20.4 | 0.0472 | | 0.7978 | 0.516 | 1.83 | | | |
| | | | 0.0292 | | | | 675 | 19 | 48 |
| $\frac{1}{2}$ | 40.8 | 0.0764 | | 0.7686 | 0.487 | 1.73 | | | |
| | | | 0.0236 | | | | 5220 | 2 | 24 |
| 1 | 81.6 | 0.1000 | | 0.7450 | 0.463 | 1.64 | | | |
| | | | 0.0411 | | | | 1305 | 9 | 12 |
| 2 | 163.2 | 0.1411 | | 0.7039 | 0.422 | 1.50 | | | |
| | | | 0.0584 | | | | 3860 | 5 | 6 |
| 4 | 326.4 | 0.1995 | | 0.6455 | 0.364 | 1.29 | | | |
| | | | 0.0701 | | | | 4068 | 2 | 3 |
| 8 | 652.8 | 0.2696 | | 0.5754 | 0.293 | 1.04 | | | |
| | | | -0.0116 | | | | | | |
| 2 | 163.2 | 0.2580 | | 0.5870 | 0.305 | 1.08 | | | |
| | | | -0.0377 | | | | | | |
| $\frac{1}{16}$ | 5 | 0.2203 | | 0.6247 | 0.343 | 1.22 | | | |

# CONSOLIDATION TEST, 3

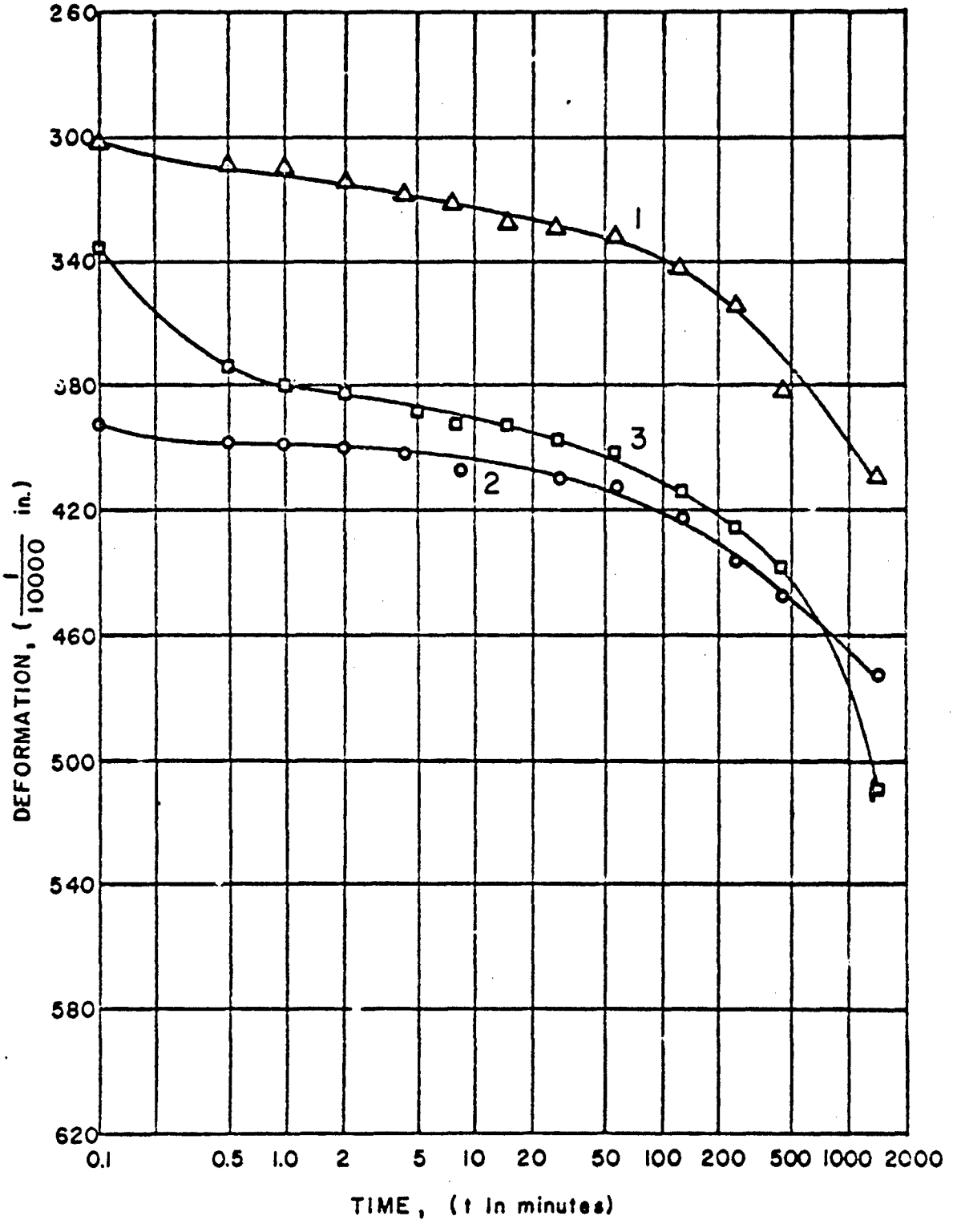
$\omega_o$ = 81.4 %

$2H_o$ = 0.278 in

| APPLIED PRESSURE ($T/FT^2$) | SCALE LOAD (lbs) | FINAL DIAL (in) | DIAL CHANGE (in) | 2H FROM DIAL CHANGE (in) | VOID HEIGHT 2H - 2H$_o$ (in) | VOID RATIO, e | $t_{90}$ (sec) | $C_v$ ($10^{-5}$ $in^2/sec$) | $a_v$ ($10^{-5}$ $ft^2/lb$) |
|---|---|---|---|---|---|---|---|---|---|
| 0 | 0 | 0.0000 | | 0.8450 | 0.5670 | 2.04 | | | |
| | | | 0.0330 | | | | 21.6 | 675 | 380 |
| 1/8 | 10.2 | 0.0330 | | 0.8120 | 0.5340 | 1.92 | | | |
| | | | 0.0180 | | | | 93.6 | 146 | 127 |
| 1/4 | 20.4 | 0.0510 | | 0.7940 | 0.5160 | 1.86 | | | |
| | | | 0.0349 | | | | 43.4 | 294 | 63 |
| 1/2 | 40.8 | 0.0859 | | 0.7591 | 0.4811 | 1.73 | | | |
| | | | 0.0081 | | | | 86.4 | 140 | 32 |
| 1 | 81.6 | 0.0940 | | 0.7510 | 0.4730 | 1.70 | | | |
| | | | 0.0413 | | | | 117.6 | 96 | 16 |
| 2 | 163.2 | 0.1353 | | 0.7097 | 0.4317 | 1.55 | | | |
| | | | 0.0557 | | | | 93.6 | 105 | 8 |
| 4 | 326.4 | 0.1910 | | 0.6540 | 0.3760 | 1.35 | | | |
| | | | 0.0759 | | | | 216.6 | 37 | 4 |
| 8 | 652.8 | 0.2669 | | 0.5781 | 0.3001 | 1.08 | | | |
| | | | -0.0117 | | | | | | |
| 2 | 163.2 | 0.2552 | | 0.5898 | 0.3118 | 1.12 | | | |
| | | | -0.0919 | | | | | | |
| 1/16 | 5 | 0.1633 | | 0.6817 | 0.4037 | 1.45 | | | |

