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Mechanical behavior of chemically treated soils, Oct. 1956

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MECHANICAL BEHAVIOR OF CHEMICALLY TREATED SOILS

by

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INTRODUCTION

On June 1, 1956 Lehigh University commenced an investigation into the Mechanical Behavior of Chemically Treated Soils. This investigation is under the sponsorship of American Cyanamid Company for the specific purpose of studying the effect of stabilizer AM-955 on the mechanical properties of soils.

The study of soil mechanics can in many respects be broken down into two studies.

1. Study of Granular soils.
2. Study of Clay-soils.

Clay-soils are fundamentally defined as follows:

"A clay-soil is a soil that exhibits elasto-visco-plastic properties at a characteristic moisture content."

This soil definition prescribes a type of mechanical behavior, and thus the definition of controlling properties is not as fundamental as it should be. Primarily for this reason any study of chemical additives to clay-soils should be preceded by fundamental investigations into clay-soil properties without additives, or be within the region of well defined soil properties.

Considering the preliminary nature of this investigation into the effects of chemical additives, it was decided to forego any study of the clay-soils, at this time.

The granular soils can be very adequately defined in terms of the geometry of the particles on a structural level of observation.

"A granular soil is an aggregation of inorganic mineral grains that will exhibit no measurable surface activity when saturated with water."

Well documented research into the behavior of granular soils (Reference 1) has proven that the mechanical properties of these materials is governed by:

1. Grain size
2. Grain shape
3. Variation of grain sizes
4. Shape of grain size distribution curve.

While the above four variables, in general control all granular soil behavior, the range of action is defined by the Relative Density (Reference 2) of the soil. Relative Density is the particular density state of a soil as referred to the loosest and densest laboratory states of packing. In terms of the voids ratio of the soil, Relative Density is formulated as follows:

$$D_R = \frac{e_L - e_i}{e_L - e_D} (100) \qquad . \quad . \quad . \quad . \quad . \qquad (1)$$

e_L = Voids ratio in loosest laboratory state.

e_D = Voids ratio in densest laboratory state.

e_i = Voids ratio in the particular condition being studied.

D_R = Relative Density (%)

Any given granular soil has finite and definite upper and lower limits of density. All the behavior of that soil, in its natural condition can be prescribed within these limits as a function of the Relative Density.

The mechanical behavior of any body is defined by the concepts of classical mechanics (Reference 3), as the response of a material body to the effects of force, time, and temperature in terms of the geometry of the material body. The measurement of the response is in terms of the deformation or strain of the body with respect to the stress, time and temperature.

Due to the complexities of theoretical mathematical solutions to these general postulations, only the simplest of these problems have been rigorously solved. The largest class of these problems are the problems in the theory of linear elasticity, which presupposes that the geometric response of a body is time independently proportional to the imposed stresses and temperatures in a linear manner. Other postulations of behavior have never been solved theoretically, with the exception of a few of the simpler problems.

An approach to the problem of inelastic behavior has been made on a one-dimensional model basis. In this approach the response of the material is postulated on the basis of a one-dimensional model where the model units represent various components of behavior. As an example, the so-called Kelvin Model of visco-elasticity uses a spring unit, for the elastic responses, in parallel with a dashpot unit for viscous component as shown in Figure 1.

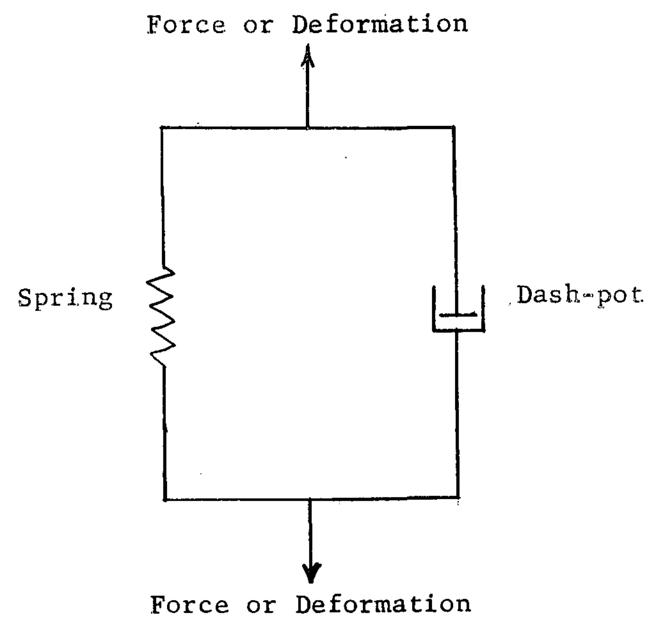


FIGURE 1

KELVIN VISCO-ELASTIC MODEL

The mechanical response of the above system to a constant force (P) is as shown in Figure 2.

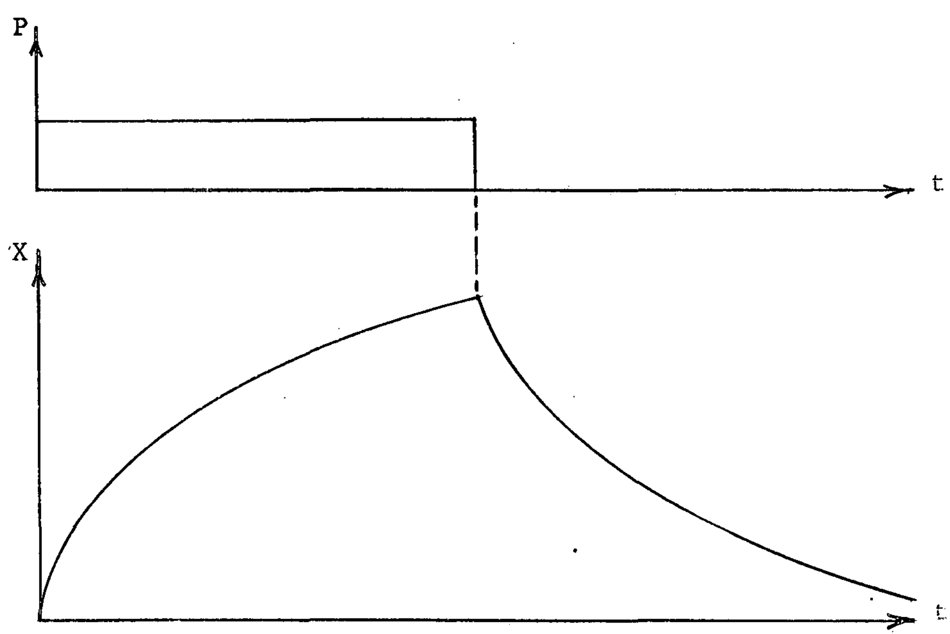


FIGURE 2

MECHANICAL RESPONSE OF A KELVIN MODEL

CONSTANT FORCE

In any fundamental study of a new material the postulation of a model of behavior becomes the basic property to seek, as this postulation enables a statement of response that becomes general under all circumstances.

Problems in soil mechanics can be discribed in many instances, as problems involving the failure of soils to sustain man made structures. Since the forces and deformations imposed on the soil are often such that the soil is on the verge of failure, it becomes necessary to examine the criteria for soil failures. These criteria, although related to the model of behavior, are defined as entities of their own.

The most useful and common failure criteria is the one proposed by Coulomb and graphically described by Mohr. The formulation of the Coulomb Hypothesis is:

$$\tau_f = c + \sigma \tan \varphi \quad . \quad . \quad . \quad . \quad . \qquad (2)$$

τ_f = shearing stress acrpss failure surface

c = maximum shearing stress under conditions of equal and oppositly sensed principal stresses.

σ = normal stress

φ = angle of internal friction.

The Mohr representation of the Coulomb Hypothesis is shown in Figure 3.

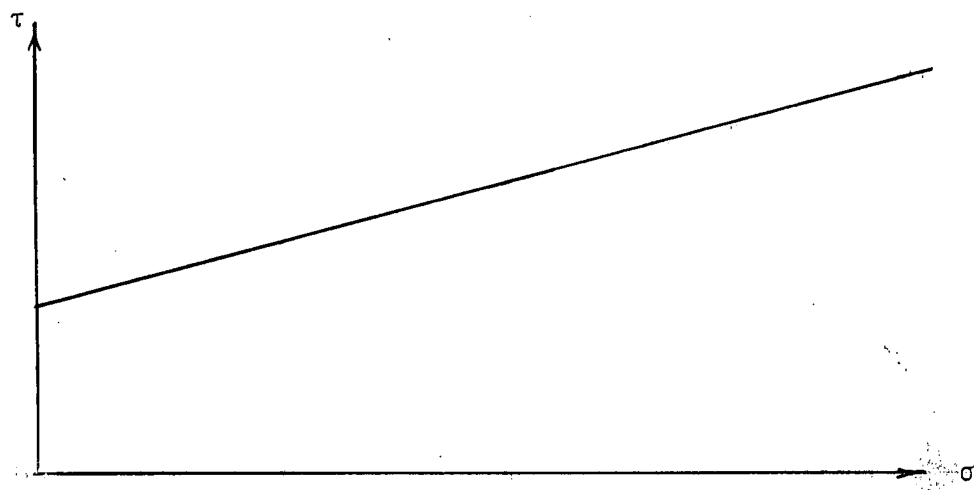


FIGURE 3

MOHR-COULOMB FAILURE HYPOTHESIS

Soil stabilization in its broadest conception, is simply a process or series of processes by which the soil properties are changed or controlled (Ref. 4). In general the stabilization can be carried out in several ways:

(1) Densification: In which the soil is altered by the introduction of external mechanical energy, such as compaction or changes in soil moisture.

(2) Electrical Stabilization: Stabilization by electrical means accomplish the end objectives of property alteration, by the introduction of electrical current to the soil mass.

(3) Additives: The use of additives has come to mean, a change in properties by adding either soil, cement or Bitumen. In these situations, the composition of the material is altered to one with more favorable properties.

(4) Chemical Stabilizers: Although the addition of chemicals to soils, can be considered to be identical with the use of additives,

their property alteration is a much more complex phenomena, and as a result can be treated separately.

Basically we can consider four mechanisms at work in the stabilized soil, any combination of which will form the agent of stabilization (Ref.5). The first type, is that by which the chemical forms a continuous matrix in the system.

Under these conditions, either the soil acts as an inert filler within the matrix, or the soil particles interact with the chemical to form a constituent part of the matrix. The end properties of this system are essentially the properties of the stabilizing agent and not the original soil, and any response to mechanical forces will be governed by the response of the chemical.

If the chemical does not form a continuous system, there are three additional types of action. The first of these is one in which the chemical alters the surface characteristics of the soil and changes the bonding mechanism between soil particles. The second method of action is that of forming a void filler. This type of action simply places mechanical constraints on the deformation of single particles, and thus alters their response to mechanical forces. The third type of action, which is the action of AM-955 on granular soils, is that of connecting soil particles. In effect the chemical adhers to the grains of soil and imposes force restraints on the entire system.

It is the broad objective of this study to establish criteria for the behavior of granular soils when treated with AM-955. The comportment considered will be for both pre-failure and failure conditions. The criteria for the response are those properties of granular soils that have been proven to control behavior in the unstabilized state. This study was undertaken to ascertain quantative information in terms of the previously described concepts of soil behavior, in such a form that, within the range of the study, this information will be of practical use in the solution of engineering problems.

RESEARCH PROGRAM

The research program (Reference 6) was designed as an experimental program based on a qualative theoretical hypothesis. The experimantal program itself is based upon a statistical hypothesis, upon which the results can be analyzed to determine the validity of the theoretical hypothesis and the probable limits of behavior.

THEORETICAL HYPOTHESIS

The general theoretical hypothesis, upon which this study was based is as follows:

The mechanical response of granular soils when treated with stabilizer AM-955 will be dependent on the following parameters:

1. Soil Effects
   a) Mean grain size
   b) Average grain shape
   c) Grain size variation
   d) Relative Density of the soil

2. Chemical Effects
   a) Concentration of monomer
   b) Age of gel
   c) Thermodynamic conditions
      A. Temperature
      B. Relative Humidity

3. Moisture Concentration of Soil

STATISTICAL HYPOTHESIS

Although a statistical hypothesis is a necessity in any experimental program, its usefulness is sometimes limited. The most efficient use of statistical methods occurs when the phenomena being studied is well established, and the purpose of the experimentation is to determine stationary response, either maximum or minimum. Under these conditions, such techniques as analysis of variance, sequential analysis, and factorial design are useful tools to most effeciently design and analyze the experiment.

