DEFORMATION CHARACTERISTICS OF GRANULAR MATERIALS SUBJECTED TO RAPID, REPETITIVE LOADING

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Distribution of Stress in Layered Systems
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Texas Transportation Institute personnel who were actually concerned with the testing, analysis and report phases of the project were Messrs. F. H. Scrivner, L. E. Stark, W. A. Dunlap, J. L. Bratton, T. L. Snow, F. L. Lynch, J. C. Armstrong, J. W. Burke and C. H. Michalak.

SYNOPSIS

The engineering behavior of soils and aggregates is a function of their basic properties and the environment to which they are subjected. In pavement design the effect of repeated loading is in important environmental condition which has received little attention, especially for granular materials. This report present results showing the influence of repetitive triaxial stressing on the deformation of granular (flexible base course) materials. The materials examined were a) hard rounded material (gravel), b) hard angular material manufactured by crushing oversized pieces of the gravel, and c) a soft crushed limestone. Each material -- except the soft -- was tested at three separate gradations representing the coarse, medium and fine ranges allowed by Texas Highway Department specifications for such materials.

The total strain characteristics of the materials were quantitatively related to the applied stresses and number of stress repetitions. It was shown that the behavior under repetitive stresses was not closely related to static shear strengths as determined by the Texas triaxial method. Rebound strains could only be expressed qualitatively.

Under repetitive stressing the rounded material, which ranked lowest in static shear tests, was at least equivalent to the angular material for the stress range expected in roadways; both were superior to the soft material. It is believed that the relative densities of the respective materials influenced their behavior more than particle shape or hardness.

Finally, recommendations were made for improving equipment and testing procedures which should simplify analysis of results and reduce experimental error in future investigations.

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INTRODUCTION

Statement of the Problem

Though flexible pavements have been constructed for hundreds of years, methods for their structural design are far from perfect. The many failures occurring in recently constructed flexible pavements are mute evidence of the inadequacy of design procedures. Each design agency seems to favor its own design methods, and it appears that no one has much in common with the other.

At a recent International Conference on flexible pavement design, Whiffin and Lister made the following appropriate comment:

> Amongst other things, a road should spread wheel loads so that the repeated stresses applied to the soil subgrade become too small to compact the soil appreciably or cause it to fail in shear, while the road itself should not experience stresses leading to failure of any of its other layers.
>
> The present methods of pavement design are ad hoc in character and based upon experience of the behavior of different types of roads over a wide range of traffic and soil conditions.

The authors went on to state:

> A more reliable method of design might be developed on the basis of the dynamic stresses produced in the road by moving vehicles, the stress/deformation characteristics of the layers in the road under repetitive loading, and the variation of these properties with time .... but it is not yet possible to present a technique for designing roads from this information (44).

Their last sentence might imply that "this information" (as stated in the preceding sentence) is available and waiting application, but such is not the case.

General Research Program

The purpose of this research program, sponsored by the Texas Highway Department in cooperation with the Bureau of Public Roads, was (a) to determine the behavior of certain flexible pavement materials under simulated traffic conditions, and (b) to calculate the theoretical traffic-imposed stresses in these materials. This was approached by developing an apparatus for subjecting these materials in the laboratory to repetitive dynamic stresses simulating the repeated traffic-induced stresses.

The materials selected for examination in this program were the near-surface granular materials, primarily base courses. This selection may at first seem puzzling, for an old axiom in pavement design is "take care of the basement (subgrade) and the roof will take care of itself." In fact, most design methods start with the subgrade characteristics, from which the properties of the subbase, base course, and surfacing are progressively determined. Yet, at the recent AASHO Road Test, only 10 percent of the rut depth in the flexible pavement sections could be attributed to reduction in the elevation of the subgrade (embankment). In Texas, much of the recent flexible pavement distress reportedly has been attributed to inadequacy of the near-surface granular materials. Apparently, the present "ad hoc" design methods are generally adequate for coping with the subgrade conditions (neglecting certain clays exhibiting large amounts of shrinking and swelling), but inadequacies exist in the design of the upper layers.

Present State of Flexible Pavement Design

In simplest terms, flexible pavement design is a process of selecting materials, estimating their behavior, and determining their required thickness in the roadway. The designer gambles that his estimate of the behavior of the materials under certain conditions is correct. An incorrect guess produces a failure or, more fortuitously (for the designer, at least), the roadway performs well but is too expensive. Overly conservative designs may well be costing the tax-paying public as much as the more noticeable failures.

Material characteristics. The term "behavior" was freely used above, and warrants definition. From a design viewpoint, it may be defined as its strength, its deformation characteristics, its stress-strain characteristics, or in numerous other ways, but however it is defined, the behavior is a function of:

a. The basic properties of the material.

b. The environmental factors to which the material is subjected.

In this respect, flexible pavement materials are unusual compared to most conventional construction materials: nature has produced an infinite number of materials with widely varying basic properties, and then provided an environment which changes rapidly and drastically at times.

The relationship between basic properties and environmental factors in establishing material behavior is one major reason for the difficulties in flexible pavement design. A material with certain basic properties may be entirely satisfactory under some environments but may perform poorly under a different set of environmental conditions. Behavior is also responsible for the difficulties in putting theoretical design methods into practice. The practical designer knows that environment changes material behavior (the theoretician considers behavior inflexible), and if he uses a particular theoretical design method, he must test his materials to obtain their most critical behavior.

This discussion could not be terminated without listing some of the basic properties and environmental factors. For simplicity, the basic properties can be considered on the basis of single grains. These properties include, but are not limited to:

a. Shape and size of particles.

b. Surface roughness or texture of particles.

c. Type of material comprising the particles.

d. Gradation of the mass of particles.

e. Plasticity of the mass of particles.

Most of these factors are determinable by standard tests.

Environment admittedly is an all-encompassing term, but those listed below are considered important in pavement design (climate, in itself, is significant, but climatic effects are included below):

a. Frequency, duration, and magnitude of applied stresses.

b. Temperature (including depth of frost penetration).

c. Moisture content (or degree of saturation).

d. Behavior of surrounding materials and their interaction with the material in question.

e. Pressures developed in the pore water and pore gas in the material.

Many of the environmental factors are interdependent; the magnitude of pore pressures is influenced by all the other factors listed, in addition to some of the basic properties. Environment can even change the basic properties, viz., repeated stress applications have been known to change the original gradation of a material.

The basic properties and environmental factors listed above are undoubtedly incomplete, nevertheless both exist, they are inseparably entwined, and they govern the behavior of roadway materials. If this concept is accepted, it will be helpful in discussing the design of roadways.

Flexible pavement design methods. Several volumes would be required for a detailed discussion of the many flexible pavement design procedures, but collectively they can be grouped into a few general classes:

a. Empirical methods.

b. Semi-empirical methods.

c. "Ideal" methods.

By far the most popular and widely used are the semi-empirical methods, and only they will be discussed. In the semi-empirical methods some behavioral characteristic(s) of the materials are related to calculated traffic-induced stresses or strains. They are of two general types:

a. Limiting stress method, where shear stresses in the roadway are limited to a safe value. (The present Texas Highway Department triaxial design procedure is an excellent example of this method).

b. Limiting deformation method, where the deformation of the pavement structure is limited to a safe value. (An example of this method is the triaxial procedure used by the Kansas State Highway Department).

To a certain extent these methods are rational*, but for the following reasons they must be modified by experience:

> Repeated applications of loads to a flexible pavement may result in sufficient cumulative deformation to cause failures, although a single application of the load would not. Present testing methods for estimating behavior of roadway materials do not consider repeated load applications, so it must be done by experience in the design procedure. It is interesting to note that most of the present design methods do not consider the number of load applications in establishing roadway design life, and those which do consider them apply the same corrections regardless of the materials involved.
>
> Traffic-induced stresses or deformations are usually calculated from elasticity theory (plastic equilibrium methods are infrequently used), although proof does not exist that roadway materials act elastically or for that matter, in any other orderly fashion. The elastic constants, modulus of elasticity and Poisson's ratio, have been the subject of considerable research on roadway materials, but the wide range of values reported for similar materials indicates that they are certainly poorly defined, if they do exist. In fact, it appears that more research effort is spent in trying to force materials to act in the rigorous framework of elasticity than in finding how they truly behave.
>
> The remaining environmental changes, such as regional climatic conditions, are accounted for by experience. Usually this is accomplished in the methods of determining material behavior, or in the arbitrary modification of design curves.

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*According to Webster's dictionary, rational is "having reason or understanding."

Improvement of existing design methods. In theory, the development of an ideal pavement design method is easy. First the magnitude of the traffic-induced stresses (or strains) throughout the roadway structure must be calculated; this should be followed by developing a method for determining the behavior of the roadway materials to these stresses and to other important environmental factors.

In practice, it is unlikely that an ideal method will ever be developed for the complex polyphase materials used in flexible pavements. The best that engineers can do is to remove some of the empiricisms that exist in the present design methods. To do this, the following steps are required:

a. A deformation law (similar to Hooke's Law for elastic materials) must be firmly established for roadway materials.

b. Based on the deformation law, the stresses within pavement structures should be calculated.

c. The behavior of the material when subjected to the calculated stresses should be ascertained.

Obviously, the environmental effect which has been most neglected is repetitive loading. Both the deformation law and the resistance or behavior of the materials should be determined under rapid, repetitive loading, not under static loads. Eventually it may be possible to evaluate the effect of many other environmental factors. But many, particularly those connected with climate, must still be handled by experience. (Meteorologists may never be able to predict day-to-day climatic variations for a typical roadway design period of 20 years.)

Specific Research Objectives

As stated previously, the purpose of this general research program is to determine the behavior of granular materials when subjected to repetitive stresses, and to calculate theoretical traffic-induced stresses. The latter objective requires establishment of a deformation law (stress-strain characteristics). In addition, the Texas Highway Department requested that the basic properties of the materials being tested vary as follows:

> Durability to range from hard (Los Angeles abrasion of 25-30) to very soft (Los Angeles abrasion of 55-60).
>
> Particle shape to range from highly angular to rounded.
>
> Gradation to range from coarse to fine as required by the Texas Highway Department for Type A, Grade 1, flexible base materials, Item 248 (42).
>
> Texas triaxial classification within the Class 1 and 2 ranges.

To meet these requirements, three basic materials, each at three gradations, were selected. In essence, this provided nine aggregates as shown below:

Shape and Hardness / Gradation	Coarse	Medium	Fine
Hard Rounded	X	X	X
Hard Angular	X	X	X
Soft (unspecified angularity)	X	X	X

For each aggregate above, it was specifically desired to:

a. Obtain its relationship between repeated loads and accumulated deformation.

b. Compare its behavior under laboratory repetitive loading to the other aggregates, to its Texas triaxial classification and, if possible, to performance in actual roadways.

c. Explain its behavior in the light of existing soil mechanics knowledge and thus extend the information gained to include other aggregates.

d. Examine its stress-strain characteristics under rapid, repetitive loads to either confirm or modify the deformation hypothesis presently proposed (17) for granular materials.

The necessary information to complete these objectives was obtained in the laboratory testing program; however, this report will be concerned primarily with the repetitive load-deformation characteristics, i.e., a., b. and c. above.

CHAPTER II

REVIEW OF PREVIOUS REPETITIVE LOADING RESEARCH

General

Studies of the stress repetition-deformation relationships on roadway materials are not new. Field studies consisting of test tracks and repeated plate bearing load tests are well documented. However, repetitive stress investigations on laboratory triaxial specimens were pioneered in the early 1950's almost simultaneously at Texas A&M University and the University of California; in time, other agencies initiated similar research. The varied objectives of these studies have produced a storehouse of knowledge concerning the environmental influence of laboratory repetitive loading on material behavior.

Any attempt to completely review and collate the many investigations would be foolhardy; instead the salient points are presented in a manner designed to show the influence of environment on pertinent aspects of material behavior, and to show some of the methods used to evaluate the test results. This writer has found that many published investigations have not included complete accounts of the test conditions -- particularly the drainage characteristics of the specimens -- and also complete information on the soil properties. It is well to remember that many of the initial investigations, especially at Texas A&M University, were concerned with small numbers of repetitions simulating proof rolling or limited traffic effects.

Review of actual results will be preceded by a section on terminology and one on metal fatigue principles. It is cogent to briefly discuss the latter subject since many repetitive stress investigations on soils have been influenced by a priori knowledge of metal fatigue and principles of metal fatigue analysis.

Terminology

As the research objectives varied, so did the terminology. Terms such as "elastic" and "inelastic" strain, "permanent", "residual", and "plastic" deformation suggest various phenomena to various investigators. In an attempt to eliminate this enigma the terminology described below will be used throughout this report (refer to Figure 2.1). The writer will freely change the nomenclature of others to conform with this terminology.

Deformation (or strains).

a. "t", total deformation obtained when the maximum load is applied to the specimen. Note that it is shown as being accumulative.

b. "p", permanent deformation retained by the specimen between cyclic load applications. It is also accumulative.

c. "r", rebound deformation or the difference between the total and permanent deformation for any particular load application. It is non-accumulative.

d. "δ", transient deformation or the deformation observed from zero to maximum stress for any particular load application. It also is non-accumulative.

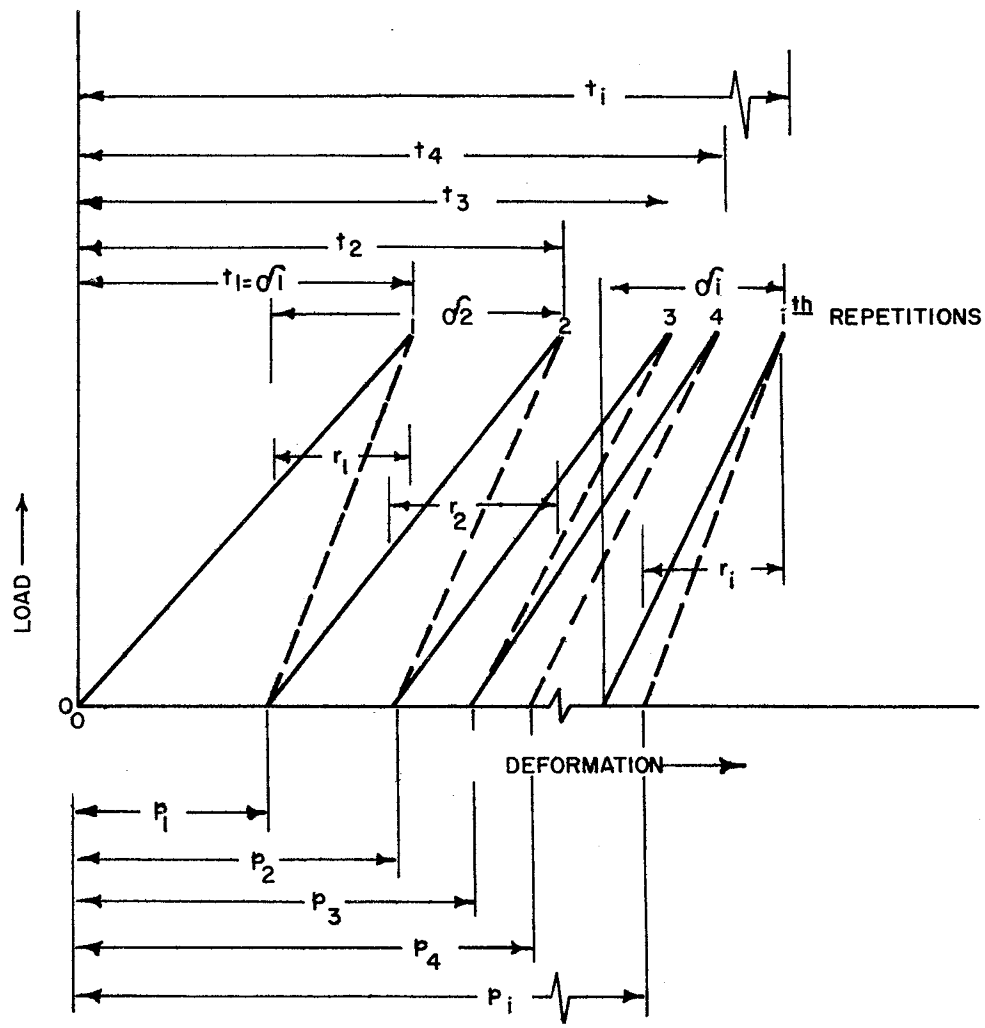


FIGURE 2.1 Hypothetical repetitive load-deformation relationship indicating suggested nomenclature for various types of deformations.

Strains may be used as well as deformations, but the terminology will remain the same, i.e., total strain, permanent strain, etc. Certain writers have used "elastic" to refer to various conditions of soil behavior under repeated loading, e.g., when either rebound or transient deformations approach constant values over several stress repetitions. To avoid conflict with the Hookean concept of elasticity, "perfectly resilient" will be used to describe the state achieved when continued repeated loading produces no further total or permanent deformation. (When this occurs, rebound would equal the transient deformation).

Stresses.

a. σ_1, total axial stress.

b. σ_3, total confining pressure.

c. $(\sigma_1-\sigma_3)$, total axial deviator stress (or stress difference).

d. σ_{1f}, $(\sigma_1-\sigma_3)_f$, total axial stress and axial deviator stress at failure, respectively.

e. $\bar{\sigma}_1$, $\bar{\sigma}_3$, $(\bar{\sigma}_1-\bar{\sigma}_3)$ etc., same as above but effective rather than total stress.

f. u_w, u_a, pore water and pore air pressure, respectively.

g. $R = \frac{(\sigma_1-\sigma_3)}{(\sigma_1-\sigma_3)_f}$, ratio of total deviator stress to deviator stress at failure.

h. $\bar{R}$, same as above but with respect to effective stresses.

Repetitions.

a. N, number of stress repetitions applied to a specimen.

Review of Principles of Material Fatigue

Fatigue of metals and solids in general has been designated as a rather abrupt failure of a specimen or part when subjected to reversed or alternating stresses (25). Bending, torsional, axial stresses or a combination thereof may be involved, with or without the application of a constant static stress. The static stress may be compressive or tensile, but in all fatigue tests the stress reversal places the component in some degree of tension.

Metal fatigue attained importance in the mid-nineteenth century with the advent of the reciprocating steam engine. High failure rates of railroad engine axles and other components which could not be attributed to excessive stresses launched investigations into reverse stressing tests, the principles of which still survive in the modern rotating-bending tests. From these tests was developed the concept of the stress-repetition relationship (the so-called S-N or σ-N curve) as exemplified in Figure 2.2.

To develop the σ-N curve specimens are subjected to the alternating stresses shown as σ_a, σ_b, σ_c, etc., and the number of repetitions to fatigue are plotted versus the stresses. For ferrous alloys there is a relatively abrupt flattening of the σ-N curve; the stress corresponding to this point is termed the "endurance limit." Stresses at this level can be applied indefinitely without producing fatigue. Such is not the case for non-ferrous metals and many other common construction materials; the stress at some large number of repetitions, say 10^8, is defined as the endurance limit.

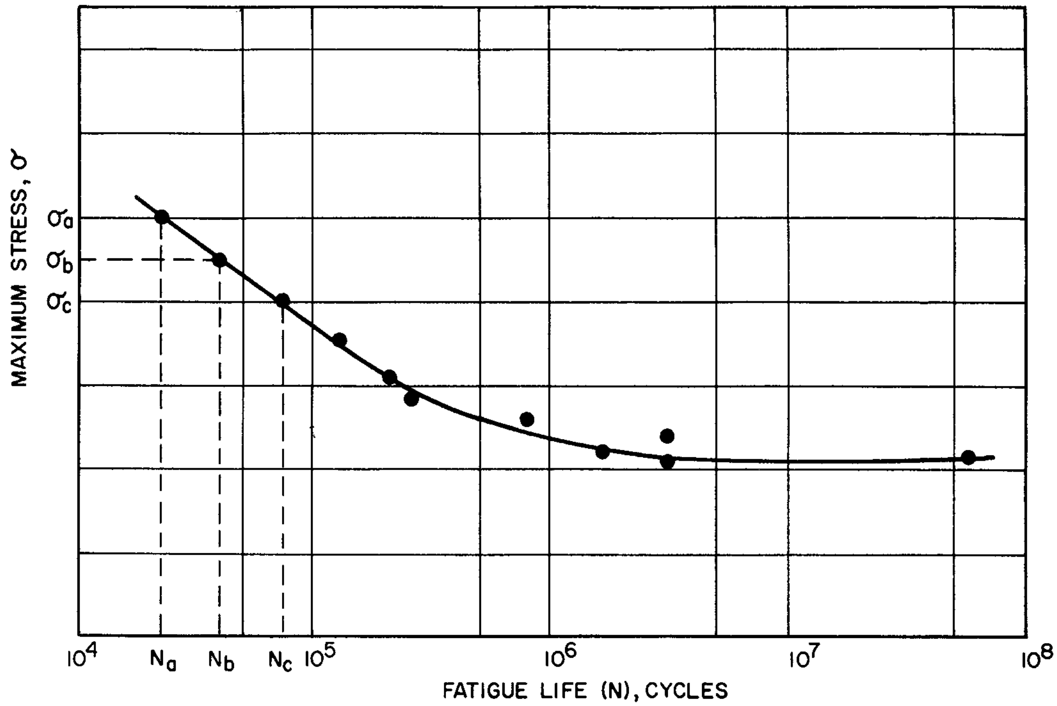


FIGURE 2.2 Typical fatigue curve for a ferrous alloy.

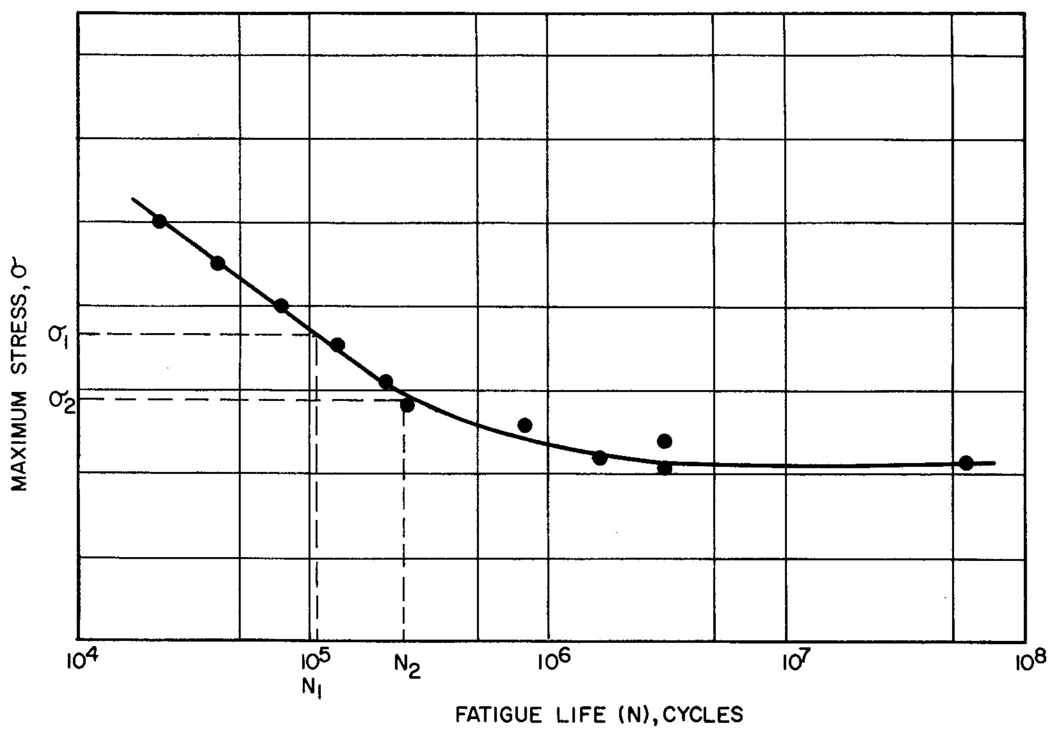


FIGURE 2.3 Illustration of the method of applying Miner's Law to assess accumulative fatigue damage to metals.

Although the fatigue relationship of metals is generally portrayed as a well defined σ-N curve, in truth a significant scatter of test data occurs in such tests, necessitating the testing of a large number of replicate specimens and the use of statistical analysis of the results. This is true even when all test coupons are obtained from a single piece of material and careful preparation is used to insure identical specimens.

Fatigue damage to metals is cumulative and as such varies greatly from the biological concept of fatigue. Periods of rest do not lead to recovery of the repeated stress effects, and understress or overstress results in cumulative damage. Thus the stress history is of extreme importance in their behavior. Miner's Law (25) is one of the best known methods of estimating metal fatigue characteristics under conditions of varying stress where the direct use of the σ-N curve is of no practical help. The assumption is made that the fractions of fatigue life used up at different stresses may be simply added to give an index of the fatigue damage. With relation to the σ-N curve in Figure 2.3, if the fatigue life is N_1 and N_2 at stress levels of σ_1 and σ_2 respectively, and n_1 and n_2 are the actual number of applications of σ_1 and σ_2 , failure under the two stresses will occur when:

$$\frac{n_1}{N_1} + \frac{n_2}{N_2} = 1 \qquad \text{Equation (2.1)}$$

For any number of stress levels, Equation (2.1) reduces to:

$$\Sigma(n/N) = 1 \qquad \text{Equation (2.2)}$$

Miner's Law is not very successful, particularly in materials where stress history influences the strengthening or weakening processes.

Others have attempted to determine more realistic relationships, but it appears that fatigue damage under varying stress conditions is difficult to assess. Strain-aging or work-hardening under repetitive stressing is always one of the major difficulties involved.

Creep is another important effect in repetitive stressing of many solids. Creep is essentially a stress-strain-time relationship while fatigue, as previously defined, is a stress-repetition relationship. Basically, it is thought that the fatigue characteristics of certain materials are also influenced by creep processes. Only recently results have been presented to show the relationship between plastic strain amplitude and the number of cycles to failure. (See typical results in Figure 2.4).

In summary, it may be said that processes contributing to metal fatigue (and creep) in solids are still poorly understood although rapid advancements in knowledge are being made.

The principles of solid state physics have done much to explain some of the factors creating metal fatigue, but most experts agree that there is much to be learned about the basic mechanisms contributing to fatigue of metals, largely because of the lack of a sound general theory and the complexity of the metallurgical effects. Stress repetitions cause irreversible changes in the material -- either work-hardening or -softening depending on the material, stress magnitude and environment. Perhaps most important in relationship to the general subject of this report is that experimental error in materials which are generally thought of as being very homogeneous is exceedingly large. To establish

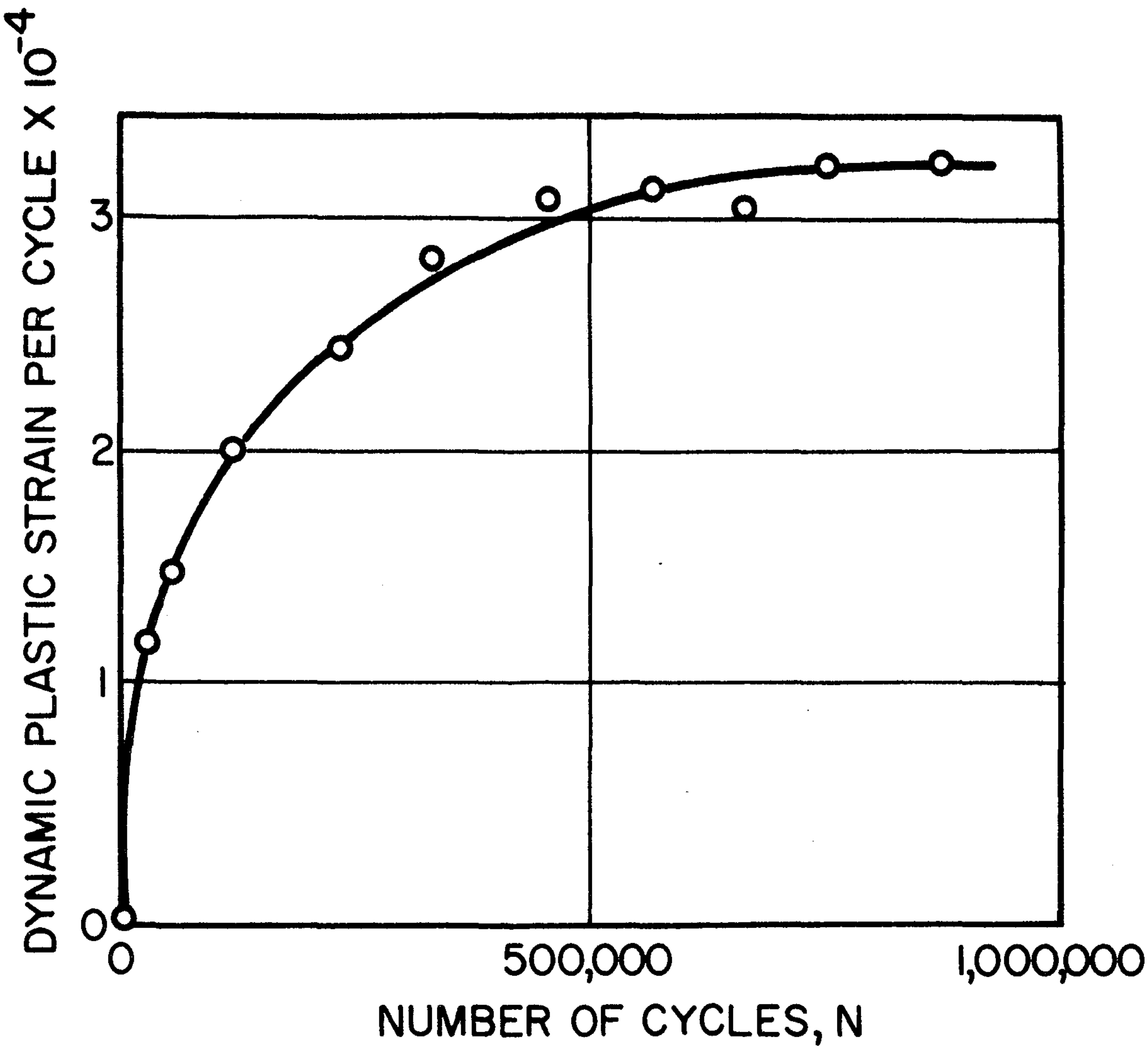


FIGURE 2.4 Influence of stress cycles on the plastic strain or creep of a metal. After Kennedy (25).

a σ-N curve for a particular metal, some three to five specimens must be tested at each stress level to obtain statistical significance.

Repetitive Triaxial Test on Soils

Chen (12) in 1948 reported on effects of repetitive loads on soil behavior; however, the initial contribution in this area is generally credited to Khuri and Buchanan (26). They tested specimens of a lean clay representing the subgrade soil for a rigid pavement test facility. The purpose of the research was to investigate the supporting capacity of the soil under loading conditions used at the test facility. As such, the repetitive triaxial stresses were applied manually, stress magnitude was small, and repetitions were applied at a rate equivalent to actual stresses imposed by the loading test cart moving 2.5 - 4 mph. Less than fifty load repetitions were applied to any one specimen. Later investigations at Texas A&M University by Werner (34), Spencer(40), Allaire (1), and Dunlap (16) were primarily concerned with the effect of proof rolling on soil behavior and they utilized a universal testing machine for the loading apparatus with the number of applications limited to 200. Beginning in 1961, Dillon (15), Armstrong (4), Hargis (20) and Wolfskill (45) applied large numbers of repetitions to granular materials with the automatic repetitive loading equipment described later in this report.

Seed and his co-workers at the University of California were concerned with the effects of repeated traffic loading on highway soils, primarily subgrades, and they were the first to develop suitable automatic equipment for applying large numbers of triaxial stress applications to soil specimens. At Purdue, Haynes (21), Johnson (24), and Larew and Leonards (27), utilized various automatic repetitive loading machines to test clays and flexible base course materials. Yamamouchi and Luo (46) at Kyushu University, Japan, utilized a repetitive loading machine to perform repetitive unconfined compression tests.

All investigators took great pains to eliminate impact loading on the specimens during repetitive loading.

Khuri and Buchanan tested undisturbed specimens; all others worked with laboratory compacted specimens. Table II.1 and Figure 2.5 summarize salient features of the investigations discussed in this review. The review will be oriented toward showing the influence of repeated loading on various behavior conditions of the soil.

Influence of repeated loading on total and permanent deformation. Khuri and Buchanan (26) found that cumulative deformation did not occur for small repetitive stresses. The stress at which deformation began to occur -- termed the elastic limit -- was approximately one-tenth of the average unconfined compressive strength. When repetitively applied stresses exceeded the so-called elastic limit, then the total and permanent strain increased with the number of repetitions.

For a given time period, repeated loading can produce more deformation than a sustained load of equal magnitude (32). This is particularly surprising as the specimen is stressed by the repetitive load for a fraction of the total loading time of the sustained load.

By appropriately varying the unit weight and moisture content, Seed

TABLE II.1

Summary of Previous Research on Repetitive Triaxial Loading

INVESTIGATORS	SOIL DATA				SPECIMEN DRAINAGE	LOADING APPARATUS
	DESCRIPTION	LIQUID LIMIT	PLASTIC INDEX	GRADATION[1]		
Chen (12)	Cohesionless sands and gravels	Non-Plastic		--	Vacuum triaxial	Platform scale manually operated
Khuri & Buchanan (26)	Lean clay (undisturbed)	38	19	--	Undrained	Manual lever operated
Werner (43)	Ottawa sand	Non-Plastic		Uniform	Drained (dry)	Universal testing machine, manually operated
Spencer (40)	Angular well-graded sand (medium)	Non-Plastic		Curve A	Drained (dry)	Universal testing machine, manually operated
Allaire (1)	Angular well-graded sand (fine)	Non-Plastic		Curve B	Drained (dry)	Universal testing machine, manually operated
Dunlap (16)	Poorly graded sand	Non-Plastic		Curve C	Drained (dry)	Universal testing machine, manually operated
Dillon (15)	Angular well-graded sand (coarse)	Non-Plastic		Curve D	Drained[3] (dry)	Automatic hydraulic machine
Armstrong (4)	Silty gravel	20.5	5.3	Curve E	Drained[3]	Automatic hydraulic machine
Hargis (20)	Silty gravel	20.5	5.3	Curve E	Drained[3]	Automatic hydraulic machine
Wolfskill (45)	Silty gravel Crushed ls.	20.5 --	5.3 --	Curve E Curve F	Drained[3]	Automatic hydraulic machine
Seed et al. (32) - (37)	Silty clay Clayey silt Silty sand	37 36 --	14 10 --	-- -- --	Undrained	Automatic pneumatic machine
Haynes (21)	AASHO gravel AASHO cr.ls.	Non-Plastic Non-Plastic		Curve G-1 Curve G-2	Undrained	Automatic pneumatic machine
Larew & Leonards (27)	Micaceous silt Ls. res. clay Sandy clay	34 65 16	3 33 1	-- -- --	Undrained	Automatic cam-operated lever machine
Johnson (24)	Ottawa sand with 14,19 & 29% binder	34[2]	14[2]	Curves H-1 H-2, & H-3	Undrained, pore pressure measurements	Automatic pneumatic machine
Yamanouchi & Luo (46)[4]	Sandy loam	55.6	16.2	--	Drained[3]	Automatic universal testing machine

1. Curve designations refer to curves shown in Figure 2.5.
2. Atterberg limits for binder.
3. Drainage allowed but specimens loaded too rapidly for complete drainage under each load repetition.
4. Unconfined specimens.

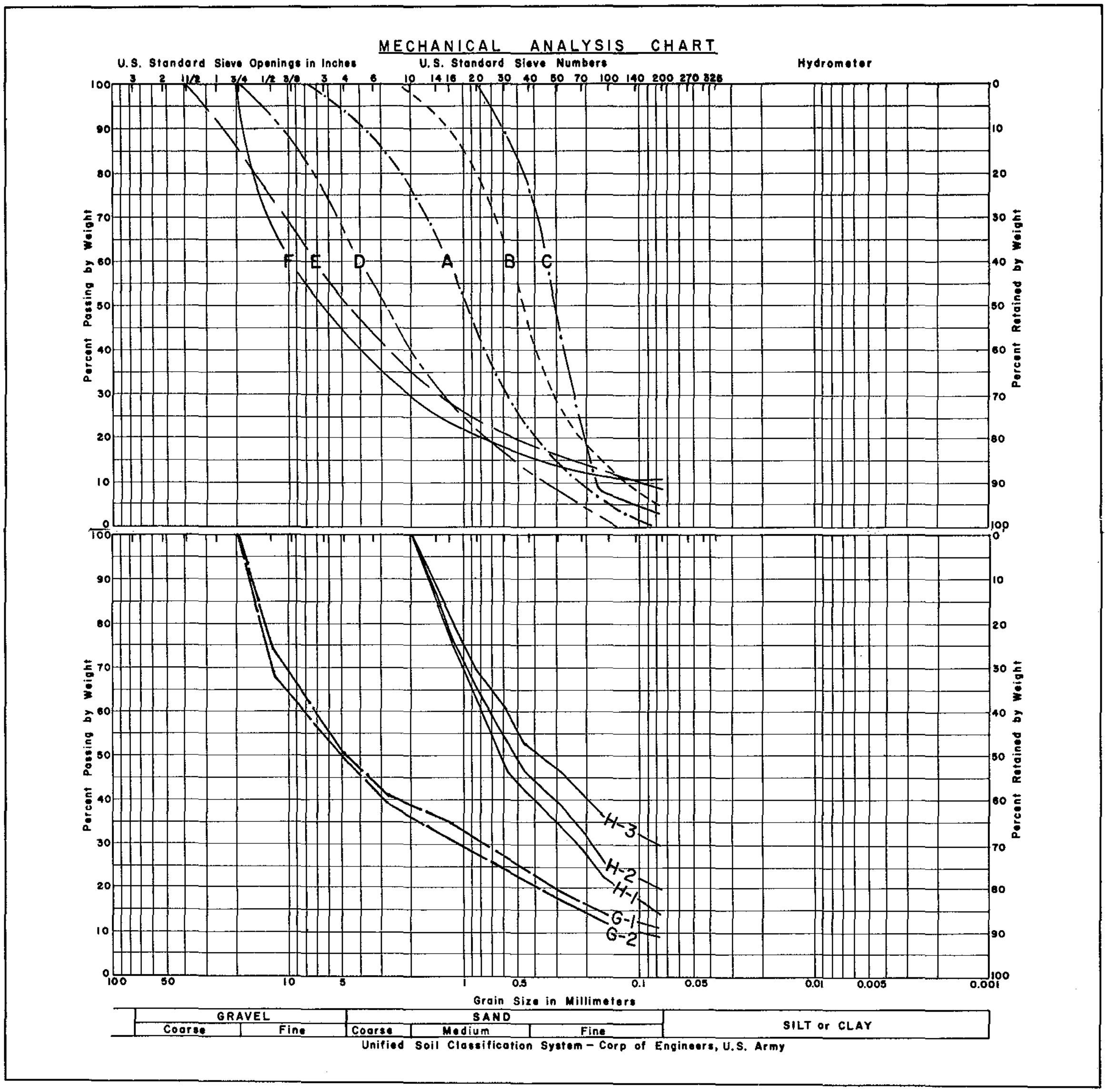


FIGURE 2.5 Gradations of materials used by previous investigators in repetitive triaxial loading research.

and McNeill (33) were able to produce specimens of a silty clay and a clayey silt which required the same stress to attain a particular strain in normal compression tests. One thousand repetitions of the same stress produced significantly greater total deformation in the clayey silt except at very high unit weights. This action is not entirely unexpected to one familiar with the properties of silts, but it does raise a question concerning the validity of extrapolating normal compression tests to conditions of repetitive loading.