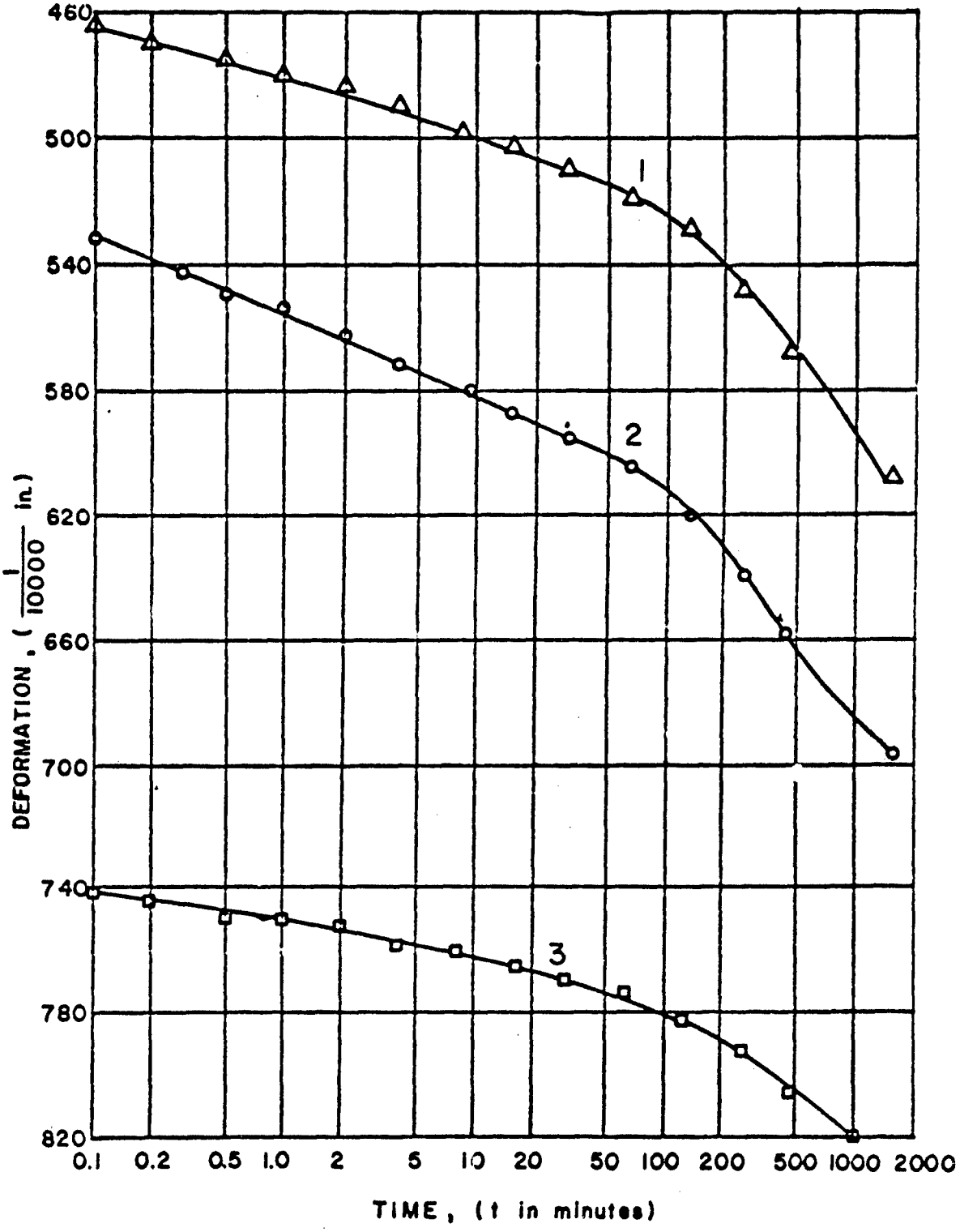
# CONSOLIDATION TEST

PRESSURE INCREMENT: $\frac{1}{8}$ T/FT² TO $\frac{1}{4}$ T/FT²



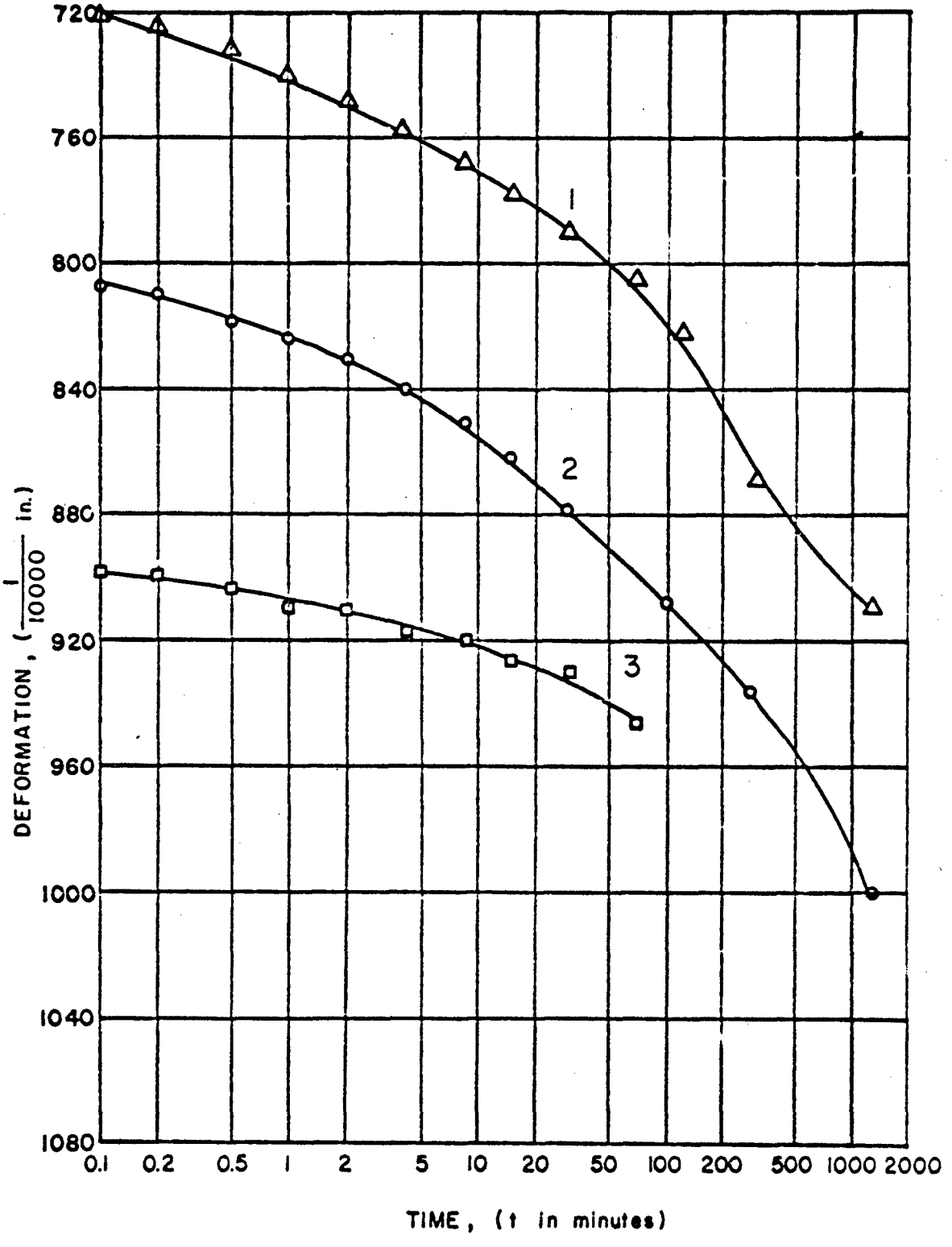
# CONSOLIDATION TEST

PRESSURE INCREMENT: $\frac{1}{4}$ T/FT² TO $\frac{1}{2}$ T/FT²



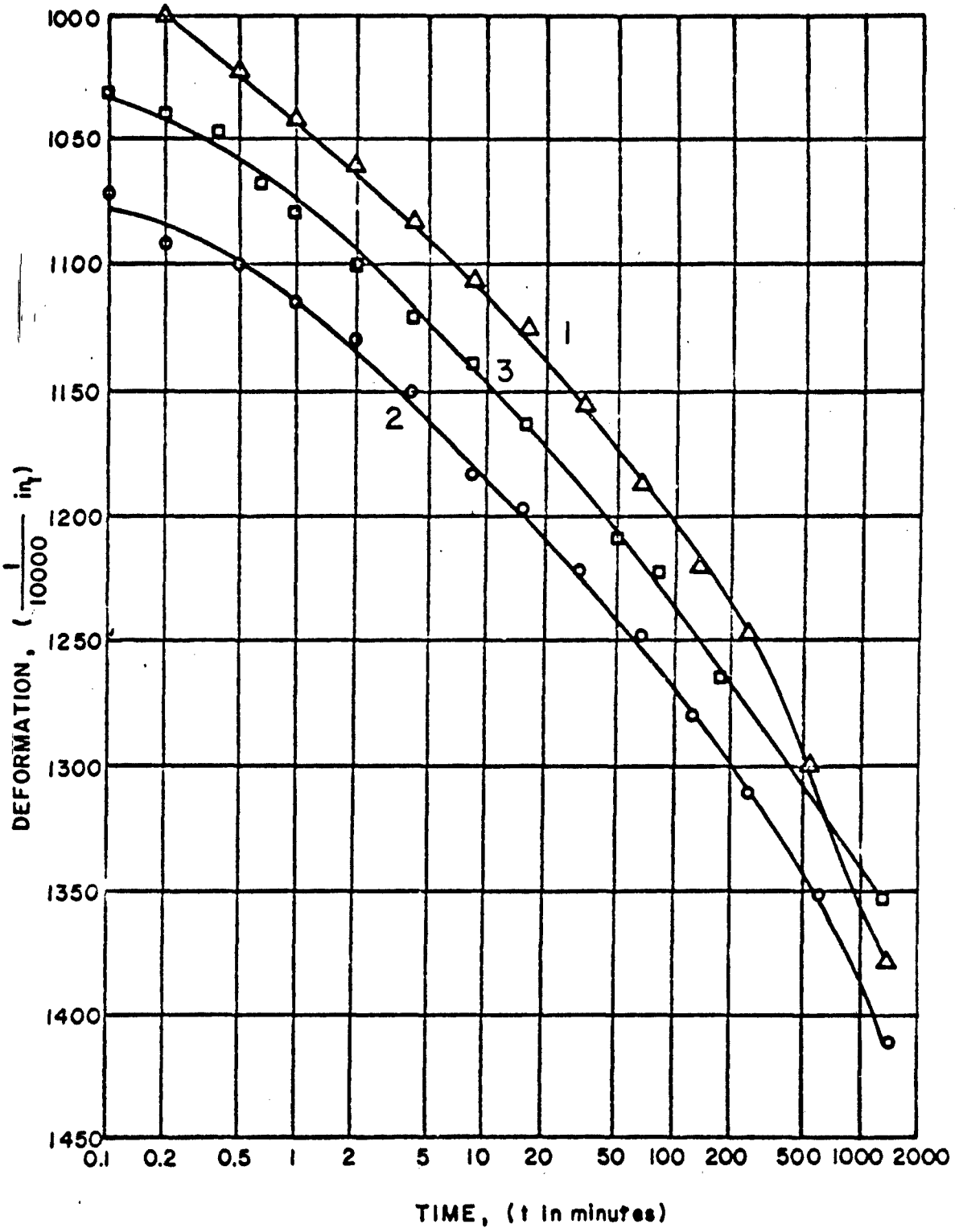
# CONSOLIDATION TEST

PRESSURE INCREMENT: $\frac{1}{2}$ T/FT² TO 1 T/FT²



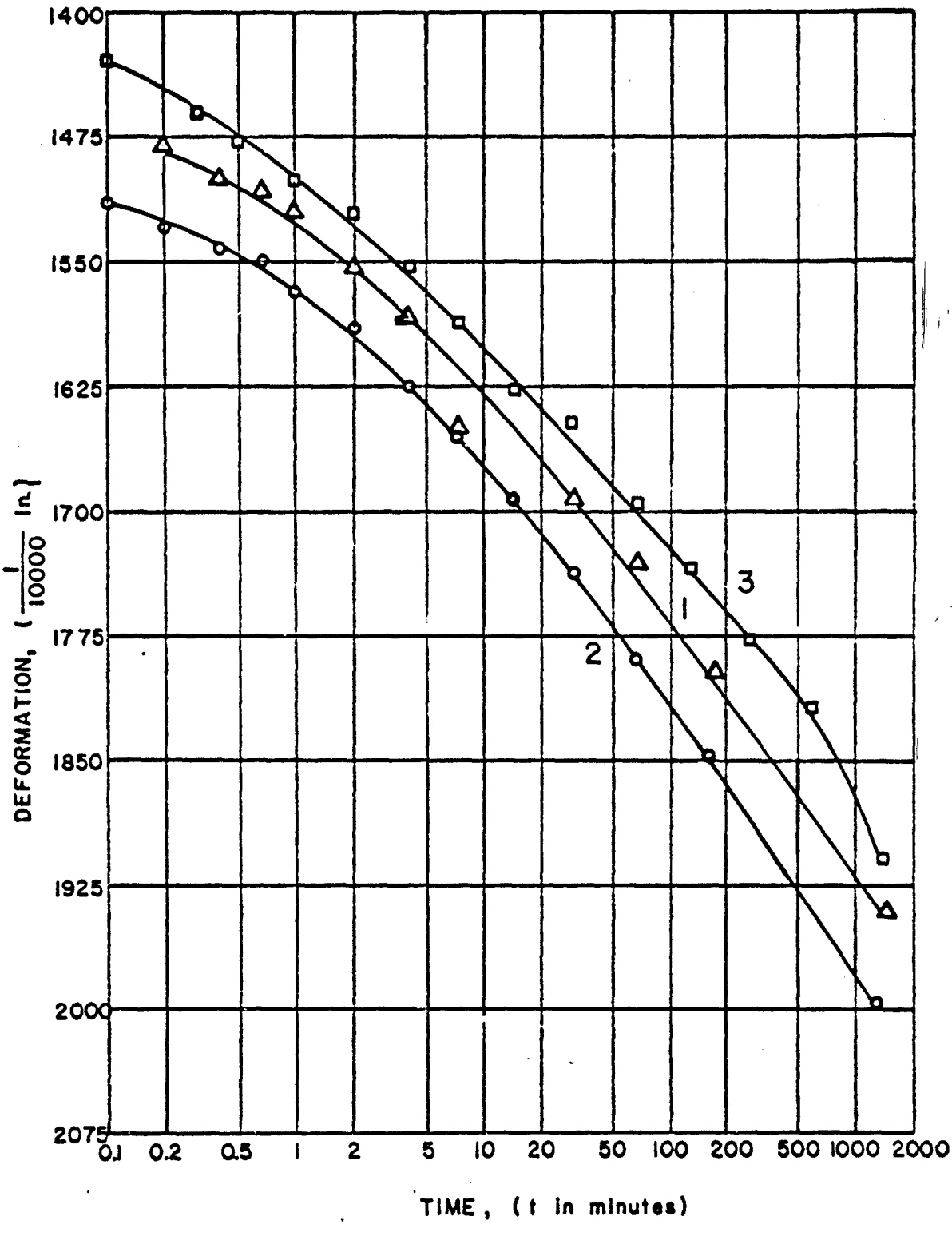