When the purpose of the experimental program is to <u>define behavior</u> over an extended area, the probelm is statistically undefined. In this circumstance the above methods are of little value, and can even be misleading and wasteful. The best use of statistical methods under these conditions is to define the trend of behavior by use of limited amounts of data combined with regression analysis. Such analysis will define the phenomena experimentally and establish probability limits for the variations of experimental replication, and service response.

With the above discussion as a basis the following statistical hypothesis was postulated.

The mechanical response of granular soils when treated with stabilizer AM-955 and determined experimentally is based on the hypothesis that each experiment is an independent event taken from a Gausian population of behavior.

Thus a predetermined number of experiments over a given range of behavior was performed and the responses determined over the full range, along with the probabilities that any future experiment will fall within computed limits.

EXPERIMENTAL PROGRAM

Due to the necessary and personnel limitations of this study the variables of the theoretical hypothesis were limited to these variables which were considered of primary importance. In this initial stage these variables were:

1. Mean grain size.
2. Grain size variation.
3. Relative Density of the soil.
4. Age of gel for a particular soil and particular state at compaction.

The constancy of the other variables were as follows:

1. Concentration of Monomer held at 7% by weight.
2. Thermodynamic conditions held at room temperature and humidity.
3. Moisture concentration of the soil held at 97-100% saturation.
4. Where gel age was not investigated it was held to the initial stage of formation.

The mechanical variables under investigation were:

1. Strain controlled unconfined compression strength at constant strain rate.

2. Strain controlled unconfined compression strength at variable strain rates for a few selected samples.
3. Volume-density changes for one soil under a single state of compaction.
4. Strain controlled triaxial compression strength at constant strain rate for all soil conditions in the newly gelled state.
5. Pilot relaxation phenomena on a few selected samples.
6. Load-unload-reload strain controlled unconfined compression phenomena at constant strain rate for a few selected samples.

The experimental program outlined above is being carried out by the authors of this report in the Soil Mechanics Laboratory at Lehigh University.

As of October 1, 1956 the laboratory phase of items (1), (2) and (3), above were substantially completed. Item (4) is currently in progress, with the item (5) and (6) being completed experimentally, and in the process of analysis.

Test Procedures

Two soils were obtained from local suppliers for testing purposes. The basic soils were river deposits composed mostly of silicates.

The soil variables in this investigation were selected as being representative of a wide range of granular soils found in nature. The natural soils obtained were hand sieved and combined in the manner shown in Table 1 and Figure 4.

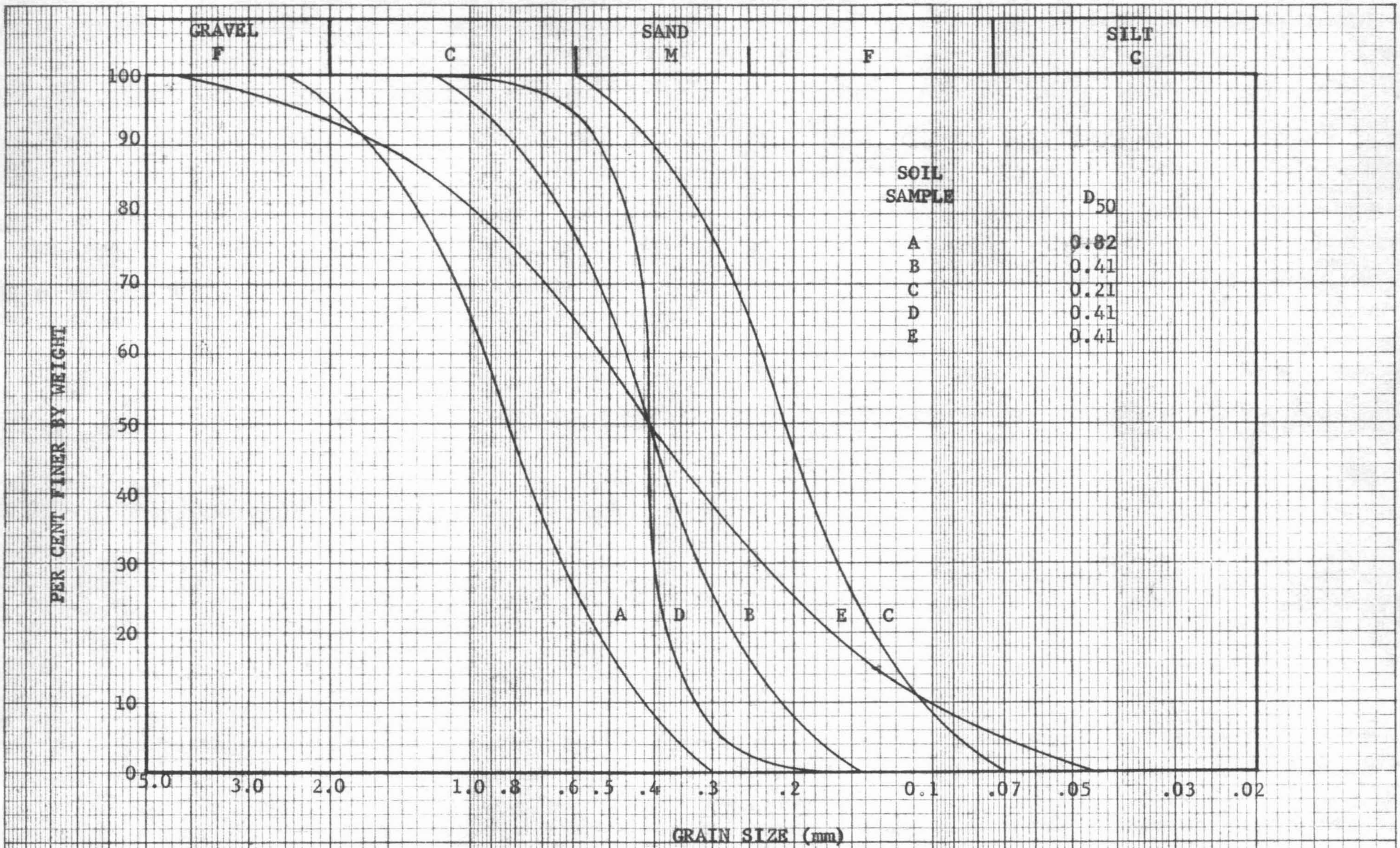


FIGURE 4
GRAIN SIZE DISTRIBUTION OF EXPERIMENTAL SOILS

TABLE 1

GRAIN-SIZE DISTRIBUTION OF SELECTED SOIL SAMPLES

Sieve Proportion (%)

Sample	Sieve Number #4	#8	#16	#30	#50	#100	#200
A		25	50	25			
B			25	50	25		
C				25	50	25	
D			5	90	5		
E	5	10	23	24	23	10	5

The soil descriptions were in accordance with the Burmister Identification System (Reference 7). The geometric mean grain size is shown on Figure 4.

A mineral analysis of the soil will be made in the future, along with a microscopic examination.

In this project specific procedures in the molding and testing of specimens were adopted. This was ordered with the view that by such actions the unknown factors or variables which would affect the strength of the specimens would be either eliminated or at least held constant.

The factors that needed consideration were as follows:

1. Method of preparing solution.
2. Method of molding specimen at different Relative Densities.
3. Cutting and Weighing specimens.
4. Mechanical testing procedures.
5. Time factors for each of the above mentioned.

1. Solution:

It had been decided earlier that the stabilizing gel should form from a solution which contained 7% by weight of the dry chemical, AM-955. Further, considering the gel time, the solution was to contain 0.7% by weight of an activator (Sodium Thiosulfate) and a catalyst (Ammonium Persulfate). This combination of chemicals would allow approximately 10 minutes to prepare the sample before the formation of gel. It was also found that impurities in tap water affected the gel time. Therefore, distilled water was used exclusively.

Another consideration was the amount of solution necessary for each specimen to assure minimum waste. For the molds used, 500 ml. of solution was found to be adequate for all situations.

The procedure adopted was as follows: AM-955 and distilled water were mixed in a weight ratio of 0.07690:1 in large quantities. The monomer was then filtered through a double layer of Oxford shirting in order to reduce as far as practically possible the undisolved residue remaining in suspension. At the time of specimen preparation, 500 ml. of the previously prepared solution was tapped from the reservoir, and at time zero 3.55 grams of each the catalyst and the activator were added, and thoroughly mixed.

2. Molding of Specimens:

Since Relative Density was one of the prime factors investigated it was necessary to mold the specimens of soil at various degrees of compactness. In order to eliminate local failures due to pockets of material of dissimilar density states than the overall sample it was necessary that each particular sample have the same degree of compactness throughout its volume.

In order to assure replication of results special molds were constructed for this program. Each mold consisted of a three-inch diameter split lucite tube, held together by brass stud bolts clamping top and bottom lucite plates. A detachable top collar was provided to eliminate possible changes in density at the top of the sample.

A photograph of a disassembled mold is shown in Figure 5.

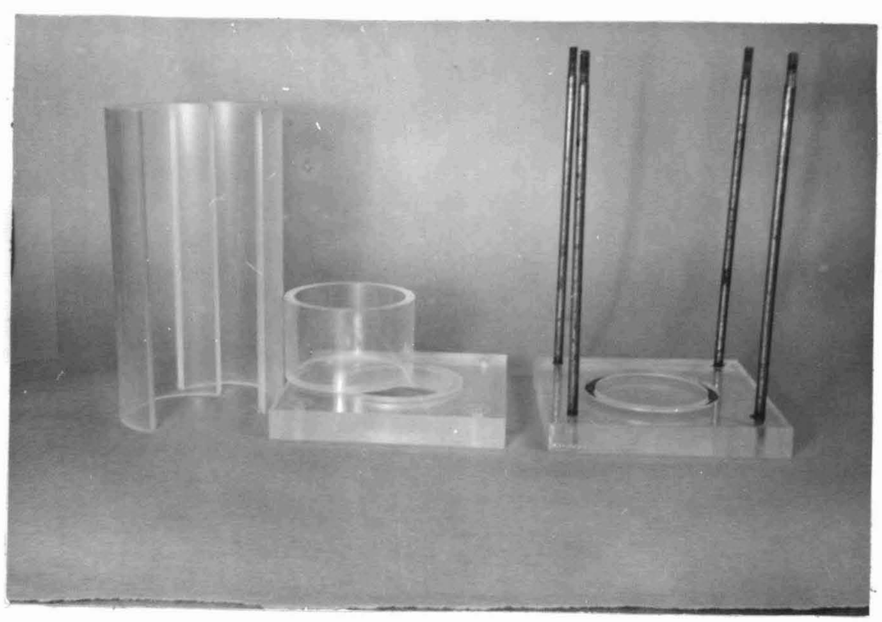


FIGURE 5

DISASSEMBLED MOLD

A photograph of an assembled mold is shown in Figure 6.

FIGURE 6

ASSEMBLED MOLD

The first step in preparing samples for the dense state was to pour approximately two inches of solution into the assembled mold. Next, the soil to be compacted was placed in the mold to a depth of about one half inch, with care so that there were minimal air voids in the sand-solution mixture. The layer of soil on the bottom was then compacted by vibration using a "Burgess Vibro-Tool" fitted with a two-inch diameter foot. The application of the "Vibro-Tool" was a function of the operator, being based on the consistency of results of a large series of preliminary compaction tests. This process was continued by the addition of solution and 1/2 inch layers of soil until the level of the soil in the assembled mold was at least 1" above the predetermined elevation of the trimmed sample.