The relationship between deformation and repetitions is rather unsure and probably varies with the type of soil. Khuri and Buchanan (26) claimed a parabolic curve existed for the total strain and the log of repetitions for a CL with less than 50 repetitions. Chen (12) found a linear relationship for total strain versus the log of the number of repetitions for several sands and gravels. Armstrong(4) and Hargis (20) used a linear log-log relationship between total deformation and number of repetitions, but agreed that this did not produce an excellent fit to the experimental data; test results indicate a parabolic relationship. Tests by Seed et al. (32), and Larew and Leonards (27) have shown a curvilinear relationship for undrained triaxial conditions.

In some instances a specimen may slowly deform under thousands of repetitions and then suffer a rather abrupt failure similar to metal fatigue (32) (27).

Thixotropic stiffening, which is time dependent, may influence repetitive test results on soils which display this phenomenon. It is certainly an important variable which must be considered in analyzing repetitive triaxial tests.

Influence of repetitive loading on rebound deformation. Hveem (23), among others, demonstrated how excessive resiliency in underlying materials can produce fatigue failure of roadway surfaces even in the absence of significant cumulative total deformation. Rebound deformation in repetitive triaxial tests may be excellent indicators of excessively resilient materials.

As a general rule, rebound deformations are initially large but decrease with the number of repetitions (26)(20). Larew and Leonards (27) suggested that the rebound reached an equilibrium value after approximately one thousand repetitions. Rebound reached a state of quasi-equilibrium after 10,000 repetitions according to Johnson (24), but other evidence indicated this is a function of the applied stresses (35).

As would be expected, the type of soil has considerable influence on the rebound deformation. For the silty clay and the clayey silt tested by Seed and McNeill (33), repeated stresses at magnitudes which produced equal strains under normal triaxial conditions produced rebound strains in the clayey silt which were significantly higher (by a factor of two in some instances) than those observed for the silty clay.

Magnitude of repeated stresses on deformations. It has been mentioned that Khuri and Buchanan (26) found a stress below which permanent deformation did not occur with repetitions. Larew and Leonards (27) hypothesized that a stress existed at which the slope of the deformation versus repetitions curve would become flat after some small initial permanent deformation, but the data did not confirm this fact.

In general, both the total and rebound deformations increase with an increase in the magnitude of repeated stresses (16)(20)(26)(45). It does not appear that any definite relationship exists between the magnitude of repeated stresses and the observed deformations (20). Very likely the type of soil -- particularly the stress-strain characteristics of a material -- has much to do with this relationship. Seed and Chan (32) did provide curves for a silty clay showing the relationship between the number and magnitude of repeated axial stress required to produce a given deformation. Khuri and Buchanan (26) produced a somewhat similar relationship for a limited number of repetitions.

Effect of unit weight (or void ratio) and degree of saturation on deformation under repetitive loading. For cohesionless materials, void ratio has significant influence on the total and rebound deformations. In general, as the void ratio decreases (or the relative density increases) both total and rebound deformations are reduced for the same magnitude of stress (16). However, previous stress history (prior application of higher or lower stresses) may influence the results. Chen's (12) tests, with slow repeated stress applications, showed that failure deformations occurred rather quickly when the soils (sand and cohesionless gravels) were close to the critical void ratio.

Hargis (20) provided very limited information on a flexible base material which showed that the total and rebound deformation were minimized at the optimum moisture content for a particular compactive effort.

Haynes (21) found that unit weight and degree of saturation affected the deformation of the gravel and crushed stone base courses used in the AASHO Road Test. A slight change in the degree of saturation had more effect on the total and rebound properties of both materials than a slight change in the unit weight. However, unit weight changes significantly influenced rebound characteristics: increased unit weights resulted in decreased rebound deformations.

Seed and McNeill (33) found that for the same number of stress applications (1,000) specimens compacted to 91 percent of Modified AASHO standards exhibited more than three times the total deformation of specimens compacted to 95 percent Modified AASHO. Rebound deformation also decreased as the degree of compaction increased. On the other hand, tests by Larew and Leonards (27) on a highly plastic clay seem to indicate that for any compactive effort total and rebound deformations are greatest at optimum moisture content and that they increase with an increase in compactive effort. On this basis they call for a critical look at existing compaction standards.

Influence of repeated loading on strength and resistance to deformation. Strength and resistance to deformation are so closely related it is appropriate to discuss these two items together. The influence of thixotropy on the behavior of clay soils makes it desireable to divide the discussion into sections on cohesive soils and cohesionless soils (including flexible bases which may be slightly cohesive).

A. Cohesive soils. In their initial investigation Seed and Chan (32) concluded that the strength of a partly saturated silty clay, after being subjected to repeated loading, was considerably greater than that of previously unloaded specimens. The greater the magnitude of the repeated load, the greater the increase in strength. Later investigations (36) showed that the amount of strength change depends also on the initial degree of saturation of the specimens: low degrees of saturation produced higher strengths. But regardless of the degree of saturation, increased resistance to deformation also developed. However, when the specimen strain exceeded about four percent, any benefits of the repeated loading on resistance to deformation were lost.

Repetitive loading under one stress made the soil more resistant to deformation if the magnitude of the repeated stress was subsequently increased (36). While some specimen densification probably occurred, which aided in creating deformation resistance, it was concluded that structural rearrangement of the particles, extrusion of adsorbed water between soil grains, and thixotropy were the primary reasons. Again it appears that any beneficial changes were lost if the deformation exceeded about four percent.

If the magnitude of the repeated stress is large enough to cause a sufficient increase in unit weight with subsequent increase in degree of saturation, then a decrease in strength will occur (36). Thus the stress history (magnitude and number of repeated stress applications) will determine whether an increase or decrease in resistance to deformation will occur.

B. Cohesionless soils. There are conflicting opinions regarding the strength and resistance to deformation of cohesionless soils subjected to repeated loading.

Chen (12), in a closely controlled experiment on sands and gravels subjected to 1,000 slowly applied triaxial stresses, concluded that dry specimens of cohesionless soil "harden" under repeated stresses which are significantly smaller than the static ultimate stress. For repeated stresses approaching the static ultimate stress, specimens weakened due to dilatancy and eventually failed at stresses less than static ultimate strength. Dunlap (16) applied 200 load applications to a fine sand subjected to a wide range of confining pressures, relative densities and repeated loads. A maximum shear strength increase of approximately ten percent was observed after repetitive loading; maximum resistance to deformation (as indicated by the secant modulus) more than doubled in some instances. Increases in strength and resistance to deformation were usually proportional to increases in confining pressure, relative density, and magnitude of the repeated load. Although the critical void ratio was not determined, its influence on behavior is noticeable, particularly at low relative densities. Allaire's (1) conclusions were comparable to Dunlap's. Under high confining pressures, Dillon (15) observed maximum strength increases of twenty-one percent after 25,000 repetitions at two-thirds of the static ultimate strength. Strengths decreased as much as 16 percent though when the confining pressure was low, which again indicated the relationship of the critical void ratio to the behavior of the materials.

Seed and et al. (36) reported that no increase in resistance to deformation occurred for a fine sand subjected to repeated loading, but later it was claimed that the duration of stress applications and the interval between applications influenced the deformation of a silty sand (see next section).

For the two flexible base course materials which he evaluated, Wolfskill (45) found a significant increase in strength and resistance to deformation resulting from repetitive loading. He attributed this to densification of the specimens under repeated loads.

Influence of duration and frequency (interval) of repeated loads on deformation. After their initial investigation, Seed and Chan (32) stated that specimen deformation depended only on the number of stress applications and was independent of the frequency of applications, at least within the range of one to twenty applications per minute. By chance, the specimens on which these conclusions were based had low degrees of saturation and little thixotropic strength gain. In subsequent research (37), the influence of duration and interval of stress repetitions on a silty clay and a silty sand was examined in greater detail.

For the silty clay an increase in the interval between applications reduced the total deformation if the duration of application was short, but if the duration was long it caused an increase in the deformation. It was suggested that this behavior was due to thixotropic effects, structural changes in the specimens, or creep. Also during unloaded periods, the clay particles may have slightly separated from each other.

These tests were performed on specimens at relatively high degrees of saturation, and the suggested reasons for their behavior, as explained above, were not effective in the earlier tests on samples of low saturation.

The silty sand had greater deformations as the duration of stress application increased. For any duration time, deformation was largest for short intervals and decreased significantly with longer intervals. It was postulated that this was a function of the adsorbed water layers on the sand particles which were driven from interparticle contact points during load application. For rapidly applied loads, the water layers were unable to return to these points, thereby resulting in high interparticle friction. In view of the recent research of Horn and Deere (22) on interparticle friction which showed that massive-grained materials usually found in sands exhibit higher interparticle friction when wet, this reasoning for the deformation behavior seems rather untenable to the writer. Instead, it is believed that stress history influenced the results more than interparticle friction.

Armstrong (4) examined the influence of frequency and duration of repetitive loading on the total deformation of a limestone gravel base course material. The results were expressed quantitatively as follows:

$$\log t = -0.2653 - 0.0091T + 0.2301L - 0.0240TL + 0.2153 \log N \qquad \text{Equation (2.3)}$$

where:

T = loading interval in seconds.

L = duration of loading in seconds.

These results also indicate that long loading durations and short intervals between loads produce larger deformations; as indicated in previously discussed research, an increase in the number of applications also will produce larger deformations.

Development of failure theories for repetitively loaded soils. Some attempts have been made to develop a strength criterion or failure theory -- similar to those postulated for metals -- for soils subjected to repeated loading.

Based on the University of California findings that soils often deform gradually for many repetitions and then fail rather abruptly, Larew and Leonards (27) hypothesized that some repeated deviator stress existed at which the slope of the deformation versus repetition curve was constant after a few repetitions. This could be interpreted as the critical strength not to be exceeded by repeated loads. The results did not indicate that this occurred as such, but they were suggestive of a possible relationship of this type.

The concept of the elastic limit as developed by Khuri and Buchanan (26) is also indicative of some type of fatigue strength criterion which might be applied to repetitively loaded soils.

Yamamouchi and Luo (46) claimed that for any particular strain, a relationship resembling a fatigue (or S-N) curve similar to metals existed. In addition, the total sum of the ratio of applied stresses to the maximum allowable stress was obtained, and it was concluded that Miner's method of assessing accumulative fatigue of metallic materials when subjected to varying loads was applicable to soils. These conclusions were based on repetitively loaded unconfined compression specimens and the results are much too limited to be convincing.

Summary

At this printing (1966) little more than a decade has elapsed since the inception of repetitive triaxial testing as applied to highways and runways. Approximately 20 separate soils have been examined which is hardly sufficient to establish characteristics of most soils or even delineate the most important variables.

The conclusions obtained from the various investigations are not always in agreement. The complex interactions which occur between such factors as loading characteristics (the form of the loading curve), frequency and interval of stress repetitions, and void ratios and degrees of saturation at which the materials are tested, may be responsible for much of the disagreement.

As mentioned in Chapter I, few methods of pavement design take repetitions into account. Those which do, do so without regard to the type of soil involved. The results from repetitive triaxial testing certainly lead one to question the efficacy of extrapolating normal compression test results to conditions of repetitive loading. Soils which have seemingly similar characteristics in normal compression tests may have widely varying characteristics, especially the rebound deformations, in repetitive triaxial tests.

It does appear that soils have some of the behavioral characteristics of metals -- strain-aging or work-hardening. In cohesionless materials, either may occur depending on the stress magnitude and the void ratio at testing as compared to the critical void ratio. In cohesive materials thixotropy, which is a function of time (and perhaps stress magnitude) for any particular soil, may be related to strain-hardening. Any beneficial effects of thixotropy will be lost under excessive strain according to the available information. Increases in unit weight under repetitive loading can also be beneficial providing the increase in degree of saturation is not significant.

Somewhat similar effects may be noted in slightly cohesive materials in which cementation may occur, e.g., certain crushed limestones. Presumably this effect is similar to thixotropy as observed in highly cohesive soils and any beneficial effects would be lost at strains which are large enough to overcome cementation at points of contact.

Attempts to determine endurance limits or strength criteria similar to those for metals have met with little success. Generally soils will continue to deform as long as they are repetitively loaded. The tests of Khuri and Buchanan are a notable exception to this. It is the writer's belief that the elastic limit which they found existed because the specimens, which were undisturbed, exhibited greater thixotropic effects than the laboratory molded specimens used by other investigators.

It is generally accepted that the major breakthrough in the Soil Mechanics field came about through knowledge of pore pressure and effective stresses in soils. Armstrong, Hargis, and Wolfskill concluded that a knowledge of the effective stresses in repetitively loaded soils was necessary to correctly explain the behavior of soils in repetitive loading. This writer agrees wholeheartedly. However, this is a formidable task, particularly for partly saturated soils. Johnson (24) did measure pore pressure in saturated to near saturated soils. The results were encouraging, although the conclusions of the investigation did not fully convey the effective stress concept as it could be related to repetitively loaded soils.

It is indeed unfortunate that most of the investigations did not use statistical concepts in the design and interpretation of the experiments. Certainly both compacted and undisturbed soil specimens are more heterogeneous than the test coupons of metals which exhibit such extreme variability in fatigue tests. Conclusions based on single specimens cannot provide any concept of experimental error or variability of results which should and will be found in soil specimens.

Repetitive triaxial tests have many limitations and should not be considered as a panacea for eliminating all problems connected with load repetitions on roadways. Of extreme importance is the fact that unequal stress distributions occur in triaxial tests (39) which may result in observed strains significantly different from those which would occur in the roadway subjected to the same stresses. This of course is a problem which exists in normal compression tests, but it may be of greater importance in repetitive tests. The manner in which the stresses are applied to the specimen is also subject to question. At a point in the roadway the vertical and radial stresses must certainly increase simultaneously as the traffic load approaches the point. With a few exceptions repetitive triaxial tests have been performed under constant confining pressure. In those few instances where the confining pressure has been increased proportionate to the repeated deviator stress the observed deformations have been larger than with a constant confining pressure (34) (35). Since stress history does have an important influence on deformation under repeated loads, and since it appears that no definite rule can be applied to account for the effects of stress history, it will be difficult, perhaps impossible, to translate laboratory test results to the roadway where mixed traffic loads and varying load intervals occur.

As important as these limitations may be, the advantages of repetitive triaxial testing definitely outweigh the disadvantages. Repetitive triaxial tests on roadway materials must be continued.

CHAPTER III

THE RESEARCH MATERIALS: SELECTION, PREPARATION AND PROPERTIES

Hard Rounded Material

Parent material. A "caliche gravel" was obtained from a stream terrace deposit on the Engler property located roughly one-half mile south of the Guadalupe River and approximately four miles south of Seguin, Texas. This material has been used for flexible base course by the Texas Highway Department both in the pit run state and as a processed material (which consisted of crushing the particles exceeding 1-3/4 inches) on several Farm-to-Market Roads and State Highways in the surrounding area. It has reportedly given excellent service for these applications (6).

Caliche gravel is a local name which belies the true nature of this material. Particles larger than about one-quarter inch were either hard limestone, or chert surrounded by a shell or "rind" of porous-appearing but hard calcium carbonate. The thickness of the rind ranged from paper-thin to a maximum of one-quarter inch. Particles finer than about one-quarter inch were either hard limestone (rounded), quartz, or soft angular particles which were a mixture of calcium carbonate, sand, silt, and small amounts of clay minerals. The soft particles satisfy the rather loose terminology of "caliche", and their cementing action in the roadway might be at least partly responsible for the excellent serviceability of this material.

The caliche gravel probably was derived from weathering of the rocks in the Edwards Plateau area, and was rounded and swept down to the lower elevations by stream action. It is thought that after deposition, the softer calcium carbonate particles were dissolved by ground water leaching through the deposit. Then, during dry periods, the carbonate-enriched ground water deposited the rind on the particles (9). Annular rings - similar to the growth rings on trees - were often observed in the rind, tending to confirm this hypothesis (See Figure 3.1).

Approximately eight cubic yards of this material were obtained from the face of an existing pit. The parent material appeared well graded with a maximum particle size of about four inches; it contained a noticeable amount of bark and small twigs.

Prepared research material. Approximately three cubic yards of the caliche gravel were dried in a 140°F oven, and then sieved into several different size ranges on a Gilson shaker. The separated material was stored in containers for later recombination of the various sizes into the desired gradations.

The shape of the larger particles can be described as rounded to sub-rounded with many elongated particles; smaller sizes contained both rounded and angular particles. (See Figure 3.2).

The prepared material was designated as material HP-27-8. For reader ease, it subsequently will be referred to as "rounded" material.

Hard, Angular Material

Parent material. To obtain a material constitutively similar to the hard rounded material but with angular particle shape, a portion of the caliche gravel was crushed.

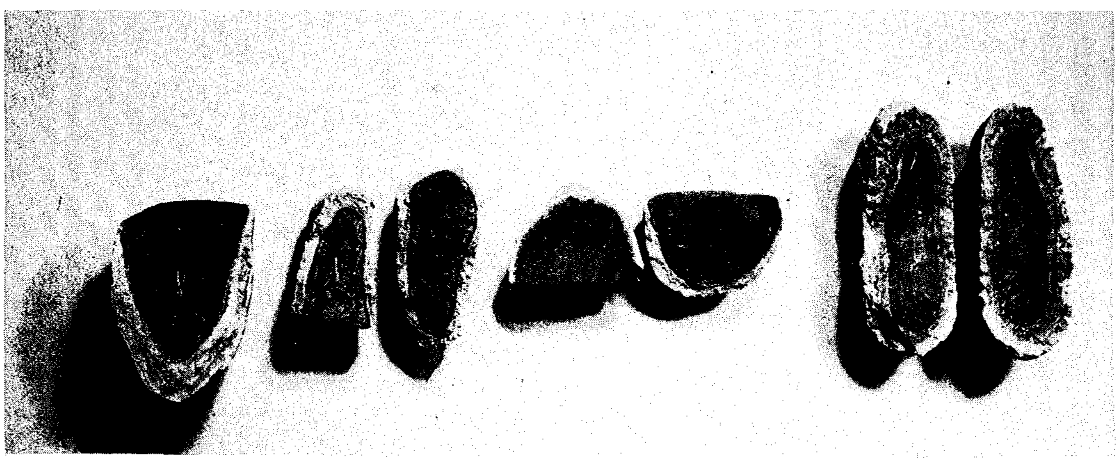


FIGURE 3.1 Typical particles of the caliche gravel split open to show the calcium carbonate rind on this material. Top picture shows particles with chert centers while those in bottom picture are carbonate centers. Note variation in thickness of the rind and the presence of annular rings in the thicker rinds.

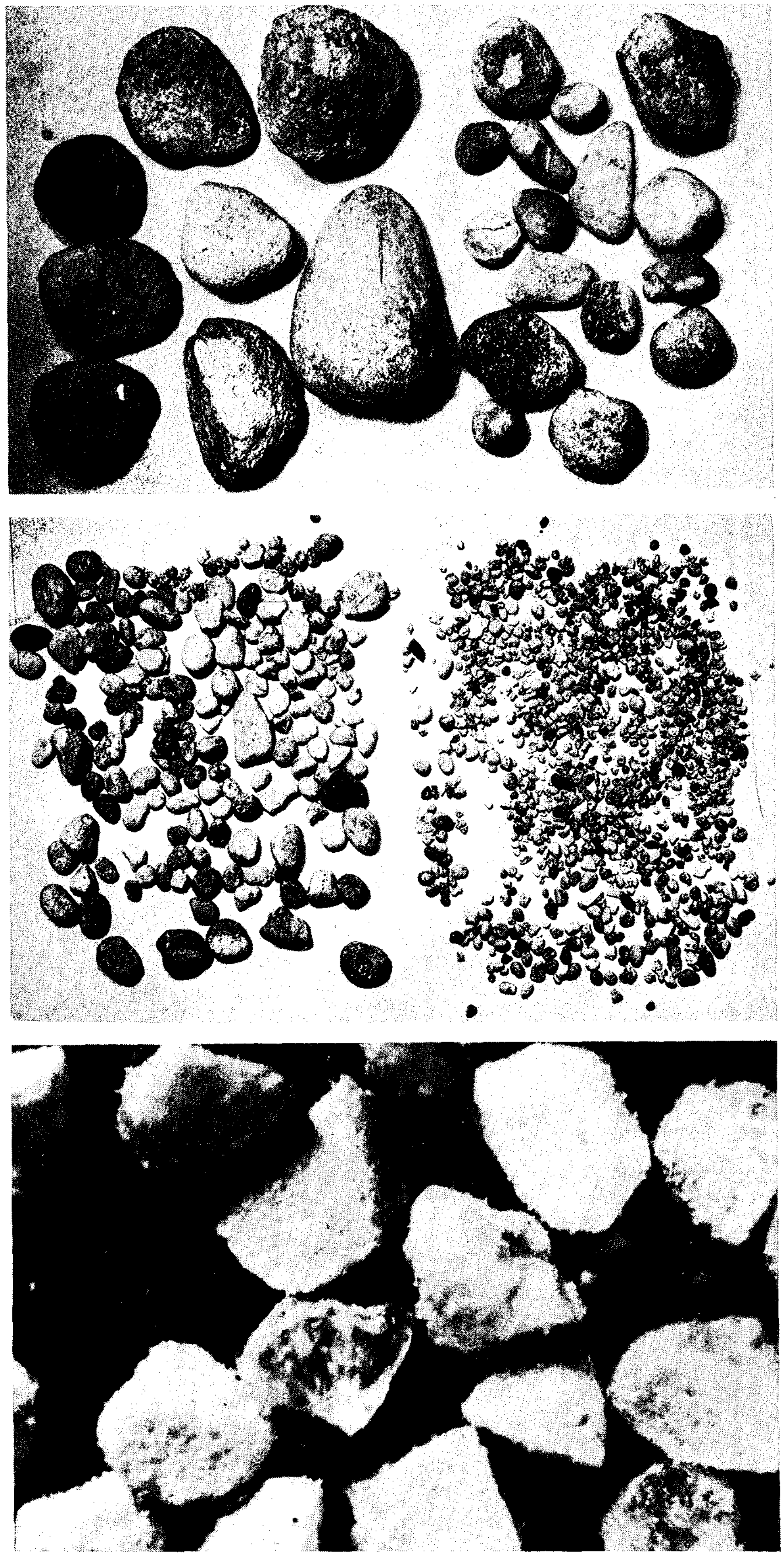


FIGURE 3.2 Rounded material. The upper picture shows particle sizes of 1-3/4 - 1/2 inches; middle picture is 1/2 inch - - #10 mesh particles; lower picture #40 - #50 mesh particles, magnified 30X. In the lower picture, quartz particles can be noted; the "fluffy" particles are soft calcium carbonate.

Prepared research material. Particles larger than one inch were scalped from the remaining caliche gravel. After oven drying, primary crushing was accomplished in a 6- by 5-inch laboratory jaw crusher. Smaller sizes were obtained with a "chipmunk crusher"; a plate grinder and a ball mill were required to obtain the very fine fraction. The jaw crusher produced roughly cubical particles, but the chipmunk crusher tended to produce elongated and flaky particles. This tendency was minimized by "crowding" or forcing material into the crusher jaws, but flaky particles were noted in the final product. In general, the crushed material was highly angular except in the larger sizes. Here, the parent gravel did not contain sufficiently large particles to produce a crushed particle with all angular faces, and often one side of the particle was rounded and the other angular. However, these particles constituted only a small percentage of the total and certainly had little effect on the overall angularity and behavior of the material. Figure 3.3 shows the particle shape of the prepared material.

The prepared, crushed material was also separated and stored as previously described. It will be designated in this paper as the "angular" material (laboratory designation was HP-27-9).

Soft Material

Parent material. The parent soft material was obtained from a quarry on the Friesenhahn property located approximately one-half mile south of Interstate Highway 35 at Selma, Texas. The quarry was not in use at the time of sampling, but it was the source of flexible base

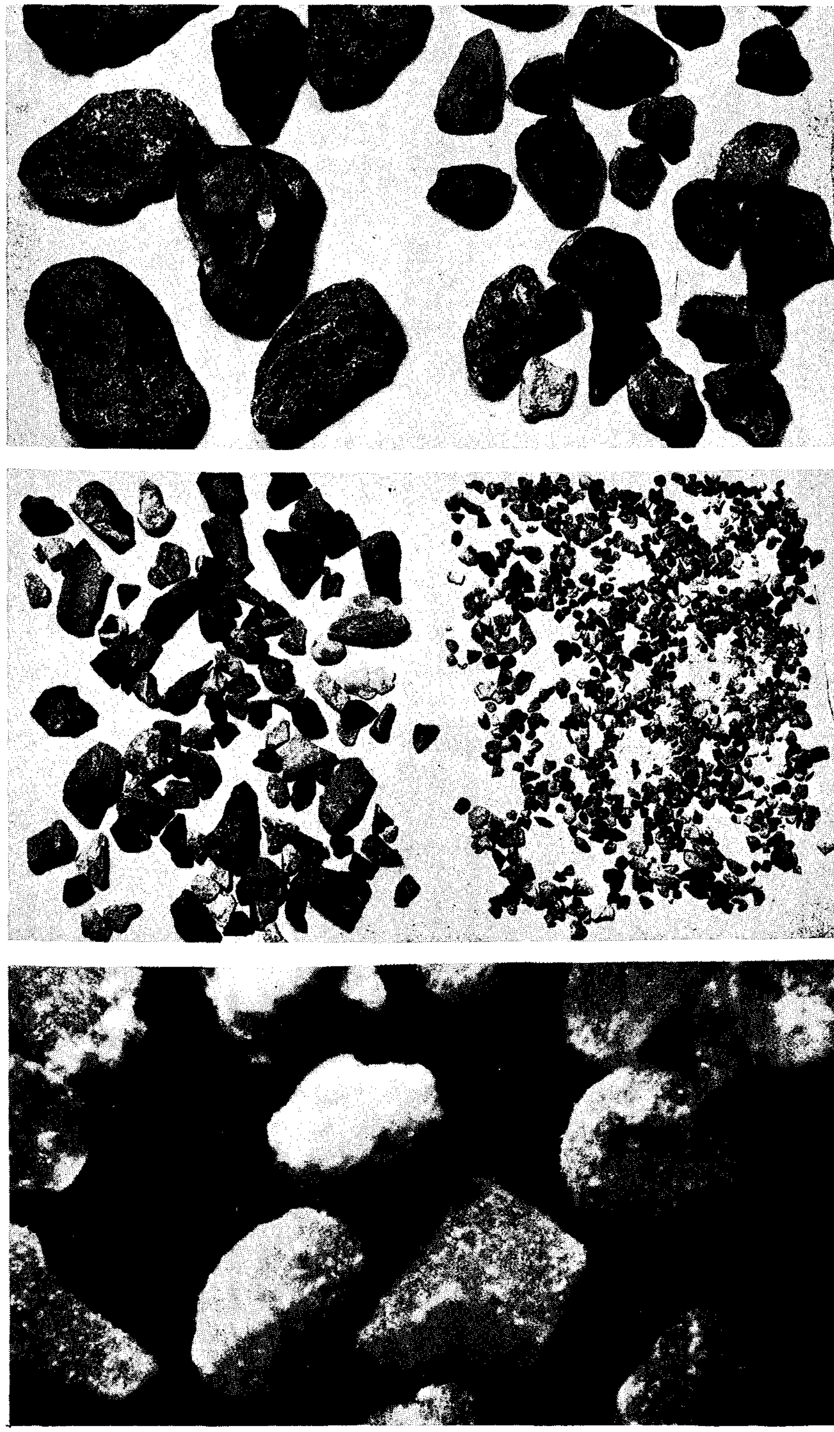


FIGURE 3.3 Angular material. The upper picture shows 1-3/4 - 1/2 inch particles; middle picture is 1/2 inch - #10 particles; lower picture is #40 - #50 mesh particles, magnified 30X. The "harsh" and usually angular particles are quartz with some carbonate sticking to the surface. The "fluffy" particles are soft calcium carbonate.

course material for a section of Interstate Highway 35 between New Braunfels and Selma. Pavement distress occurred rapidly on the section, necessitating repairs which consisted mainly of stabilizing the Friesenhahn quarry material with cement.

This quarry is located in a moderately fossiliferous outcrop of the lower Navarro group -- probably the Corsicana Marl (38). The quarry face showed layers of limestone with varying hardness typical of this formation. Since it was desired to obtain a relatively soft material for this research, about four cubic yards of large soft fragments were selected from the rubble remaining on the quarry floor.

Prepared material. The large fragments were broken to fit into the laboratory crushers, and then dried. Laboratory crushing of the soft limestone produced no particular problems although it was again necessary to use several varieties of crushers to obtain the desired range of sizes. Due to the softness of the material, the particles ranged from angular to subangular; the shape varied from roughly cubical to some elongated particles (See Figure 3.4).

The prepared, crushed material was sieved and then stored. The soft, angular material was designated material HP-27-10, or simply "soft" material.

Aggregate Gradation

As described in the Specific Research Objectives (Chapter I), it was desired to test each material in the coarse, medium and fine ranges of the gradation allowed by the Texas Highway Department for Type A, Grade I, flexible base materials.

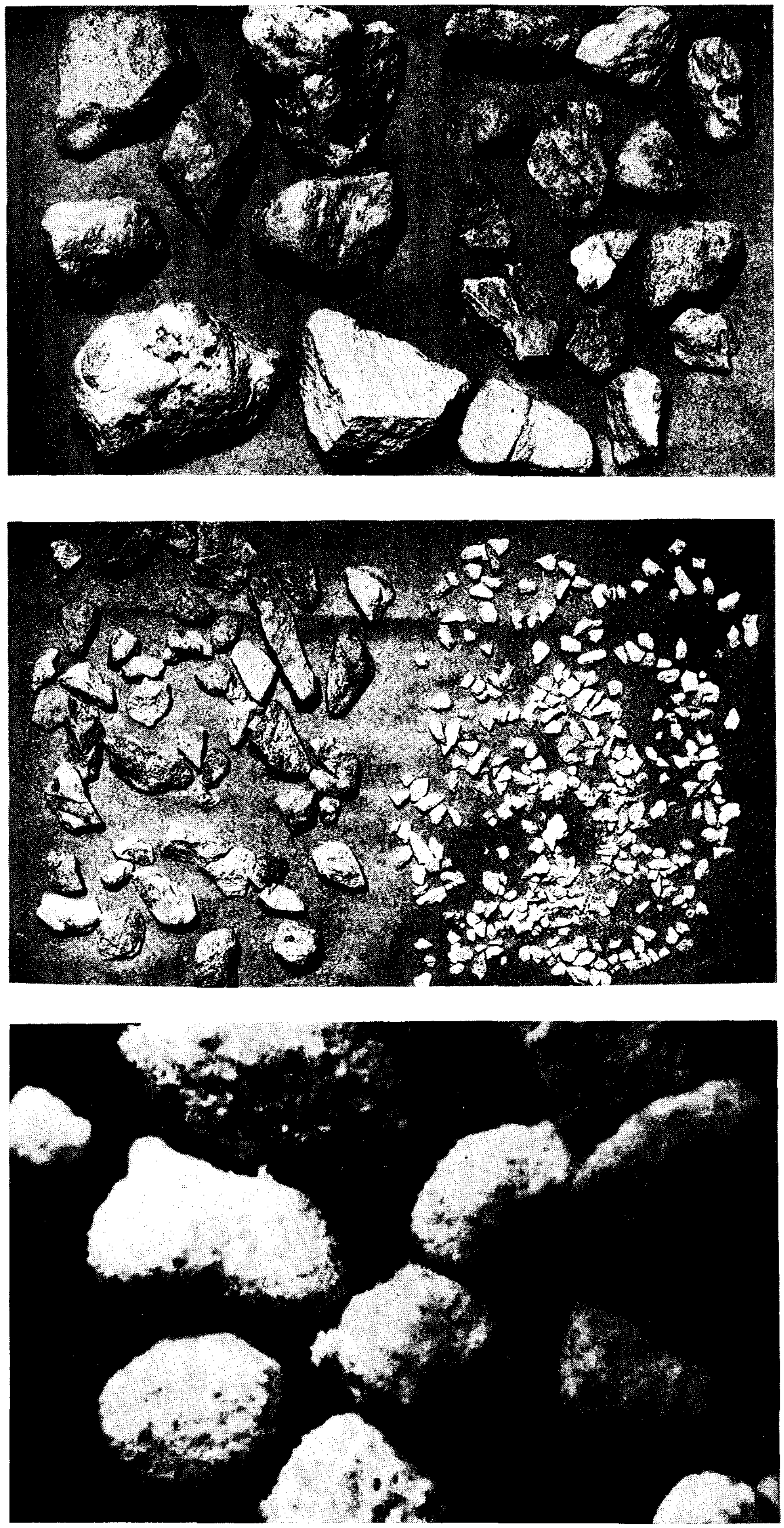


FIGURE 3.4 Soft material. The upper picture shows particle sizes of 1-3/4 - 1/2 inches; middle picture is 1/2" - #10 mesh particles; lower pictures are #40 - #50 mesh particles, magnified 30X. Small dark spots noted on many particles are glauconite.

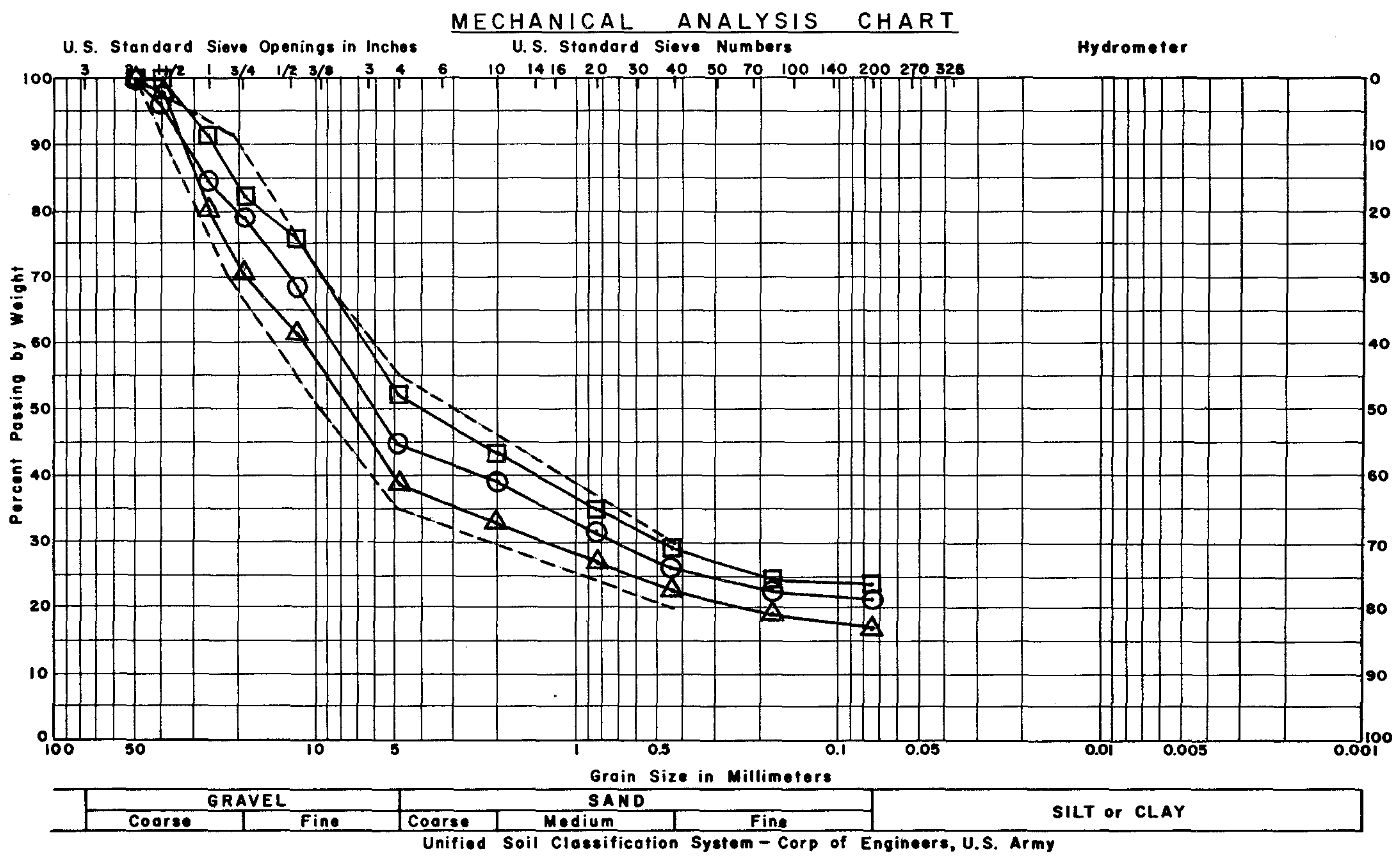
The gradations actually obtained by recombining the separate sizes for each material are shown in Figures 3.5, 3.6, and 3.7. There are always differences between supposedly identical specimens, and these figures represent the average results from several replicate specimens at each gradation. It should be emphasized that these specimens were not compacted or stressed in any manner. These gradations -- and all others presented in this report -- are "washed" gradations on specimens slaked for 24 hours before sieving. Tests proved that one hour of slaking was sufficient for the uncompacted specimens, but the 24-hour period was used throughout the research in anticipation of the difficulty of slaking compacted specimens later in the research program.

Optimally, all three materials should have the same coarse, the same medium, and the same fine gradations. This was physically impossible to achieve, but the variations which did occur between materials were minor.