# CONSOLIDATION TEST

PRESSURE INCREMENT: 1 $T/FT^2$ TO 2 $T/FT^2$



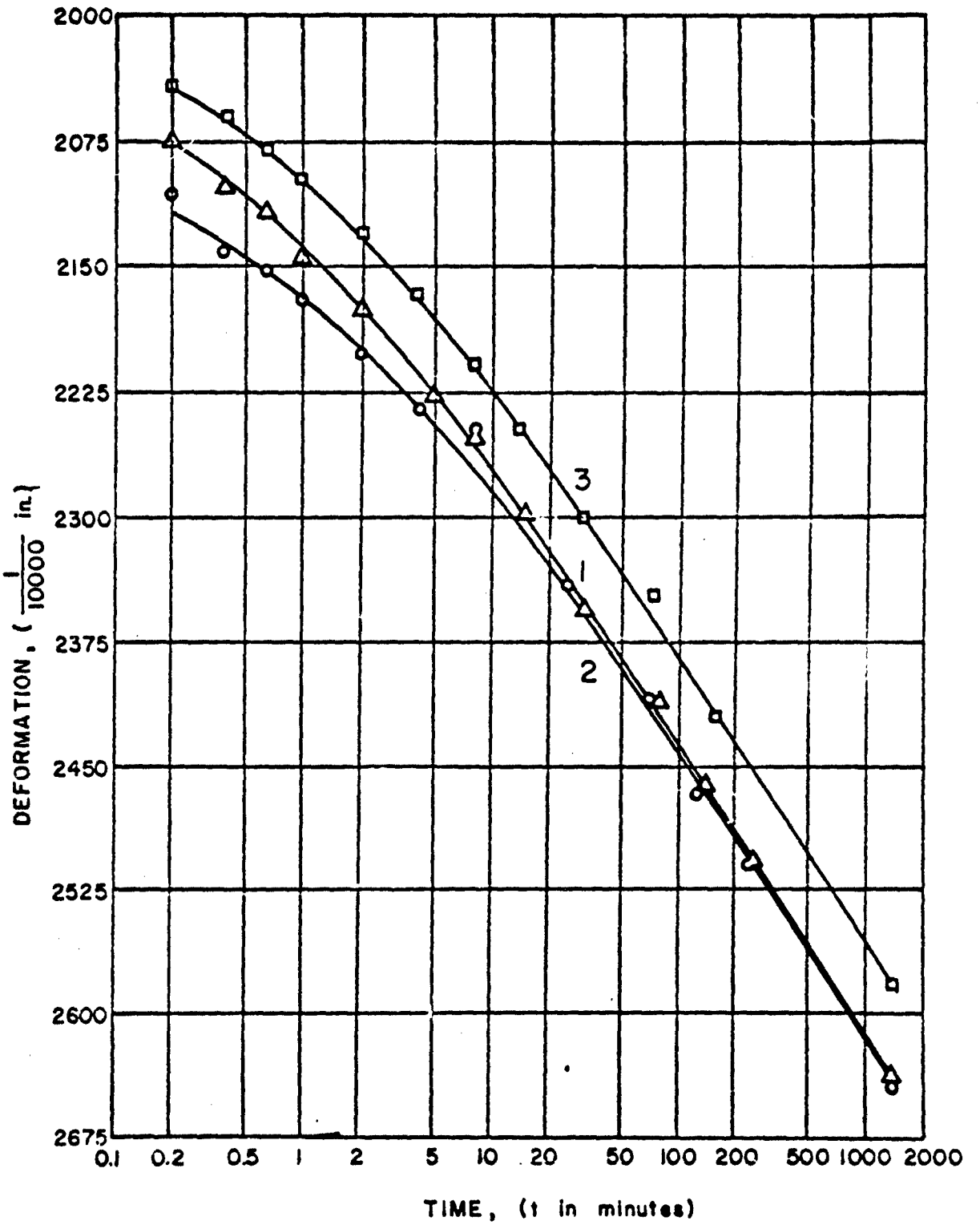
# CONSOLIDATION TEST

PRESSURE INCREMENT: 2 $^{T}/_{FT^2}$ TO 4 $^{T}/_{FT^2}$