The formation of the loose state started by pouring approximately 2 inches of solution in the assembled mold. Soil was then gently and slowly dropped into the mold from the top. This was done either by means of a funnel in which case the tip is kept just above the level of the solution or by slowly shaking soil from a spoon held over the top of the mold. Employing either method, the level of the solution was continuously kept about 2 inches above the level of the soil. The process of adding fluid and soil was continued until the level of the specimen was at least one inch above the final elevation of the sample.

A photograph of the compaction devices along with a partially compacted specimen is shown in Figure 7.

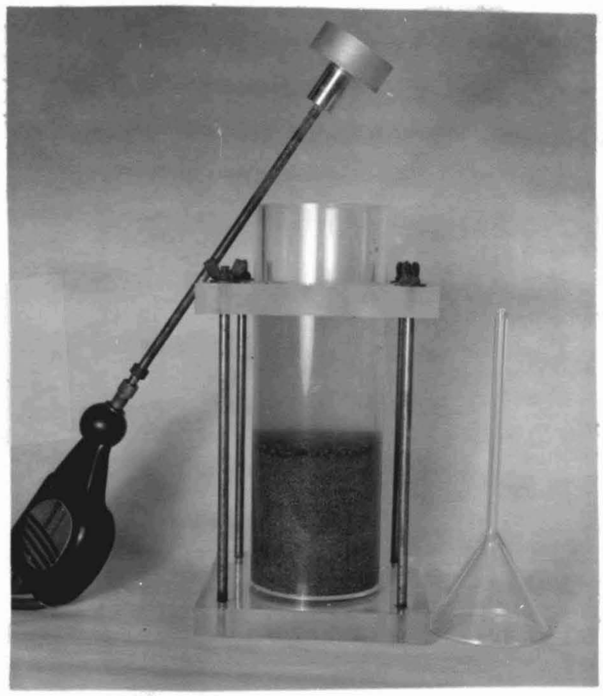


FIGURE 7

COMPACTION DEVICES

Specimens to be compacted in intermediate density states were first formed in the loose condition. The desired Relative Density was then obtained by imparting impact energy to the assembled mold and contents.

This energy was induced by blows from a wooden mallet applied evenly in mumber and intensity to both the base and the collar plates of the assembled mold. The degree of Relative Density was controlled by the number and intensity of the applied blows.

3. Trimming and Handling of Specimen:

After the gel had formed, the top portion of the mold and the collar were removed. The specimen then had about 1-1/2" of soil-gel mixture protruding above the bottom part of the mold. This portion was trimmed carefully with a feathered edge knife, cutting away small pieces only, until the specimen was exactly flush with the top of the mold. Final screeding was performed with a fine hacksaw blade. The split mold was then removed and the specimen weighed. The mold was then replaced and was not removed until the stabilized sample was placed in the machine for the compression tests. In this manner, loss of weight by evaporation and disturbance effects were minimized.

4. Unconfined Compression Tests:

All compression tests were performed on a Tinius-Olsen Electomatic Universal Testing Machine. Prior to testing the machine was calibrated with regard to accuracy of weighing system, and precision of strain-rates. In the region of the applied loads, the machine was found to be accurate to 1/4 of a pound, or within .05% of the lowest ultimate load. The precision of the strain-rates used was within 1% of the rated velocity. The compression tests were performed at a predetermined strain rate, the load recording being started after a seating load of ten pounds or 0.102 tsf of normal stress.

A photograph of the specimen under test is shown in Figure 8.

FIGURE 8

UNCONFINED COMPRESSION TEST OF STABILIZED SOIL

A photograph of the testing machine in operation is shown in Figure 9.

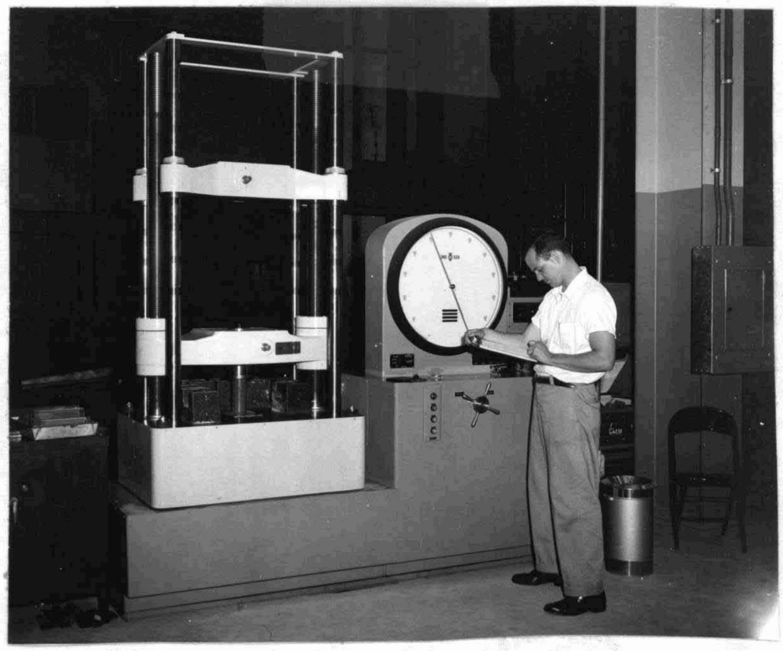


FIGURE 9

STABILIZED SAMPLE IN TESTING MACHINE

5. Volume Measurements:

The volume measurements were performed on samples of various sizes and volumes all compacted initially in the dense state. The volume change measurements were made in two different ways. The volume change by caliper measurements proved to be superior to the use of mercury displacement. A photograph of the samples used is shown in Figure 10.

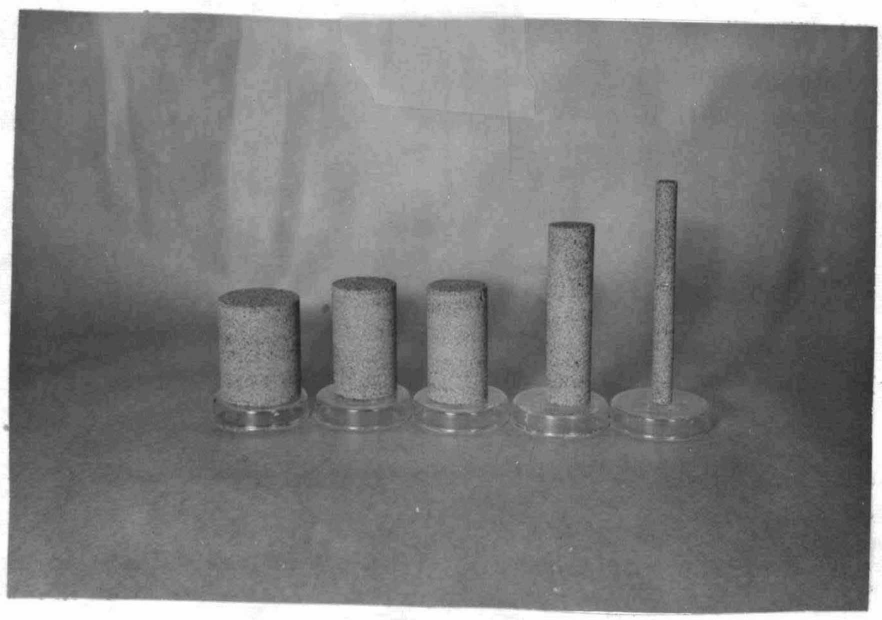


FIGURE 10

VOLUME MEASUREMENT SAMPLES

EXPERIMENTAL RESULTS

The experimental results presented in this progress report are only partial results, and in many instances qualitative. For reasons of maximum usage of personnel and equipment, detailed analysis is being delayed pending the completion of experimentation. Although qualitative interpretations are made in the course of the testing program to check the hypothesis and to chart future experimentation, a detailed analysis of results is being delayed until a later date.

1. Soil and Chemical Characteristics

The principal characteristics of the soil and the chemical which influence the behavior are listed below.

Specific gravity of soil particles = 2.68

Specific gravity of fluid monomer = 1.005

Specific gravity of Gel = 1.036

Per cent saturation of soil samples = 97-100%

2. Unconfined Compression Strength

Unconfined compression tests of stabilized soil were run on all soil groups at various relative densities. Typical stress-strain curves for the various soils considered are shown in Figures 11a - 11e.

A summary of the data obtained and analyzed as a "least squares" fit of relative density versus unconfined compression strength, is shown in Figure 12.

A pilot study of the effect of strain rate on the maximum unconfined compression strength was made on soil B. The results of that study are presented in Figure 13.