Engineering Properties of the Materials

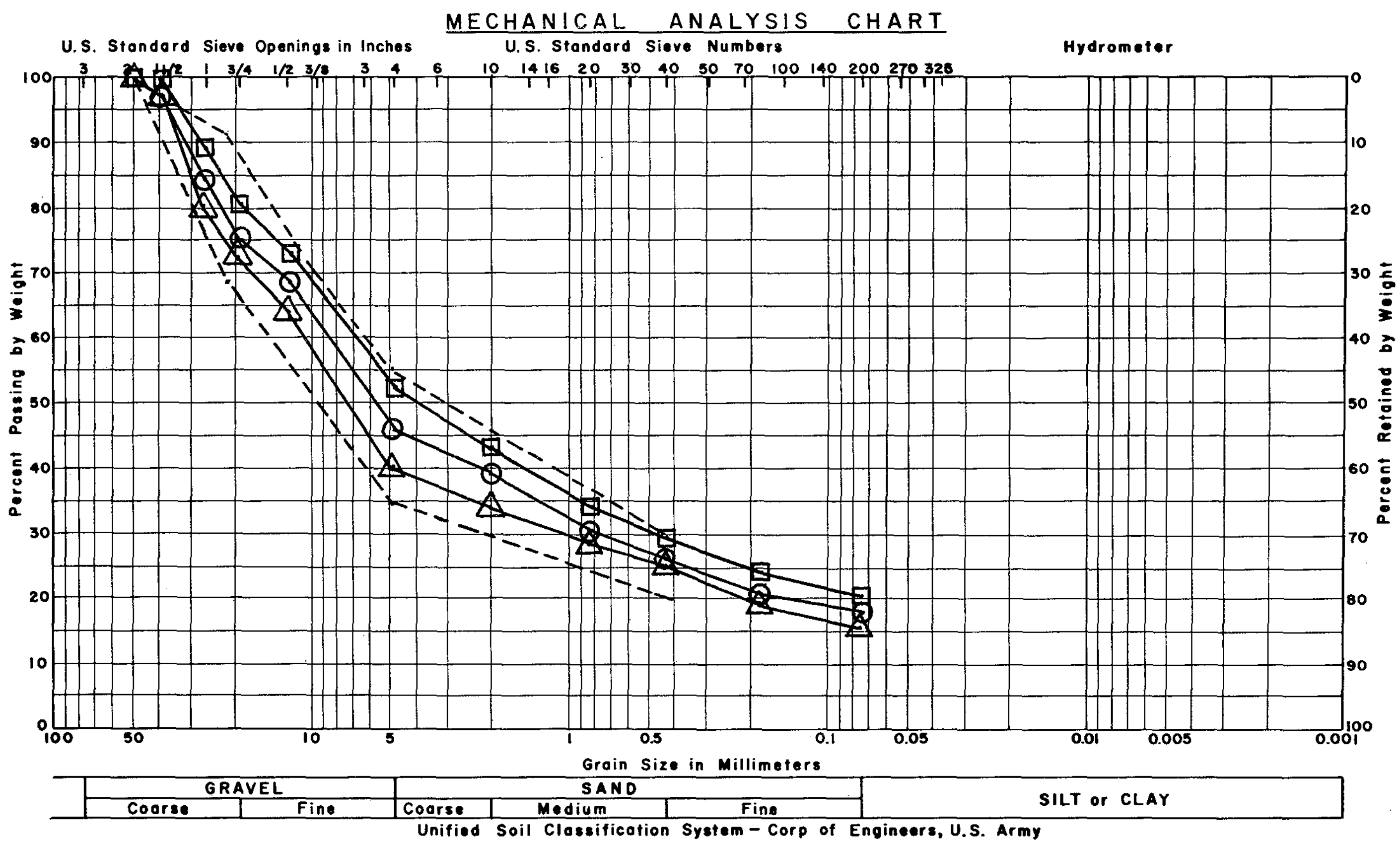
Physical tests were performed to classify the materials and obtain other data which might be helpful in analysing their behavior. Table III.1 contains a summary of the more important tests. Brief comments on the procedures and test results are given below:

Compaction characteristics. (Test Method Tex-113-E)(41). Optimum moisture contents and maximum unit weights for a compactive effort of 13.26 ft.lbs./cu.in. were obtained for each material. In this procedure 6-inch diameter by 8-inch high specimens are compacted in four equal layers. Since maximum aggregate size of two inches is allowed in the



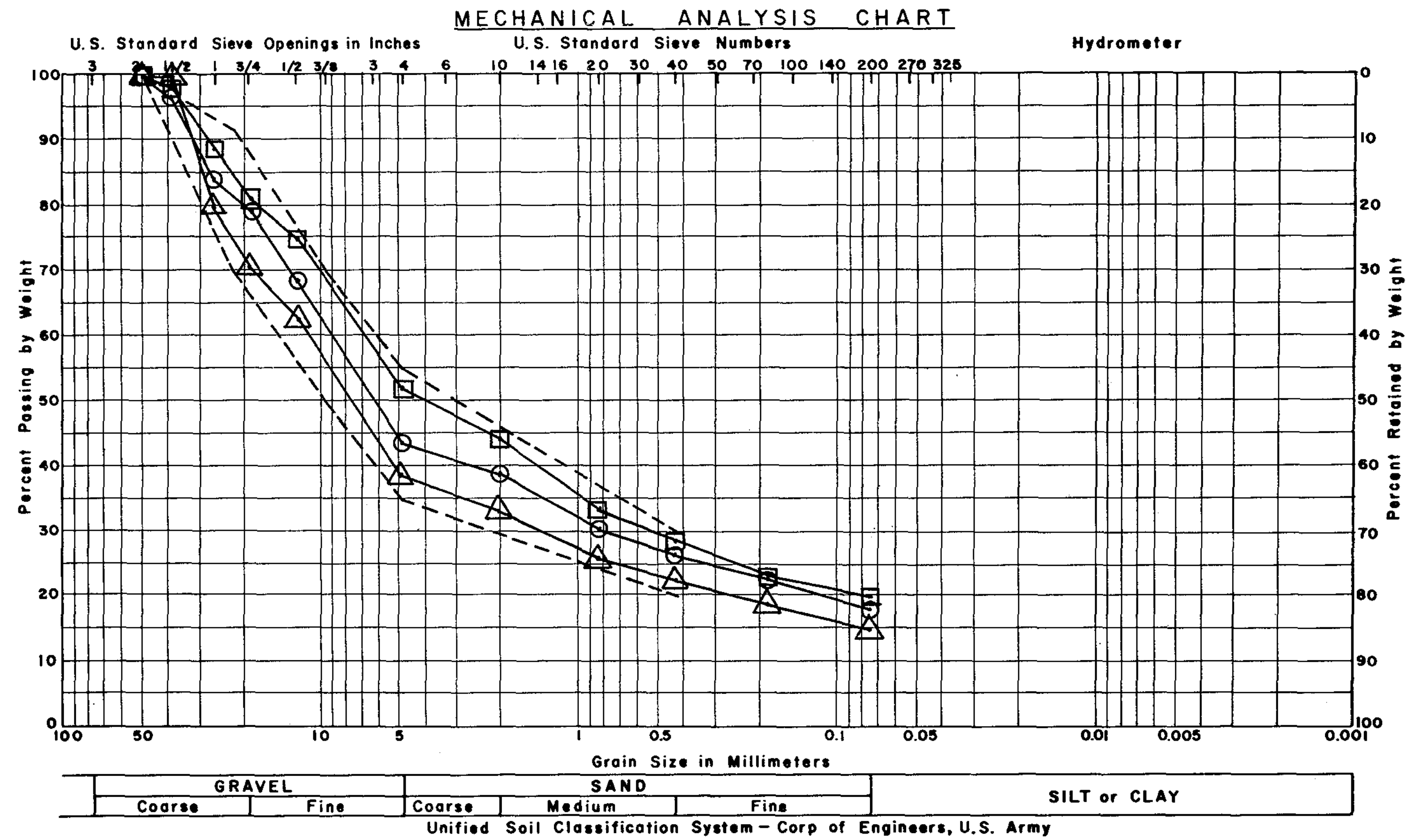
____ ______ ____ Specification Limits, Type A, Grade 1, Flexible Base Materials (42)

FIGURE 3.5 Gradations for unstressed specimens of <u>rounded</u> material.



_____ _____ _____ Specification Limits, Type A, Grade 1, Flexible Base Materials (42)

FIGURE 3.6 Gradations for unstressed specimens of <u>angular</u> material.



– – – Specification Limits, Type A, Grade 1, Flexible Base Materials (42)

FIGURE 3.7. Gradations for unstressed specimens of soft materials.

TABLE III.1

Summary of Characteristics of Research Materials

Designation	Description	Gradation	Compaction Characteristics[a]		Actual Unit Weight, D_A 100% Compaction Ratio p.c.f.	Texas Triaxial Test				Plasticity			Los Angeles Abrasion ("A" Grading)		Texas Wet Ball Mill	Classification		Specific Gravity	Permeability ft/day
			Optimum Moisture %	Maximum Unit Weight p.c.f.		Average Moisture After Capillarity %	Failure Stress At Indicated Lateral Pressure, psi: 0 psi	Failure Stress At Indicated Lateral Pressure, psi: 15 psi	Triaxial Class	Liquid Limit	Plasticity Index	Linear Shrinkage	100 rev.	500 rev.		Texas	Unified		
		Fine	7.3	133.9	-----	7.4	18.1	147.7	3.0						37.2	Type B, Grade 3	GMd	2.64	-----
HP-27-8	Rounded	Medium	6.8	135.4	133.8	7.0	23.8	158.7	2.8	21.3	7.4	5.6	7.2	27.3	36.2	Type B, Grade 3	GMd	2.63	0.006
		Coarse	6.7	135.2	-----	7.3	23.2	161.9	2.7						32.0	Type B, Grade 3	GMd	2.65	-----
		Fine	7.3	133.9	-----	7.0	42.1	223.2	1[b]						39.0	Type A, Grade 2	GMu	2.64	-----
HP-27-9	Angular	Medium	7.0	136.0	136.0	6.8	62.1	246.9	1[c]	17.8	2.3	2.4	6.8	25.3	36.1	Type A Grade 1	GMu	2.63	0.003
		Coarse	6.8	137.7	-----	5.9	57.7	270.7	1[d]						33.5	Type A, Grade 1	GMu	2.64	-----
		Fine	11.9	124.2	-----	11.5	28.8	169.5	2.5						50.3	Type A, Grade 2	GMu	2.67	-----
HP-27-10	Soft	Medium	11.9	124.2	124.3	11.6	52.0	167.8	2.1	20.2	4.8	2.7	19.0	57.9	47.8	Type A, Grade 2	GMu	2.67	0.002
		Coarse	11.9	124.2	-----	11.3	48.2	175.4	2.1						48.1	Type A, Grade 2	GMu	2.67	-----

a. Compactive effort = 13.26 ft. lbs. per cu. in.
b. Lowest classification of HP-27-9
c. Medium classification of HP-27-9
d. Highest classification of HP-27-9

test, it was not necessary to eliminate or replace the larger aggregate sizes as is common in many other compaction procedures.

The first compaction data were highly erratic; it was difficult to precisely establish the optimum moisture content, yet this was of major concern since in the repetitive loading program it was planned to compare all materials at their respective optimum moisture contents. Previous experience with granular materials proved that even minor variations in the moisture contents greatly influenced their repetitive loading characteristics. Thus the compaction data were checked using the following precautions and techniques:

a. The separate sizes of the materials were recombined in the desired proportions to obtain enough material for a single compaction layer (two inches high) which was handled throughout as a separate layer. By experience, the layer weights for each specimen were adjusted so the finished height of the compacted specimen was 8.0 ± 0.1 inches. This procedure assured constant gradation between specimens and within each specimen; it also maintained constant compactive effort per unit volume of soil for all specimens.

b. The desired amount of water, taking into account hygroscopic moisture and evaporation during mixing and compacting, was added to each layer, it was mixed for 3 minutes in a Lancaster PC counter-current batch mixer, and then "tempered" in an air-tight can for 24 hours before compaction.

c. Only one technician performed compaction tests on any particular material. The advantage gained here, since the actual compactive effort was applied automatically with a Rainhart tamper, was in "finishing" the surface of the compacted specimen. When using large maximum size aggregates, the specimen surface must be smoothed by hand, and there is often considerable difference in the finishing techniques between various technicians.

d. The entire specimen was dried to obtain the molding moisture content. On many specimens wet of optimum -- especially in the coarse gradations -- molding water seeped from the specimens during compaction. The effective compaction moisture for these specimens probably lies between the measured final moisture, which is plotted on the graphs, and the initial moisture added to the specimens.

With the above techniques, the resulting compaction data (Figure 3.8) were better than the original attempts, but were still somewhat erratic. However, it was possible to obtain fairly reliable values of optimum moistures and maximum unit weights from them.

Gradation influenced the compaction characteristics of the rounded and angular materials; finer gradations increased the optimum moisture contents and reduced the maximum dry unit weights. No perceptible difference was observed in the maximum values for the three gradations of the soft material. Sieve analyses on several specimens compacted at or near the optimum moisture contents are presented in Table III.2. They show that the compaction process produced minor gradation changes in the rounded and angular materials, but in the soft material it produced considerable particle breakdown throughout the entire gradation range. This degradation must have masked the influence of the original variation in gradation.

The compaction characteristics for the medium gradings were also obtained at 25 and 125 blows per layer (or 6.67 and 33.16 ft.lbs./cu.in. respectively). These "families" of compaction curves are shown in Figure 3.9. It is generally accepted that plots of maximum dry unit weight versus logarithm of compactive effort will produce straight lines, at least for cohesive soils. Plots of this nature, shown in Figure 3.10, show that this is apparently not true for the granular materials used in this research. In particular, the change in unit weight due to increasing the compactive effort from 50 to 125 blows per layer was minor, especially for the soft material. Again this is explainable by the amount of degradation occurring under compaction, as shown in Table III.3.

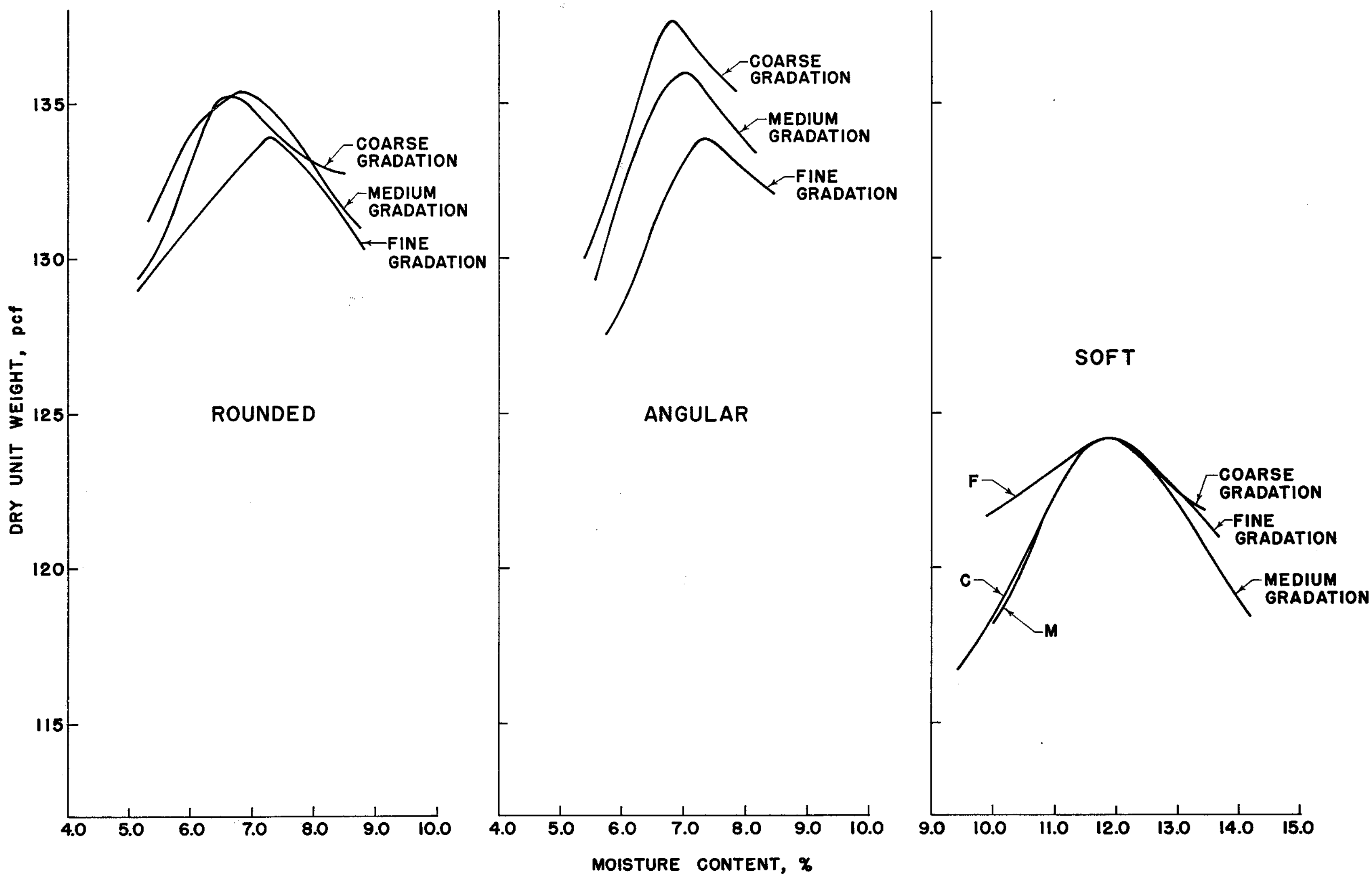


FIGURE 3.8 Compaction curves for research materials; compactive effort = 13.26 ft.lbs./cu.in. (50 blows/layer).

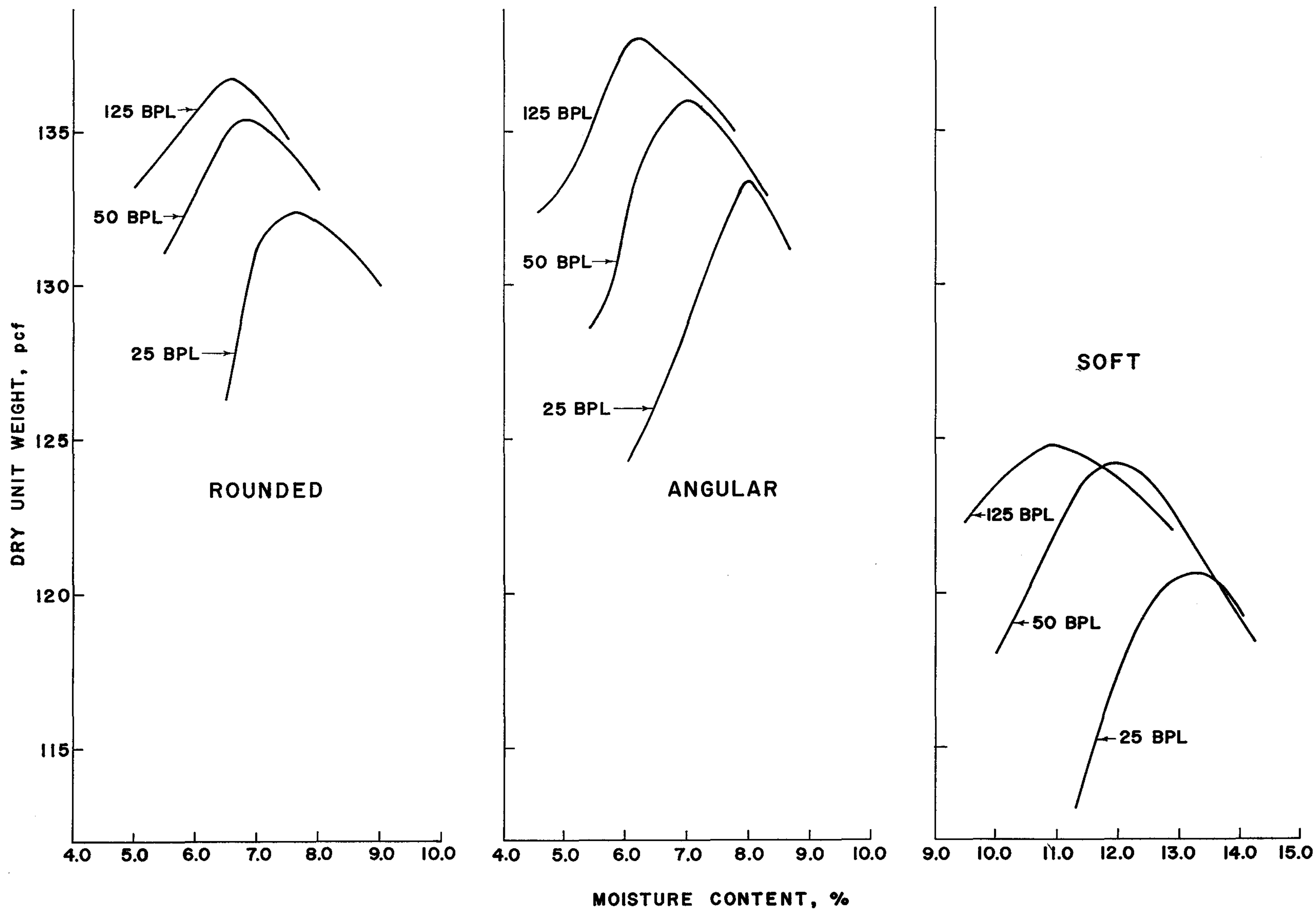


FIGURE 3.9 Family of compaction curves for medium gradations; compactive effort is shown in curves.

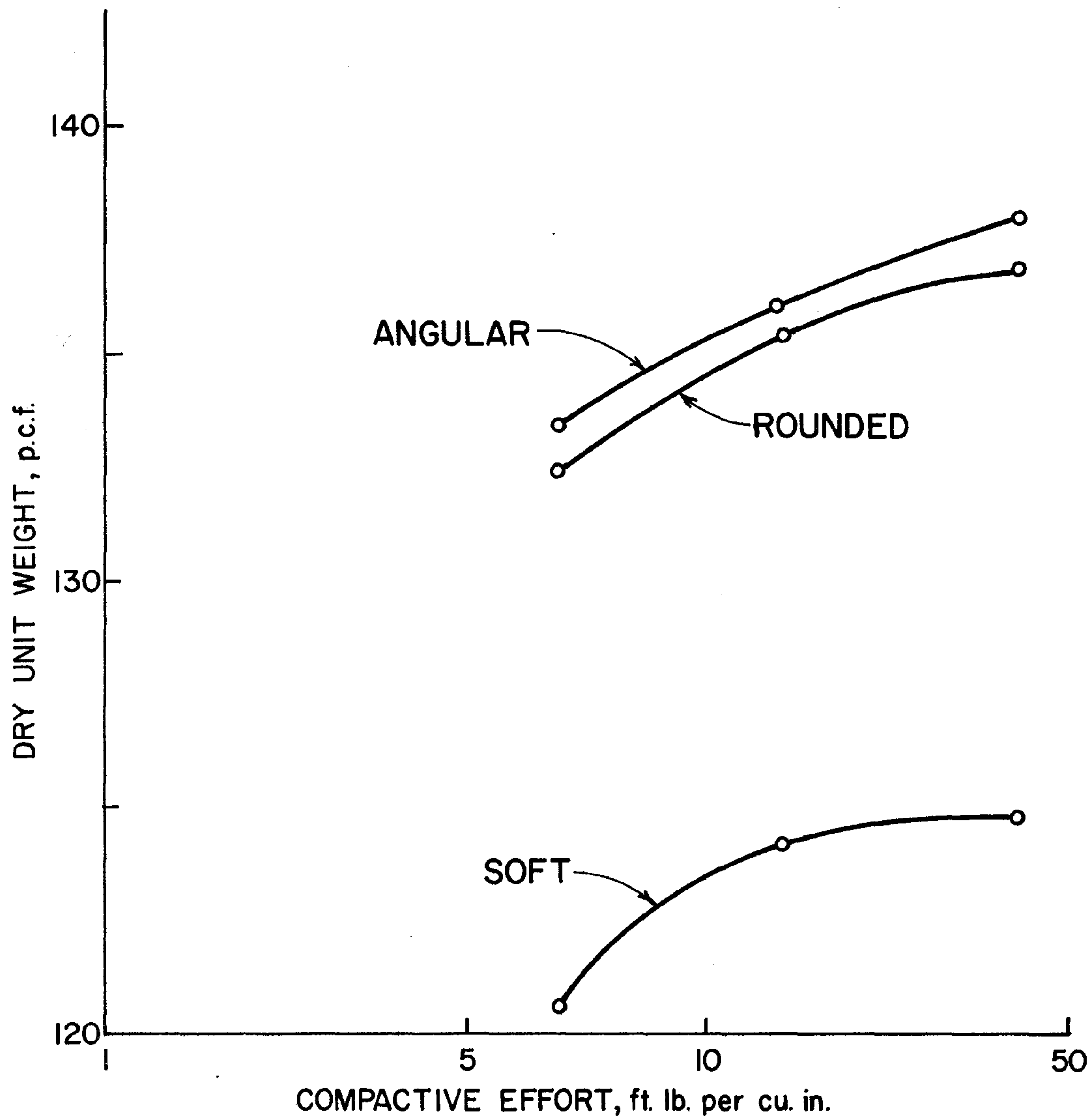


FIGURE 3.10 Relationship between maximum dry unit weight and compactive effort for medium gradations.

TABLE III.2

Summary of Gradation Tests on Specimens
Compacted at 50 Blows/Layer

Sieve	Coarse		Medium		Fine	
	Uncompacted % passing	Compacted % deviation	Uncompacted % passing	Compacted % deviation	Uncompacted % passing	Compacted % deviation
			Rounded Material			
1-3/4	100.0	0.0	100.0	0.0	100.0	0.0
1-1/2	97.6	-2.9	96.0	4.5	100.0	-4.0
1	79.9	0.3	84.4	2.5	91.2	0.9
3/4	69.5	2.5	78.3	0.1	82.2	4.5
1/2	62.7	0.0	68.1	1.0	75.8	1.6
#4	38.4	1.4	44.8	1.0	52.0	1.3
#10	33.6	1.5	38.2	0.9	43.3	1.4
#20	27.1	2.4	31.7	1.2	34.8	2.0
#40	23.9	1.2	26.7	1.4	28.9	2.2
#80	17.2	3.1	22.5	1.0	24.6	1.8
#200	14.4	2.7	21.4	0.0	23.5	1.8
			Angular Material			
1-3/4	100.0	0.0	100.0	0.0	100.0	0.0
1-1/2	97.4	1.6	96.9	3.1	100.0	-2.3
1	80.0	0.7	84.5	-1.4	89.1	0.2
3/4	72.3	0.2	75.2	2.0	80.7	1.6
1/2	62.1	2.4	68.6	1.0	73.6	3.0
#4	38.7	1.4	46.2	0.4	52.3	2.9
#10	32.9	1.4	39.1	0.5	43.7	2.9
#20	27.4	1.2	30.6	1.2	34.2	3.5
#40	24.2	0.7	26.6	1.4	29.6	3.7
#80	18.4	1.6	20.5	1.0	24.1	3.5
#100	13.4	3.0	17.9	0.8	20.2	3.6
			Soft Material			
1-3/4	100.0	0.0	100 0	0.0	100.0	0.0
1-1/2	100.0	-2.1	96 6	0.1	97.5	-0.6
1	80.2	6.4	84 3	3.4	88.5	3.3
3/4	71.1	8.6	79 1	2.4	81.0	3.7
1/2	62.9	10.5	68 2	4.5	74.8	2.8
#4	38.6	8.3	43 9	6.2	51.8	6.0
#10	33.5	6.4	38 4	4.6	44.3	5.5
#20	25.9	6.6	30 7	4.3	33.3	7.9
#40	22.4	6.1	26 6	4.2	28.0	8.2
#80	18.7	5.1	22 4	2.0	22.9	8.4
#100	14.8	4.7	17 8	3.5	17.8	12.1

TABLE III.3

Summary of Gradation Changes Due to
Change In Compactive Effort - Medium Gradation

Material	Sieve	Original Per-cent Passing	25 Blows/Layer	50 Blows/Layer	125 Blows/Layer
	1-3/4"	100.0	0.0	0.0	0.0
	1-1/2"	96.0	1.4	4.5	1.4
	1"	84.4	0.0	2.5	2.3
	3/4"	78.3	0.5	0.1	1.5
	1/2"	68.1	1.8	1.0	2.9
Rounded	# 4	44.8	0.7	1.0	3.1
	#10	38.2	0.3	0.9	2.1
	#20	31.7	1.5	1.2	6.3
	#40	26.7	1.7	1.4	5.9
	#80	22.5	-0.5	1.0	1.7
	#200	21.4	-2.3	0.0	0.8
	1-3/4"	100.0	0.0	0.0	0.0
	1-1/2"	96.9	3.1	3.1	-4.2
	1"	84.5	1.6	-1.4	-1.7
	3/4"	75.2	4.2	2.0	2.2
	1/2"	68.6	2.6	1.0	1.5
Angular	# 4	46.2	0.5	0.4	1.7
	#10	39.1	1.8	0.5	2.1
	#20	30.6	3.4	1.2	4.0
	#40	26.6	3.8	1.4	4.8
	#80	20.5	3.5	1.0	4.2
	#200	17.2	1.9	0.8	2.7
	1-3/4"	100.0	0.0	0.0	0.0
	1-1/2"	96.6	-3.1	0.1	3.0
	1"	84.3	-2.3	3.4	8.3
	3/4"	79.1	-5.4	2.4	8.7
	1/2"	68.2	-1.7	4.5	13.8
Soft	# 4	43.9	1.7	4.5	15.7
	#10	38.4	0.8	4.6	11.6
	#20	30.7	3.6	4.3	11.3
	#40	26.6	4.7	4.2	10.9
	#80	22.4	5.2	2.0	10.0
	#200	17.8	4.9	3.5	8.9

A more detailed discussion of the nature of the laboratory compaction difficulties with these materials would serve little useful purpose in this report. It is probably sufficient to say that others (see references (14) and (19) for example) have reported similar difficulties on like materials. It is the author's belief that much of the compactive energy in the impact compaction method is spent in degrading the material (until it no longer bears resemblance to the original gradation) and in friction loss between the particles and side of the mold, rather than in compacting the material. Field compaction equipment produces some degradation during compaction also, but in the field there is more tendency for the particles to move into closer contact by sliding over each other instead of being vertically forced into position. The mold effect is absent also.

Under the author's direction, research was conducted using the Vicksburg gyratory compactor on a material very similar to the angular material used in this report (29). The gyratory compactor produced unit weights 9 pcf higher than those achieved in the 50 blow per layer Texas compaction method; this increase would have been impossible with impact compaction and results from the field-like kneading or shearing action of the gyratory compactor.

While the compactor techniques and precautions discussed above did not solve the compaction difficulties, they did eventually serve a useful purpose by providing the technique necessary later in the project to produce numerous repetitive triaxial specimens within a small tolerance of unit weights and moisture contents.

Compaction ratio. (Test Method Tex-114-E)(41). For so-called "density control" projects, the Texas Highway Department uses the Compaction Ratio Method to determine the desirable degree of denseness to which soils or base materials should be compacted in the roadway. This test requires -- for flexible base materials -- the determination of: a) the loose density (D_L) which is the unit weight of a representative specimen rodded in three equal layers in a one-half cubic foot measure, and b) the dense density (D_D) which is the maximum dry unit weight at a compactive effort of 30 ft.lbs./cu.in.

D_L was obtained for the medium gradation of each material according to the prescribed procedure; D_D was interpolated from Figure 3.10 at 30 ft.lbs./cu.in.

D_A, the actual dry unit weight which would be specified in the roadway, was calculated for the desired compaction ratios. The results in Table III.1 show that the dry unit weight which would be specified in the roadway is essentially the same as the maximum unit weight obtained at 13.26 ft.lbs./cu.in. compactive effort except for the rounded material which could be compacted in the field at a slightly lower unit weight.

Texas triaxial tests. (Test Method Tex-117-E)(41). Nine 6- by 8-inch specimens (only seven specimens are required in the standard procedure) at each gradation were prepared at their optimum moisture content and maximum unit weight using the same procedure detailed for the compaction tests.

The specified curing procedure was followed*, and the specimens were tested in compression at lateral pressures of 0 (2 specimens), 3, 5, 10, 15, and 20 psi. The remaining two specimens were used as replicates wherever the results appeared anomalous or where difficulty in testing occurred. Test results are portrayed in terms of Mohr's failure envelopes, Figure 3.11. The triaxial classifications and the failure stresses at 0 and 15 psi lateral pressure are shown in Table III.1.

While gradation influenced the overall shape of the failure envelopes, the effect was not so pronounced on the Texas classification chart, which uses only the initial portion of the failure envelope and classifies the material according to its lowest portion (between 3 and 25 psi normal stress) on the chart. For the rounded material, the triaxial classification progressively increased ** (from Class 3.0 to Class 2.7) as the gradation became coarser. The same trend (with a somewhat greater range in classifications) occurred for the angular material, but all gradations were in the Class 1 area. The trend was similar in the soft material with the fine gradation having the lowest classification (Class 2.5), but the medium and coarse gradations were both Class 2.1.

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*Specimens were covered with membranes, placed in the laboratory overnight, then dried to about 40 percent of their molding moisture content, allowed to cool overnight, and subjected to 10 days of capillary moisture absorption.

**In the terminology used by the Texas Highway Department, "high" classification means a strong material, or a low classification number. For example, Class 1 is "higher" or stronger than Class 5.

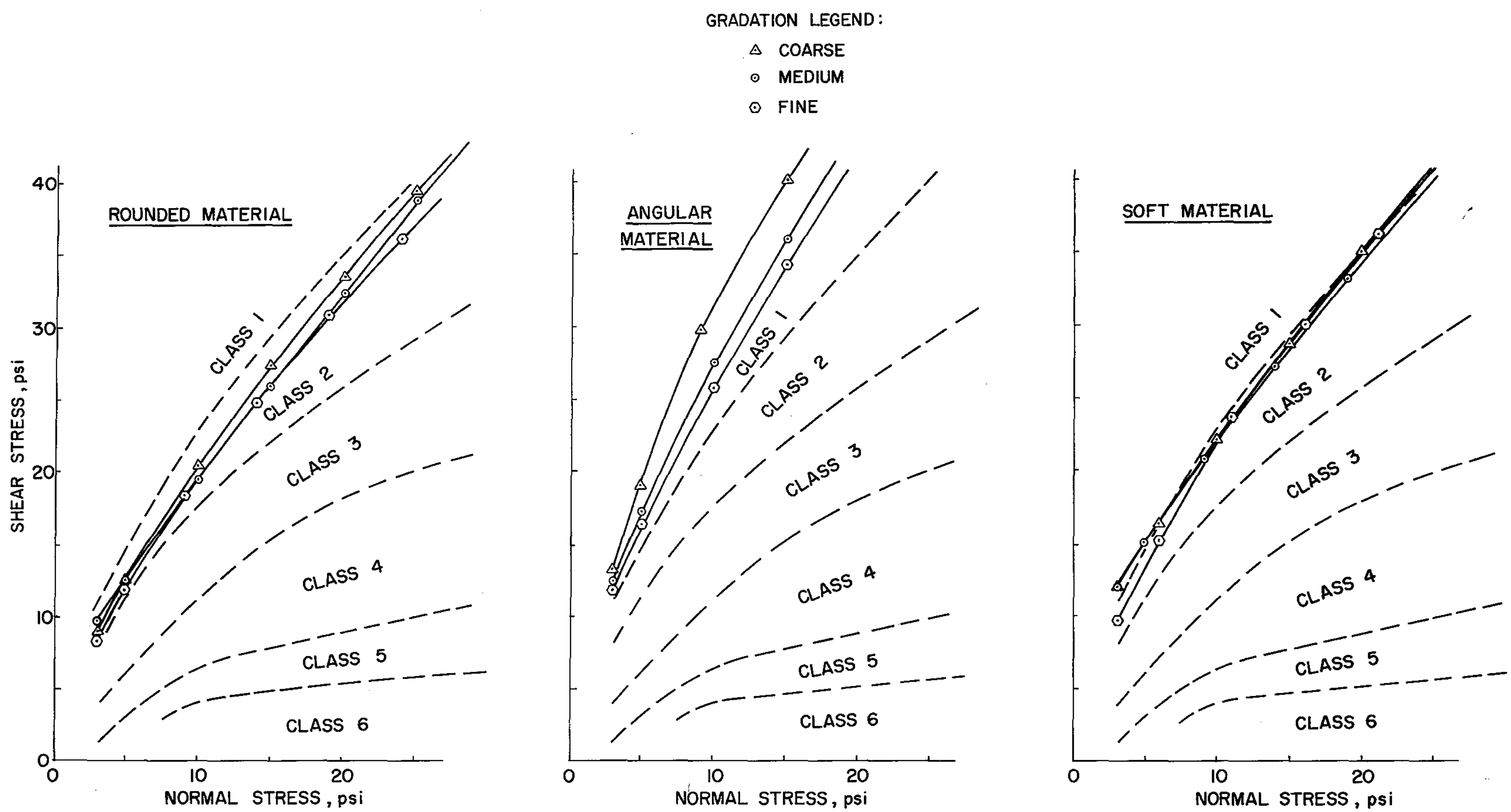


FIGURE 3.11 Mohr failure envelopes for research materials plotted on Texas classification charts; compactive effort = 13.26 ft.lbs./cu.in. (50 blows/layer).

In all materials, the highest unconfined compressive strength was obtained with the medium gradations although it would seem that this should occur in the finest gradations due to the cohesive action of the fines. This may be attributable to the high moisture contents in the fine gradations at the time of testing (see next paragraph). On the other hand, the coarse gradations always had the highest strengths at 15 psi lateral pressure. This is not surprising since the reduction of fines in granular materials generally increases their angles of internal friction.

The comparison of molded moisture content to the moisture content after capillarity (which is also the moisture content at the time of testing) is interesting. The capillary soaking process is intended to bring the materials to roughly the moisture content they would attain during critical climatic conditions during their life in the roadway. The moisture contents after capillarity for the angular and soft materials were <u>less</u> than the molded moisture contents, and the difference generally increased as the gradation became coarser. For the rounded material, the moisture content after capillary soaking was <u>greater</u> than the molded moisture content, and the difference increased as the gradation became coarser. It might be expected that these trends would be a reflection of the plasticity characteristics of the fines, and this certainly seems to be the case; the material of highest plasticity (rounded) gained water during capillary soaking, while the material of lowest plasticity (angular) lost water.

It should be mentioned that no specimens exhibited measureable swell at the termination of the curing period.

Los Angeles Abrasion. (ASTM Designation C 131-55) (3). This test was performed on the three materials using the "A" grading. Abrasion values were determined following 100 revolutions and after the standard 500 revolutions. Comparison of these two values gives a rough idea of the uniformity of hardness within the materials. For example, with the rounded and angular materials, the value at 100 revolutions was roughly one-fourth of the 500 revolution value, indicating relative uniform particle hardness for both materials. That the same parent material was used is reflected in the closeness of the abrasion values. The abrasion value of the soft material at 100 revolutions was roughly one-third of the 500 revolution value, indicating particles of varying degrees of hardness. This is consistent with the layered characteristics of the parent quarry.

The abrasion value of the rounded and angular materials -- roughly 26 -- is indicative of hard, durable materials. The value of 58 for the soft material is well above the values accepted for near-surface granular materials.

Texas Wet Ball Mill. (Test Method Tex-116-E)(41). This test uses representatively graded specimens which were produced by recombining the separated sizes in the manner used to produce compaction specimens.

Unlike the Los Angeles abrasion value which is a measure of the attrition produced during the test, the wet ball mill test, by using a graded specimen of the material, reflects the amount of fines initially in the specimen plus those produced by attrition during the test. It is claimed that this test is a better indicator of attrition under traffic since the specimens are kept wet during the test, and the fines function by "cushioning" the abrasive effect as they would in the roadway.

The test results are consistent with the initial gradings for the rounded and angular materials, i.e., the values decrease as the original gradation becomes coarser. With the soft material, the same trend exists although the medium gradation appears somewhat out of line.

It was noted that the range of wet ball mill values was much less than the range of Los Angeles abrasion values.

Plasticity characteristics. The Atterberg limits were obtained using ASTM Designation D 423-54T and D 424-54T (3). The minus 40 mesh material was obtained by wet-sieving recombined specimens of the rounded material; dry-sieving was considered adequate for the two crushed materials.

The rounded material containing the fines from the native caliche gravel had the highest plasticity (Plasticity Index = 7.4). The angular and soft materials, whose fines were derived from crushing of large particles, had low plasticity (Plasticity Index = 2.3 and 4.8 respectively).

Linear shrinkage tests were also performed using Test Method Tex-107-E (41). Based on a relationship between plasticity index and linear shrinkage developed by the Texas Highway Department (41), there is good agreement between the two properties.