# CONSOLIDATION TEST

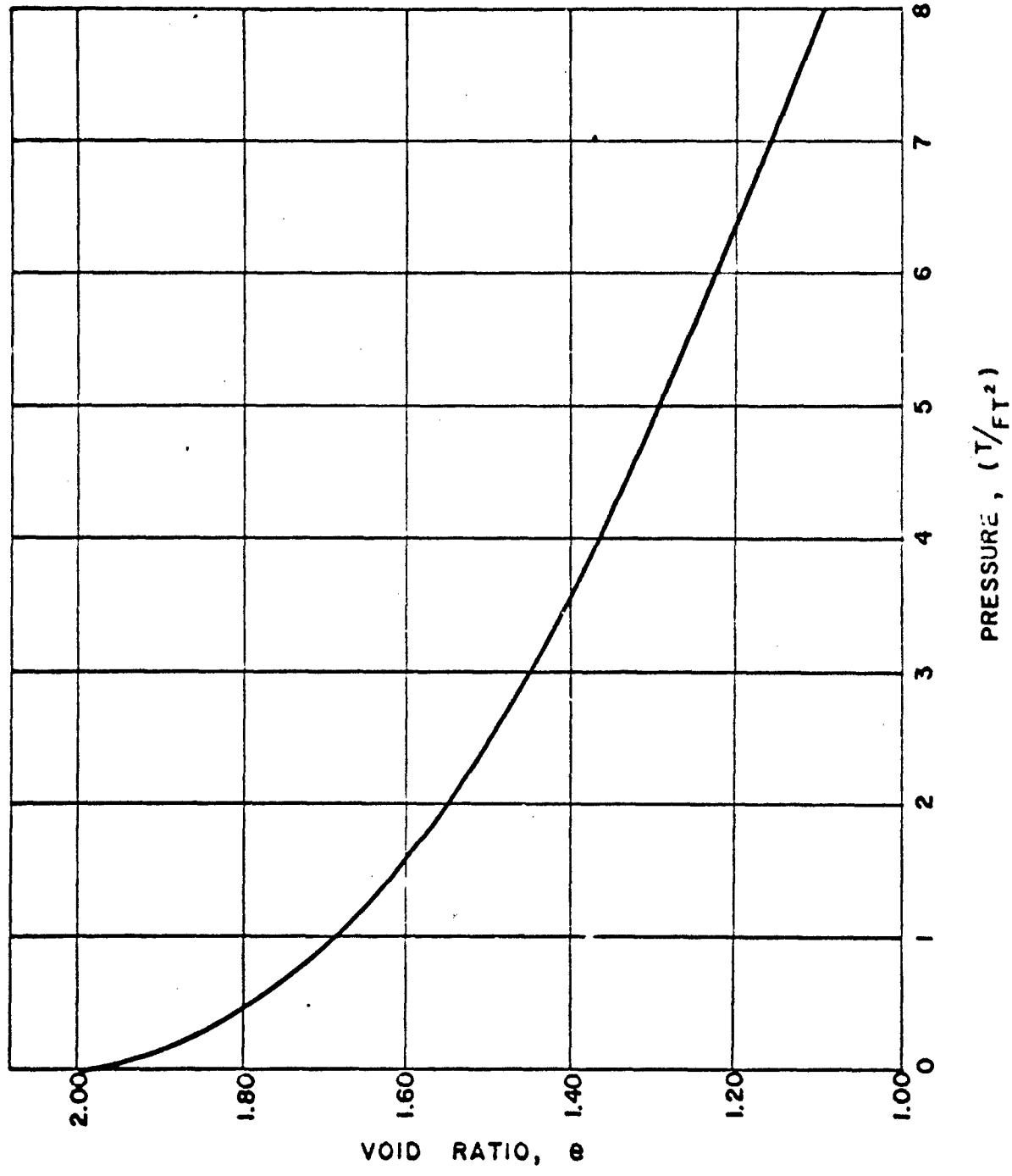
PRESSURE INCREMENT: 4 $^{T}/_{FT^2}$ TO 8 $^{T}/_{FT^2}$



# TYPICAL CURVE

## VOID RATIO VS PRESSURE

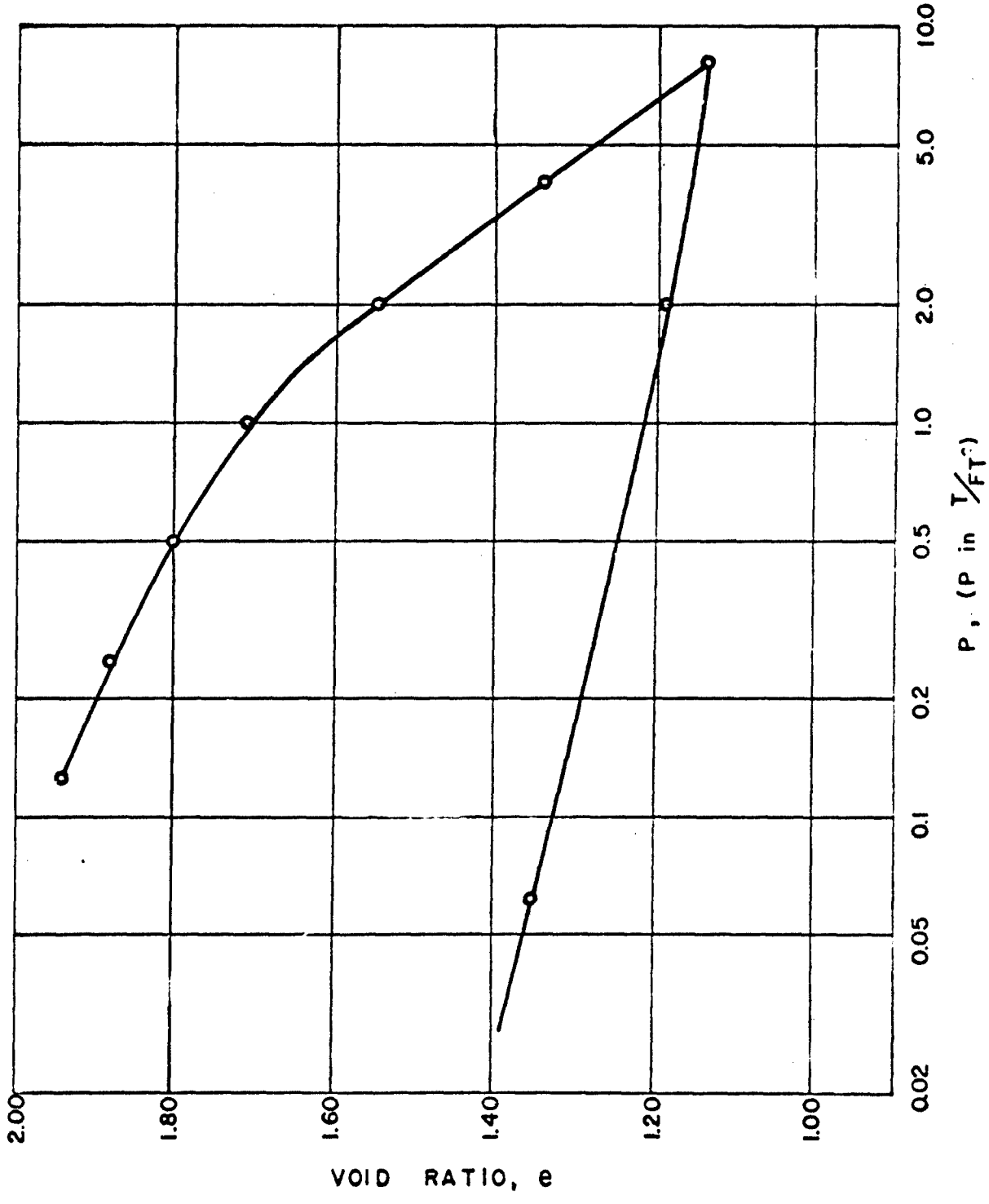
## (FROM CONSOLIDATION TEST DATA)