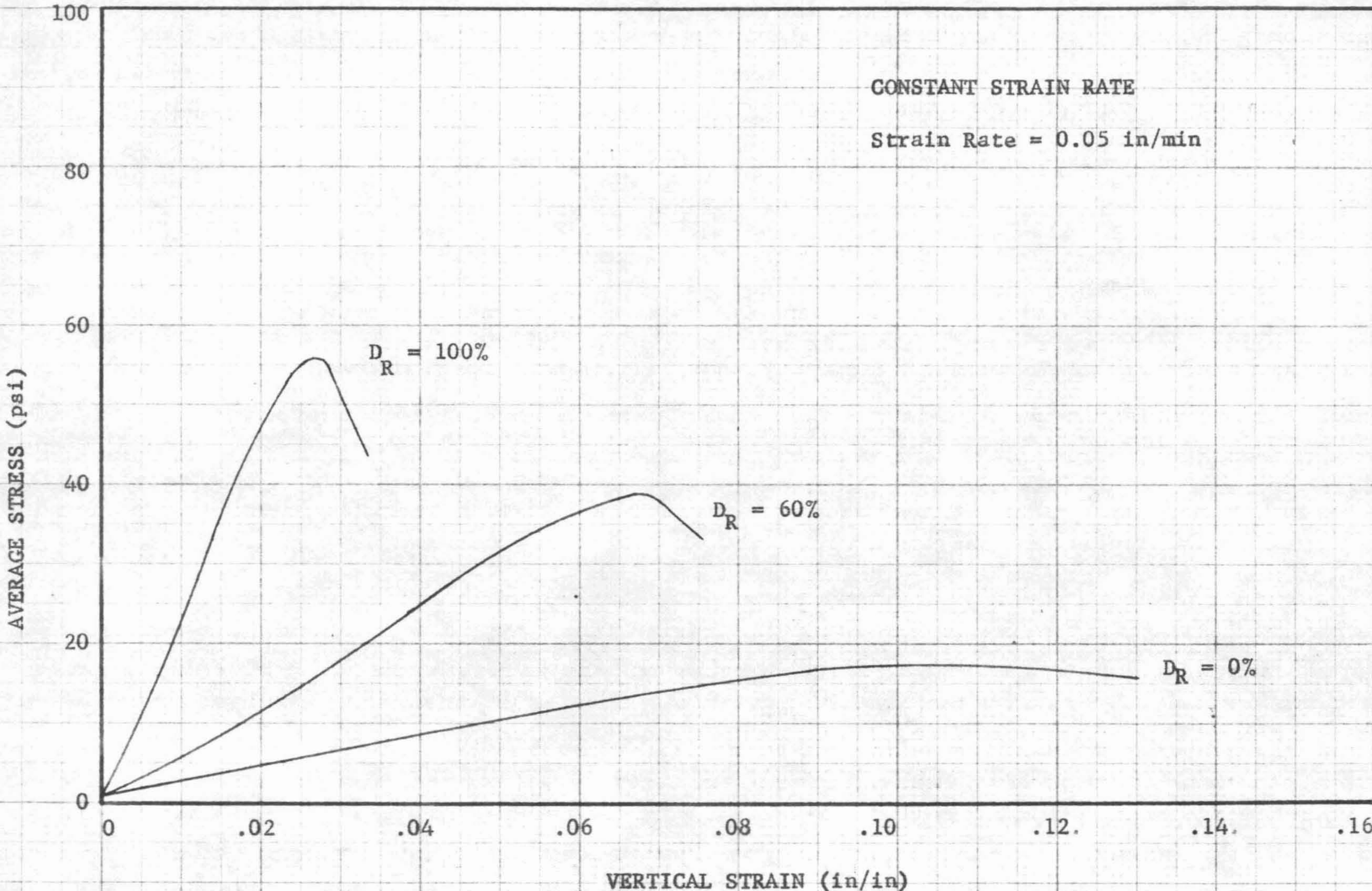


FIGURE 11A

TYPICAL UNCONFINED COMPRESSION TEST
STRESS-STRAIN CURVES - SOIL A

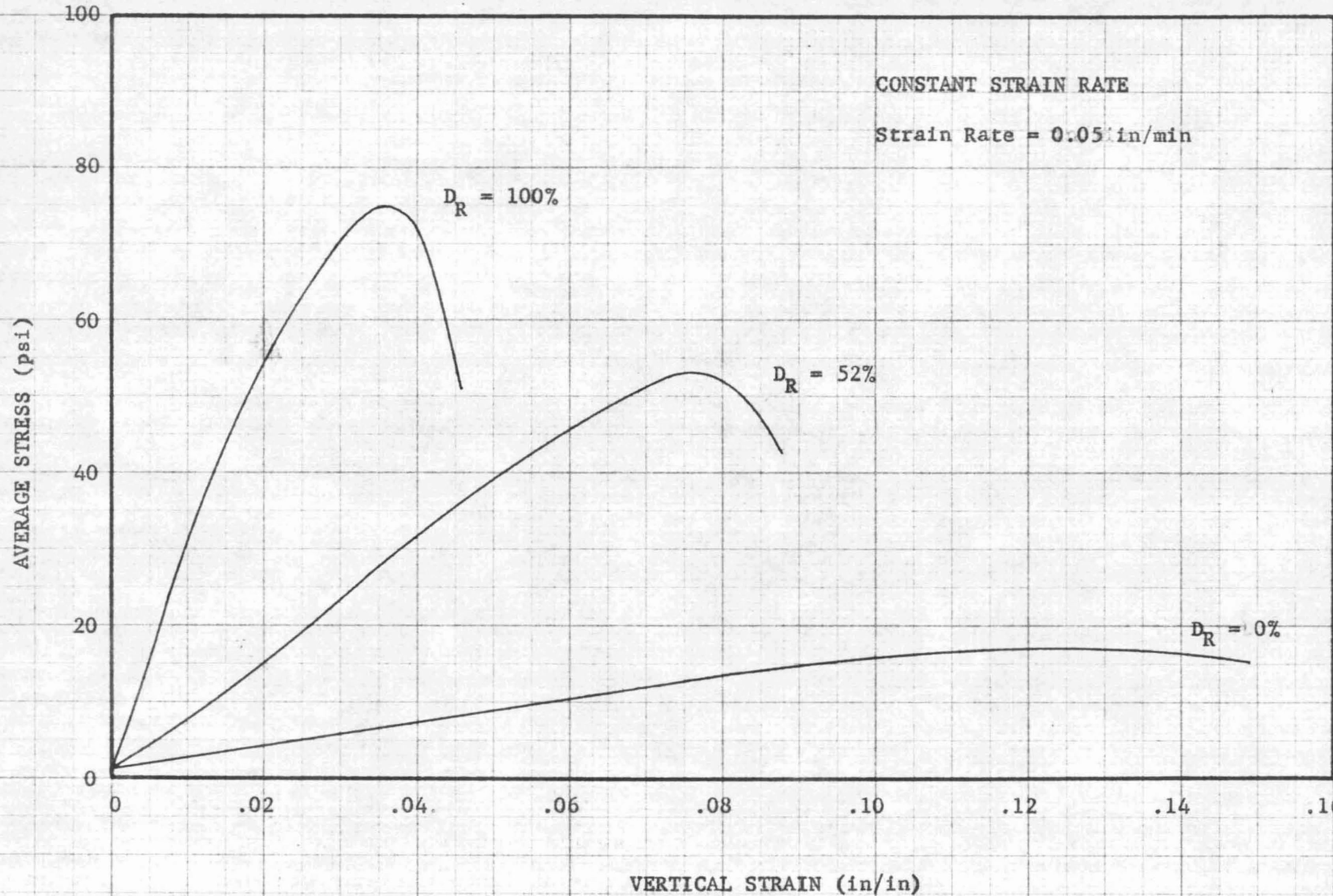


FIGURE 11B

TYPICAL UNCONFINED COMPRESSION TEST
STRESS-STRAIN CURVES - SOIL B

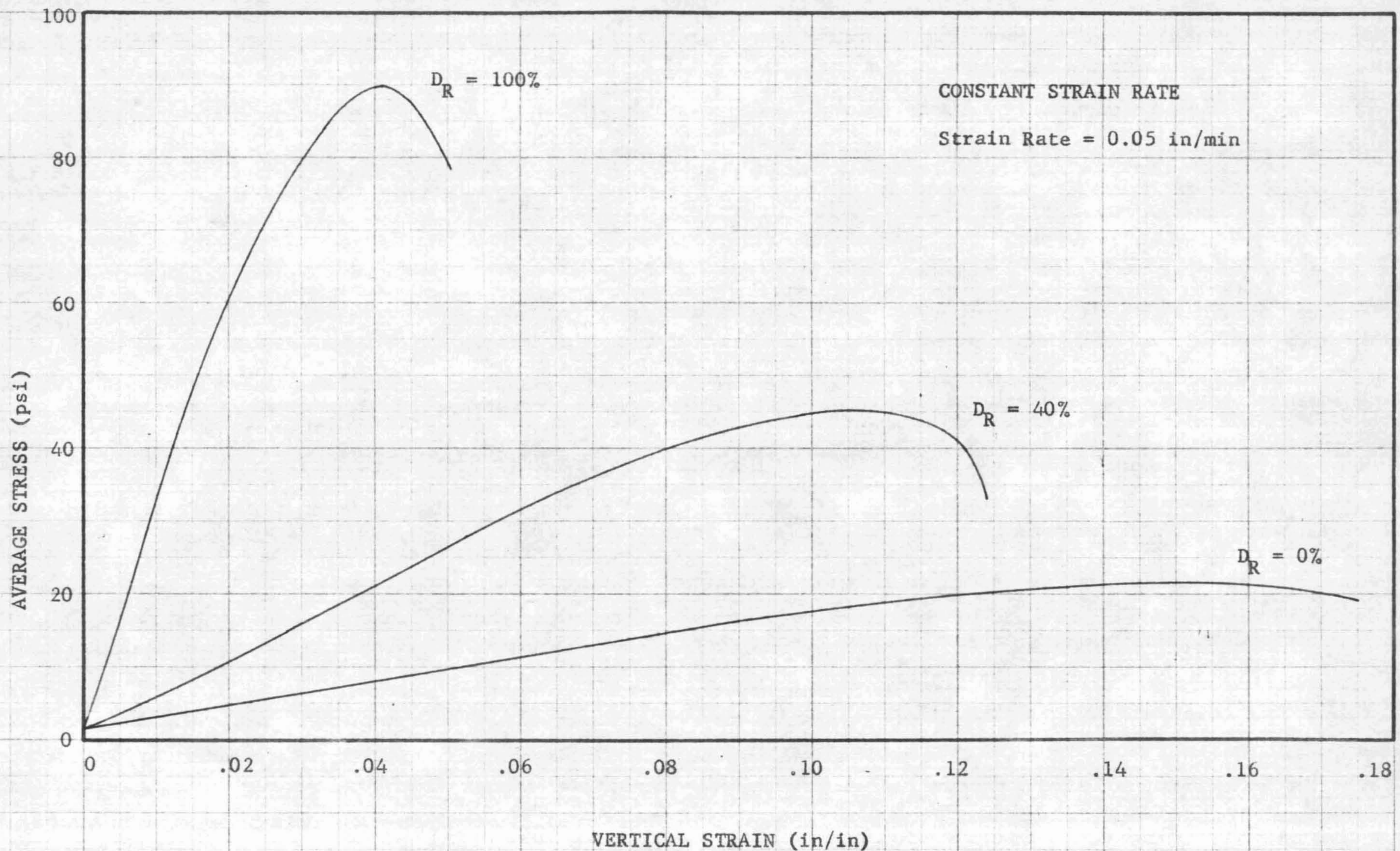
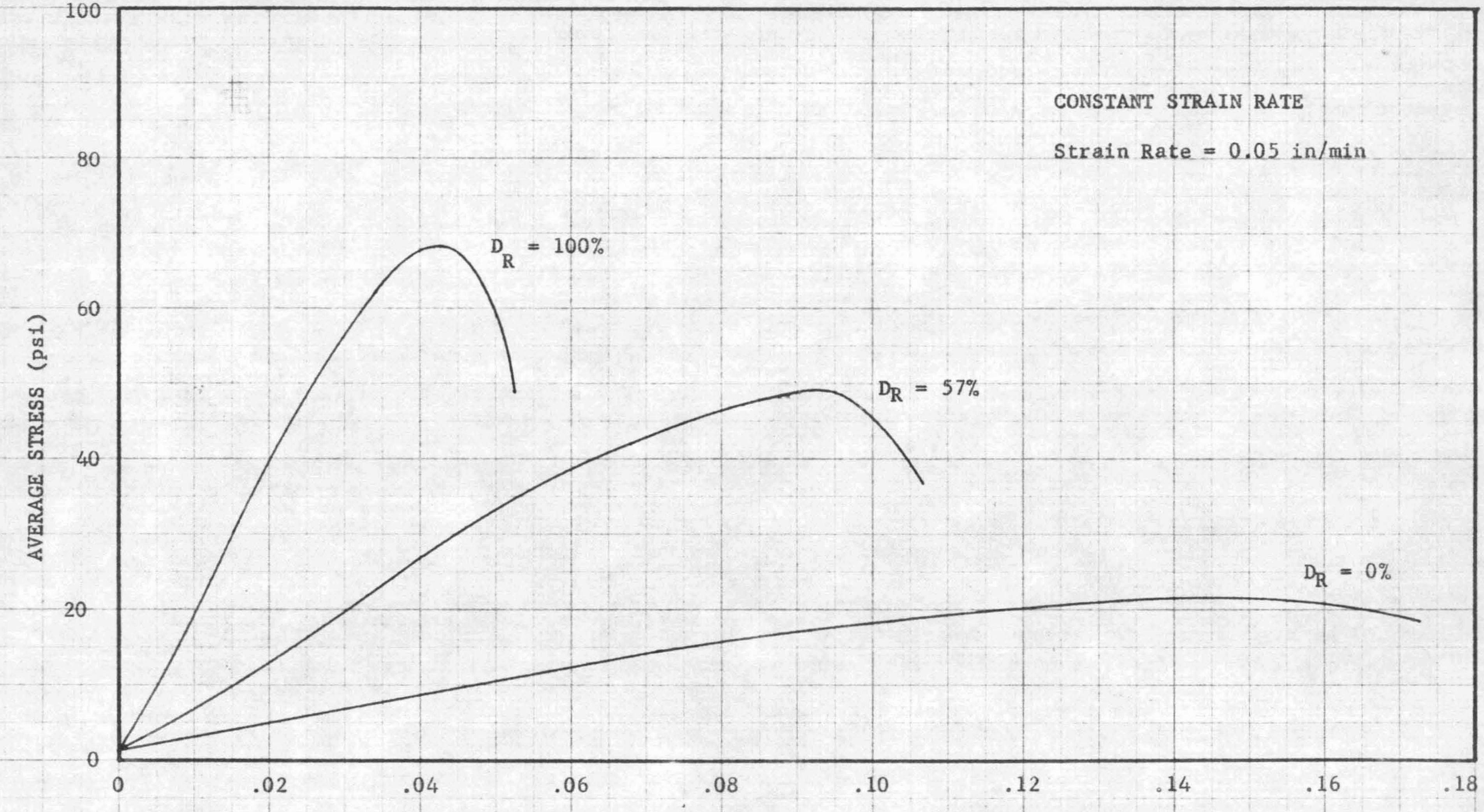


FIGURE 11C

TYPICAL UNCONFINED COMPRESSION TEST
STRESS-STRAIN CURVES - SOIL C



VERTICAL STRAIN (in/in)