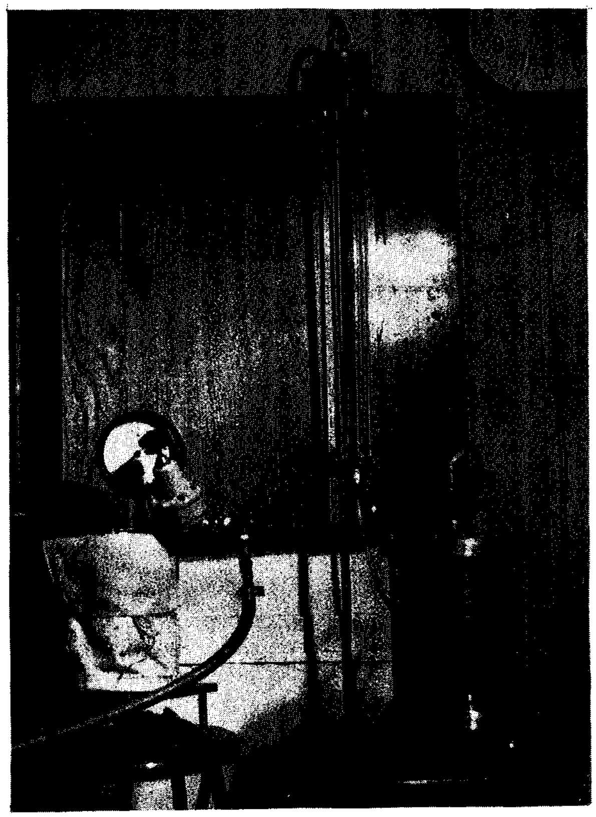
Permeability. The permeability of the three materials at their medium gradation was determined on 6-inch high specimens compacted at their respective maximum unit weights for a compactive effort of 13.26 ft. lb. per cu. in. compactive effort. A falling head permeameter was developed which utilized the compaction mold as the soil container. The apparatus is shown in Figure 3.12.

The mold, containing a compacted specimen, was bolted between the end plates, and the specimen was allowed to soak up distilled, deaired water by capillarity. Following this, vacuum was applied and deaired water was pulled through the specimen until no air bubbles were observed in the outflow tube. To further insure saturation, the permeability was obtained while a back pressure of 80-90 psi was maintained on the specimen. The inflow of water to the specimen was measured using a volume change device similar to the type reported by Bishop and Henkel (7).

The measured permeabilities were surprisingly low, ranging from 0.006 to 0.002 feet per day. This is the order of magnitude expected of silts and clays of low plasticity. As shown on Figure 3.13, these results compare favorably with values reported elsewhere for compacted granular materials having a high percentage of fines.

It should be noted that gradations <u>prior</u> to compaction are shown in Figure 3.13. The degradation of the soft material during compaction is probably one reason why it exhibits the same permeability as the angular material although the soft material has a lower unit weight.

The Texas Highway Department includes in their test procedures Test Method Tex-123-E, Method for Determination of the Drainage Factor of Test Materials. This is a measure of unsaturated flow which may



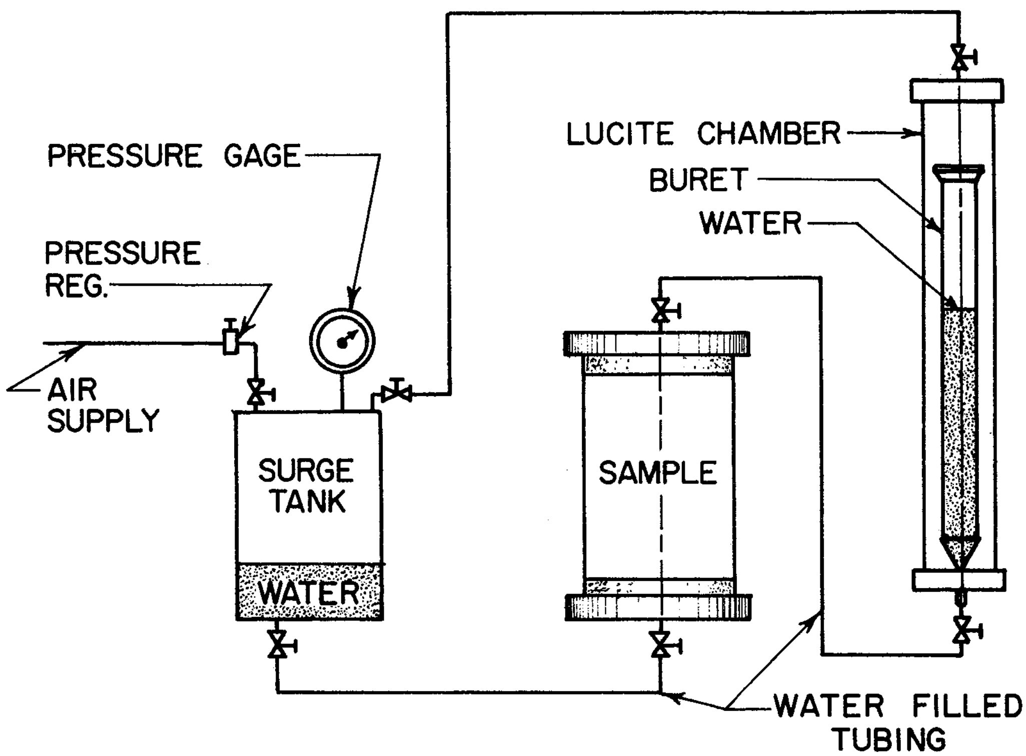
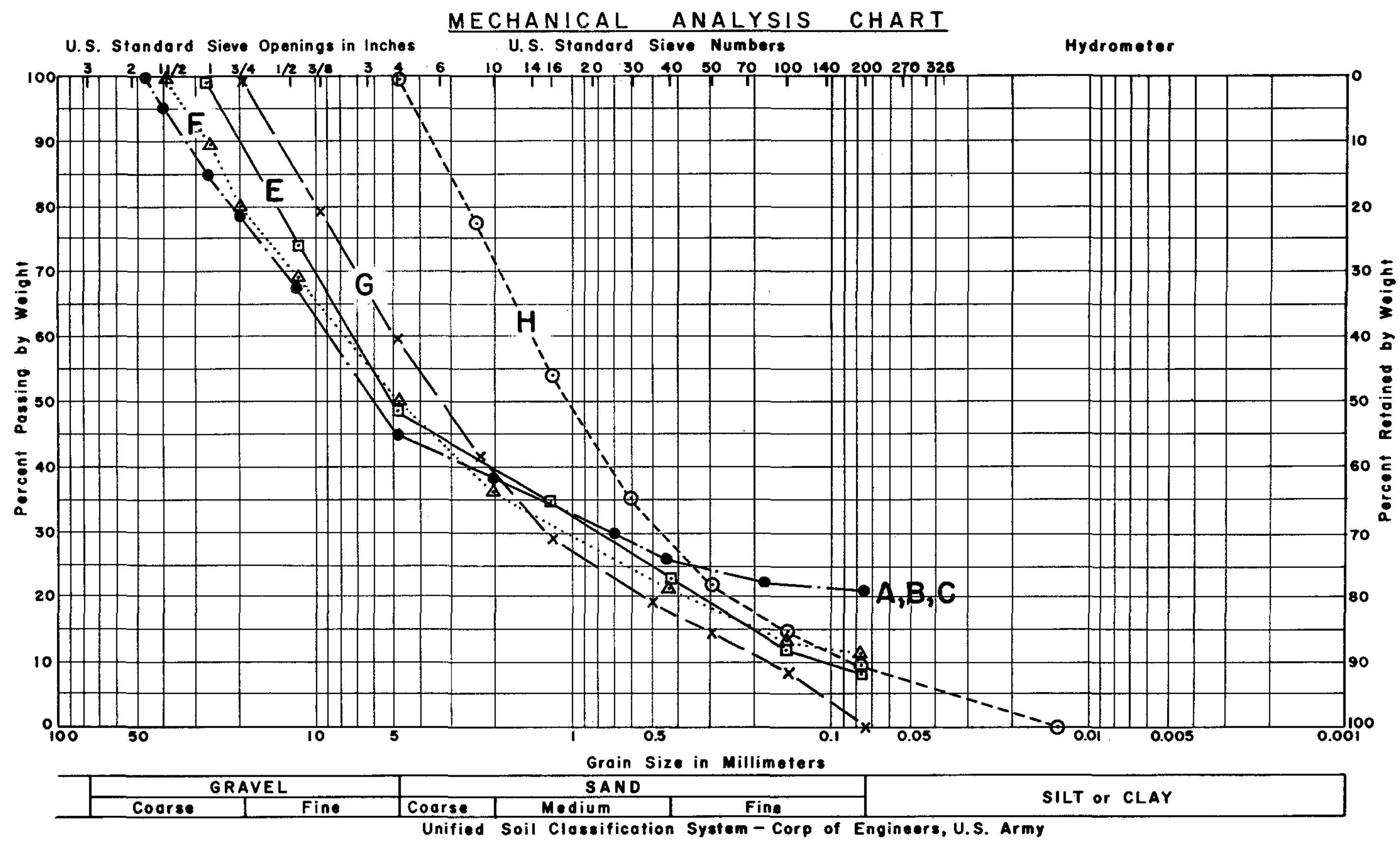


FIGURE 3.12 Picture and schematic diagram of falling head permeameter.



Legend	Description	γ_d pcf	Permeability ft/day	Ref.	Legend	Description	γ_d pcf	Permeability ft/day	Ref.
A	Angular	136.0	0.003	--	E	AASHO gravel	144.2	0.002	(21)
B	Rounded	135.2	0.006	--	F	AASHO cr.ls.	144.9	0.800	(21)
C	Soft	124.2	0.002	--	G	--	--	10.0	(28)
D	Sand & gravel	--	7.60	(11)	H	--	--	1.0	(28)

FIGURE 3.13 Permeabilities of research materials and other typical base courses.

be a better indicator of field performance than a permeability test. However, it was thought that a true value of permeability was a closer measure of basic material properties.

Specific Gravity. (Test Method Tex-201-F)(41). Although not related to material behavior, the specific gravity of the aggregates was necessary for certain calculations later in the research program. A representatively graded specimen of each gradation was tested using the method specified above except the apparent -- rather than bulk -- specific gravity was obtained.

Summary of Classification Tests

The materials selected appear to be ideally suited for the objectives of this research. There is a significant range in Texas triaxial classifications (from Class 1 to 3) and a moderately good range exists within the Class 1 materials. The materials also represent a wide range of hardness as measured by the Los Angeles abrasion test (from 25 to 58).

As might be expected, particle angularity greatly influenced the triaxial classification; for the same parent material, rounded particles were approximately Class 3 while angular particles were well into the Class 1 range. Not all of this difference can be attributed to particle shape, because the more plastic fines in the rounded material resulted in it being tested at a higher moisture content than the angular material. Perhaps a more valid comparison of the two materials would have resulted had they been produced at the same plasticity index; however, this was not the basic objective of the material selection.

The soft material, which is known to perform poorly as flexible base, was surprisingly stronger (about one-half of a class) than the rounded material which has provided good performance. In fact, it almost rates as a Class 1 material, despite its high molding and testing moisture content. Of course, after laboratory preparation neither the soft nor rounded material corresponds (in gradation or plasticity) to the parent materials actually used in the roadway, although due to the nature of the parent quarry, the soft material has in its favor the fact that it contained some harder particles when used in the roadway. It does appear that for these particular materials the Texas Triaxial classification is not severely influenced by aggregate hardness or durability.

The triaxial test was responsive to gradation; in all materials, the coarsest gradation produced the highest triaxial class.

There are factors other than triaxial class which influence material suitability. The Texas Standard Specifications (42) classify flexible base materials into grades and types based on these factors which presumably are the culmination of field observation of the behavior of many materials. In Table III.1 the classification of the materials by grade and type is shown. The rounded material classifies as Type B, Grade 3, the crushed material is Type A, Grade 1 (Grade 2 for the fine gradation), and the soft material is Type A, Grade 2.

By the Unified Soil Classification System (13), the rounded material would be classified as GMd; both angular and soft material would be classified as GMu. All materials are well-graded.

Due to the high percentage of fines and the low permeabilities, none of the materials can be classified as "free-draining", a requirement many engineers feel is important for satisfactory flexible base materials.

While conducting the classification tests, the single most important deficiency observed was the need for a suitable method of determining the compaction characterisitcs of slightly cohesive, granular materials. (It has already been mentioned that this is a universal problem). The materials cannot be treated as cohesionless materials and densified by simple vibration for they require a certain optimum amount of water for compaction. With impact compaction -- particularly under high compactive efforts -- much of the compactive energy is spent in degrading the materials, which further complicates the problem by changing one of their basic properties. For these materials, the Texas Highway Department compaction method is probably more suitable than most others; at least it uses a large compaction mold and a representatively graded specimen.

It is the author's opinion that if the true compaction characteristics of the rounded and angular materials were known, this would be another measure of their field performance. This is based on the belief that the 50 blow per layer compactive effort brings the rounded material much closer to its ultimate density (or minimum void ratio) than the same effort does for the angular material. This cannot be proved with the present laboratory compaction techniques, but it is inferred from the field observations and the gyratory compactor experiments which were briefly mentioned in this chapter.

CHAPTER IV

REPETITIVE TRIAXIAL TEST PROCEDURES

General

The basic testing equipment used in this research was developed earlier, but to accomplish the specific research objectives outlined in Chapter I, certain auxiliary equipment was added to the basic apparatus. Essential details of the equipment as now used have been previously described by the author (18). As a matter of completeness, and to promote understanding of the test results, a brief description of the apparatus will be included below. The detailed test procedure will then follow, plus a description of certain special tests.

Triaxial Loading and Testing Equipment

Loading apparatus. The repetitive loading apparatus (Figure 4.1) is basically a hydraulically-operated testing machine with the speed of loading and frequency of load applications controlled by timer-actuated solenoid valves. By exchanging gears on the timers, a wide range of loading speeds and frequencies is possible. The shape of the loading curve is adjusted with various valves in the hydraulic circuit.

Four testing stations are available on the loading apparatus; the hydraulic loading cylinders in these stations can be replaced by others of varying diameters to obtain optimum loading characteristics. The apparatus was designed for a maximum repetitive load of 14,000 pounds.

Figure 4.2 is a schematic diagram of the loading apparatus.

Triaxial cells. The triaxial cells (Figure 4.3), designed for 6-inch diameter by 12-inch high specimens, follow conventional design except for the loading piston arrangement. A one-inch diameter hardened stainless steel loading piston travels through two linear ball bushings mounted in the heads of the triaxial cells to minimize friction and to insure that the entire applied load reaches the specimen. Conventional drainage grooves are machined in the loading caps and pedestals; both are covered with porous corundum stones. Imperial-Eastman Poly-Flo tube fittings and Circle Seal 9500 series plug shut-off valves were used on the cells during this research.

Protective membranes. It was found that the thin latex membranes usually employed in triaxial testing were unsuitable for this particular application. Commercially available 6-inch diameter membranes leaked badly (nearly all contained small pinholes), and they were easily punctured by the granular materials -- even when two membranes were used. Latex membranes are also permeable to air, which could have an important effect on the outcome of long-term tests (30). A more suitable membrane material from the standpoint of permeability is butyl rubber, but thin, tubular sections suitable for triaxial membranes are not commercially available. However, the Carlisle Tire and Rubber Company, Carlisle, Pennsylvania, kindly provided several 6-inch diameter membranes of 1/32-inch thick butyl rubber. These membranes were formed from sheet butyl resulting in a longitudinal lapped seam which caused some difficulties as will be described shortly. However, they

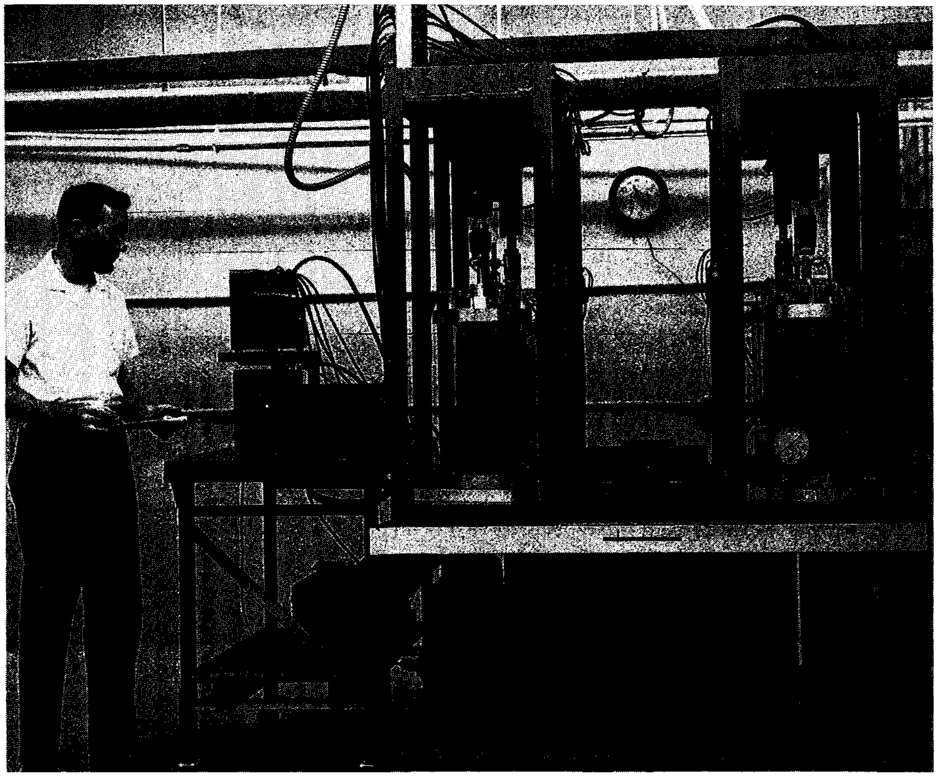


FIGURE 4.1 Repetitive loading apparatus. Triaxial cells are shown in place in the loading stations.

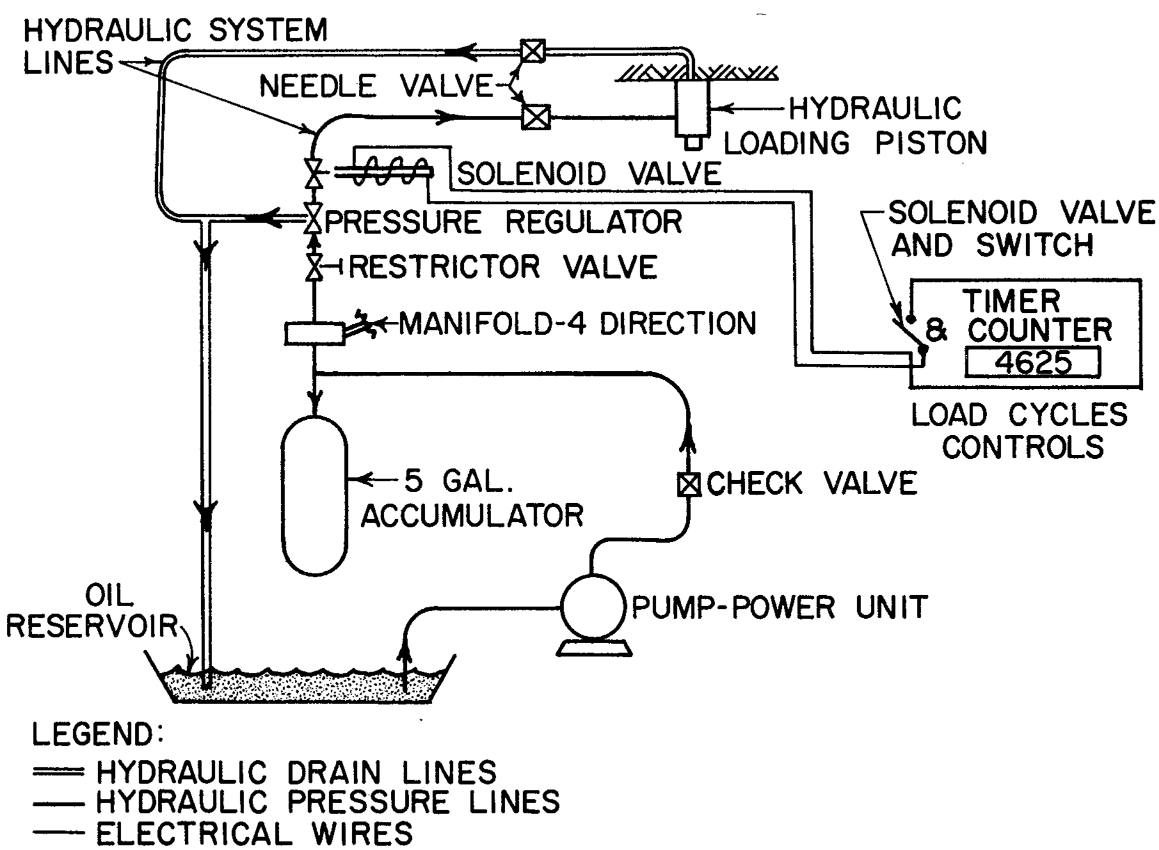


FIGURE 4.2 Schematic diagram of repetitive loading apparatus.

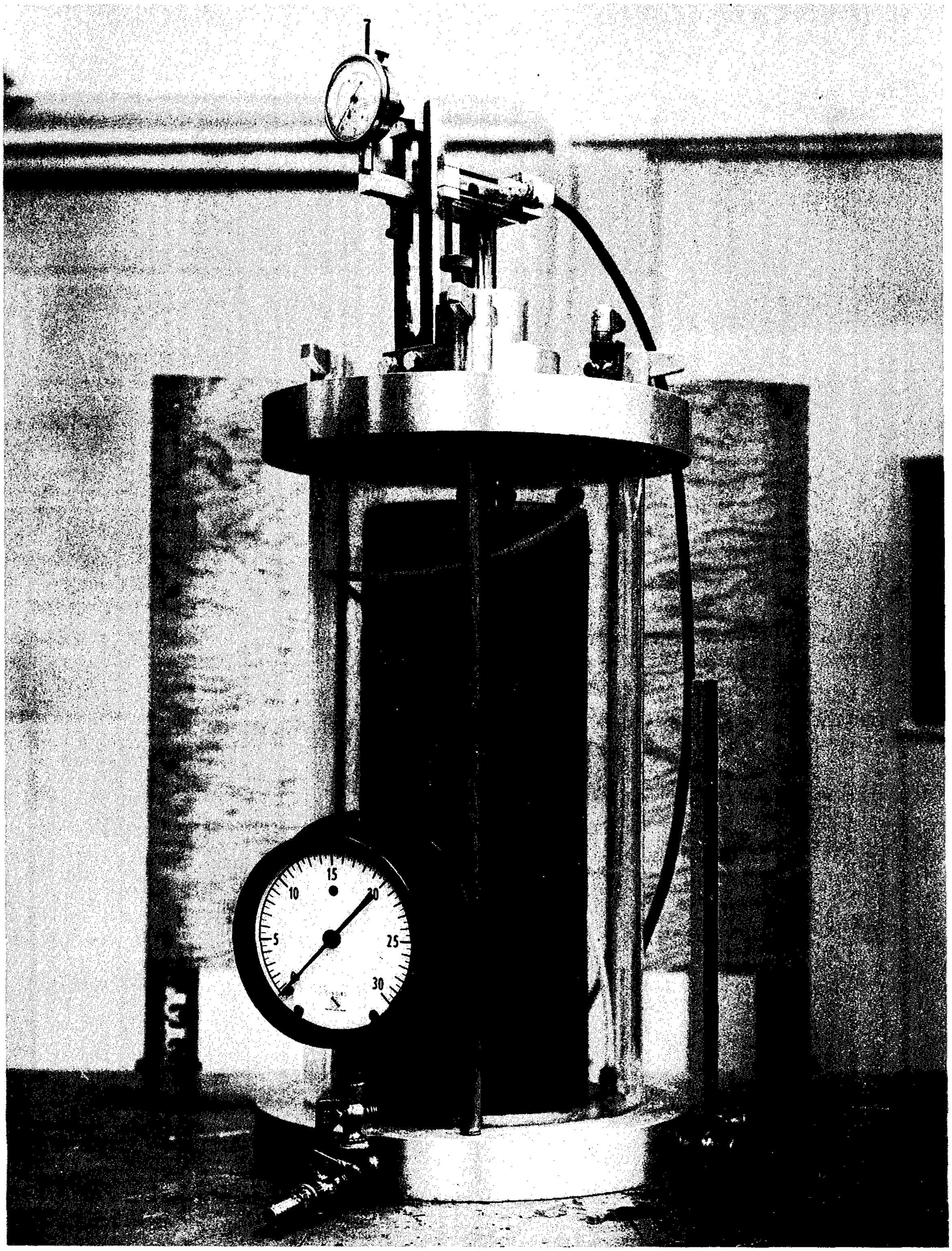


FIGURE 4.3 Triaxial cell for 6-inch diameter by 12-inch high specimens. Stopcock valves shown were replaced by plug shut-off valves for this research program.

have served the purpose admirably and seem to last almost indefinitely. Occasional punctures developed when testing the angular material, but these were easily repaired with ordinary tire patching kits. The confining effect of the relatively thick butyl membranes (the latex membranes are only one-third as thick) on the compressive strength of the specimens has been investigated (18). The butyl membranes will add 0.1 psi to the specimen compressive strength for each percent of specimen strain. This amount is insignificant compared to the stresses which were applied to the specimens.

Sealing the membranes to the triaxial pedestal and loading cap proved to be a problem because of the lapped seams and because the end diameters of the membranes increased slightly with usage. Many techniques were tried, but the most successful consisted of sealing the membrane to the pedestal and cap with silicone rubber (Dow Corning Silastic RTV) and then stretching a 1-inch wide strip of rubber innertube over the seal in place of the rubber O-rings commonly used for this purpose.

Special tests were performed to determine the permeability and leakage of the entire confining system including membrane, membrane contacts, fittings, and valves. The method used and the results obtained are presented in Appendix A. The butyl membranes were found to be permeable to the air dissolved in the confining fluid, but the permeability was slight and its effect could be easily corrected.

Measuring System

For this program, it was desired to measure or control:

A. The magnitude of the applied vertical load.

B. The complete vertical deformation pattern of the specimen at certain specific applications.

C. The total, permanent, and rebound deformations of the specimen during its entire life.

D. The pore-air and -water pressures developed during and between load applications.

To measure the vertical load, force transducers were constructed which attach directly to the piston of the hydraulic loading cylinders and bear on the loading piston of the triaxial cell. These are hollow aluminum cylinders instrumented with four temperature-compensated strain gages in a full bridge arrangement.

To obtain the deformation pattern for any particular load application, transducers were developed which consisted of two beryllium copper cantilever blades with temperature-compensated strain gages mounted on both sides of the two blades to form a full bridge arrangement. One end of the deformation transducers was attached to the loading piston and the other rested on an adjustable platform on the head of the triaxial cell.

The geometries of the force and deformation transducers were selected because of their high natural frequency of vibration, an important requirement for dynamic measurements.

The accumulative deformations were measured with dial gages of 0.001 inch sensitivity which were attached to a yoke on the head of the triaxial cell and rested on an extension clamped to the loading piston. It was surprisingly easy to obtain the maximum and minimum dial gage readings during each load application, and they provided a good check against the deformation transducer readings.

Figure 4.4 illustrates the method of affixing the deformation transducer and dial gage to the triaxial cell assembly. With this method, movements of the entire cell do not influence the transducer or dial gage readings, although deformations within the cell proper, such as strain produced in the loading piston, are reflected in the measurements.

The output from the deflection and load transducers was displayed on a Model 1508 Honeywell Visicorder; two Model 82-6 Honeywell Bridge Balance units were used to balance the transducers and to adjust the bridge voltage, supplied by either a regulated power supply or dry cell batteries. The recordings from the Visicorder provided permanent records of specimen load and deformation pattern. A typical oscillograph is shown in Figure 4.5. Later in the program, a Model 130C Hewlett-Packard oscilloscope was added in the circuit. The oscilloscope shows instantaneous load or deformation patterns, or, since it is an X-Y type oscilloscope, it will show instantaneous load-deformation curves.

Transducers were calibrated frequently under static conditions to maintain a high degree of accuracy. Usually, the calibration curves were slightly non-linear, primarily due to the characteristics of the recorder.

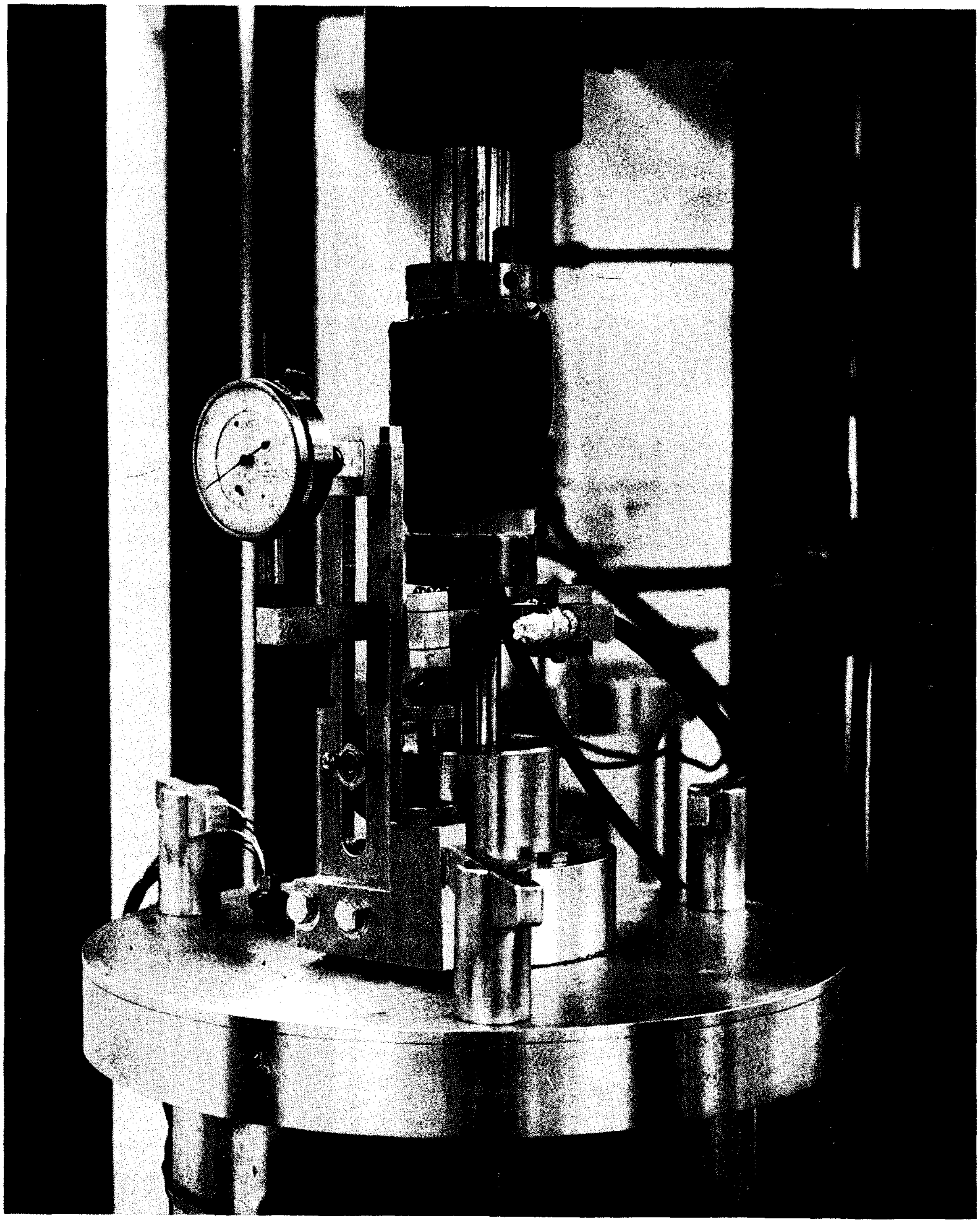


FIGURE 4.4 Close-up of head of triaxial cell showing mounting arrangement of dial gage force and deformation transducers. Note the safety switch on the yoke which can be set to turn off the repetitive loading apparatus at any desired sample deformation.

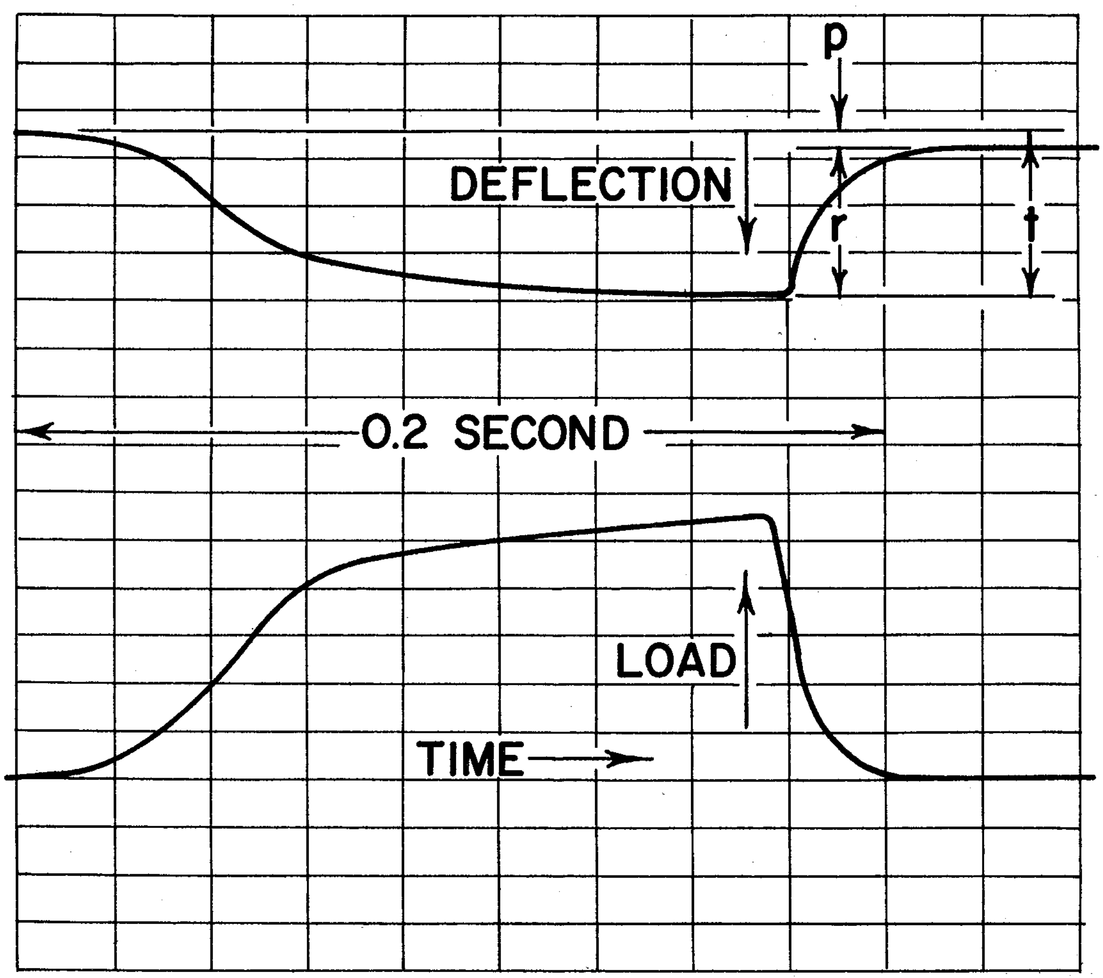


FIGURE 4.5 A typical load-deflection oscillograph.

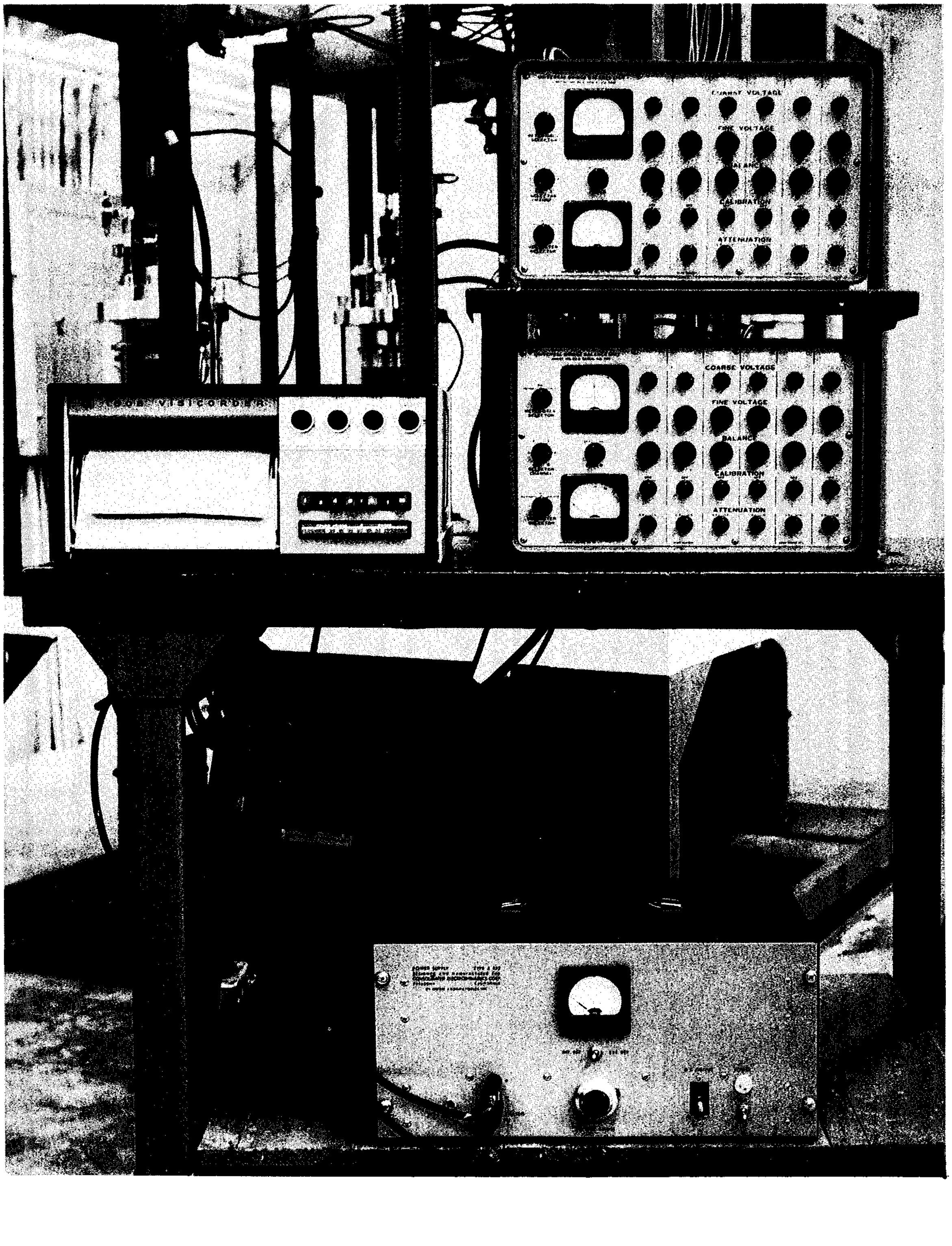


FIGURE 4.6 Instrumentation used to obtain recordings from load and deformation transducers.