# TEST 1

## e VS LOG P

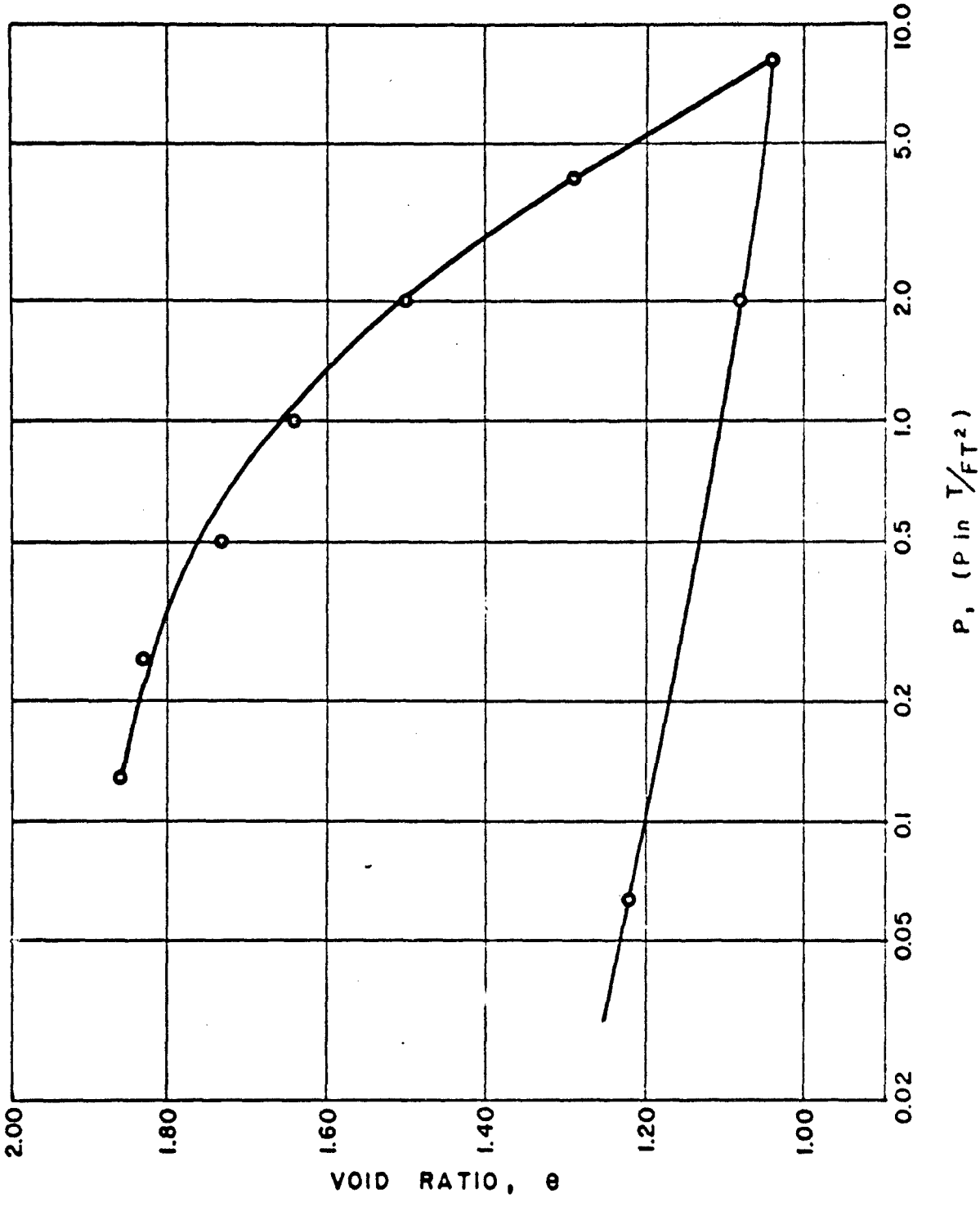
$\omega_o = 61\ \%$ $C_c = 0.68$



# TEST 2

## e VS LOG P

$\omega_o = 74\%$ $C_c = 0.83$



# TEST 3

e VS LOG P