FIGURE 11D

TYPICAL UNCONFINED COMPRESSION TEST
STRESS-STRAIN CURVES - SOIL D

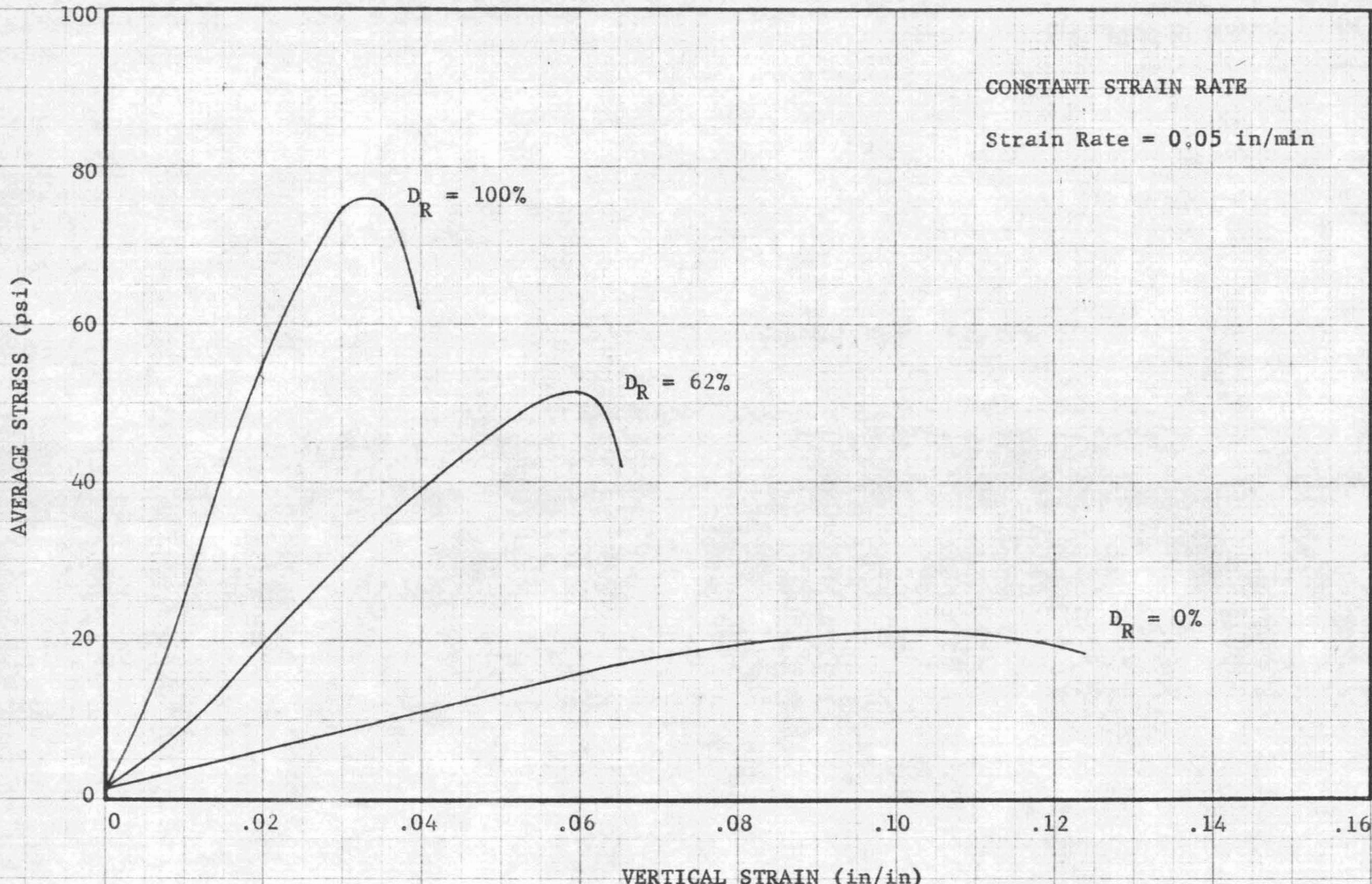


FIGURE 11E

TYPICAL UNCONFINED COMPRESSION TEST
STRESS-STRAIN CURVES - SOIL E

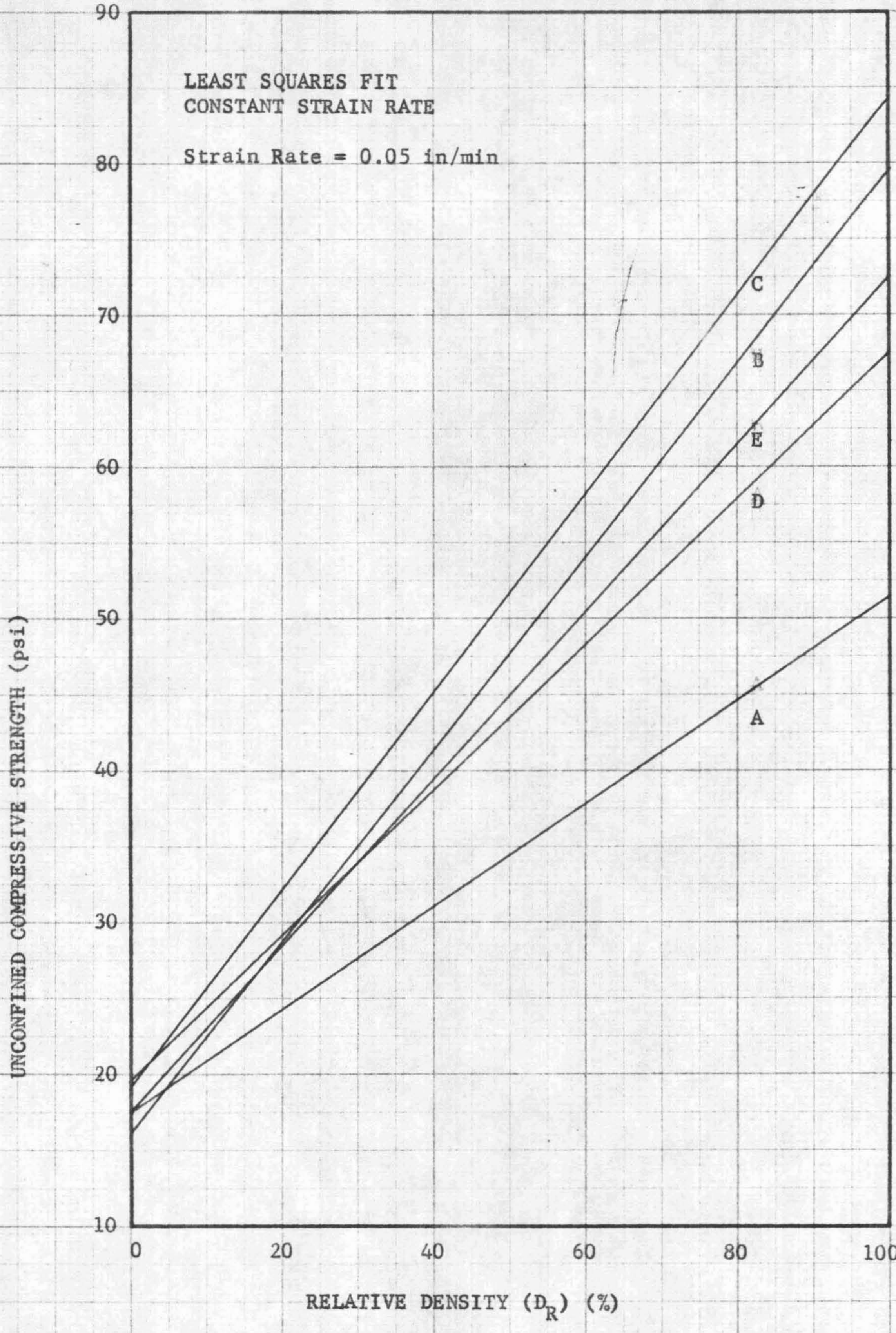


FIGURE 12

UNCONFINED COMPRESSIVE STRENGTH - RELATIVE DENSITY RELATIONS

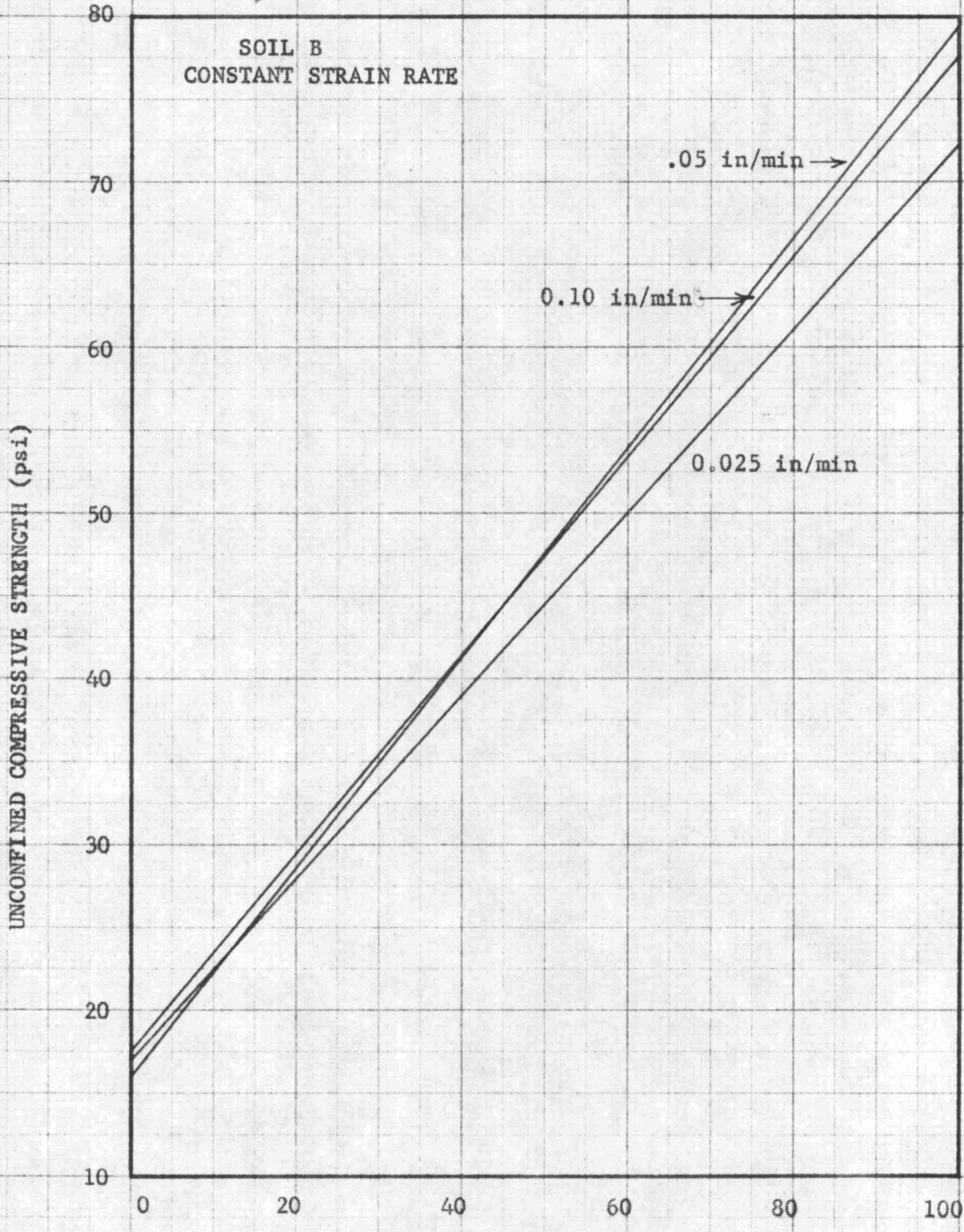


FIGURE 13

INFLUENCE OF STRAIN RATE ON UNCONFINED COMPRESSIVE STRENGTH - RELATIVE DENSITY RELATIONS

3. Effect of Aging

A pilot study was made of the effect of aging on the stabilized soil under conditions of laboratory temperature. The samples were aged in air at an average temperature of 75°F and an average relative humidity of 52%. Soil E, in the dense state, was used exclusively for these tests. Strength tests, by unconfined compression were performed, in addition to measures of changes in volume and density of samples of varying volume and surface area. Although density changes in the stabilized soil were noted, there were no measurable changes in volume. In addition, no changes in the specific gravity of the gel (measured by water displacement) were observed, although marked gel shrinkage was observed.