Pore pressure measurements could provide valuable insight to the behavior of the repetitively loaded specimens, but since the specimens were partly saturated, both pore air and pore water pressure would have to be measured (or controlled). This has recently been accomplished for specimens subjected to slow (static) rates of loading (8), but it is such a difficult technical problem for dynamically loaded specimens that -- to the author's knowledge -- it has never been attempted. In lieu of pore pressure measurements, the air and water volume changes in the specimen interior were measured with the device shown in Figure 4.7. This apparatus is attached to the drainage ports of the triaxial cell such that the water expelled from the specimen is retained and measured in the water trap. The 100 ml. burette and plastic tubing form a water manometer; by raising or lowering the plastic tubing to keep the water level at the same height as in the burette, the volume of air expelled from the specimen, plus the air permeating through the butyl membrane, can be measured in the burette. The measured volume changes can be used to calculate changes in unit weight and degree of saturation as the test progresses. The water measurements also provide an integral check on leakage in the system: the water expelled plus the moisture in the specimen at the end of the repetitive loading should equal the original moisture in the specimen. Observation of the water level fluctuations in the plastic tubing during each loading cycle also provided a qualitative impression of the rapidly induced pore pressure in the specimen.

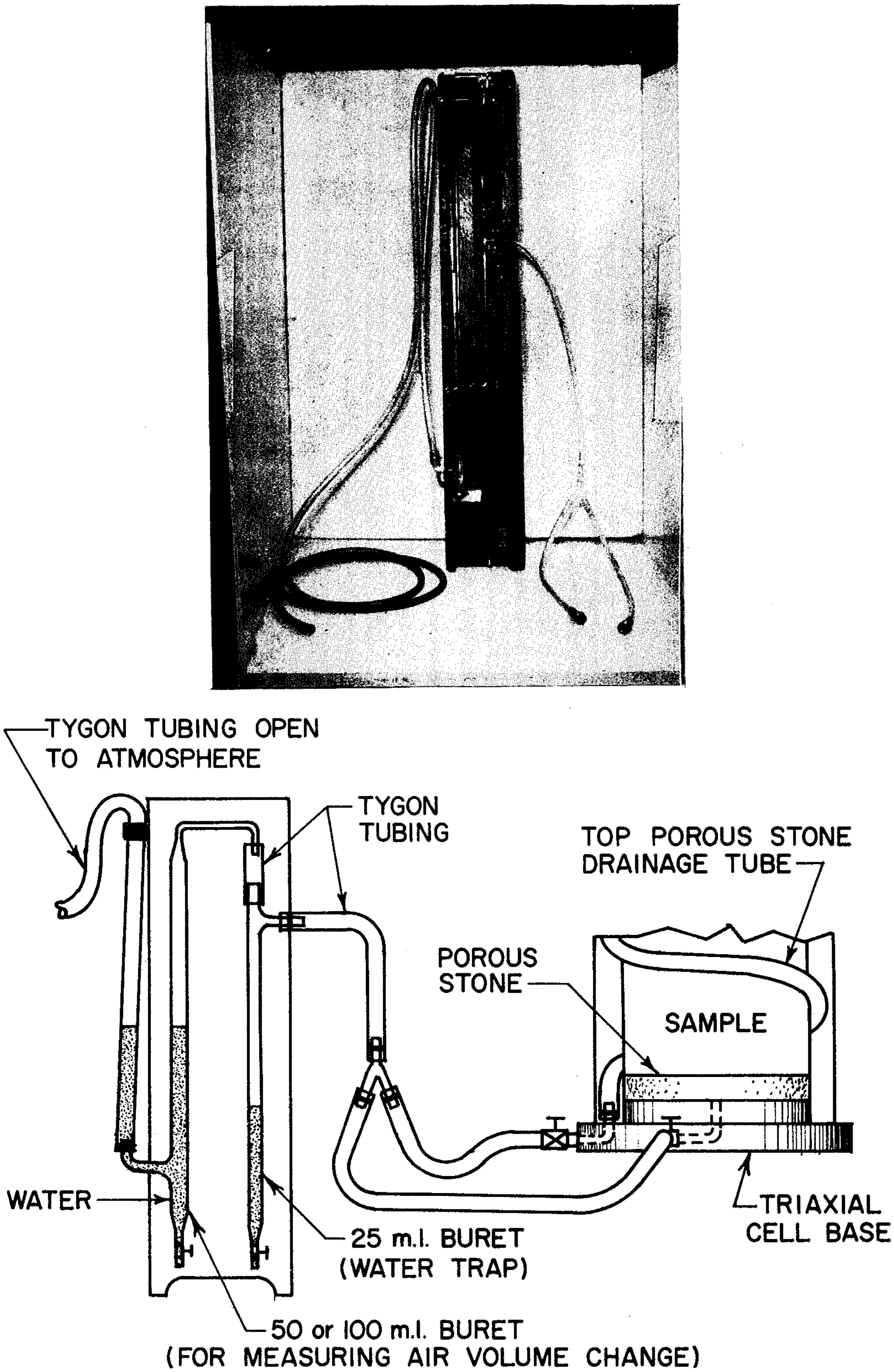


FIGURE 4.7 Picture and schematic diagram of specimen volume change apparatus.

Experiment Design

For this type of research, a well developed experiment design is necessary, not only for selecting an orderly arrangement of applied stresses but also for ease in analysing the results. In similar research conducted by the author or under his supervision (20), repetitive specimens were subjected to stresses as shown in the factorial experiment below:

σ_3, psi \ R	.25	.50	.75
3	X	X	X
11.5	X	X	X
20	X	X	X

Note: Circles denote replicate specimens.

The ultimate strengths, R, were those obtained from Texas triaxial tests. But the shearing resistance under rapid repetitive loads was so high (compared to the Texas triaxial shear resistance) that even those specimens subjected to the highest stresses lasted very long. Analysis of this factorial proved troublesome, also. It appeared that a more desirable experiment was one in which some (highly stressed) specimens failed in a relatively short time, but where even the least stressed specimens failed in a reasonable length of time, say 7-10 days.

A new factorial experiment was developed which is described below (see Figures 4.8 through 4.10):

A. From the Texas triaxial test results, vertical stresses at failure, σ_1, were plotted versus the confining pressures, σ_3. It has been determined from previous research that a curve can be statistically fitted to these points with a high correlation coefficient (31). This could be termed a "failure envelope" although not in the Mohr-Coulomb concept.

B. Three straight lines were fitted beneath the failure envelope; these lines have the general equation:

$\sigma_1 = X(A + B\sigma_3)$

where: $X = 1.5, 2.25$, and 3.0

A & B = constants for each aggregate

To determine A and B, the uppermost line ($X = 3.0$) was arbitrarily drawn near and roughly parallel to the failure envelope; A equals one-third of the intercept of this line on the σ_1 - axis, while B is one-third of the slope of the line.

Once the general equations of the lines were determined, they were solved to find the repeated stresses for three values of X and σ_3. Confining pressures of 3.0, 11.5, and 20.0 psi were retained for this program. The resulting stresses are shown in Table IV.1 for each material and gradation. To aid the reader in visualizing these stresses, they are shown graphically in the form of Mohr's circles in Figures 4.11, 4.12, and 4.13.

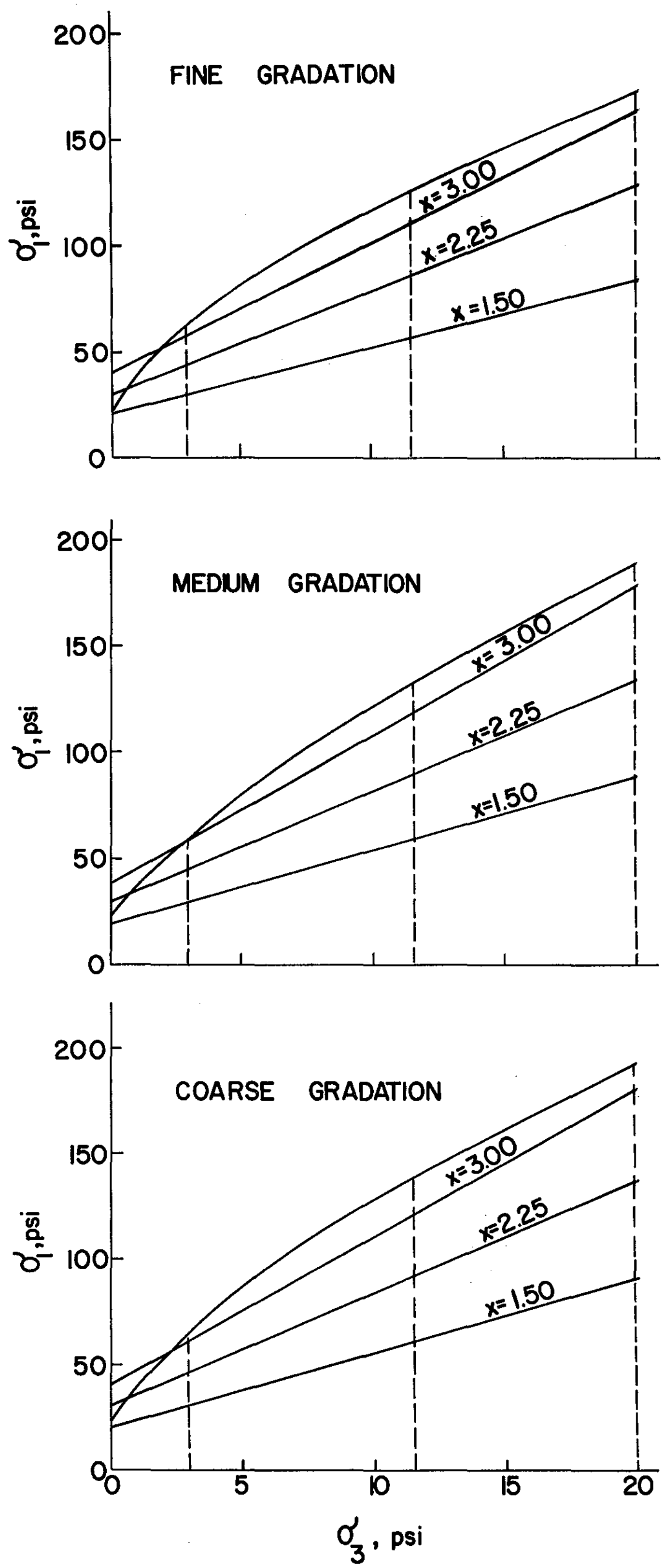


FIGURE 4.8 Determination of applied repetitive stresses (σ_1) for rounded material.

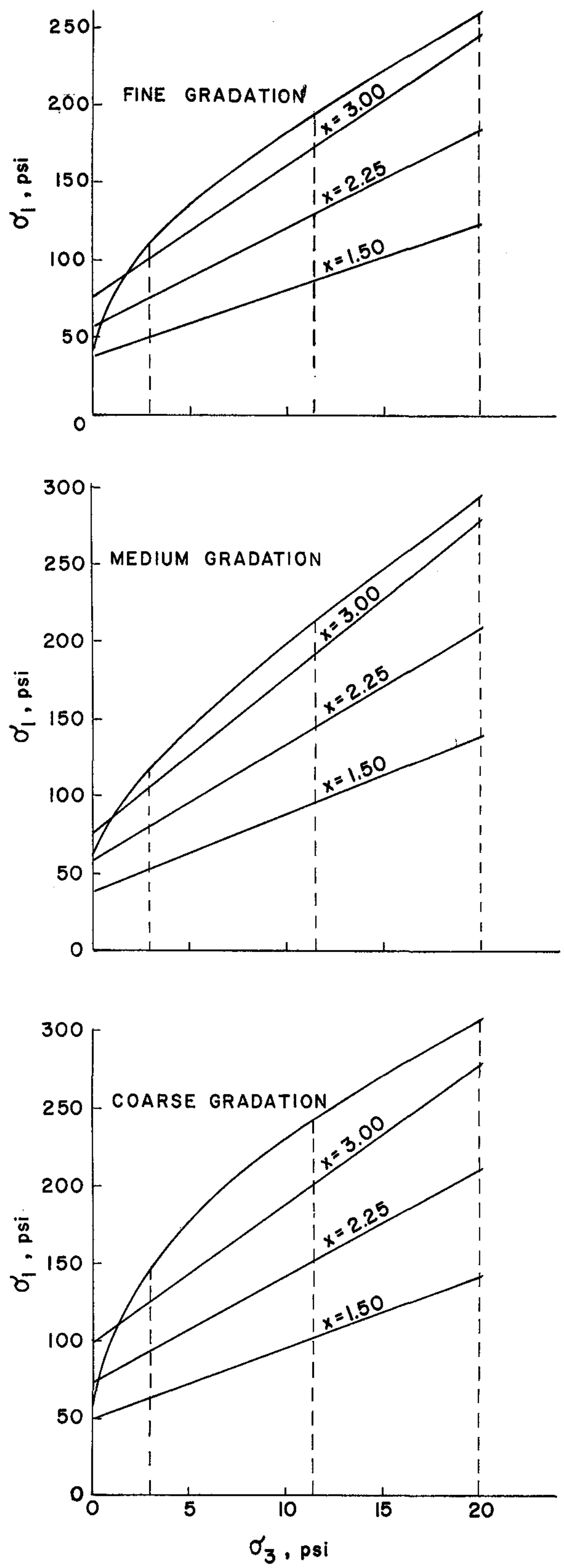


FIGURE 4.9 Determination of applied repetitive stresses (σ_1) for angular material.

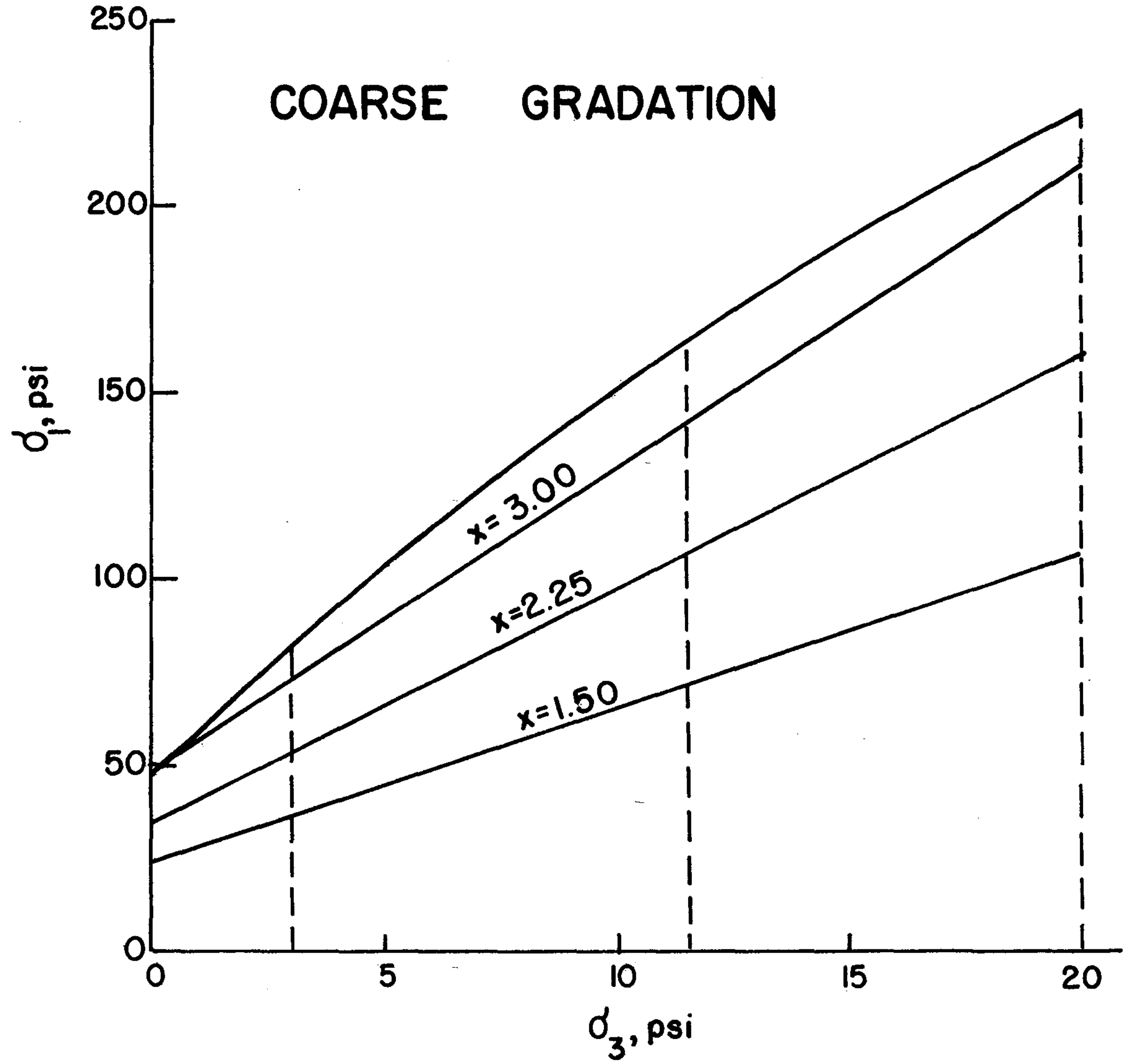


FIGURE 4.10 Determination of applied repetitive stresses (σ_1) for soft material.

TABLE IV.1

Stresses for Repetitive Triaxial Specimens

Rounded Aggregate

Coarse

σ_3, psi \ X	1.50	2.25	3.00
3.0	30.75	46.13	61.50
11.5	61.30	91.84	122.46
20	91.69	137.54	183.39

Medium

σ_3, psi \ X	1.50	2.25	3.00
3.0	28.89	43.34	57.78
11.5	59.75	89.61	119.49
20	90.60	135.90	181.20

Fine

σ_3, psi \ X	1.50	2.25	3.00
3.0	29.85	44.78	59.70
11.5	57.78	86.67	115.56
20	85.70	128.54	171.39

Angular Aggregate

Coarse

σ_3, psi \ X	1.50	2.25	3.00
3.0	63.20	94.70	126.30
11.5	101.60	152.40	203.20
20	140.10	210.20	280.20

Medium

σ_3, psi \ X	1.50	2.25	3.00
3.0	52.71	79.07	105.42
11.5	95.81	143.70	191.61
20	138.90	208.40	277.80

Fine

σ_3, psi \ X	1.50	2.25	3.00
3.0	50.24	75.35	100.47
11.5	86.33	129.50	172.65
20	122.40	183.60	244.80

Soft Aggregate

Coarse

σ_3, psi \ X	1.50	2.25	3.00
3.0	36.30	54.30	72.60
11.5	71.15	106.72	142.30
20	106.00	159.00	212.00

Note: Numbers in boxes refer to repeated vertical stresses (σ_1) in psi.

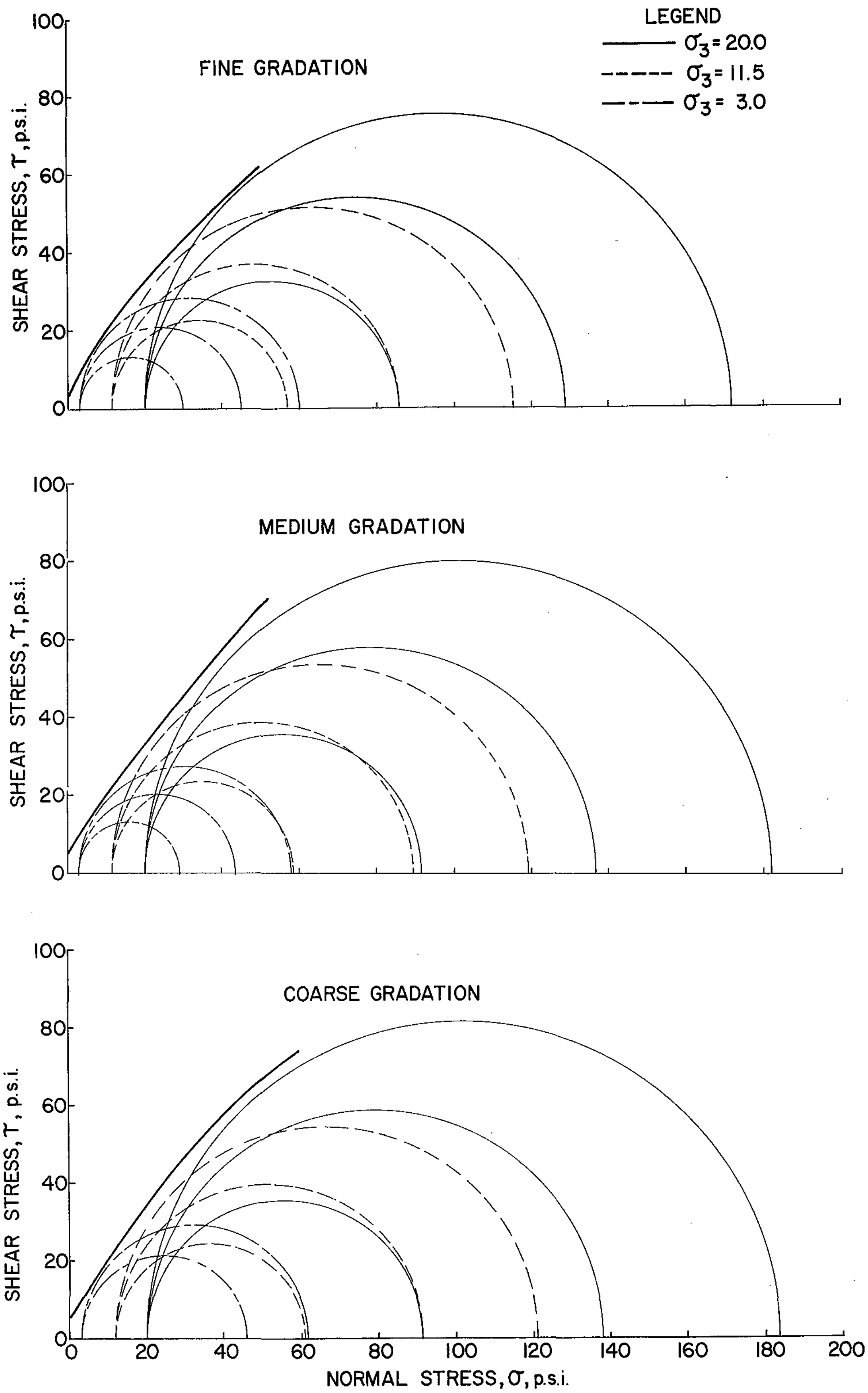


FIGURE 4.11 Mohr's circles for repetitive stresses applied to rounded material.

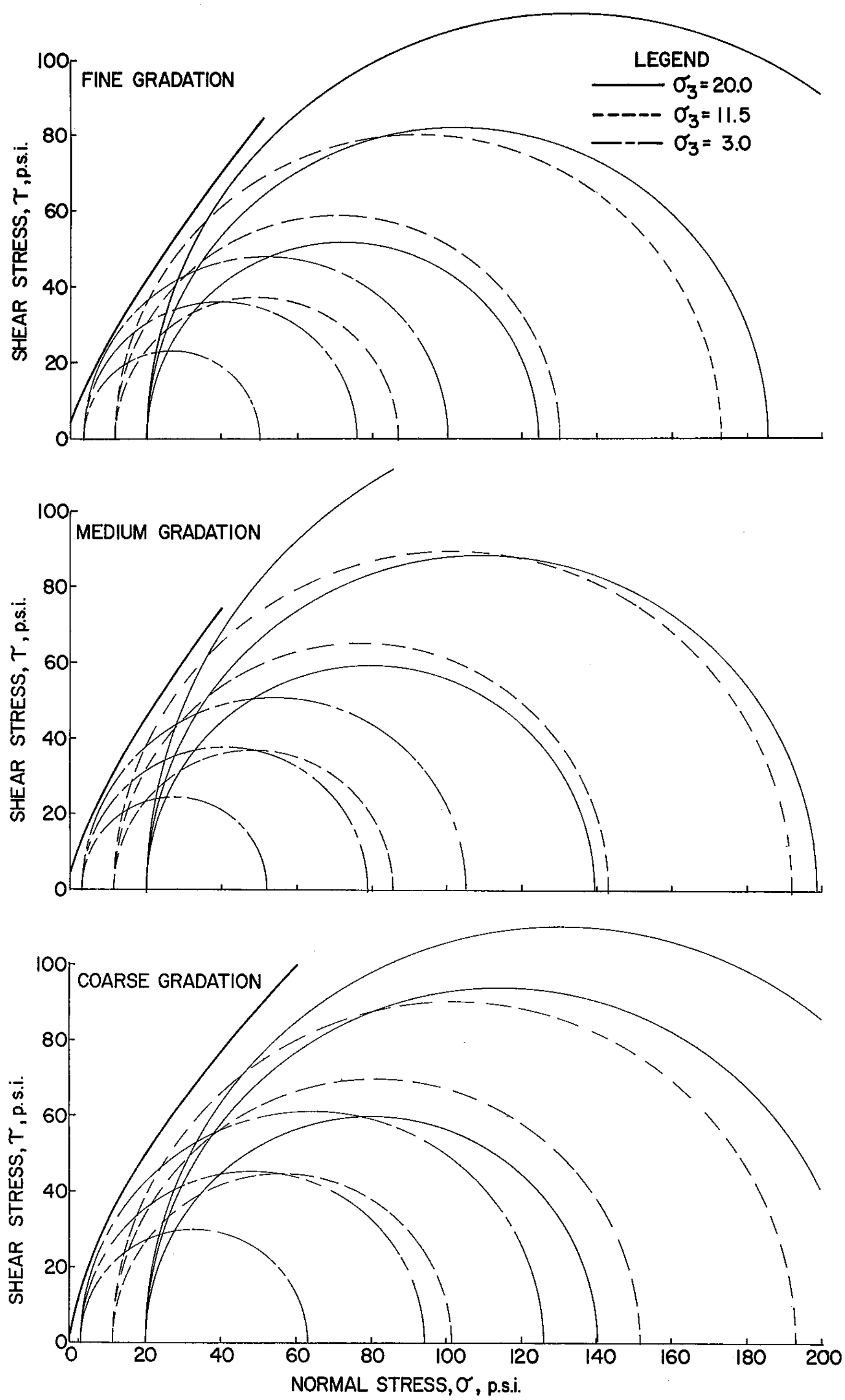


FIGURE 4.12 Mohr's circles for repetitive stresses applied to angular materials.

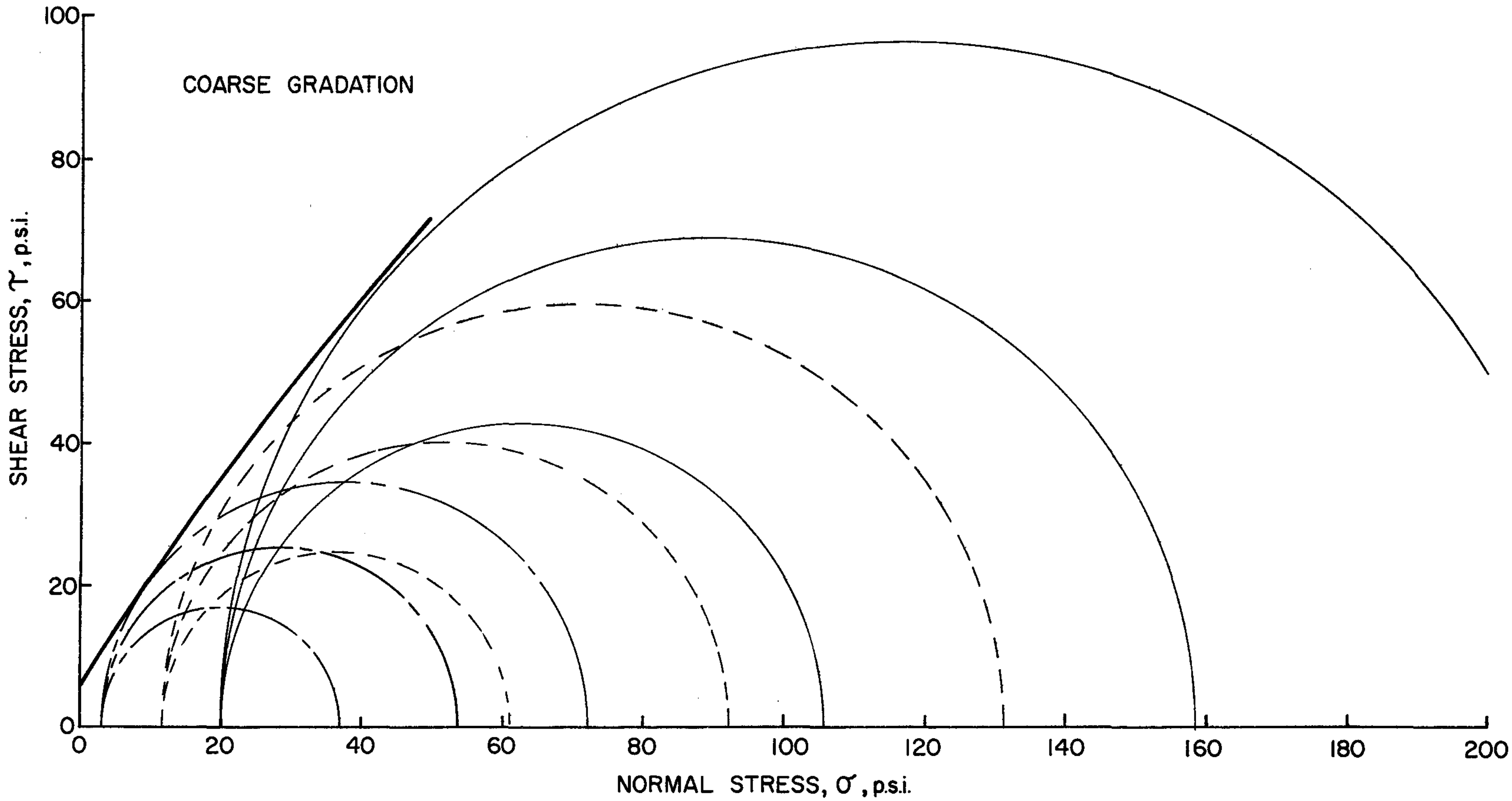


FIGURE 4.13 Mohr's circles for repetitive stresses applied to soft material.

Because the soft material -- due to degradation by compaction -- exhibited similar characteristics regardless of the initial gradation, it was deemed expedient to continue the testing program on only the coarse gradation.

This method of selecting the magnitude of the repetitive stresses is admittedly arbitrary. It should be emphasized that the term "X" is merely a mathmatical convenience, rather weakly related to material strength. In retrospect, it appears that the new method of establishing repeated stresses has as many disadvantages as the previous method. Some specimens lasted too long, many failed too quickly. Some of the specimens were subjected to repeated stresses which appear considerably higher than the actual stresses expected even in heavily loaded roadways. On the other hand, to use stresses low enough to be representative of the lowest stresses encountered in the granular materials in a roadway might result in specimens which lasted for months or even years.

It may also be argued that the confining pressures used are low and do not cover the range of actual radial stresses which might be encountered in the near-surface layers of heavily stressed pavements.

Frankly, it appears that the only suitable method by which the magnitude of repeated laboratory stresses can be determined is to have prior knowledge of the range of stresses expected on the material in the roadway. Until this can be done, a more sophisticated experiment design is not warranted.

The reader can observe from Figures 4.11 through 4.13 that the materials were at least subjected to a wide range of shear stresses.

Characteristics of Repetitive Loading

Once the magnitude of the repetitive stresses had been determined, the next problem was to determine the manner in which they should be applied. Here again, lack of knowledge concerning actual stresses in near-surface granular materials hinders the solution.

The three characteristics of interest are: (a) shape of the loading curve, (b) duration of load application, and (c) "rest" period between loadings, or frequency of load applications.

As reported in Chapter II, many other investigators (32) have used triangular or square wave loading curves, primarily because they are easy to control and simplify analysis of the results. It appears that a sinusoidal curve would be more representative of actual field conditions, but attempts to produce a curve of this shape on the repetitive loading apparatus were fruitless. The desired shape was obtained on the loading portion of the curve, but the unloading part was almost instantaneous, regardless of the manipulation of the control valves.

Armstrong's (4) research was the basis for establishing the duration and frequency of load application, even though his study was limited to a single material and two durations and frequencies. The most destructive combination found was a loading period of one second applied once every ten seconds. Nearly as critical was a 0.2 second loading period applied every two seconds. The latter combination, particularly the loading period, seemed more appropriate for present highway loading conditions, and it was selected for this research.

For the general shape of the loading curves used throughout this research, the reader is referred back to Figure 4.5.

Specimen Preparation

The repetitive triaxial specimens were prepared and compacted in the manner described in Chapter III for the compaction and Texas triaxial specimens except all 1-3/4 -- 1-1/2 inch material was replaced with 1 -- 1-1/2 inch sizes. Six rather than four layers were required to form the 12-inch high specimens. A special 6-inch diameter by 12-inch high compaction mold was supplied for the Rainhart automatic compactor by the Rainhart Company. Great care was taken in finishing the top surface of each specimen so it was level and square to the longitudinal axis (even slightly tilted heads significantly hastened the failure of repetitively loaded specimens) and to prevent variations in unit weight within the specimen. The latter requirement can be particularly important: if any portion of the specimen were looser than the remainder -- and this could easily occur on the finished surface -- it would be manifested in initial specimen deformation which would not be typical of the entire specimen. Unfortunately, there is no sure way of checking uniformity of unit weight throughout a granular specimen, particularly at the relatively thin hand-finished surface.

The compacted specimen was extruded from the mold, weighed, and encased in a butyl membrane. It was sealed in a triaxial cell, a volume change device was attached to the drainage ports and a 3.0 psi confining pressure was applied. This pressure forced the relatively stiff butyl membrane against the specimen and expelled air entrapped between the membrane and specimen. Then the initial readings on the volume change device were recorded. Following this, the confining pressure at which the specimen was to be tested (3.0, 11.5, or 20.0 psi) was applied and maintained for approximately 24 hours during which periodic volume change readings were made. If these readings indicated that the confining system was leak-proof, the specimen was ready for testing.

Repetitive Testing Procedure

One of the early problems encountered in the testing program was maintaining a constant repetitive load on the specimen during the initial stages of loading; the load was usually low for several repetitions. One possibility for this behavior was entrapped air in the hydraulic loading cylinder; it is also possible that the specimens were initially so resilient that the loading cylinders were unable to move fast enough to apply their full load during the short 0.2 second loading period.

This problem was minimized, but seldom eliminated, by a "pump down" procedure in which a dummy triaxial cell, constructed of pipe, was loaded repetitively for 15-30 minutes prior to loading the specimen. Some resilient material such as rubber gasketing or plywood, was placed between the dummy cell and the load transducer to simulate specimen deformation. This removed any air trapped in the cylinder and also allowed adjustment of the shape of the loading curve and magnitude of load. By experience, the load was set somewhat higher than desired to compensate for the initial specimen resilience.

After adjustments, the dummy cell was removed and the triaxial cell was quickly placed in the loading station and centered beneath the load transducer. Initial readings were made and the repetitive loading started.

Dial gage and volume change readings were made at the following repetitions: 1, 5, 10, 20, 40, 80, 160, 325, 650, 1300, 2500, 5000, 10,000, 20,000, 40,000, 80,000, etc. Dynamic load and deformations were also recorded at these intervals starting with 325 repetitions. In general, the readings after 10,000 repetitions were taken at convenient times close to the desired number of repetitions. The load magnitude was recorded for at least the first 100 repetitions and load adjustments were made when needed. Four to five technicians were required to start a repetitively loaded specimen; usually two could handle the task after the 325th repetition.

The repetitive loading was continued until the specimen underwent five percent permanent strain, or until it was apparent from the slope of the strain-repetition curve that the specimen would not attain five percent strain within several million load applications. Before the specimen was removed from the repetitive loading apparatus, dynamic load and deformation were recorded at the lateral pressure (termed "nominal" lateral pressure) used during the test. The lateral pressure was then increased to 30 psi (while holding the repeated deviator stress constant), a few hundred repetitions were applied while the volume changes adjusted, and dynamic load and deformation were recorded at the new lateral pressure. This procedure was repeated in 5 psi decrements to zero lateral pressure.

The specimen was then removed from the triaxial cell, stripped of the butyl membrane, and weighed. It was photographed, then split into six layers along the original compaction planes. Each layer was weighed and dried separately to determine whether moisture gradients existed within the specimen. The dried material was then slaked for 24 hours and wet-sieved to determine the post-loading gradation.

The area of the specimen at various strains can be computed using the well-known formula:

$$A_c = \frac{A_o}{1-\varepsilon}$$

where: A_c = the corrected area

A_o = the original area (28.28 square inches)

ε = the vertical specimen strain

For the maximum strain of 5 percent used in this research, the corresponding change in corrected area would amount to an increase of approximately 5 percent. Thus, by maintaining a constant repeated load, the repeated deviator stress ($\sigma_1 - \sigma_3$) would continually decrease during the life of the specimen. For this small amount, there seemed to be little reason for increasing the deviator load to compensate for changes in diameter, especially since the true cross-sectional area of a bulged specimen can be obtained only by direct measurement. Also, many of the specimens in this research densified under the repeated loading, and the vertical strain was not necessarily manifested in a significant diametral increase of the specimen.

Special Tests

Several special tests were conducted to: (a) determine characteristics of the equipment used in the research, and (b) examine certain properties of the material under repetitive loading. These tests are described or referred to below.

Tests on equipment. When loads were applied to the specimens, the stresses produced deformations in the triaxial cells, even though they were designed to minimize this problem. These deformations were manifested on the transducers and dial gages as specimen deformation. When it became apparent that the cell deformations might be significant -- especially for those specimens which were subjected to high stresses -- the triaxial cells were calibrated. The procedure and results of these tests are presented in Appendix B.

Appendix B also contains the results of tests conducted to determine whether significant friction existed in the loading piston under dynamic loading conditions. The linear ball bushings which the piston traveled in were intended to eliminate this problem, but there was concern whether the "break out" friction of the O-ring pressure seal around the piston and eccentric loading would noticeably reduce the load transmitted to the specimen. The limited tests which were performed indicate that piston friction was very minor.

The response time of the deformation and load transducers was also of interest. Theoretically, this should pose no problem; the geometries of the transducers were satisfactory, and the recording system has a minimum undamped frequency of 120 cps -- well above that required for the loading rates used. These tests, also described in Appendix B, confirmed the adequacy of the system response plus the accuracy of the system under dynamic conditions.

Tests on specimens. As the test series progressed, certain problems arose concerning the test conditions and results. Each of these could easily develop into separate extensive investigations, but they were studied only briefly in this program. The test procedures are discussed below.

A. Effect of drainage on specimen life. By the very nature of the volume change device, the excess pore pressures in the repetitive specimens dissipated as rapidly as the permeability of the material and the loading frequency would permit. Actually, the test conditions fall somewhere between drained and undrained conditions. Sufficient material remained after the regular tests to conduct undrained tests on one specimen of the angular material at the coarse and one at the fine gradation. These specimens are identified by the circles in the experiment design extracted from Table IV.1 and shown below.

Angular Material

σ_3, psi \ X	Coarse 1.50	2.25	3.00
3.0	63.20	94.70	126.30
11.5	101.60	152.40	203.20
20	140.10	210.20	280.20

σ_3, psi \ X	Fine 1.50	2.25	3.00
3.0	50.24	75.35	100.47
11.5	86.33	129.50	172.65
20	122.40	183.60	244.80

The undrained tests were conducted in the usual manner except the valves at the drainage ports were closed during repetitive loading. After failure the valves were opened and volume change readings were made when the excess pore pressures had dissipated.