$\omega_o = 81\ \%$ $C_c = 0.90$

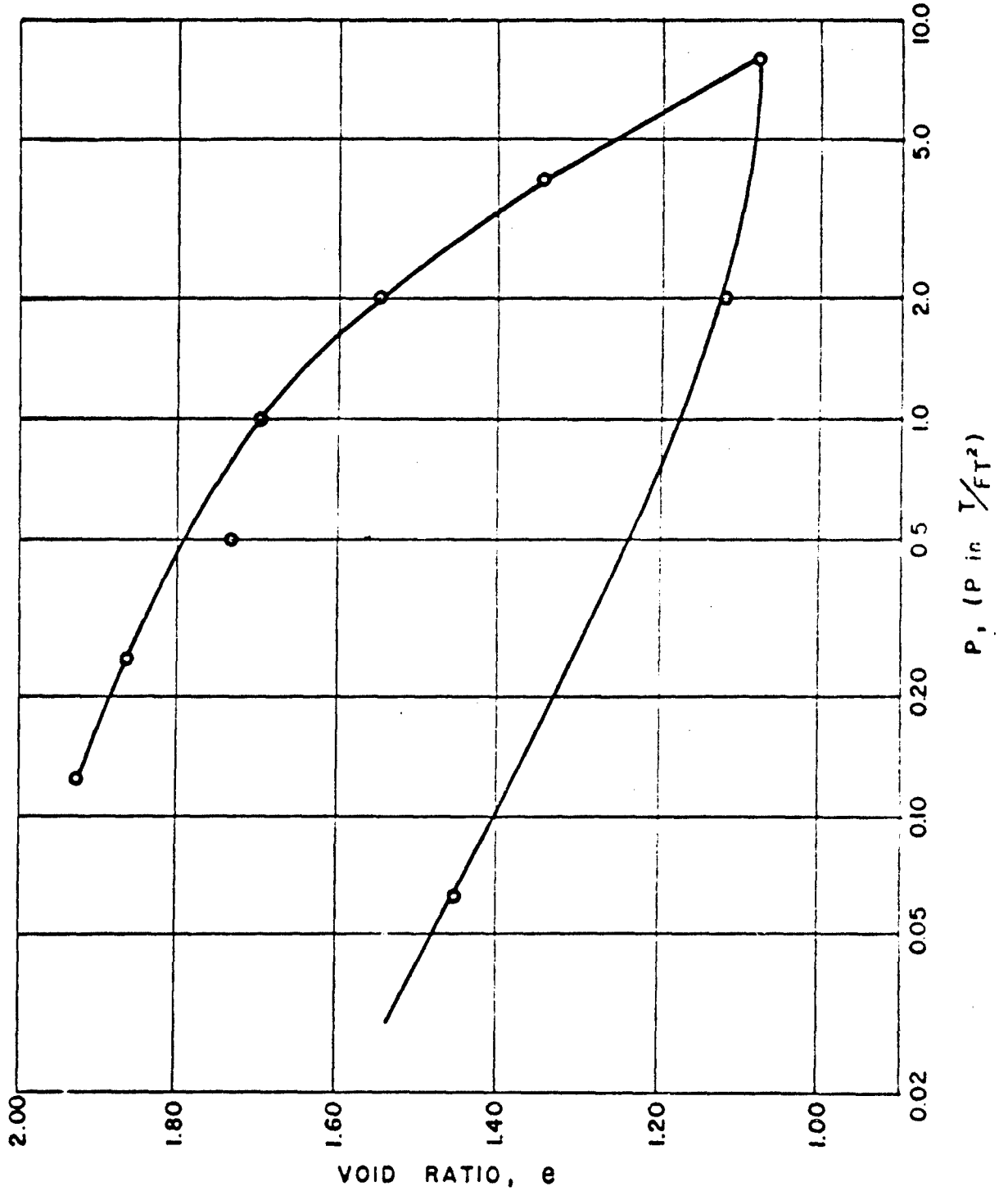


FIGURE 1

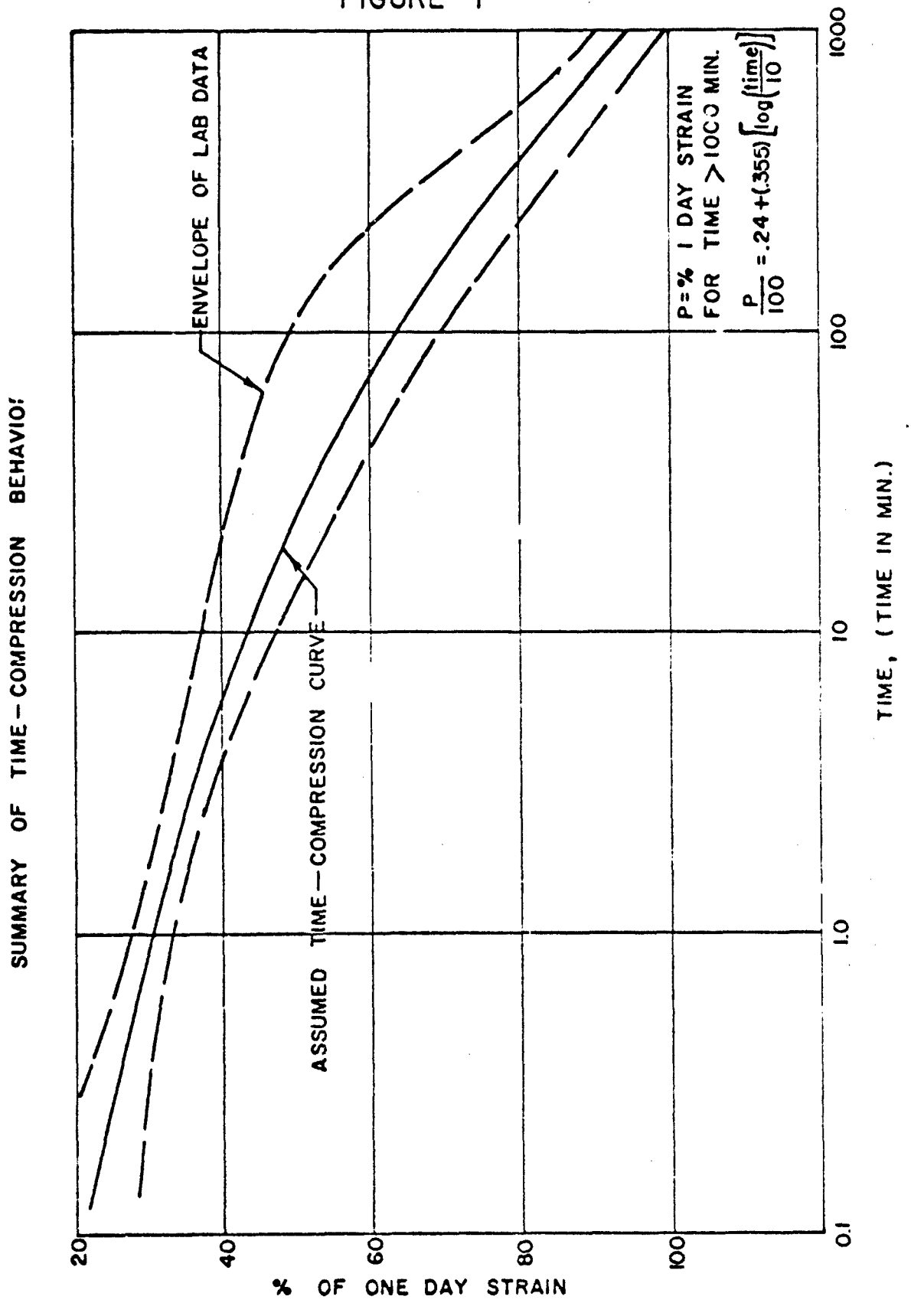
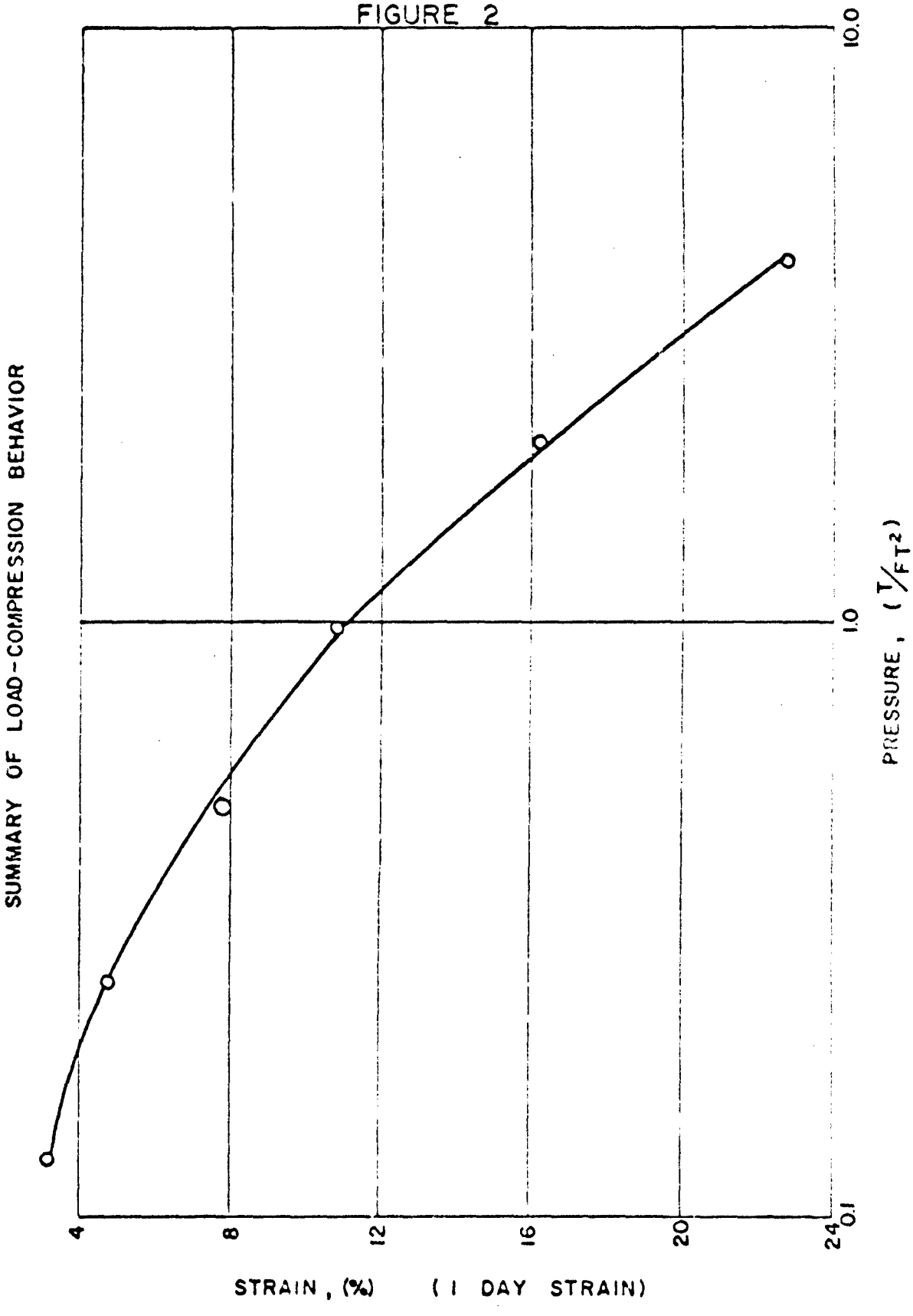