The curves in Figures 14 and 15 indicate the trend of the dry density change with age, on the basis of the gel specific gravity remaining constant.

Typical unconfined compression stress-strain curves are presented in Figure 16 for the samples at various ages. Figure 17 indicates the trend of the change in strength with aging of the sample.

INTERPRETATION OF TEST RESULTS

Interpretation of the test results, as herein presented, are qualitative, based on observed behavior and the quantitative data obtained to date. A quantitative interpretation will be made at a later date, when all the experimentation connected with this study is completed.

NEWLY FORMED GEL

Several factors of behavior can be ascertained from the stress-strain results, of the unconfined compression tests, in terms of stabilized soil characteristics.

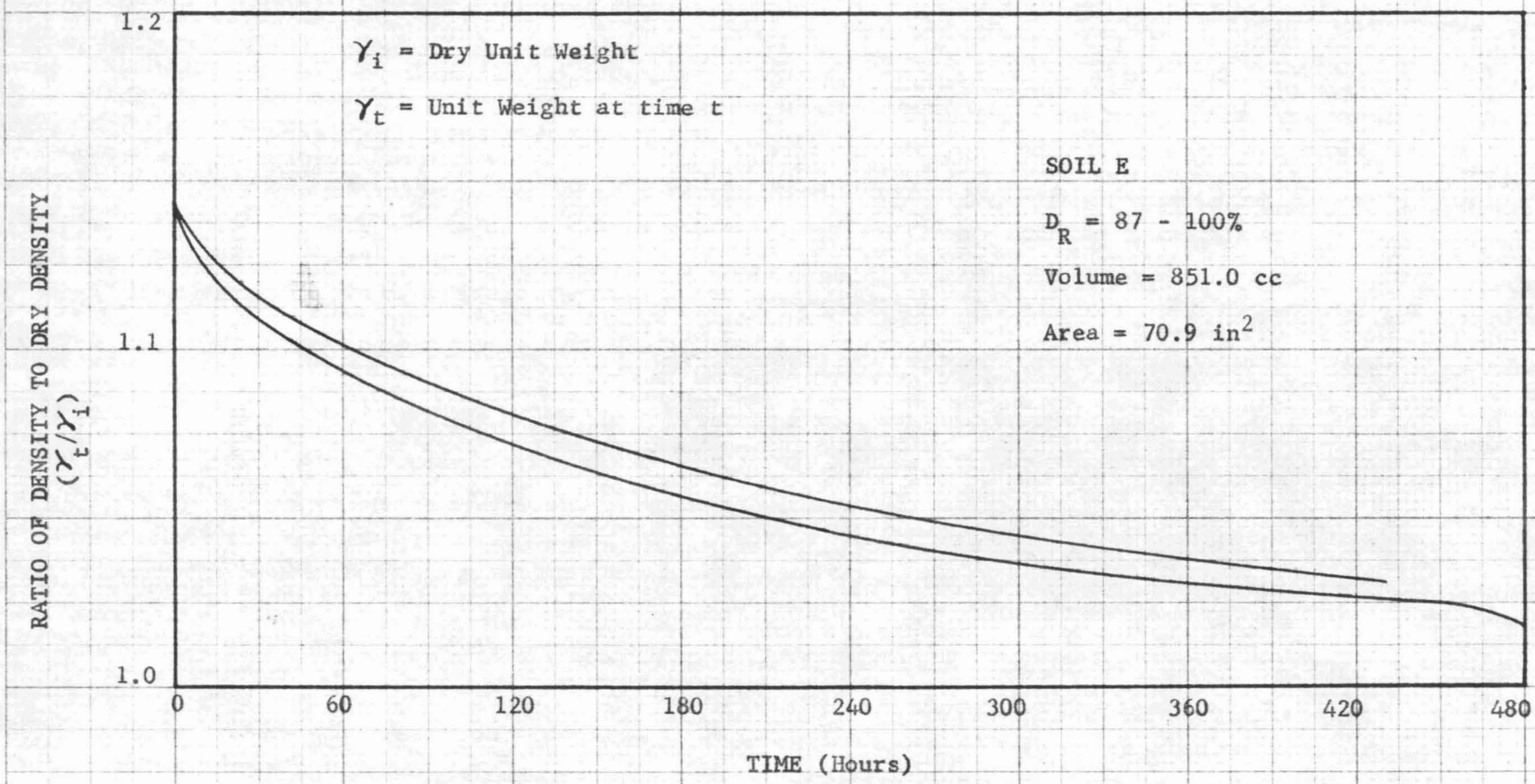


FIGURE 14

CURING RATE OF STANDARD CYLINDER

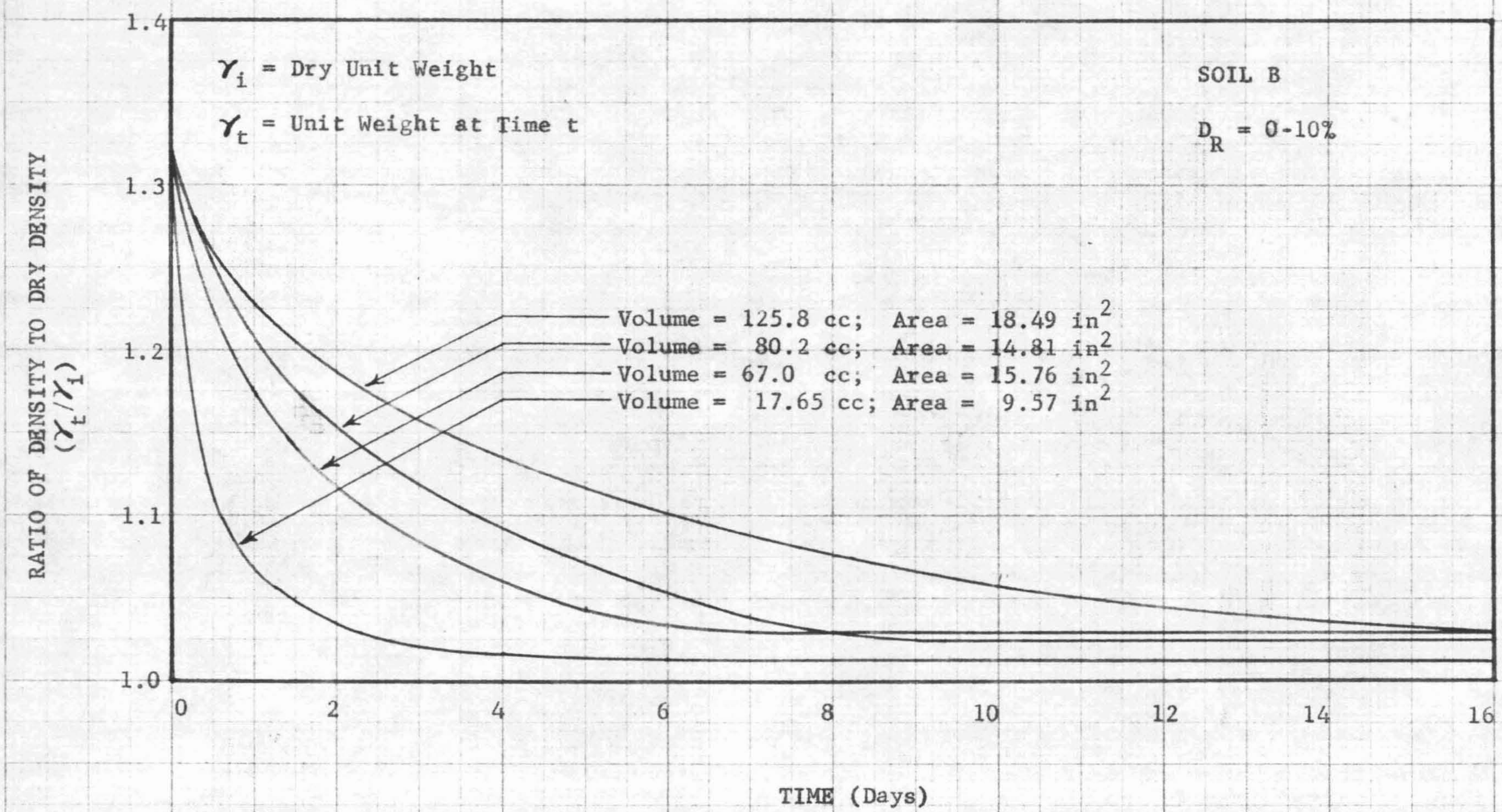


FIGURE 15

CURING RATES AS A FUNCTION OF VOLUME SURFACE RELATIONS

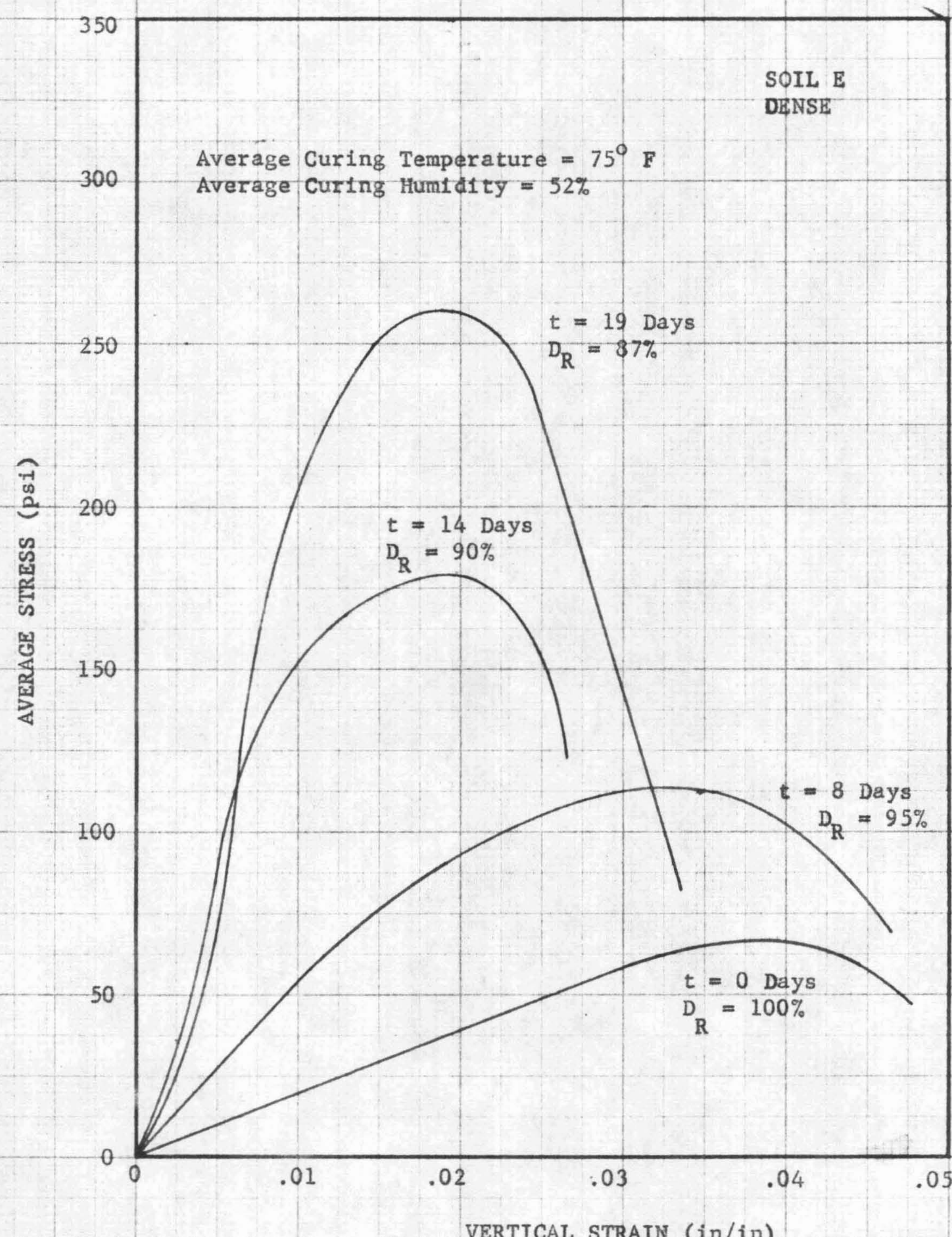


FIGURE 16

UNCONFINED COMPRESSION STRESS-STRAIN
CURVES AT VARIOUS CURING STAGES

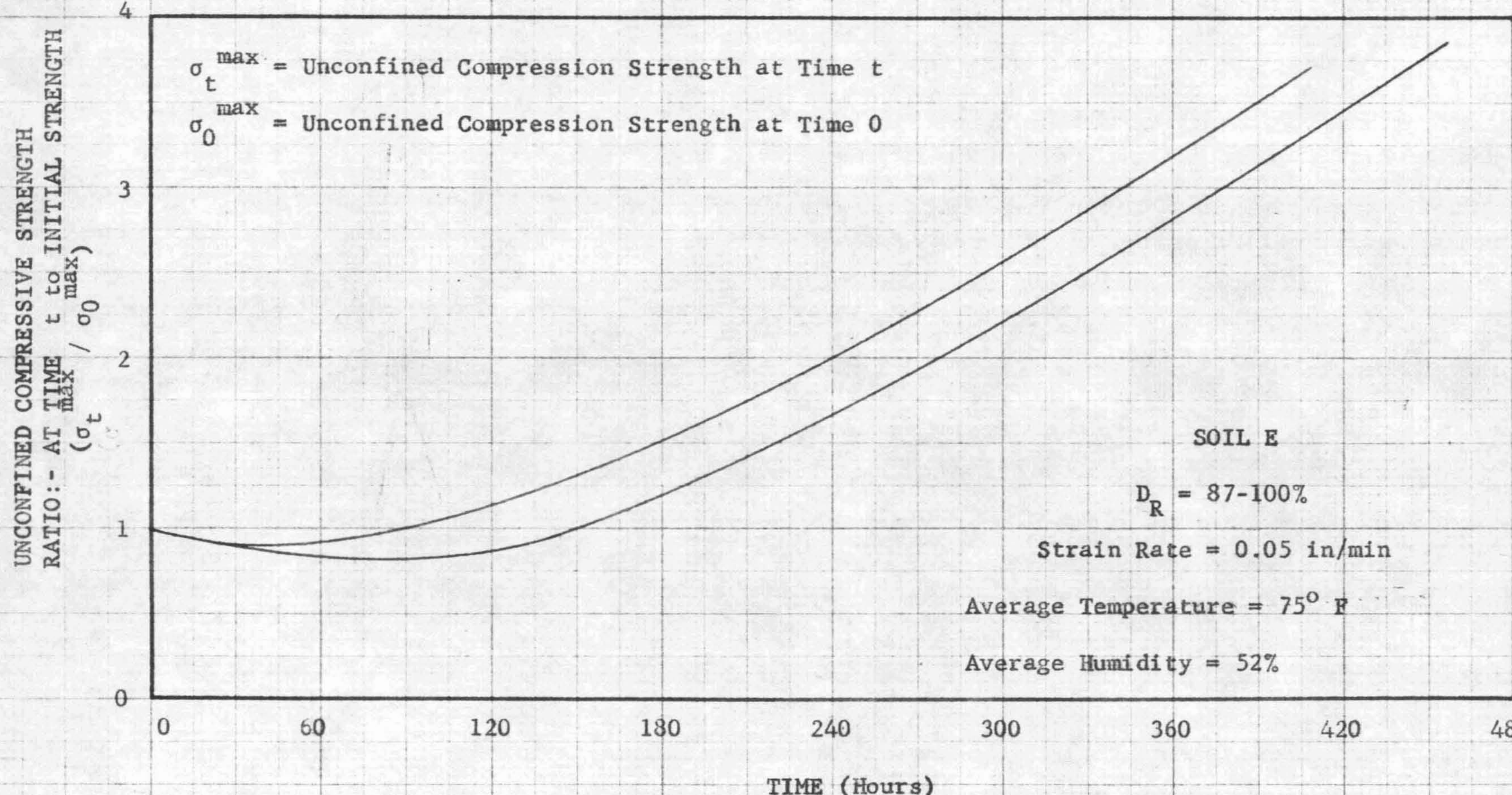


FIGURE 17

EFFECT OF AIR CURING ON UNCONFINED COMPRESSIVE STRENGTH

An examination of the stress-strain curves of Figures 11a through 11e indicate several significant features with regard to the fineness of the soil, and the range of grain sizes.

The general character of the stress-strain curves is similar to that of an unstabilized granular soil with several major differences. In the first place, the comparison is between unconfined compression of stabilized soil and triaxial compression of unstabilized granular soil. This is not an outlandish comparison, as the action of the gel is to pull the grains together and impose an internal tension within the sample