B. Effect of compaction on specimen life. The degree of densification almost certainly influences the repetitive strain characteristics of granular materials. This effect is difficult to assess in the laboratory mainly because of the problems in compacting the materials (refer to discussion on this subject in Chapter III). However, two specimens of the angular material were tested at their optimum moisture contents and maximum unit weights for a compactive effort of 33.16 ft. lbs. per cu. in. (125 blows per layer). These specimens are also identified by the circles in the experiment design below.

Angular Material

σ_3, psi \ χ	Medium 1.50	2.25	3.00
3.0	52.71	79.07	105.42
11.5	95.81	143.70	191.61
20	138.90	208.40	277.80

Note: Two specimens tested.

The specimens were tested by the regular repetitive testing procedures.

Data Reduction and Processing

Repetitive strains. Figure 4.14 is presented to aid the reader in visualizing the strain-repetition relationship. (Basically this is another method of portraying the information shown in Figure 2.1). The original height, h_o, of the specimen was established by direct measurement; t_o represents the original dial gage reading after the specimen was installed on the triaxial cell. Using the terminology established in Chapter II, deformation under the maximum load was termed the total deformation, "t"; after the load was removed, the deformation remaining in the specimen was "p", the permanent deformation; the difference between the total and permanent deformation (t - p) was the rebound deformation, "r".

The total and permanent deformations, t and p, were obtained from the dial gage readings. Starting with the 325th repetition the rebound deformations could also be obtained from the deformation transducer recordings. After several repetitions, depending on the life of the specimen, the dial gage rebound deformations were influenced by experimental error, and the deformation transducer recordings were considered more reliable indications.

The total deformation, t, is considered the most reliable of all deformations, primarily because factors such as seating errors, slack in moving parts, etc. were constant when the maximum load was attained. The permanent and rebound readings, p and r, were influenced by many factors, primarily by seating error and the tendency of the loading

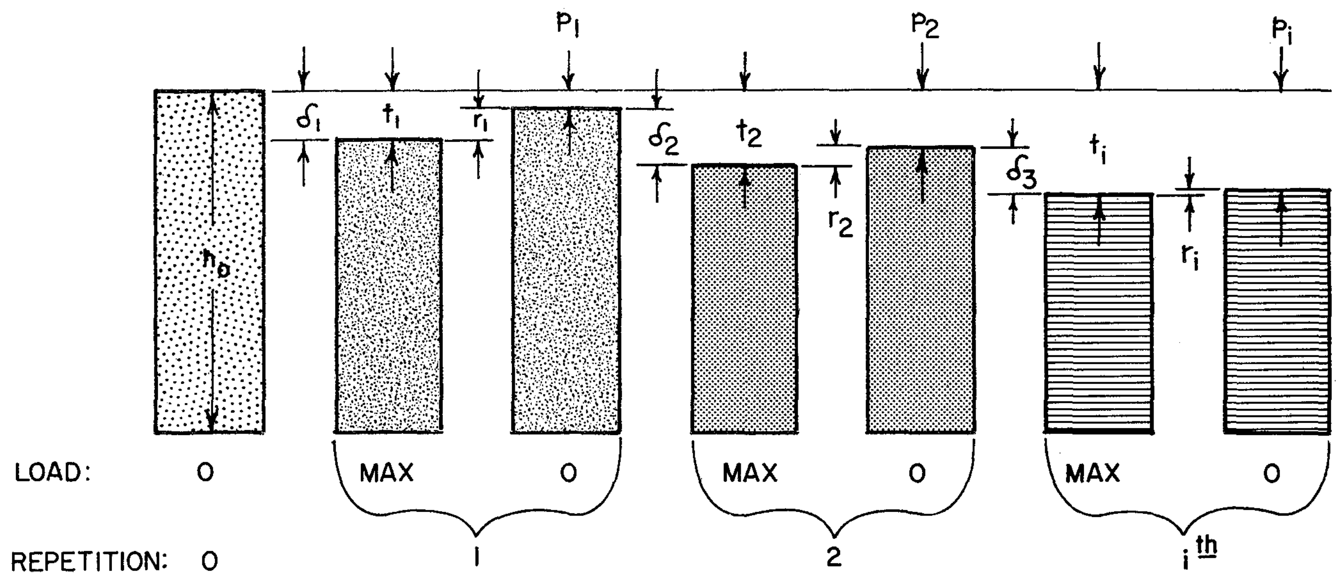


FIGURE 4.14 Diagrammatic representation of changes in specimen lengths during load and unload cycles.

piston to be "blown" out of the triaxial cell when the higher lateral pressures were used. Strong rubber bands were attached to the loading piston to counteract the latter problem, but they were not always successful.

All deformation readings were translated into strains by dividing them by the original specimen height, h_o, after the deformation readings were corrected for strain in the triaxial cells.

Unit weight and degree of saturation. After the specimens were compacted, measured, and weighed, their initial unit weights and degrees of saturation were computed. From the water and air volume changes (with corrections made for the air which diffused through the membranes), the changes in unit weights and degrees of saturation were computed during the lives of the specimens.

CHAPTER V

TEST RESULTS AND ANALYSIS

Presentation of Test Data

The large quantity of test data obtained during the research program makes it impractical to present the results of each separate test in graphical form. These results are available in tabulated form upon request for interested parties. Compacted unit weights and gradations for all specimens tested on the project (except some Texas triaxial specimens) are presented graphically in Appendix C.

A typical example of the "working" curves drawn for each specimen is shown in Figure 5.1. The lower curve is a log-log plot of total strain (t) versus repetitions (N), while the upper group of semi-log curves show rebound strains (r), degrees of saturation (S) and dry unit weights (γ_d) versus log N. (Permanent strain, p, may be obtained by subtracting r from t). The data were also plotted arithmetically.

Analysis of Total Strain

As explained in Chapter IV, the total strain was considered the most reliable strain observation and was, therefore, selected for quantitative analysis.*

There is little value in explaining the many and varied methods

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*The reader is reminded that total strains in the triaxial tests are analogous to the strain observed at a point in the roadway during application of the wheel load. Permanent strains, on the other hand, might represent the "rut" remaining upon removal of the load.

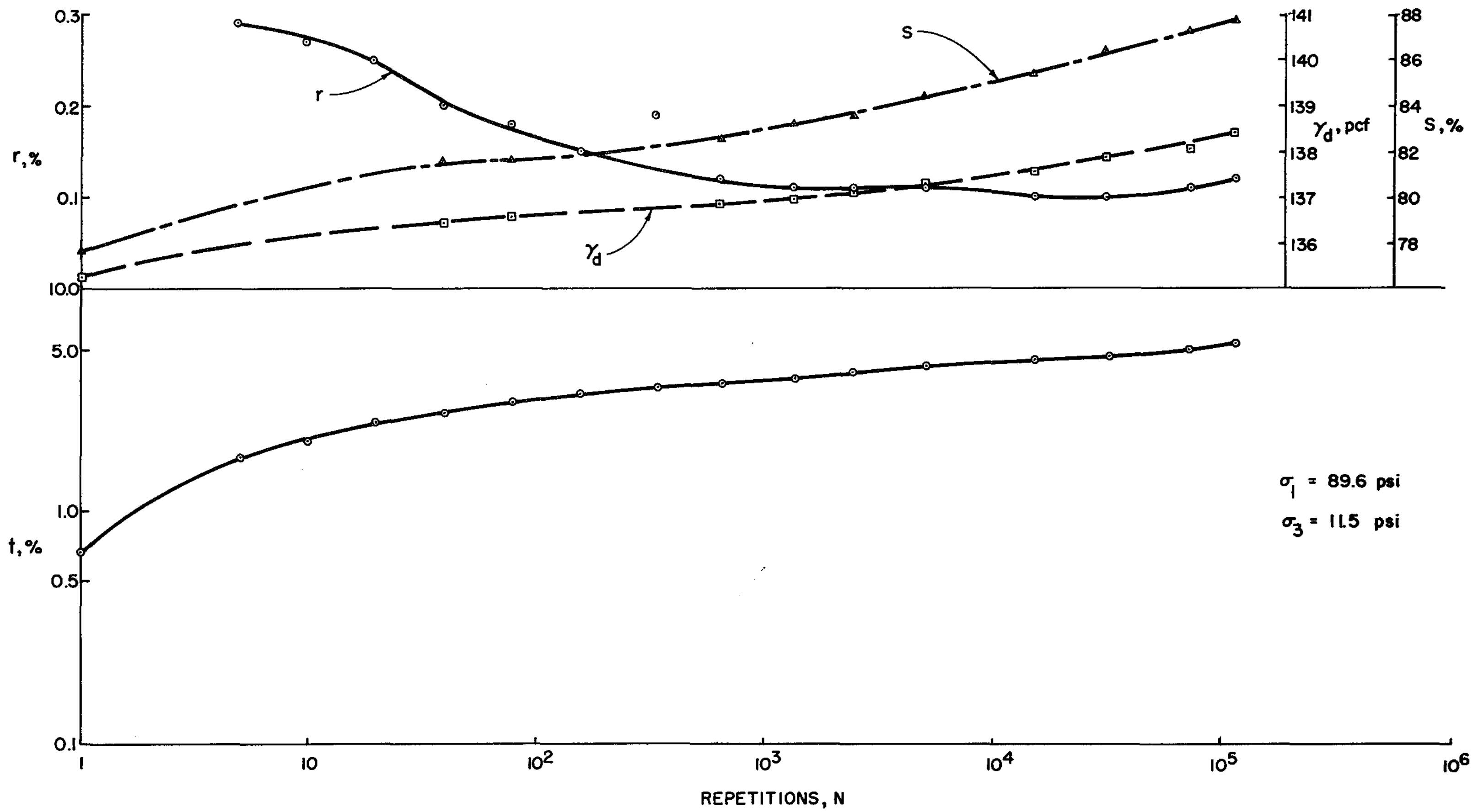

FIGURE 5.1 Rounded medium aggregate. Example of data obtained from a repetitive triaxial test on one specimen.

which were attempted in analyzing these data. On the other hand, a brief review of some of the shortcomings which were encountered may be of value to future researchers.

Obviously, the analysis could be simplified by linearizing the total strain-repetition relationship. This would also facilitate interpolations and extrapolations of the data. However, all attempts to produce suitable linearity, including log-log, semi-log, arithmetic, inverse relationships, etc. were fruitless. Reasons for this will be discussed later, but for the present it will suffice to say that the mode of deformation required to attain failure (or 5 percent strain) varied for different specimens.

For example, Figure 5.2(a) shows the log t-log N relationship, while the same information is plotted arithmetically to two different repetition scales in Figures 5.2(b) and 5.2(c). Comparing Figures 5.2(a) and 5.2(c), it is obvious that the log-log plot severely distorts the data. Of particular importance is the slight upward trend in Specimen 924 near the end of its life as shown in Figure 5.2(a); actually, as shown in Figure 5.2(c) this trend starts approximately halfway through the life of the specimen. The latter figure also vividly portrays the large amount of deformation which occurred initially in a relatively short period of the life of the specimen.

Another serious difficulty in analyzing the data stemmed from the experiment designs. It was anticipated that they would spread the life of the various specimens over a wide range of applications. This worked well for the rounded material, but the angular and soft materials reached

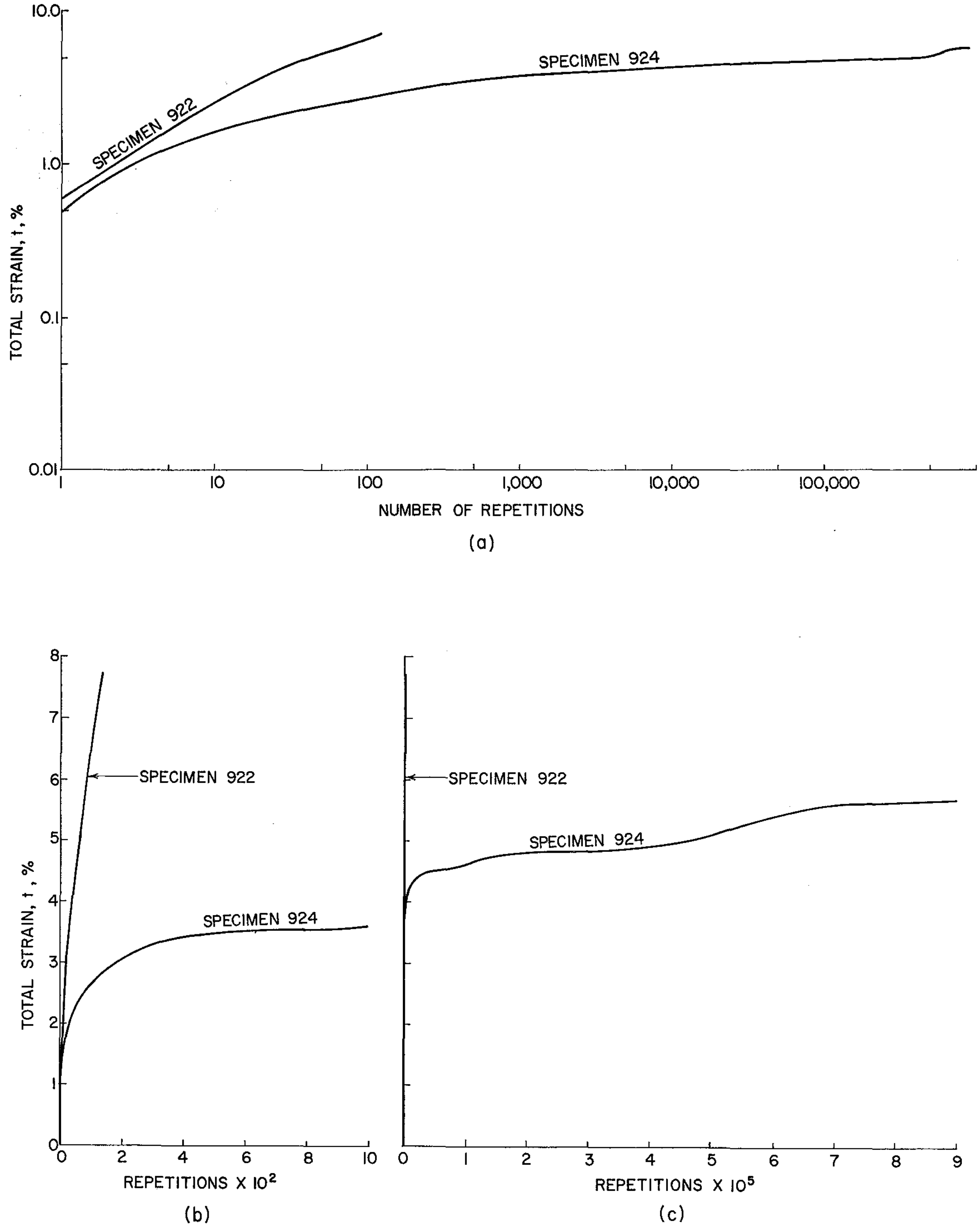

FIGURE 5.2 Total strain for two specimens plotted on three different scales.

the failure strain much quicker than desired. In addition, the method of determining the repeated stresses based on the Texas triaxial strengths resulted in such a wide range of repetitive stresses being applied to the various materials that it was immediately impossible to compare the performance of the materials at the same combination of principal stresses.

In view of these difficulties, two separate analyses were conducted. One was entirely graphical, the other was strictly statistical.

Analysis A (Graphical Method). In this method data from all specimens in each factorial experiment (Table IV.1) were plotted arithmetically with the repeated stress (σ_1) as the ordinate, and total strain (t) as the abscissa. The number of repetitions was placed by each data point. Simple contours of repetitions were then plotted for each lateral pressure as exemplified in Figure 5.3. It was thus possible to obtain values of t and N for the entire range of σ_1 even though only three levels of σ_1 were applied for each lateral pressure. Repetitions less than 100 were not considered significant enough to plot.

It should also be noted that these plots graphically indicated the magnitude of the replication error.

A fair visual analysis of the results could be made simply by placing transparencies of one contour plot over another; however, the amount of data is rather overwhelming. Also, the direct use of these plots for design purposes would be rather difficult. To place the data in useable form, two additional plots were made. First, the observed values of σ_1 for selected values of N (100, 1000, 10,000, and 100,000), t (1, 2, 3, 4 and 5 percent) and σ_3 were plotted. An example is shown in Figure 5.4.

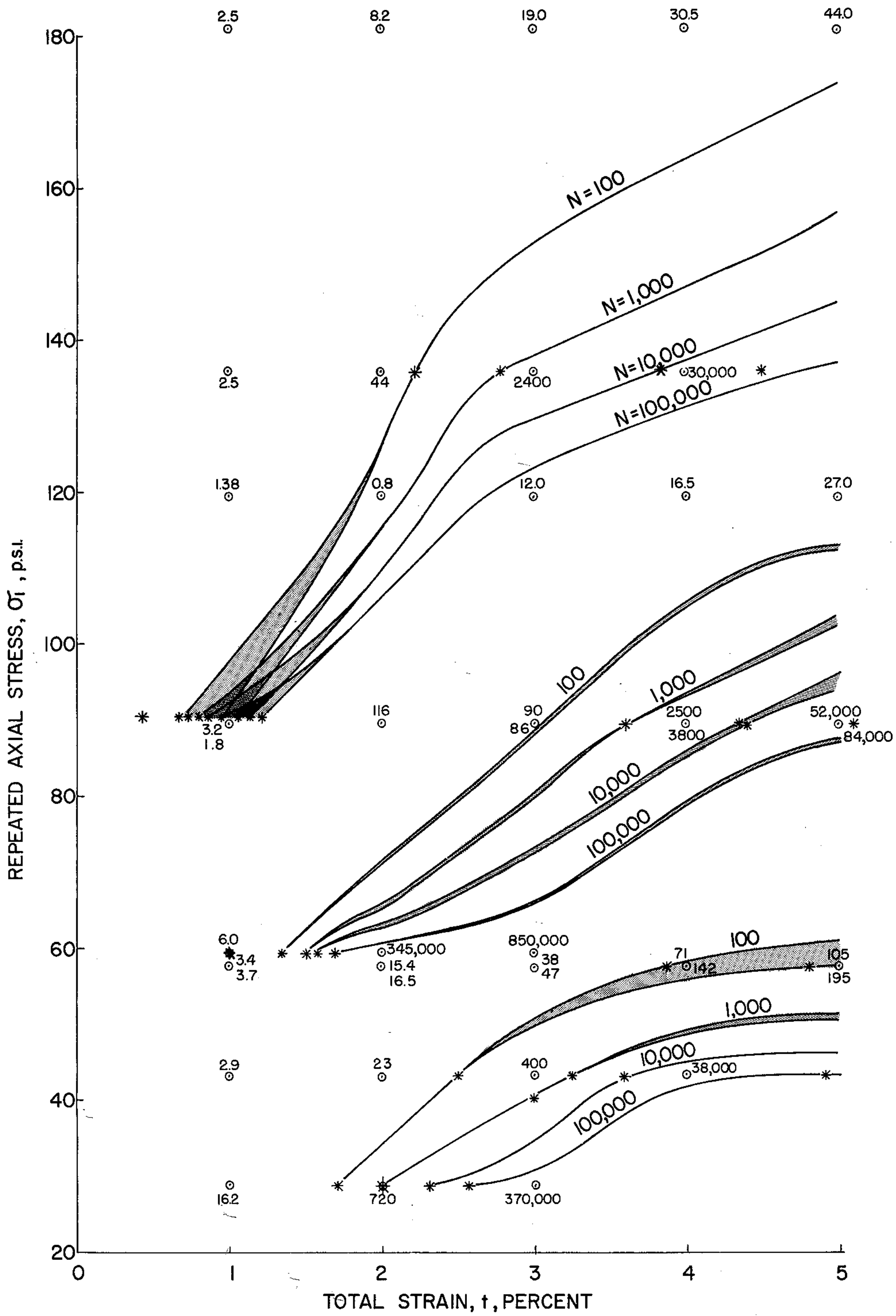

FIGURE 5.3 Rounded medium aggregate. "Contour" curves of total strain for all specimens of this aggregate in factorial experiment.

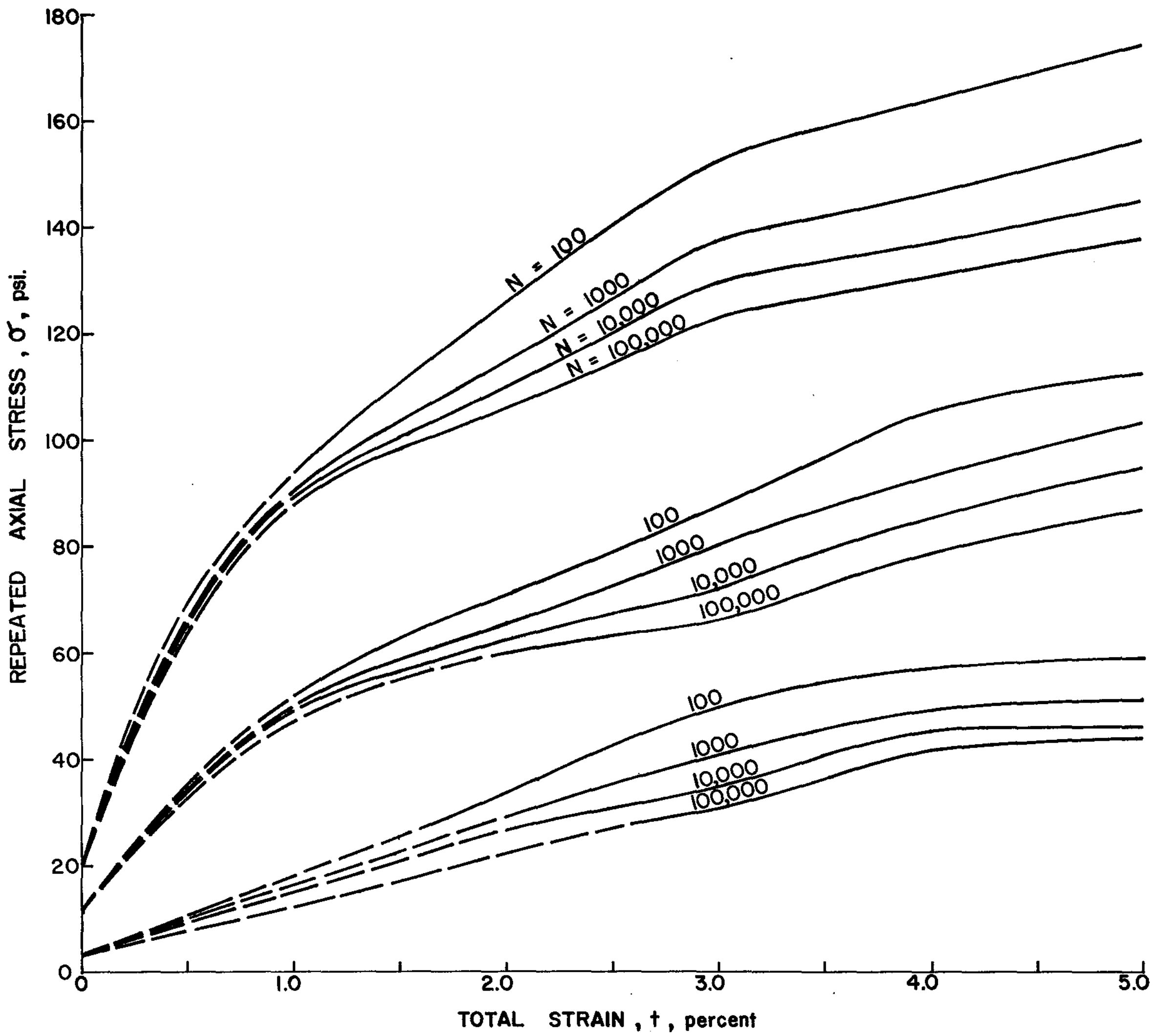

FIGURE 5.4 Rounded medium aggregate. Relationship of total strain and repeated stress to confining pressure and number of repetitions.

They can be thought of as stress-strain curves, but for various values of repetitions rather than for one repetition (or a single load to failure). Some of the curves were incomplete, but there is an obvious boundary condition which aids in extrapolation of the curves: they must originate at σ_1 equal to σ_3 (the stress condition when t = 0). With this boundary condition known, and based on the shape of the curves which were complete, the incomplete curves were completed. Extrapolations and interpolations are shown as dotted lines. An average curve was drawn through the zones of replication error.

To complete the analysis, the data from the "stress-strain" curves (as exemplified in Figure 5.4) were plotted as shown in Figures 5.5 through 5.11. These curves can be readily used to compare the various aggregates, and they can be used directly for design purposes, e.g., if the principal stresses (σ_1 and σ_3) in the roadway were known, the total strain to be expected for 100, 1,000, 10,000, or 100,000 repetitions could be determined.

In this analysis the curves are not as smooth and well-ordered as might be desired. However, actual data were used throughout, and experimental error obviously had considerable influence on the shape of the curves.

Analysis B (Statistical Method). In this method a second degree response surface was fitted to the data for each aggregate using normal multiple regression processes. Only sufficient data points to adequately express the shape of each t-N curve were used. Using t as the dependent variable, the equation was:

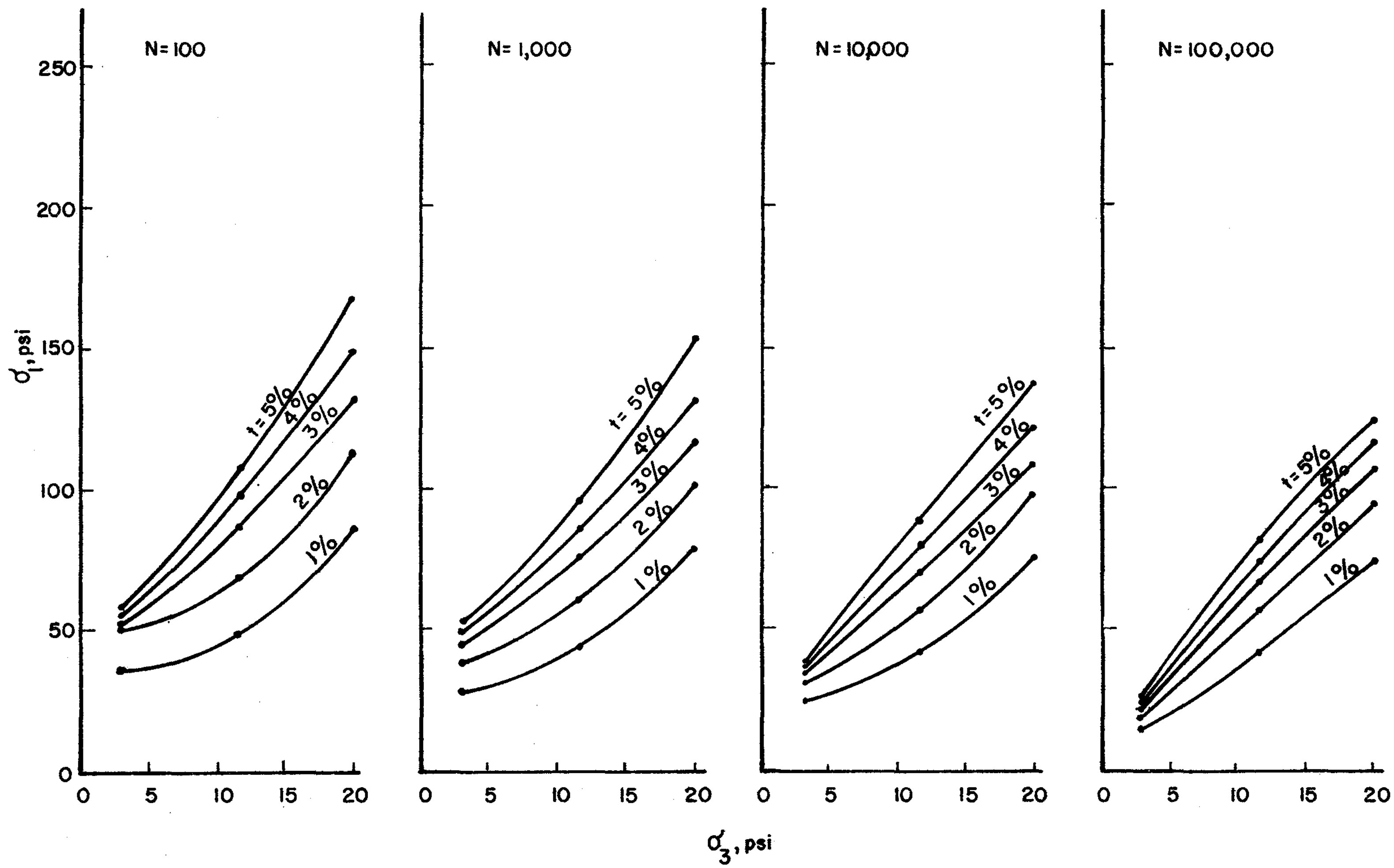

FIGURE 5.5 Rounded coarse aggregate. Results of Analysis A (graphical method).

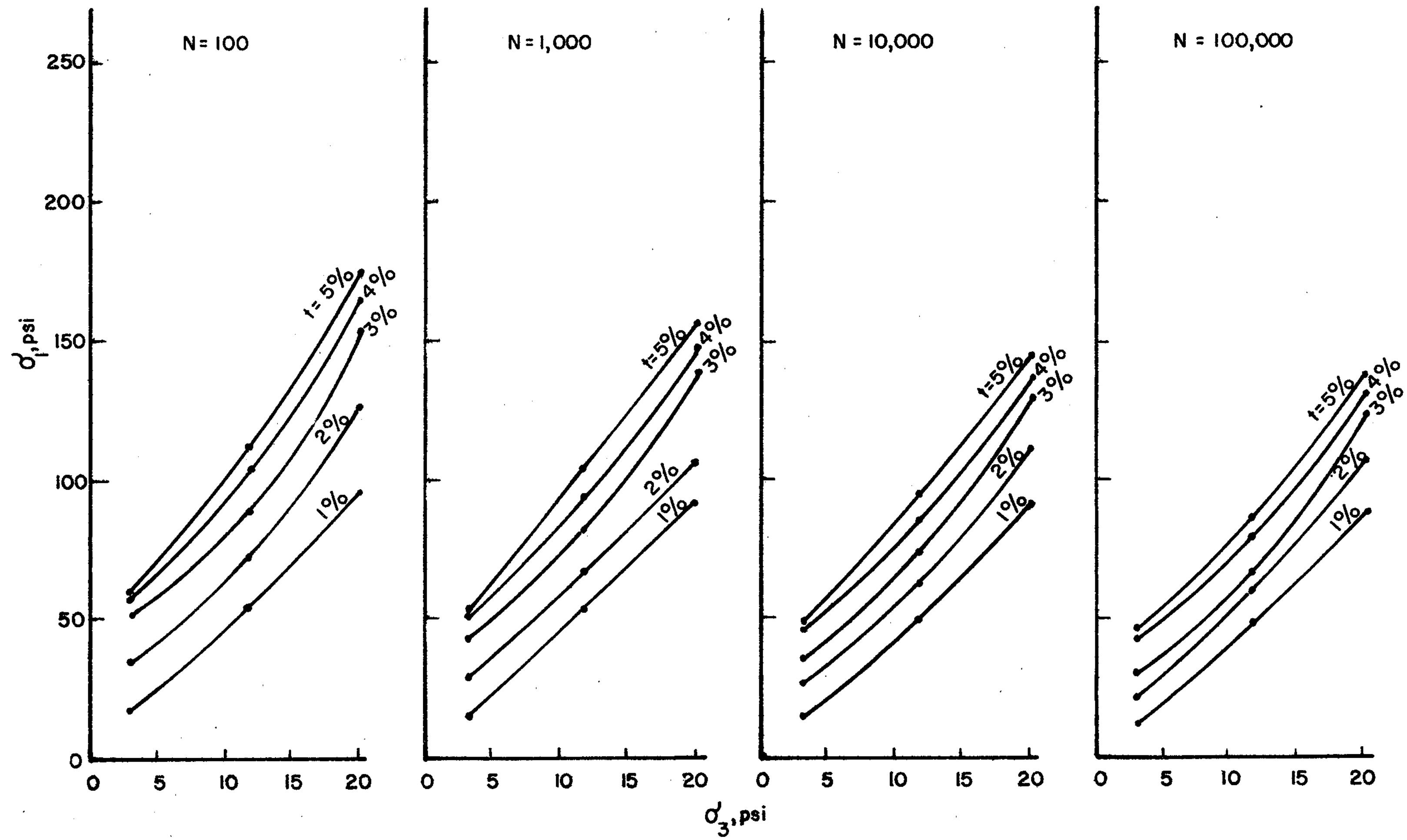

FIGURE 5.6 Rounded medium aggregate. Results of Analysis A (graphical method).

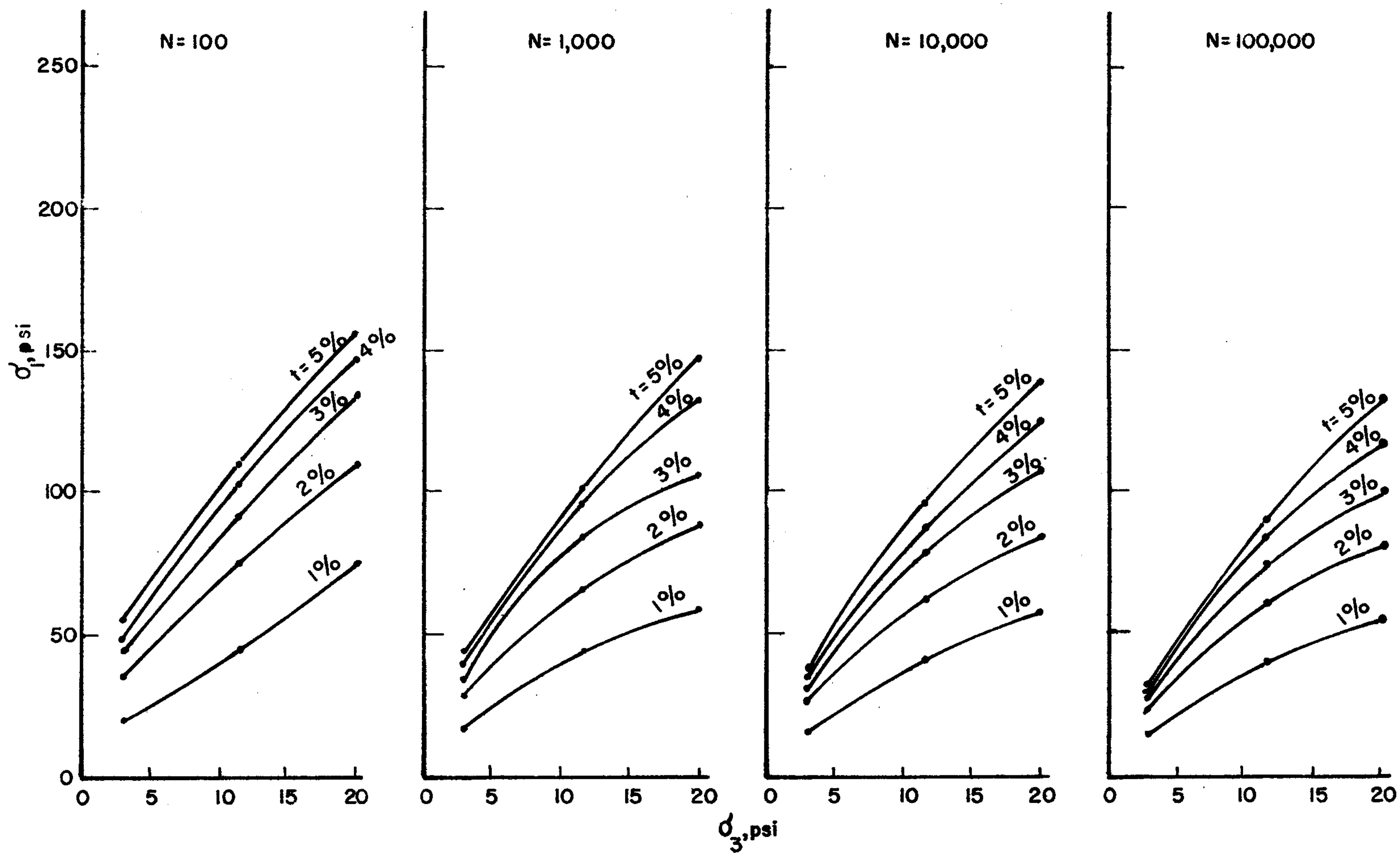

FIGURE 5.7 Rounded fine aggregate. Results of Analysis A (graphical method).

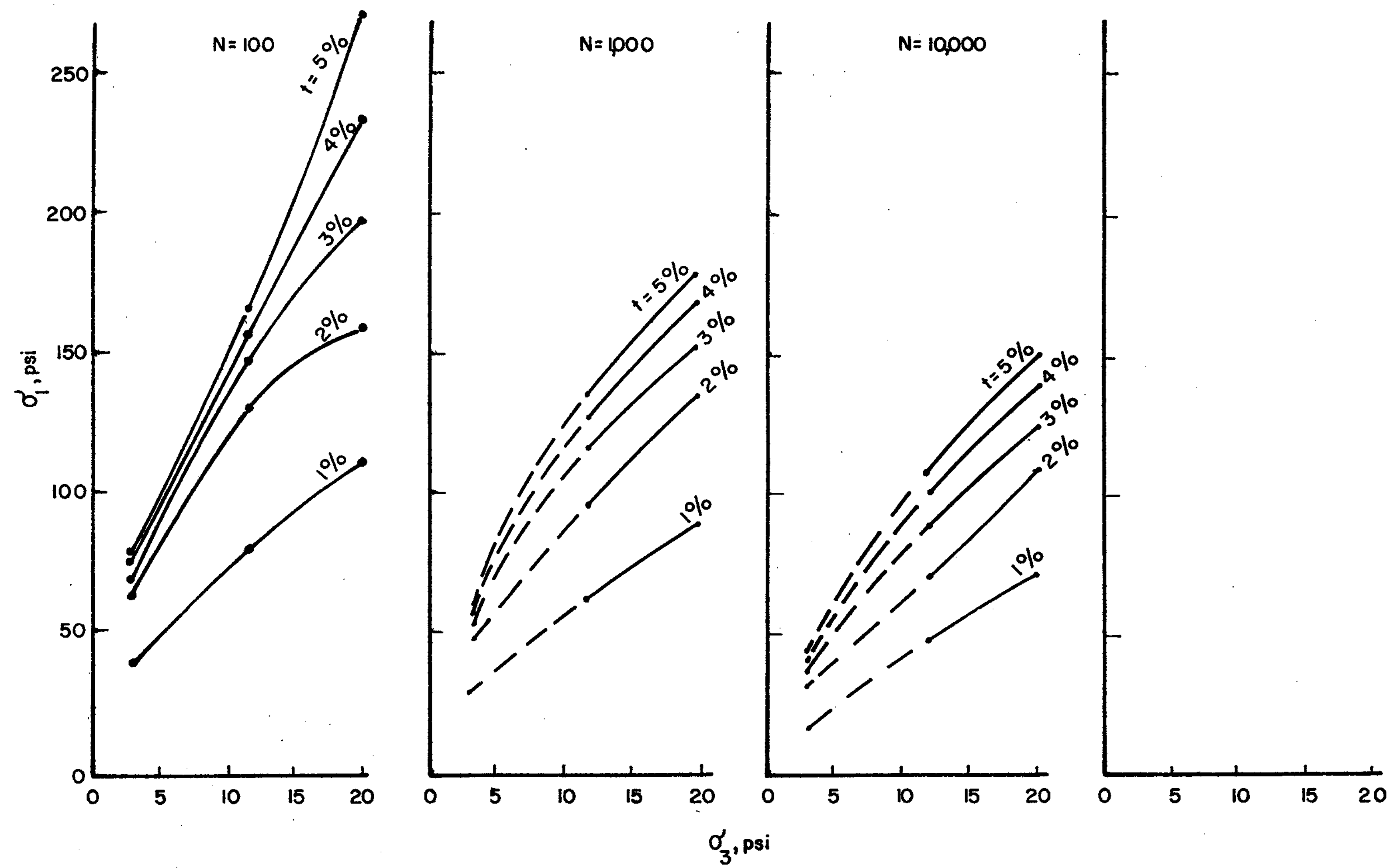

FIGURE 5.8 Angular coarse aggregate. Results of Analysis A (graphical method).