SUMMARY OF TIME-COMPRESSION BEHAVIOR
ENVELOPE OF LAB DATA
ASSUMED TIME-COMPRESSION CURVE
P=% 1 DAY STRAIN FOR TIME >100 MIN.
$\frac{P}{100} = .24 + (.355)\left[\log\left(\frac{time}{10}\right)\right]$
20
40
60
80
100
% OF ONE DAY STRAIN
0.1
1.0
10
100
1000
TIME, (TIME IN MIN.)

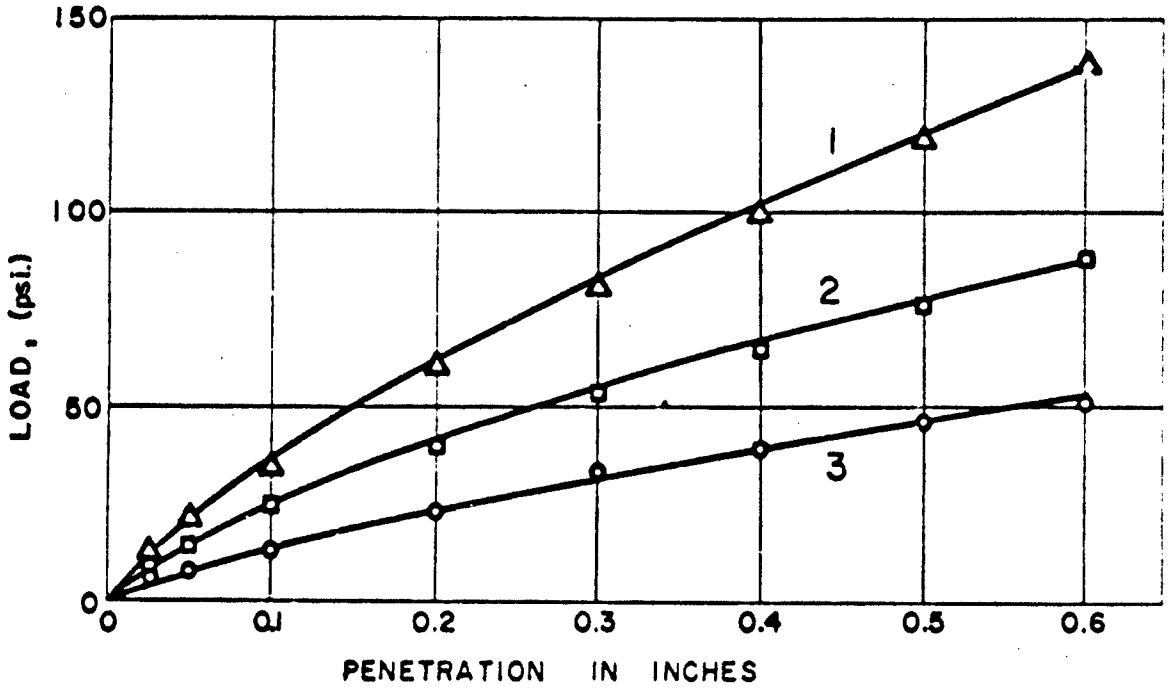
FIGURE 2

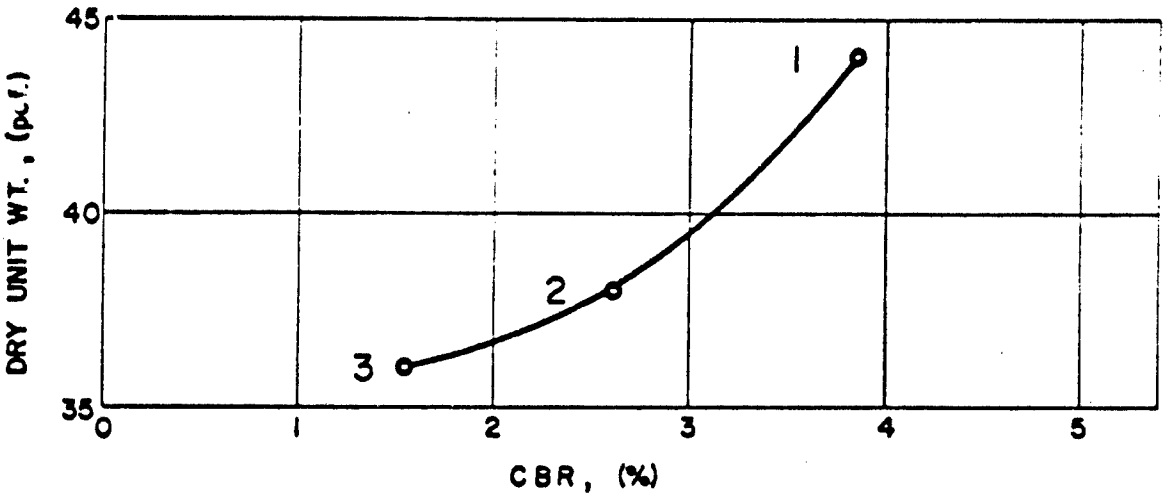

SUMMARY OF LOAD-COMPRESSION BEHAVIOR
10.0
1.0
0.1
PRESSURE, (T/FT²)
4
8
12
16
20
24
STRAIN, (%) (1 DAY STRAIN)

# APPENDIX F

## Results of California Bearing Ratio Test

## CALIFORNIA BEARING RATIO




FINAL CBR INFORMATION

| TEST NO. | CBR (%) | $\gamma_w$ (pcf) | $\gamma_o$ (pcf) | $\omega_o$ (%) |
|---|---|---|---|---|
| 1 | 3.86 | 79.4 | 44.1 | 80 |
| 2 | 2.62 | 68.4 | 38.0 | 80 |
| 3 | 1.55 | 64.8 | 36.0 | 80 |

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