The difference between the stabilized soil and unstabilized soil prior to failure, is the greater degree of linearity in the stress-strain behavior for the stabilized soil. Any mechanical system can be represented by springs, dashpots, and friction units. The viscous elements in the gel, tend to neutralize the friction elements between grains and to hold the grains in contact. Thus the action is, in the early stages, largely the deformation of grain upon grain, which is predominately elastic. There certainly are slight grain friction slips which will account for the minor concavity of the stress-strain curve. The initial concavity of the stress-strain curve is the action of the grains moving into contact in the gel medium.

The behavior of unstabilized granular soils beyond the peak of stress is that of a drop-off and then leveling out, as shown in Figure 18.

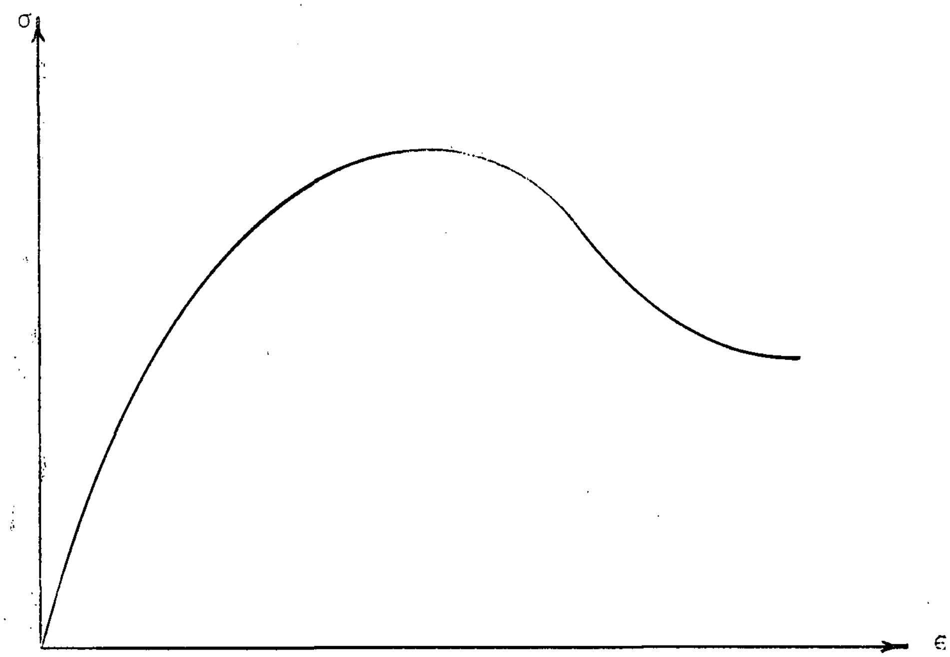
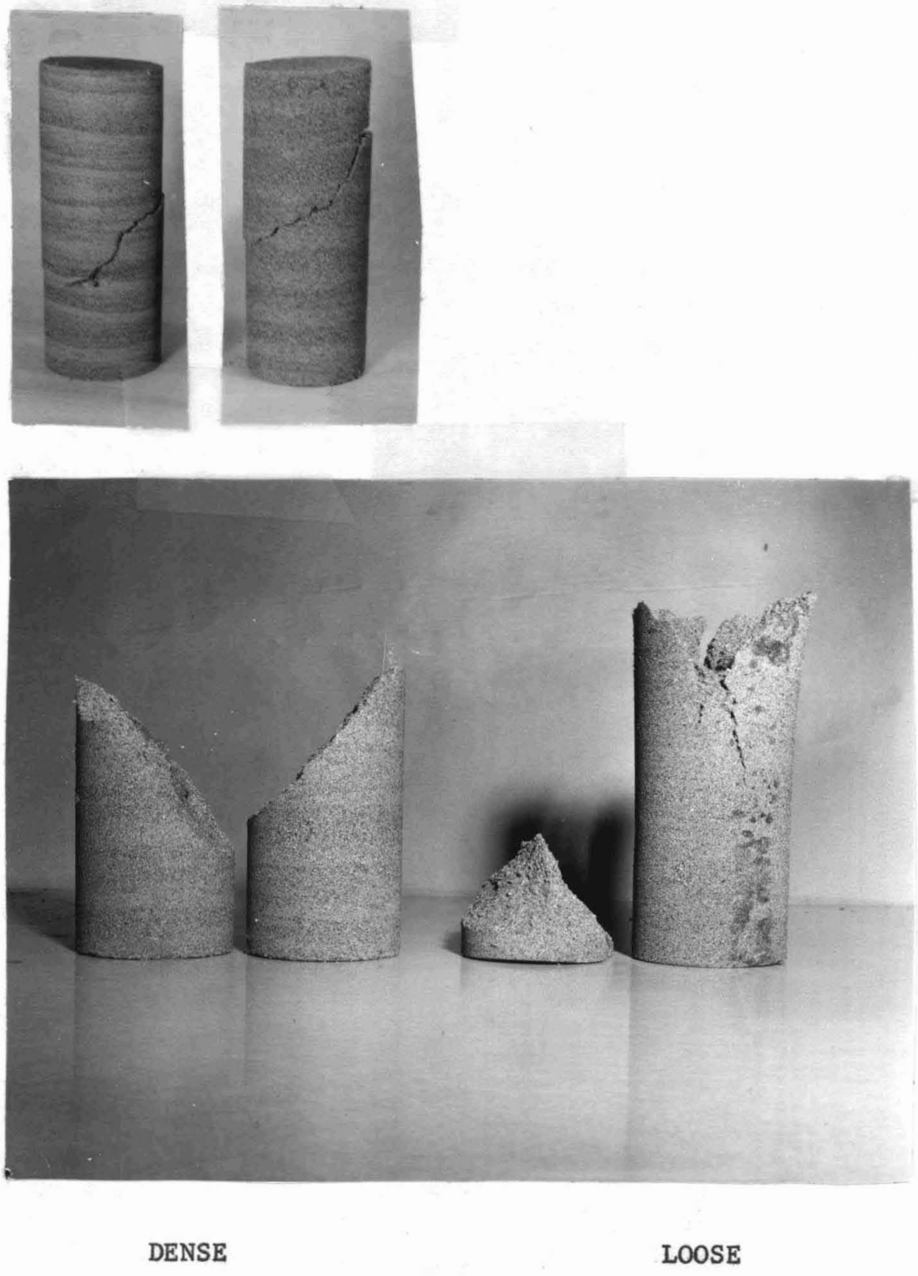


FIGURE 18

TYPICAL STRESS-STRAIN BEHAVIOR
OF DENSE GRANULAR SOIL

This drop-Off effect is not apparent in the stabilized soil. Probably, when the peak is reached, the presence of the gel prevents any grain readjustment, and a crack forms. From this point on, the failure is progressive, and thus a continual decrease in stress.

The difference in behavior in the loose and dense state is implied by the differences in the failure conditions of these state of compaction. Photographs of these failures are shown in Figure 19.



DENSE LOOSE

FIGURE 19

FAILURE OF STABILIZED SOIL

The dense state is characteristic of an intergranular fracture at a definite angle. The loose ~~state is more complex~~, ~~starting~~ with a split duê to the Poisson effect, and then developing to a failure along a plane of shear. In the dense state, the gel content is at a minimum and the mechanism of failure is frictional, grain-to-grain. The loose state, however, contains more voids, filled with gel, and the failure is more a failure of the gel than in the dense state. Extrapolating, to the gel without soil, the failure should be a split along the weakest system of cross-links.