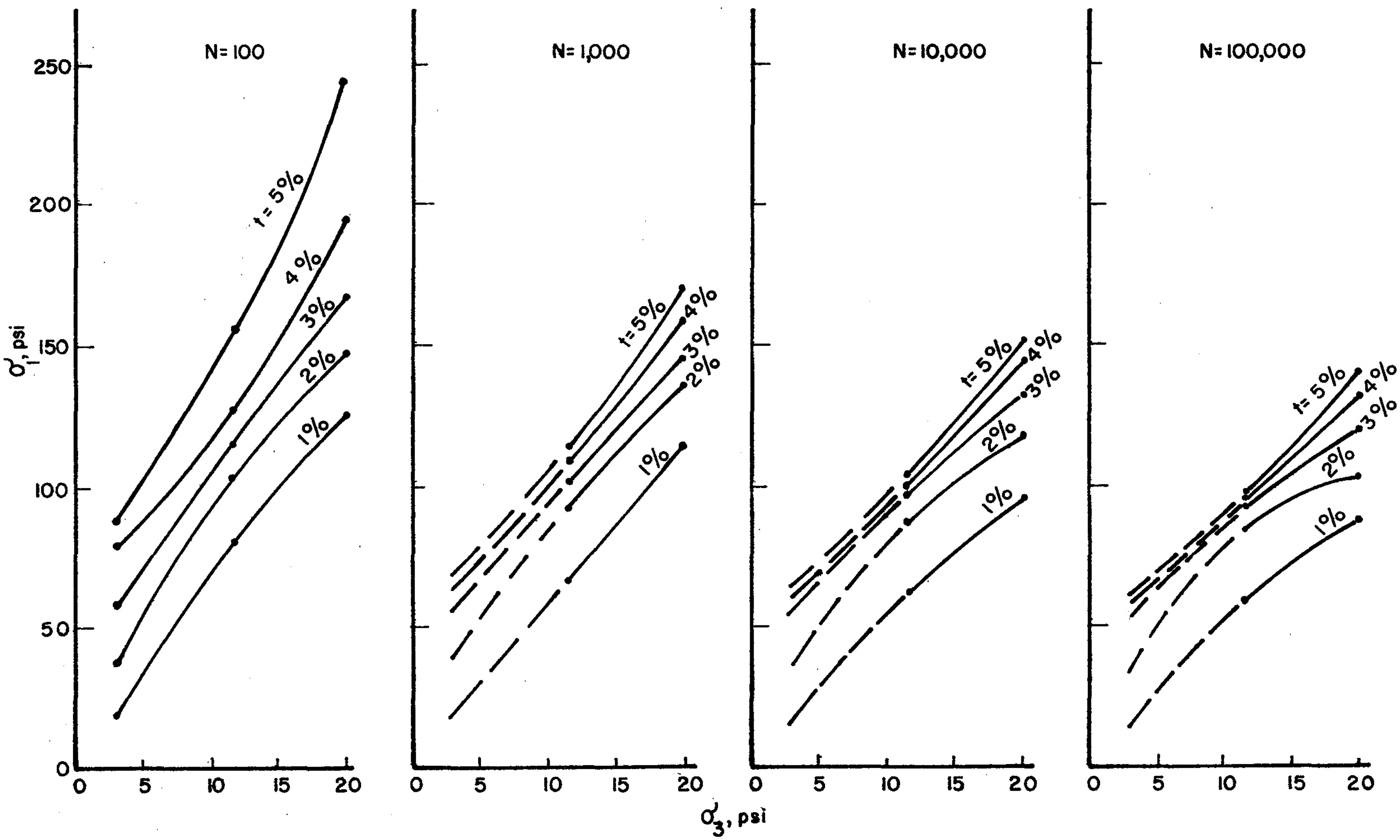

FIGURE 5.9 Angular medium aggregate. Results of Analysis A (graphical method).

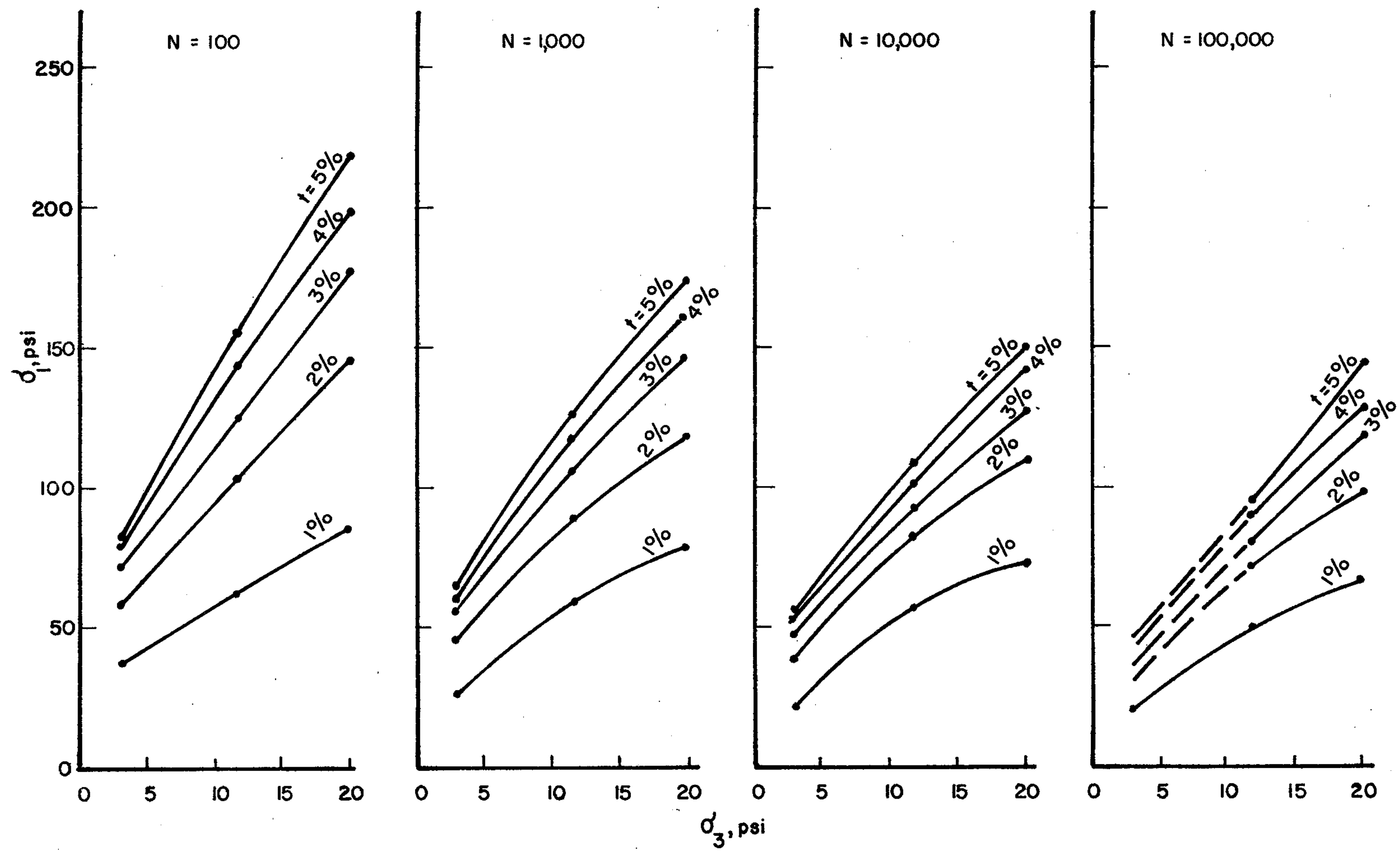

FIGURE 5.10 Angular fine aggregate. Results of Analysis A (graphical method).

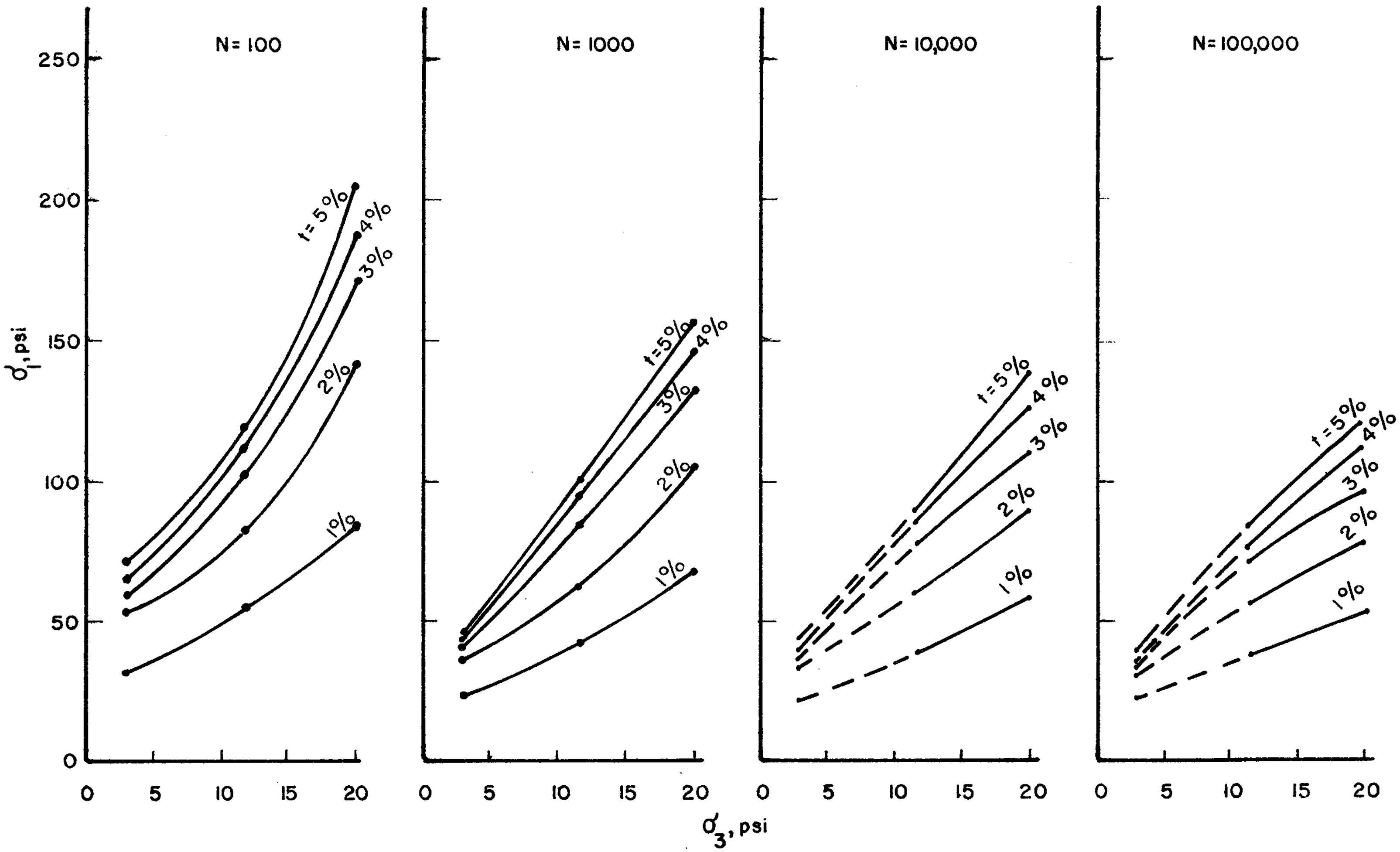

FIGURE 5.11 Soft coarse aggregate. Results of Analysis A (graphical method).

$$t = A_0 + A_1\sigma_1 + A_2(\sigma_1)^2 + A_3\sigma_3 + A_4(\sigma_3)^2 + A_5 \log N + A_6(\log N)^2$$
$$+ A_7\sigma_1 \log N + A_8\sigma_3 \log N + A_9\sigma_1\sigma_3 \,. \qquad \text{Equation (5.1)}$$

Actually the analysis was performed using N and log N; log N proved to be far more significant. It was also done once by calculating A_0, and again by forcing A_0 to be zero. The latter produced a better fit to the data and it seems to be more acceptable from a physical standpoint. The coefficients, A_1, A_2, ..., A_9 and pertinent statistical data are presented in Table V.1.

The squared multiple correlation coefficients, r^2 , are generally quite acceptable*, but they are also somewhat misleading. For engineers, standard deviation might present a better picture since it indicates the variation in t which one might expect from the regression equation. In a few cases, the standard deviation approaches one percent strain, a rather high value.

Although the results are not shown herein, the data were tested to determine whether the variance of the regression equation from the actual data could be attributed to lack of fit of the equation or to replication error. The results indicated that both contributed about equally, thus replication error, which was initially considered to have an important influence on the results, did not particularly bias the regression equation.

The regression equations and corresponding coefficients are rather cumbersome for engineering use. They can be reduced to quadratic equations

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* To those uninitiated in statistics, an r^2 of unity would indicate perfect fit of the equation to the data.

TABLE V.1

Coefficients from Multiple Regression, Analysis B

Material	Gradation	A_1	A_2	A_3	A_4	A_5	A_6	A_7	A_8	A_9	r^2	r	Standard Deviation
Rounded	Coarse	0.0286	0.0002	0.1376	0.0293	-1.2392	0.2481	0.0355	-0.2308	-0.0075	0.9595	0.9795	0.7876
	Medium	-0.0373	0.0014	0.2494	0.0407	0.5334	-0.0856	0.0272	-0.1082	-0.0165	0.9887	0.9943	0.3795
	Fine	0.0930	-0.0007	-0.6495	-0.0048	0.8415	-0.0874	0.0044	-0.0086	0.0068	0.9590	0.9793	0.8013
Angular	Coarse	-0.0160	0.0003	0.4637	0.0522	-1.2404	0.4872	0.0373	-0.3685	-0.0084	0.9717	0.9858	0.6961
	Medium	-0.0064	-0.0000	0.0671	0.0166	1.0847	-0.0838	0.0259	-0.2055	-0.0014	0.9857	0.9928	0.4773
	Fine	0.0251	-0.0002	-0.3499	-0.0077	0.1895	-0.0122	0.0193	-0.0605	0.0027	0.9834	0.9917	0.5386
Soft	Coarse	0.0480	-0.0001	-0.0827	0.0008	-0.3147	0.0456	0.0163	-0.0507	-0.0010	0.9384	0.9687	1.0077

$$t = A_0 + A_1\sigma_1 + A_2(\sigma_1)^2 + A_3\sigma_3 + A_4(\sigma_3)^2 + A_5 \log N + A_6(\log N)^2 + A_7\sigma_1 \log N + A_8\sigma_3 \log N + A_9\sigma_1\sigma_3.$$

Equation (5.1)

and expressed in familiar terms as follows:

$$\sigma_1 = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \qquad \text{Equation (5.2)}$$

where: $a = A_2$

$b = A_1 + A_7 \log N + A_9\sigma_3$

$c = A_3\sigma_3 + A_4\sigma_3 + A_5 \log N + A_6(\log N)^2 + A_8\sigma_3 \log N - t$

By placing values of N (100, 1,000, 10,000 and 100,000 repetitions), t (1, 2, 3, 4 and 5 percent), and σ_3 (ranging from 3 to 20 psi) in the above equation, and solving for σ_1, information was obtained for the series of graphs presented in Figures 5.12 through 5.17. These are the same type of curves shown in Figures 5.5 through 5.11.

Dashed curves represent areas where the term under the radical sign was negative resulting in imaginary roots. In these areas the shapes of the curves were based on surrounding curves, or else sufficient positive values in the area were calculated to adequately define the curves.

When the radical term was positive, the appropriate root could be easily selected.

Insufficient positive terms were obtained to draw curves for the angular material, medium gradation.

It should be emphasized that Equation (5.1) is not a physical mathematical model which attempts to accurately describe the behavior of the materials. In fact, no attempt was made to impose known boundary conditions on the independent variables, which is probably why imaginary terms existed when Equation (5.2) was solved. The prime value of Equation (5.1) lies in the numerical description of replication errors.

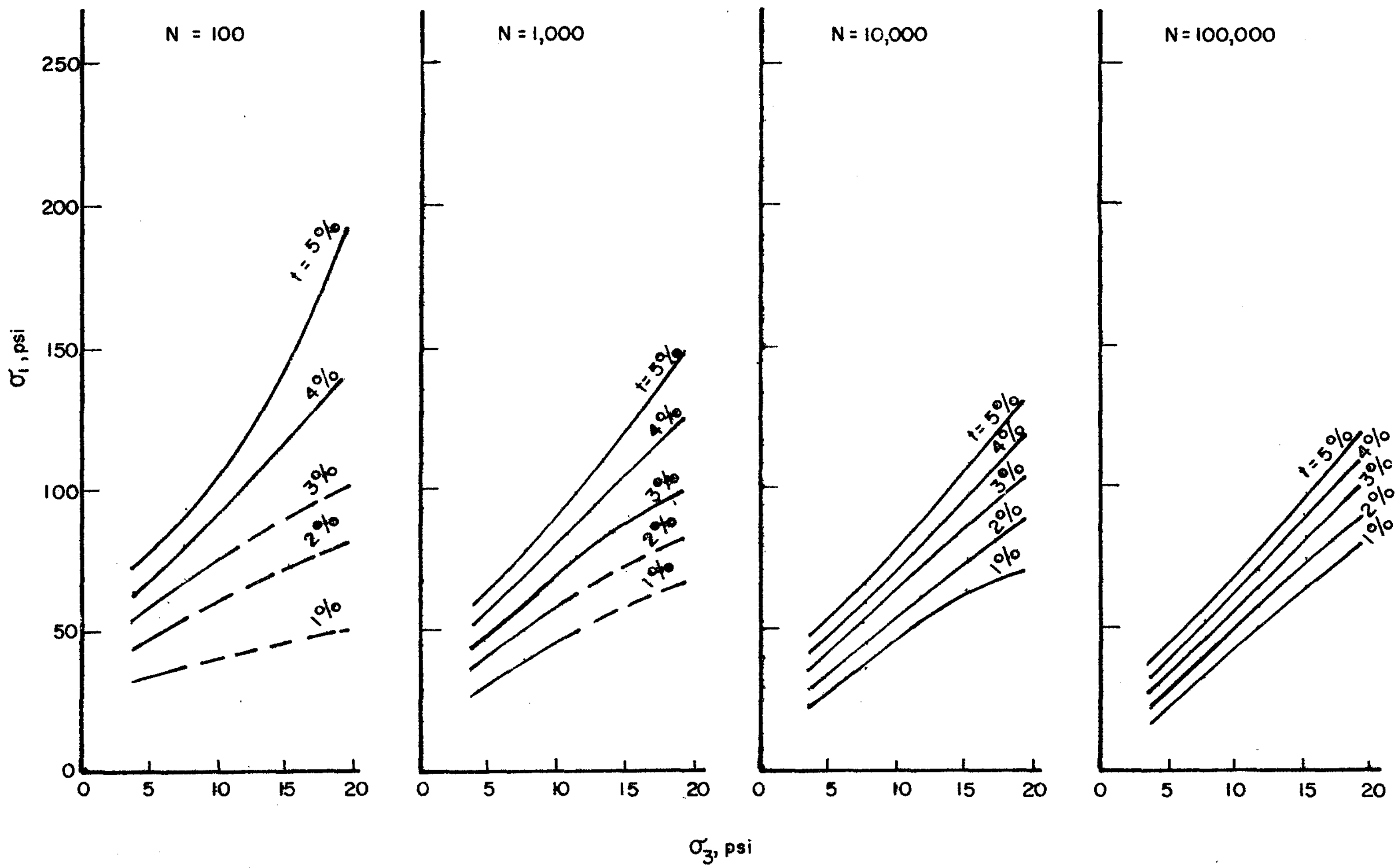

FIGURE 5.12 Rounded coarse aggregate. Results of Analysis B (statistical method).

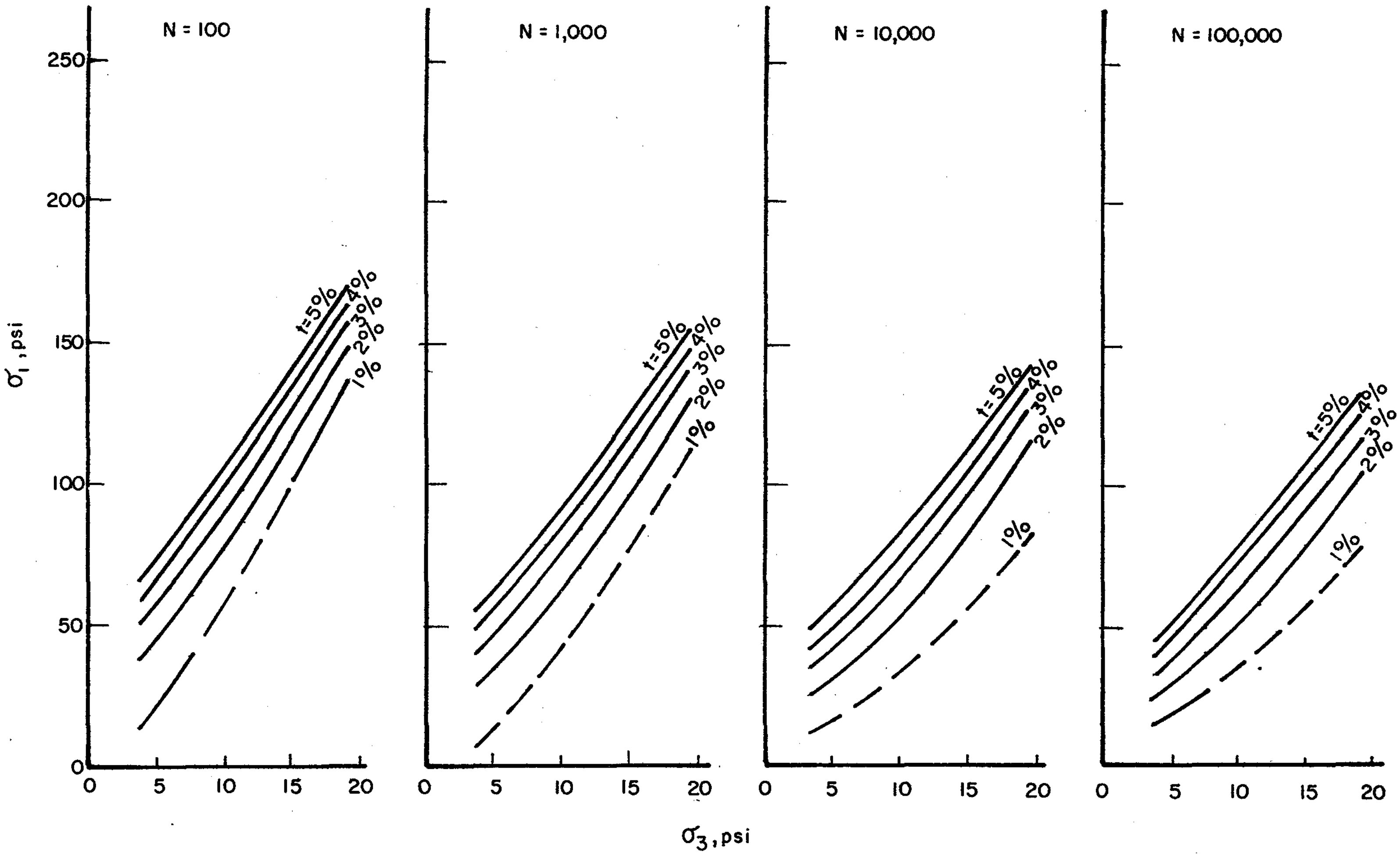

FIGURE 5.13 Rounded medium aggregate. Results of Analysis B (statistical method).

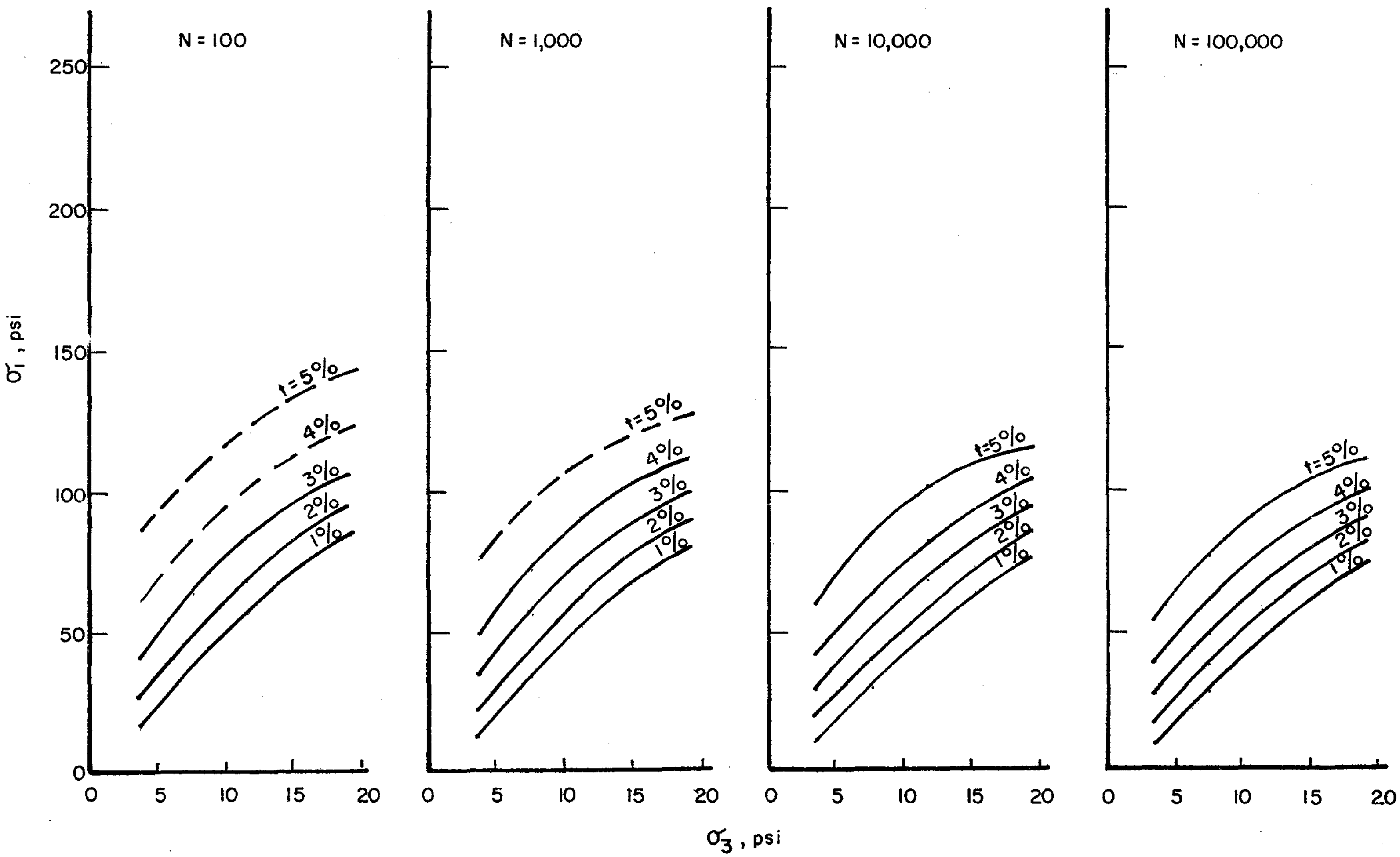

FIGURE 5.14 Rounded fine aggregate. Results of Analysis B (statistical method).

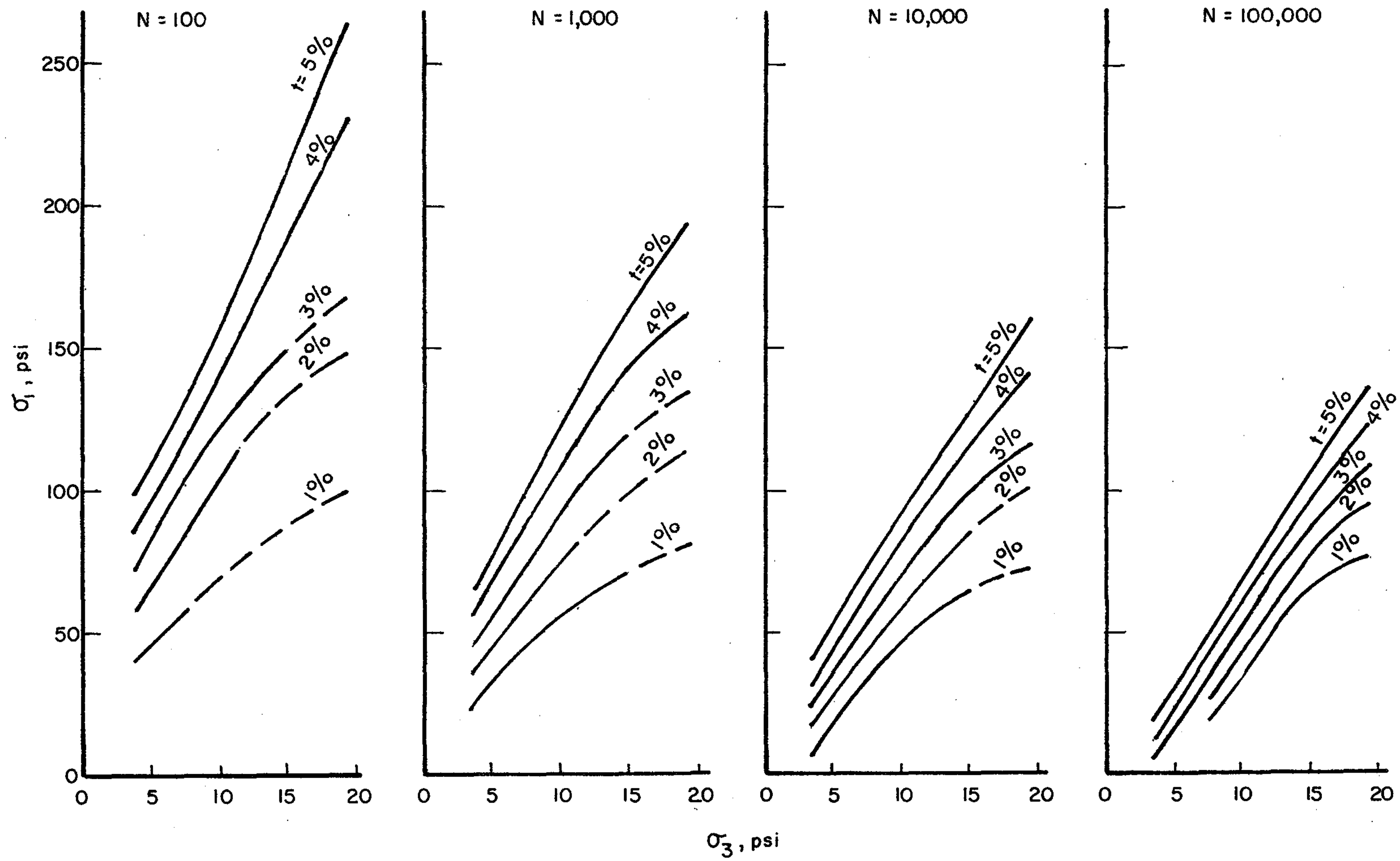

FIGURE 5.15 Angular coarse aggregate. Results of Analysis B (statistical method).

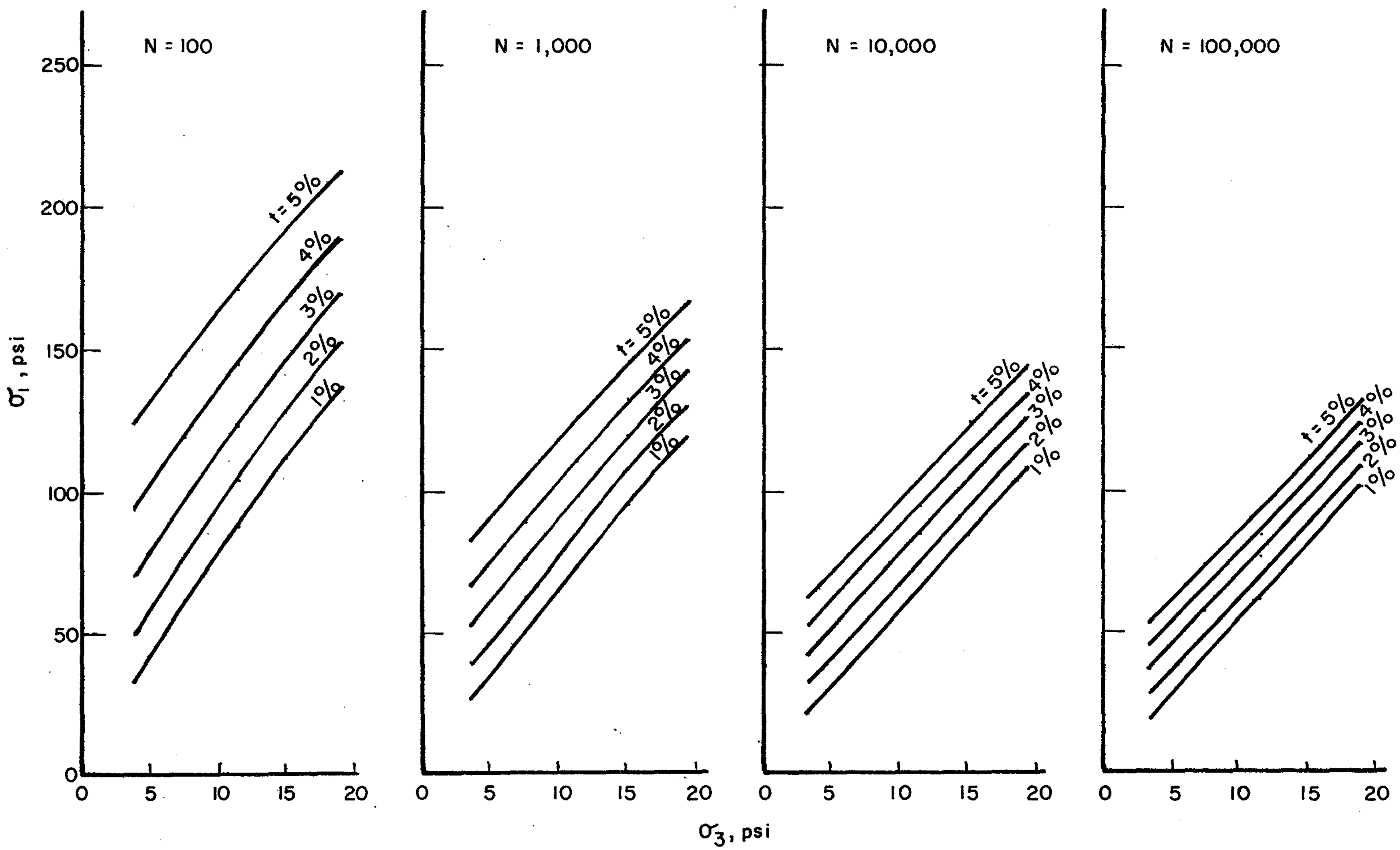

FIGURE 5.16 Angular fine aggregate. Results of Analysis B (statistical method).

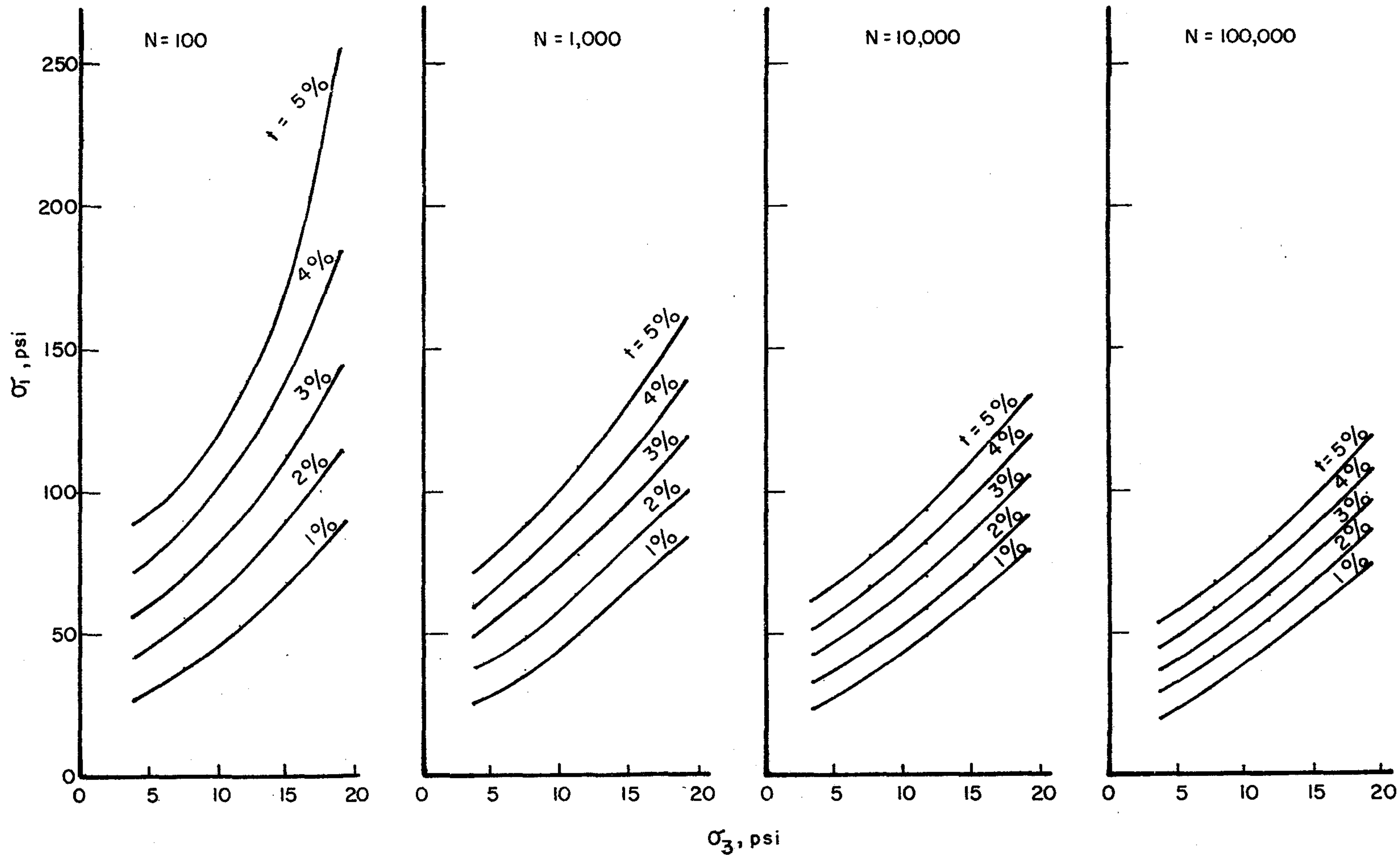

FIGURE 5.17 Soft coarse aggregate. Results of Analysis B (statistical method).