The failure criteria for the loose soils is further indicated by the relationship between maximum compressive strength and relative density. At zero relative density, the maximum shearing strength is independant of the soil type, thus indicating that the failure is predominately through the gel, and that the soil, in this state, acts predominately as a binder.

The relations of strength against relative density as presented in Figure 12, are most significant. Firstly, the change in strength with soil relative density is a linear one, and varies in percentage increase from 160 to 320, on the basis of the minimum strength.

The basis of relative density, in this report, is that of the 0% state and the 100% state, being the minimum and maximum density obtainable, converted to a dry basis. These are based on saturated experiments. There is some reason to believe that density as set by a dry experiment, may be a more significent measure. The final report will cover this analysis in detail.

With regard to the relation of the soil properties vs. the strength at 100% relative density, the results are somewhat different than would be expected of the unstabilized soil. In the first place, the strength increases with the decrease in fineness (D_{50}), all other variables being constant. Secondly, the strength relation for the constant fineness is not directly related to the size dispersion. Both these facts have roots in the same basic phenomena, governing the behavior of the stabilized soil. The soil itself exhibits a certain strength, due predominately, to the frictional resistance preventing grain slip. The introduction of the gel introduces internal tensile stresses holding the grains together, and raises the level of frictional slip. The gel performs this action by coating the grains of soil and filling the voids. If, however, the void spaces are large, failure will occur within the gel, and the binding mechanism will be secondary. Thus the coarser soils, with larger void spaces, result in lower strengths than the finer soils containing smaller voids. The influence of size dispersion is somewhat more complex. The weakest series is the most uniformly sorted series which has the largest void spaces. The fact is, however, that between the three sieve size soils, and the soil with more than six sieve sizes, the three-size soil exhibits definitely better strength properties than the better graded soil. The effects of segregation are probably responsible for this apparent contradiction. Soil deposits resulting from the sedimentation action of wind or water are laid down in thin layers of uniform size, making up a mass deposit of wide grain size dispersion. The same phenomena was noticed in the preparation of the laboratory samples. The densely formed samples were size segregated in bands, as shown in Figure 19. These bands were narrowly graded and within each band the void spaces are characteristic of the grading of the band and not the total soil. Thus the failure criteria must be that of the weakest band, initiating failure, and not the total soil. The three-sieve size soil, apparently is the best graded material in detail, and thus the strongest.

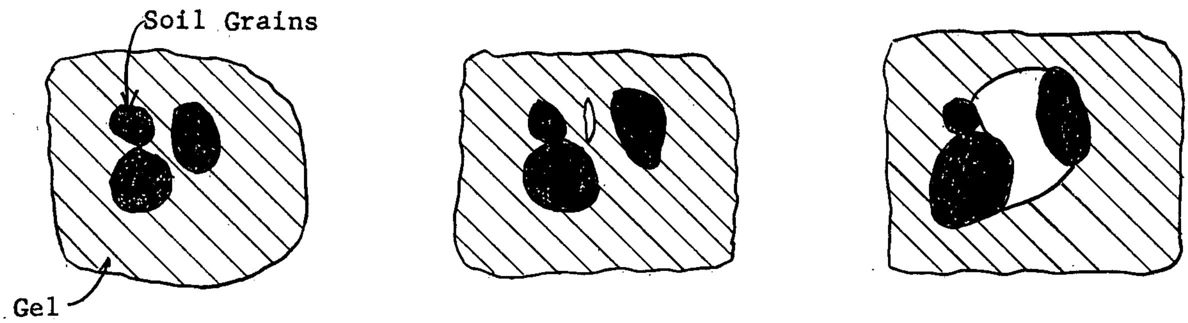
Although the statistics of the data analysis are based on a best fit of all the data, and this best fit is a linear one, the observations on the intermediate states indicate that these intermediate states are not an consistant as the extreme states, with relatively poor replication, and many local failures due to non-uniformity of density. Thus a more detailed analysis may indicate the necessity of re-evaluating the analysis to eliminate the experimental bias. A future analysis will also establish probability limits for replication of future results.

The pilot study of the effect of strain-rate on the strength properties for the median soil, indicates that quantitatively there is a very small influence of strain rate on the test results, within the range studied. A complete interpretation of the effect of strain rate will, however, depend on the probability spread of the rest of the data.

AGING EFFECTS

The effect of aging and curing of the stabilized soil was very marked and indicative of the long term behavior of this material.

The first fact that was noted was the trivial change in volume with aging. This fact is contrary to the experience of the Cynamid group of investigators. This difference in the two results was due to the manner of soil-stabilizer formation. The Cynamid group formed their samples by a random pouring of relatively large units of soil into the ungelled stabilizer. The effect was to produce a mechanically disordered state. From the known fact that gel shrinkage occurs with age, the stress conditions produced by shrinkage in a mechanically disordered state of stabilized soil, will be unbalanced, causing a grain readjustment and a total volume change. For a fully saturated system, such as was investigated, the gel shrinkage forces were uniform in all directions, resulting in a self-equilibriating force system. Thus there was a minimal unbalanced force system on the grains, and insignificant volume change. The effect of the gel shrinkage is not to decrease the volume, but to decrease the unit weight. As the gel ages, a drying process occurs due to the loss of free water. Instead of pulling the grains in drying, the gel tends to crack in the void centers and shrink to the particle sides. Schematically, this phenomena is shown in Figure 20.



(a) Unshrunk Stabilized Soil (b) Initial Shrinkage (c) Final Shrinkage

FIGURE 20

SHRINKAGE EFFECTS

The net result is twofold. In the first place, air voids are formed in place of gel-filled voids. The shrinkage occuring from the inside of the void space outward to the grains, tends to introduce a capillary force system holding the soil grains in place, and markedly increasing the strength.

The density changes, as indicated in Figures 14 and 15, is not an instantaneous, nor a homogeneous phenomena. The drying is dependent on the exposed surface, and as such, the drying effect varies with variations in the exposed surface area. As would be expected, the drying proceeds from the outside towards the center. A distinction must be made between what can be called elemental drying and mass drying. The elemental drying process, being a thermodynamic phenomena, starts off at the exposed surface, that being the only one with a thermodynamic unbalance of moisture vapor pressure. As the outer surface dries out, the gel shrinkage forms air voids which establishes the unbalance for the next layer of particles. This continues inward in concentric rings (mass drying), until the entire sample reaches an equilibrium condition. The drying time, being dependent on the air-exposed surface, becomes a multiphase system. Consider the outer unit surface in relation to an adjacent inner surface. The outer portion will dry at a certain rate. As the voids increase in air space, the rate of drying of the inner surface will increase, but will always lag behind the adjacent outer surface. Thus, before the inner surface has dried out, the outer portion will have reached an equilibrium condition. Thus, by the time the center has dried, the entire sample is in thermodynamic equilibrium. This phenomena is indicated by the asymptotic behavior of the drying curves of Figures 14 and 15. The shift in the curves is due exclusively to the amount of exposed area.

A photograph of the concentric drying is shown in Figure 21.

TWO DAYS | THREE DAYS | EI HT DAYS | NINE DAYS

FIGURE 21

CONCENTRIC DRYING PHENOMENA

The strength characteristics with age showing the effects of this concentric drying, are shown in Figs. 16 and 17. The stress-strain behavior is not a simple unaxial compression problem. Essentially, the phenomena consists of a uniform displacement over a two-phase material, with complete continuity of radial and tangential stresses, and vertical and radial displacements, along the cylindrical boundary between the two materials. This mathematical problem, although of interest, has never been solved, and it is doubted that within the time limitations of this investigation, a rigorous solution can be achieved. A simplified one-dimensional solution is feasible, and will be presented in the final report. The character of the failure criteria is shown in a photograph in Figure 22.

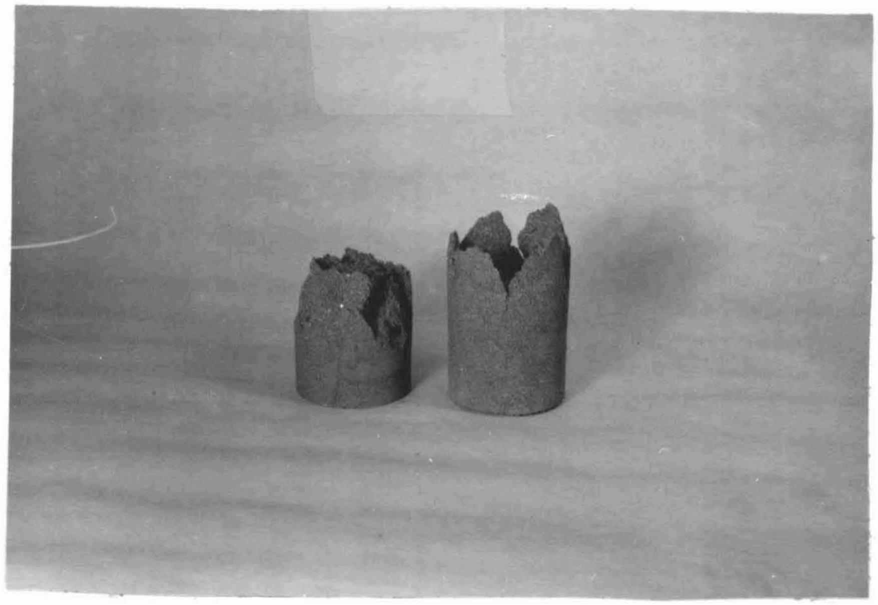


FIGURE 22

TWO-PHASE FAILURE

Qualitatively, and at a given strain level, the outer surface is approaching failure, while the inner softer material is not. Thus, a constrained failure will start at the outer surface in the usual manner. Once the outer skin has ruptured, the tendency is for the bond between the two phases to be broken and the inner portion to behave in the normal manner. However, since the outer skin does not completely remove itself, the net effect is that of a lateral strain restraint. This increases the load-carrying capacity of the specimen. As the skin becomes thicker, the force to break the skin becomes larger, and the lateral restraints also greater, resulting in a net increase in ultimate strength. For early stages of curing, the strength decreases, since the skin effect is slight, and thus the failure load decreases. In these stages there are no lateral restraining effects. Since the basis of the strength is the original area, the ultimate strength will appear smaller than for the uncured specimen.

The aging-strength properties statistically showed a large variation. Since a method of sample preparation, other than normal, was used for this series, there is some question in the authors' minds as to the quality of the results. This question is under continual study, and will be reported on at a later date.

CONCLUSIONS

Although it is premature at this stage to draw any conclusive conclusions, certain general effects can be stated. Firstly, the behavior of granular soils, when treated with AM-955, is dependent on the soil properties. This dependency, however, does not follow the exact pattern of the untreated granular soils, but is contingent on the mechanism of a soil-gel system. In addition, the aging and forming effects have a marked influence on the strength of the stabilized soil. In general, the curing tends to increase the strength of saturated soils. The effect of partial saturation is, however, unknown, but is not believed to be the same as for saturated soils. A third significant conclusion, is the effects of strain rates which are negligible within the areas investigated.

Further and more quantitative conclusions will be presented in the final report.

FUTURE PROGRAM

The future research program under this contract will consist of two phases. These phases are, the remainder of the research to complete this preliminary study, and such additional studies that may be carried on by graduate students in the academic year 1956-57.

PRELIMINARY STUDY

In order to complete the program previously outlined in Report No. 1, a series of triaxial compression tests will be performed on samples in the densest and loosest states, and sufficiently varied lateral pressures to establish the Mohr-Coulomb or other suitable failure criteria.

ADDITIONAL STUDIES

It is hoped that there will be sufficient interested graduate students to engage in any or all of the following three additional investigations.

1. Detailed study of aging effects of the gel, including a theoretical study of two-phase compression.

2. Variations in strength with variations of monomer concentration.

3. Effects of capillary saturation, and the use of hydrophillic agents in developing high degrees of saturation.

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