Influence of Repetitions on Rebound Strain, Changes in Unit Weight, and Changes in Degree of Saturation

Because rebound strain, unit weight and degree of saturation are closely related, they will be discussed together.

The rebound strain is significant insofar as roadway behavior of the aggregates is concerned. On the other hand, changes in unit weight and degree of saturation are of interest in explaining why the materials acted as they did.

The method of contour plots used in evaluating total strains was not suitable for analysing rebound strains or the changes in unit weights and degrees of saturation. These variables had a tendency to change erratically with the number of repetitions so that suitable boundary conditions were not obtainable. The only evaluation method which could be devised was comparison of the aggregates at similar stresses, and, fortunately, this could be accomplished for a limited number of specimens. For example, at a confining pressure of 20 psi, one specimen of the rounded coarse aggregate was subjected to a repeated vertical stress of 137.5 psi, and a specimen of the angular coarse aggregate was stressed at 140.1 psi. These values are close enough to permit a comparison of the two materials, and although this method of analysis uses a limited number of the total specimens tested in the program, it is believed to give a valid representation of the overall behavior of the aggregates.

The data for the selected specimens are shown in Figures 5.18, 5.19 and 5.20. The term, $\Delta\gamma_d$, is the change in the dry unit weight;

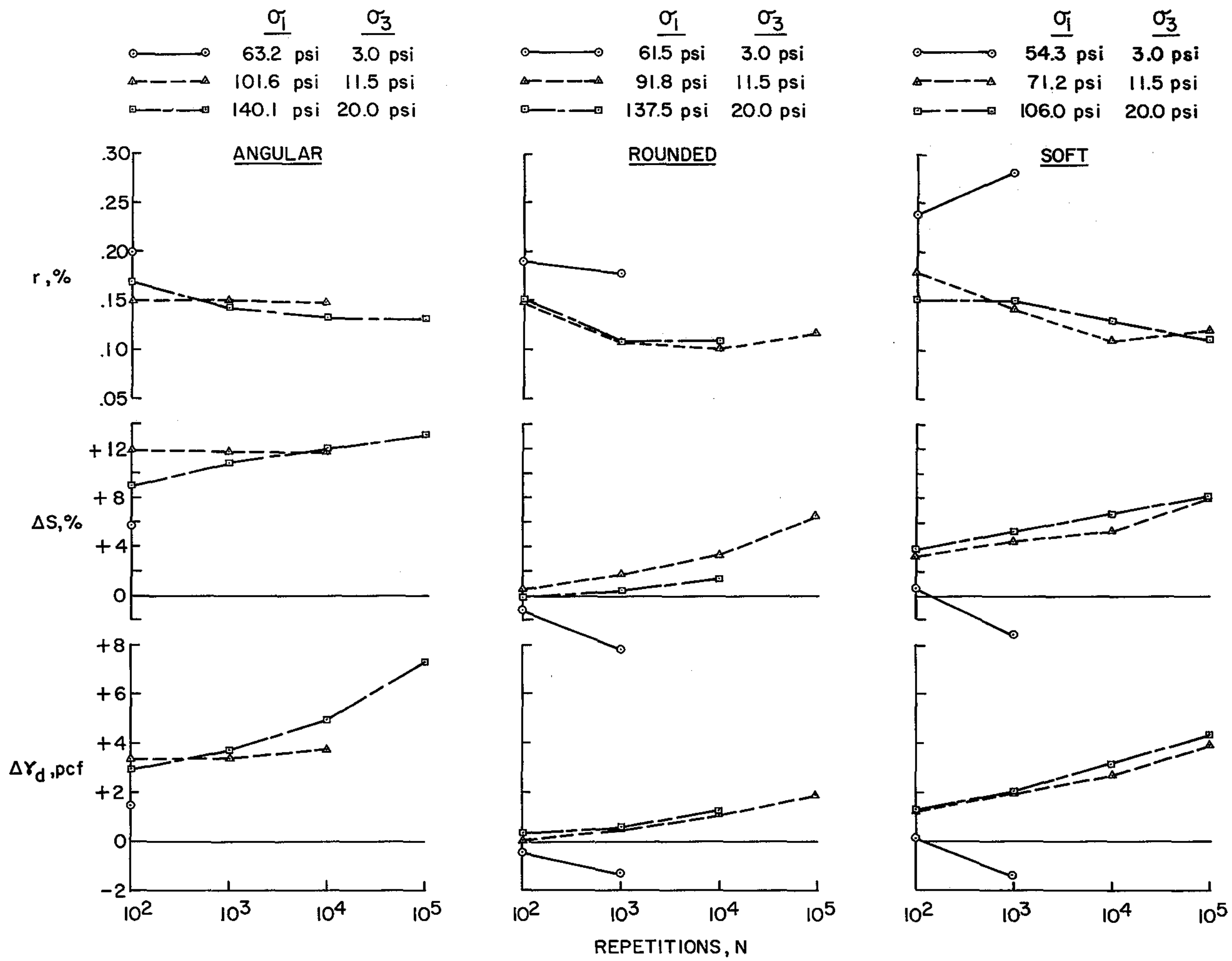

FIGURE 5.18 Coarse gradations. Comparison of rebound strains, and changes in dry unit weights and degrees of saturation at similar stresses for the research materials.

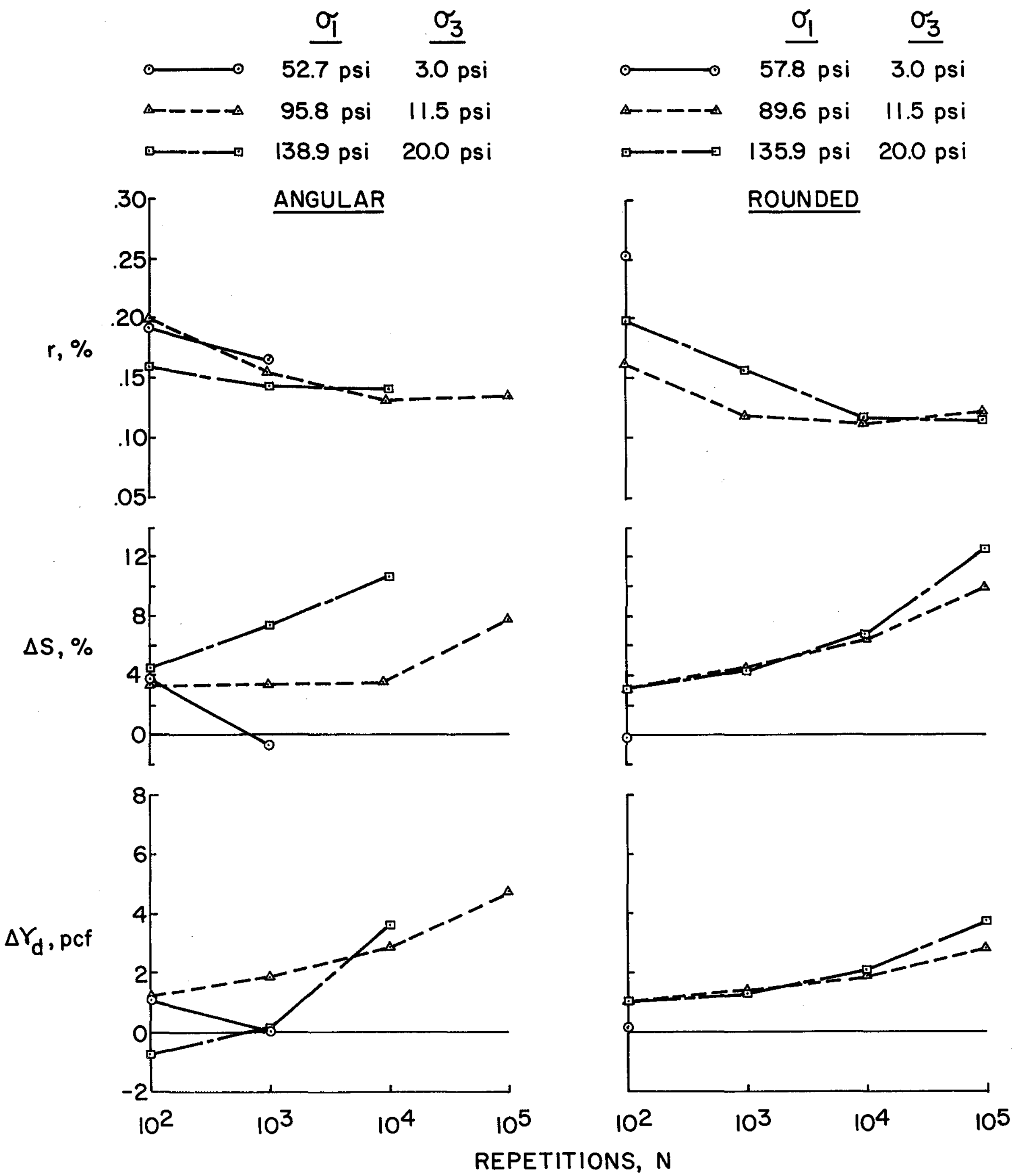

FIGURE 5.19 Medium gradations. Comparison of rebound strains, and changes in dry unit weights and degrees of saturation at similar stresses for the research materials.

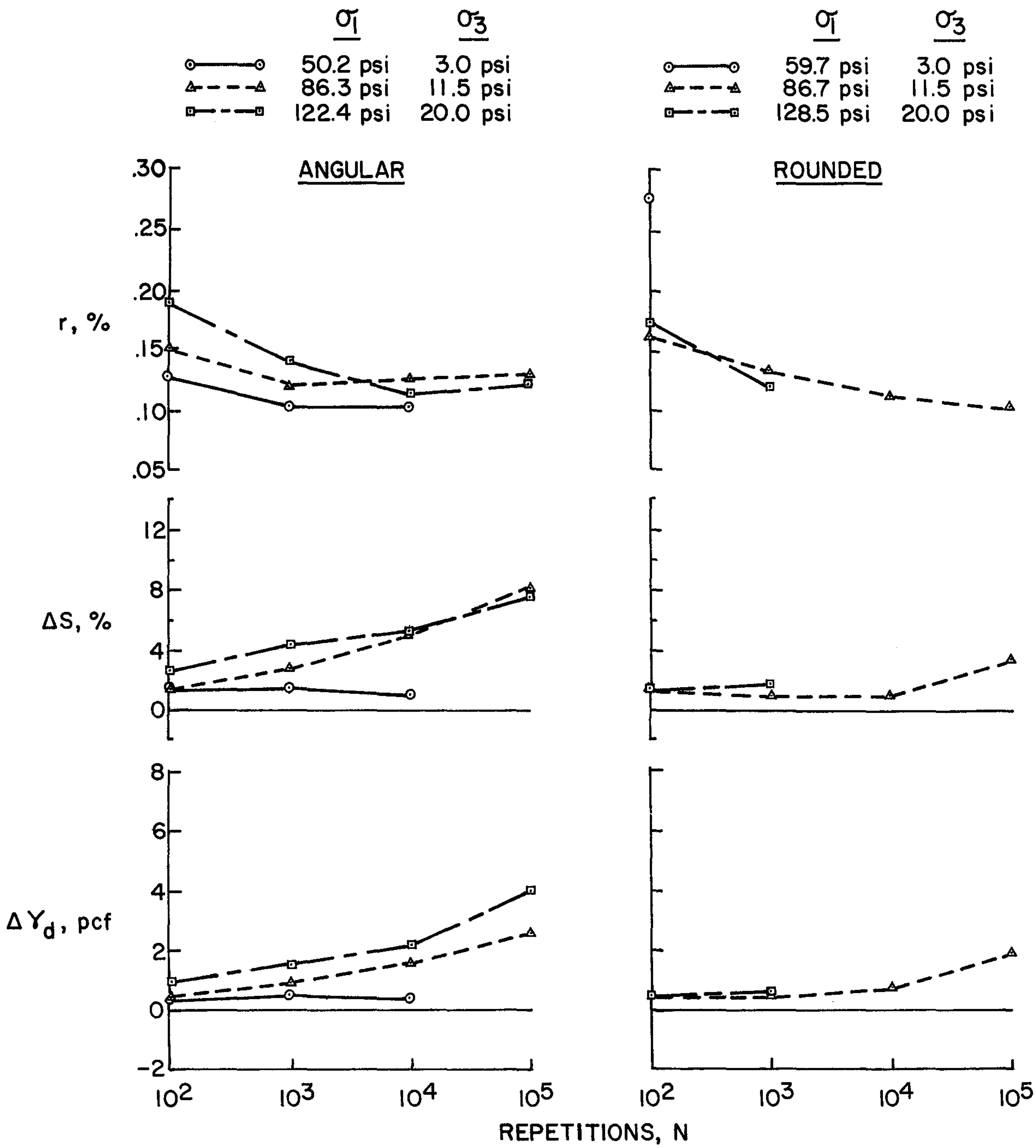

FIGURE 5.20 Fine gradations. Comparison of rebound strains, and changes in dry unit weights and degrees of saturation at similar stresses for the research materials.

ΔS denotes the change in the degree of saturation; r is the rebound strain, in accordance with previous definition.

Effect of Drainage on Repetitive Stress Characteristics

In Figures 5.21 and 5.22 the strain-repetition characteristics of drained and undrained specimens of the angular material are compared. Values shown for the drained specimens are the average of two specimens (replication error was very small in both cases) whereas only one undrained specimen at each gradation was tested.

All specimens reached a total strain of five percent. For the coarse gradation this required 1,325 repetitions in the undrained and 80,000 repetitions in the drained condition. Corresponding values for the fine gradations were 8,000 and approximately 1,000,000 repetitions. Thus, by eliminating drainage the specimen's life was reduced to roughly one percent of the life of the drained specimen (actual ratios were 0.8 and 1.6 percent for the coarse and fine gradations, respectively). Rebound strains were somewhat erratic during the initial loading stages, i.e., the first 100-200 repetitions, but then the undrained specimens rebounded roughly twice as much as the drained specimens.

The number of repetitions required for the two gradations to achieve five percent total strain varied considerably. This may have been partly a function of gradation, but also the repeated stress was about 18 psi higher on the coarse gradation specimens.

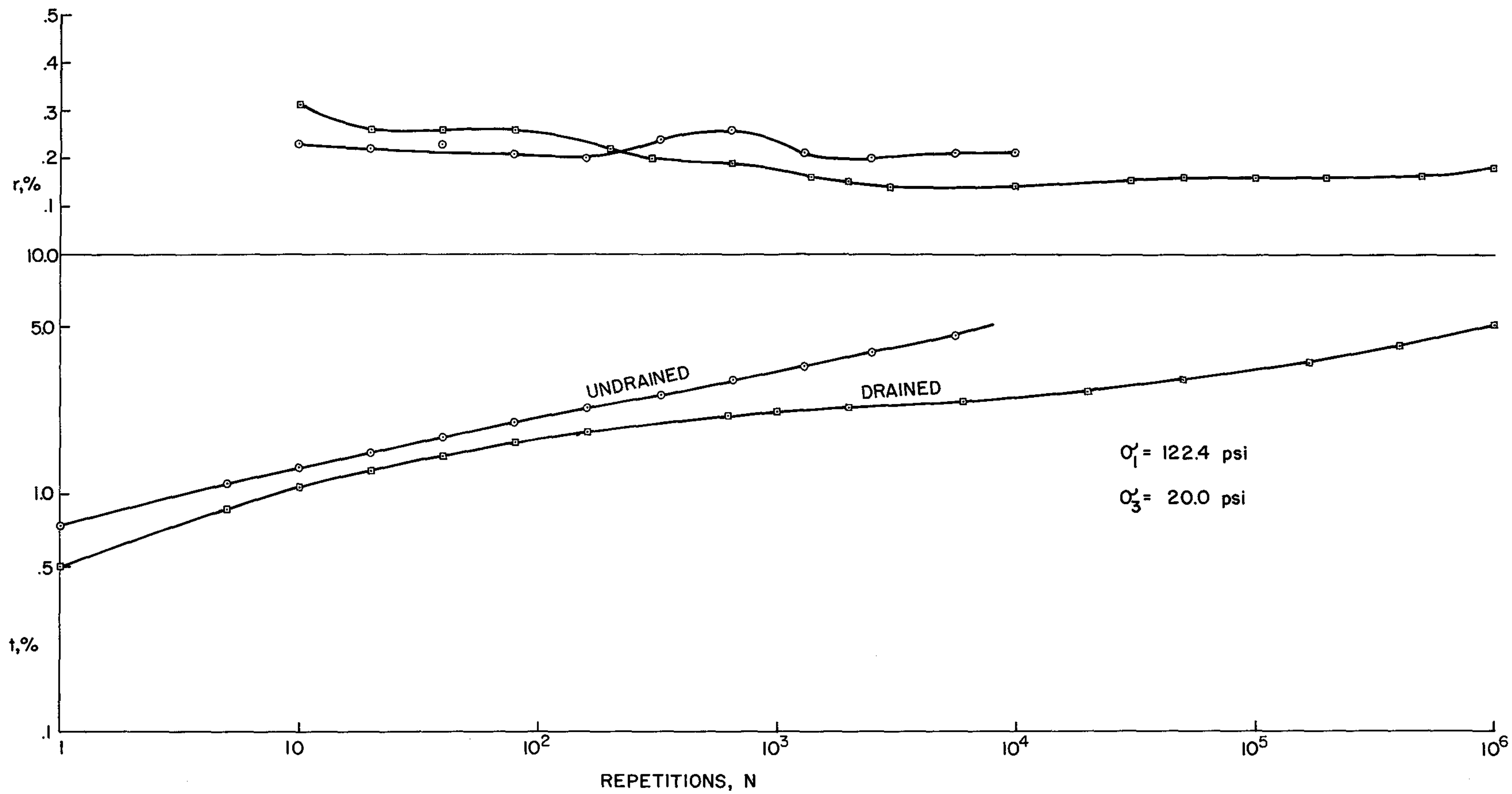

FIGURE 5.21 Angular fine aggregate. Influence of specimen drainage on total and rebound strain.

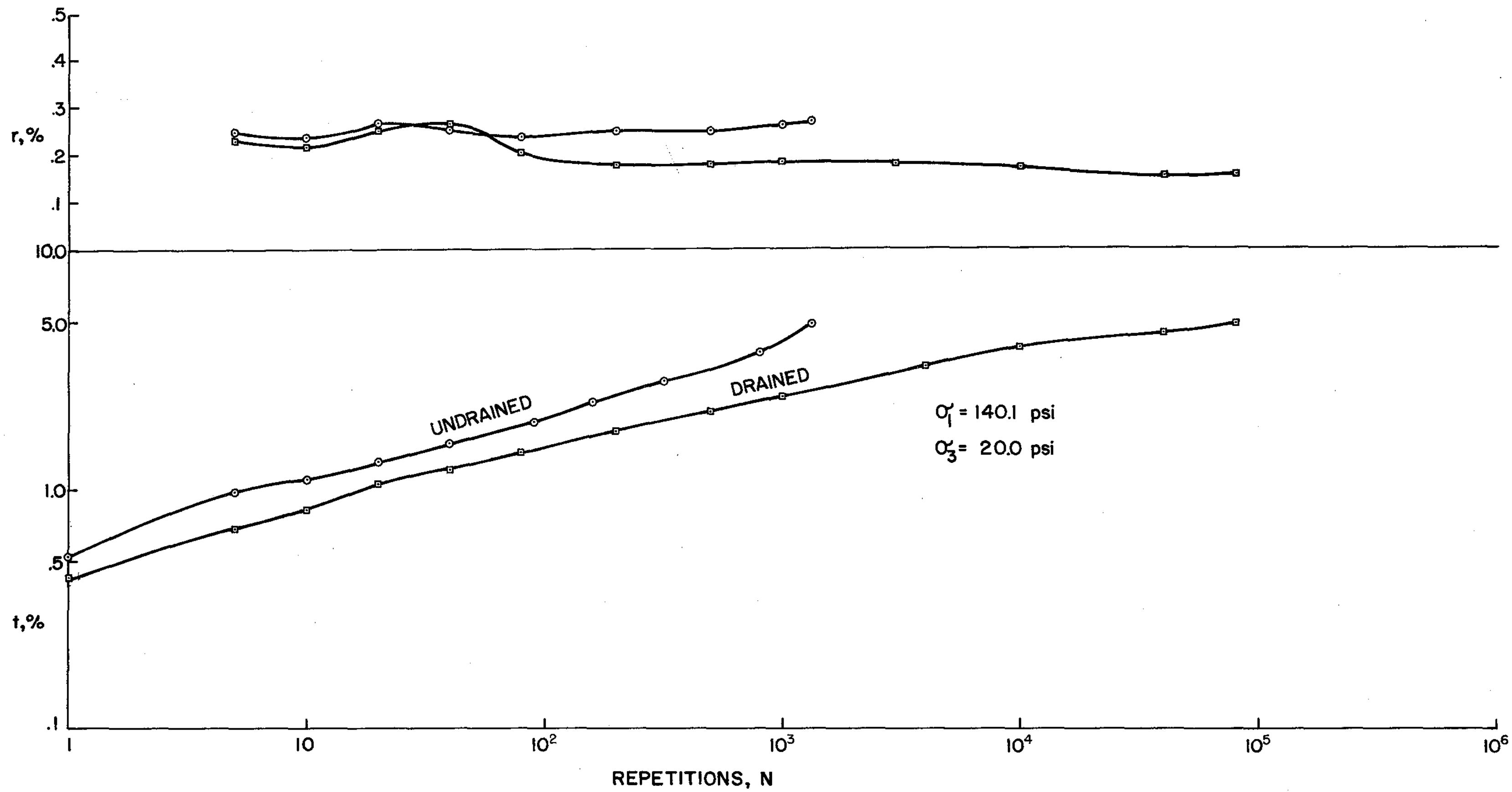

FIGURE 5.22 Angular coarse aggregate. Influence of specimen drainage on total and rebound strain.

Effect of Compactive Effort on Repetitive Stress Characteristics

In this comparison the angular medium aggregate was used. Two specimens each at 50 blows per layer and two at 125 blows per layer were tested at the same repeated stress and confining pressure. Unfortunately, one specimen at each compactive effort developed membrane leaks during loading and their results are not considered reliable. The test results for the two reliable specimens are shown in Figure 5.23.

Increasing the unit weight approximately three pcf had astonishing influence on the strain characteristics of the materials. For example, it reduced the rebound strain about 50 percent. The exact influence on the life of the material cannot be assessed simply because the test on the highly compacted specimen was terminated at 1,400,000 repetitions at which time it had achieved only 2.6 percent total strain. The specimen compacted at 50 blows per layer reached five percent total strain at 120,000 repetitions.

Analysis of Degradation Under Repetitive Loading

To check the amount of degradation that occurred under repetitive loading, the gradation analyses of repetitively loaded specimens were compared to control specimens of the same aggregate which were compacted but unstressed. The latter are listed in Appendix C as "Compaction" specimens. In general, the degradation was so minor that detailed analysis, e.g., relating applied stress magnitude and number of repetitions to degradation, was not required. Instead the mean gradations of the repetitively stressed specimens were compared

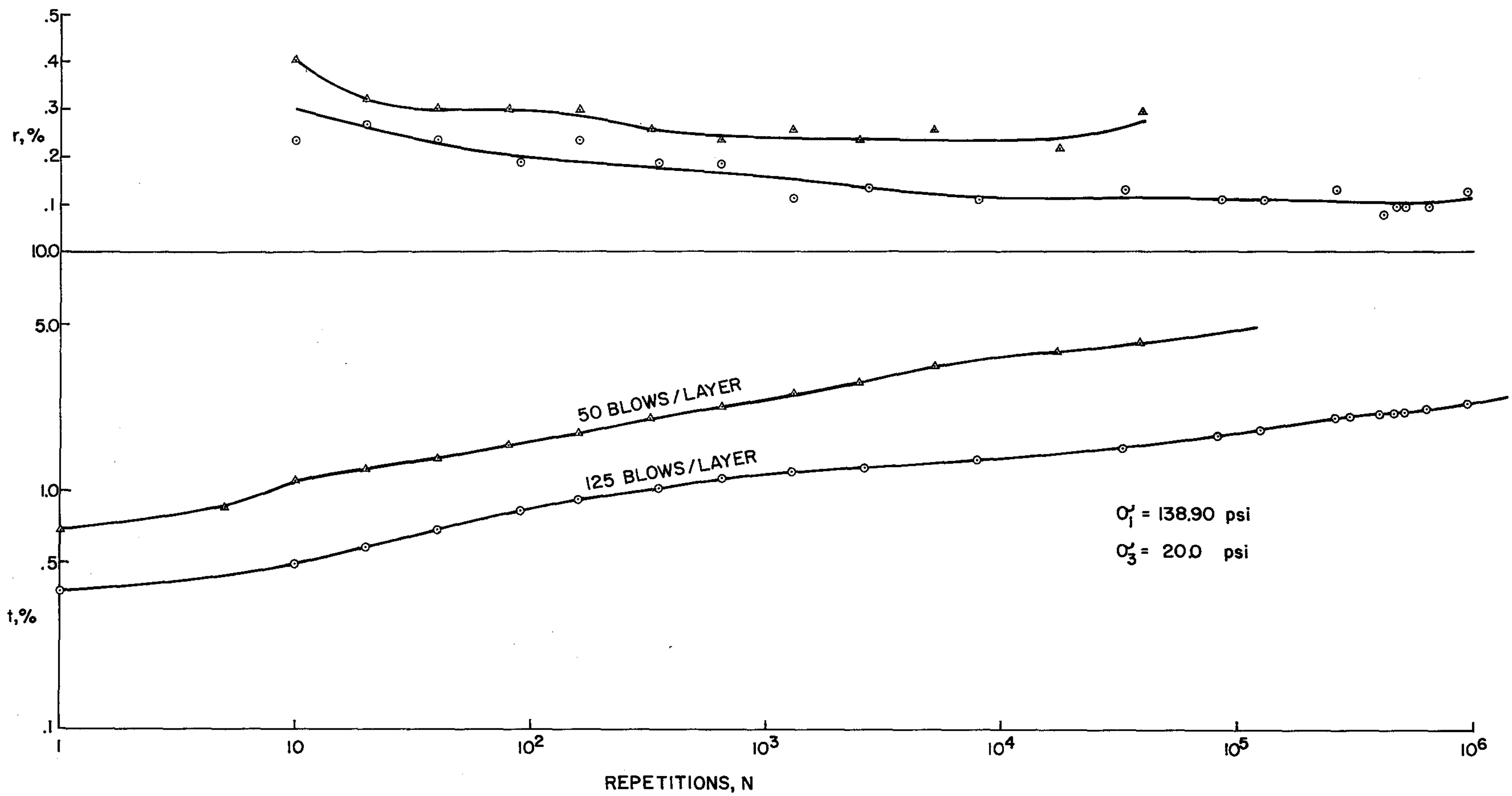

FIGURE 5.23 Angular medium aggregate. Influence of compactive effort on total and rebound strain.

to the mean of the control specimens. This information is presented in Figures 5.24, 5.25 and 5.26.

The soft material suffered the only significant degradation. This occurred in the No. 10 and finer sizes and averaged about three percent.

While the mean gradations indicated little degradation, it will be noted from the figures in Appendix C that for the stressed specimens the range of gradations on any one sieve was occasionally large -- about seven percent in one case. However, this occurred on a coarse sieve and may have resulted from particle breakup during compaction. For the fine sizes (<No. 4 sieve) the maximum range was roughly two percent.

In some cases the mean gradations for the stressed specimens fell below those of the control specimens, which indicated that experimental errors in recombining sizes to produce specimens was of greater significance than degradation due to stressing.

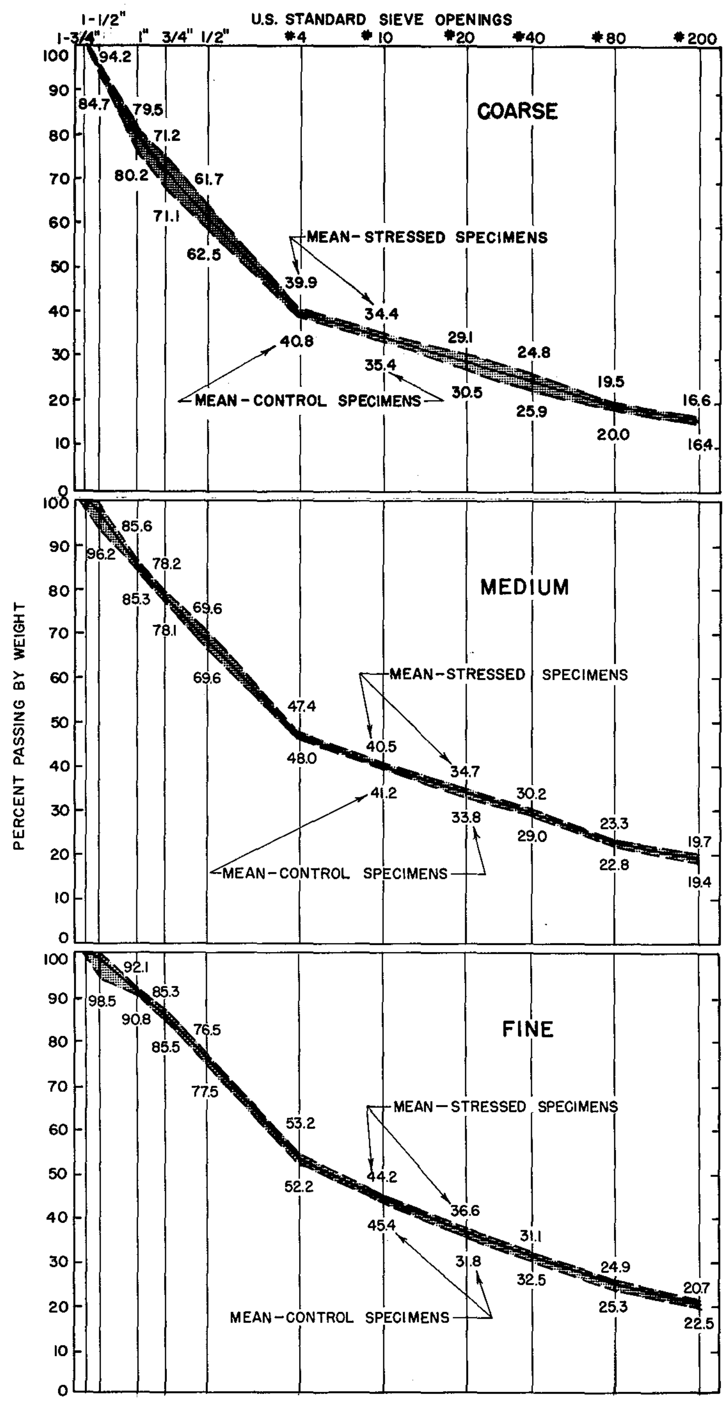

FIGURE 5.24 Rounded aggregate. Degradation produced by repetitive loading. Band and solid line show range and mean, respectively, of repetitively stressed specimens.

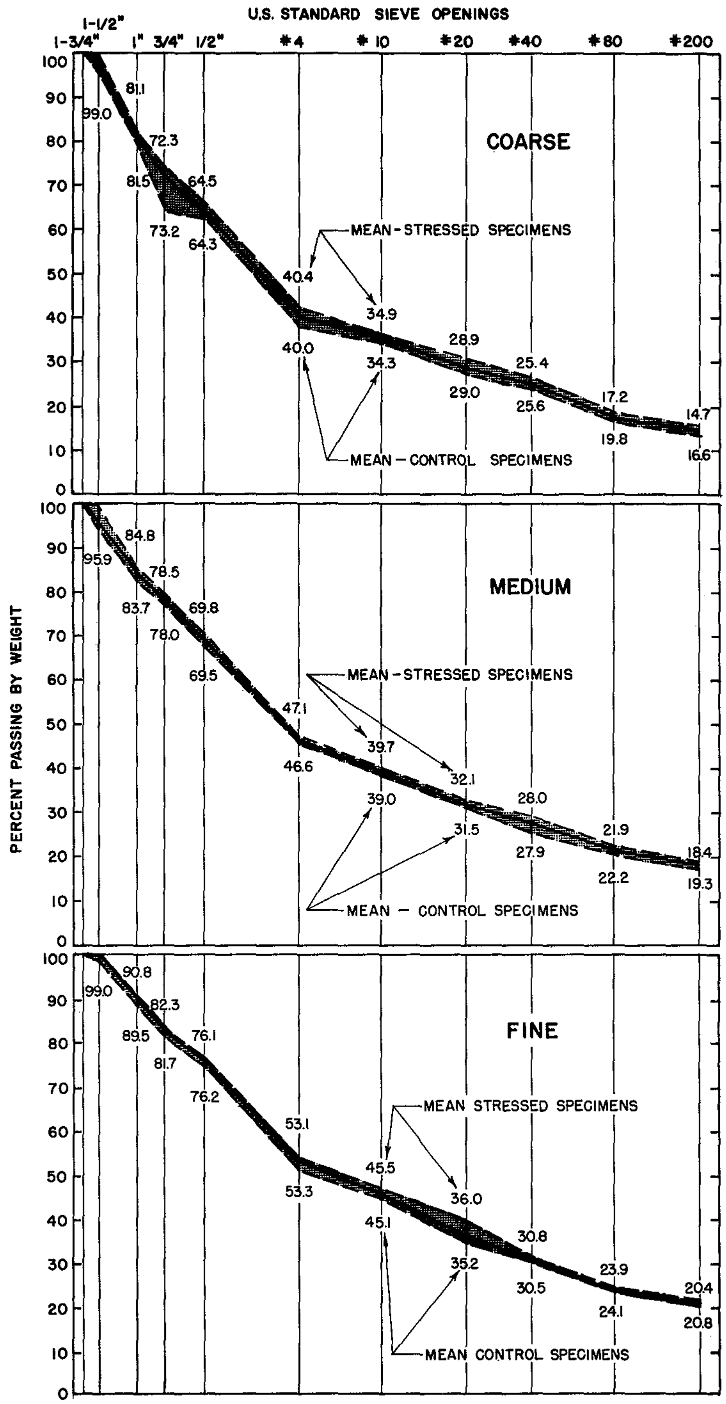

FIGURE 5.25 Angular aggregate. Degradation produced by repetitive loading. Band and solid line show range and mean, respectively, of repetitively stressed specimens.

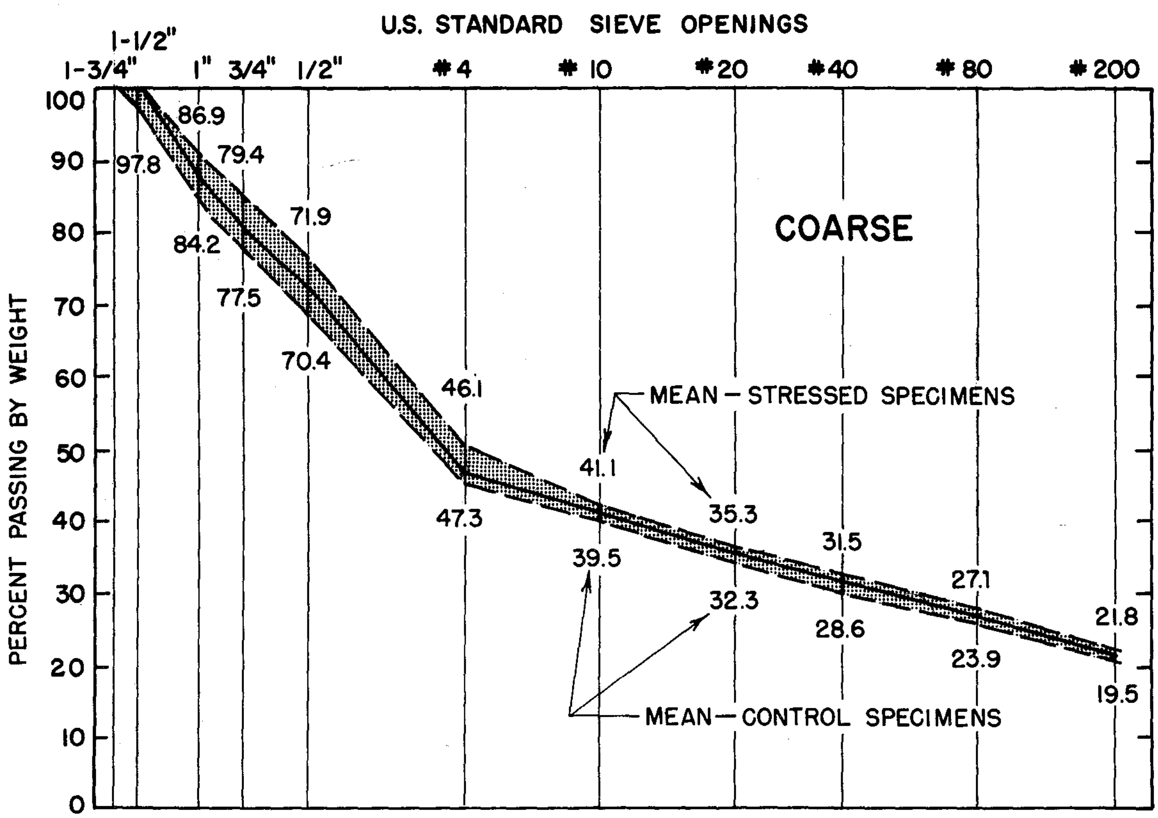

FIGURE 5.26 Soft aggregate. Degradation produced by repetitive loading. Band and solid line show range and mean, respectively, of repetitively stressed specimens.

CHAPTER VI

DISCUSSION OF EQUIPMENT, TECHNIQUES AND TEST RESULTS

General

Some idea of the effort expended in this research can be grasped by realizing that approximately nine cubic yards of material were utilized in the experimental phase; of this, about five and one-half cubic yards were crushed in the laboratory. All material was dried, separated into various sizes and later recombined in the correct proportions.

Information on previous repetitive loading research on granular mateials is so sparse that this research must still be considered preliminary in nature. The development and operation of the equipment, the development of the test procedures, and many other innovations contributed so significantly to the complexity of the analysis that it seems appropriate to discuss them first.

Test Equipment

Repetitive loading equipment. At the time the repetitive loading apparatus was developed no similar equipment for handling large diameter specimens of granular materials was available. Considering its initial cost, it operated well. However, the equipment had certain limitations, most important of which was its inability to consistently apply the correct load during the initial loading phases. The pumpdown procedure described on page 93 helped, but it never completely eliminated the problem. On lightly stressed specimens, the correct loads could usually be obtained in two to five repetitions, but on some highly stressed specimens, so much manipulation of the pressure regulators was required that the specimen often failed before a significant number of correct loads could be applied. Because the specimens are so susceptible to deformations during their early life, even slight deviations in the desired load could influence their behavior. This is a manifestation of the stress history effect discussed in Chapter II.

If the repeated load varied during an unattended period this also affected the specimen behavior. Referring to Figure 5.5 , it is obvious that changes of one or two psi in the repetitive stress could change the number of repetitions required to produce a strain of one percent by several thousand.

The shape of the loading pattern applied to the specimen also influences its deformation characteristics. To reproduce a particular loading pattern required almost infinite patience in adjusting pressure regulator and flow control valves. Actual deviations in load patterns between replicate specimens may have been partly responsible for replication error.

Commercial repetitive loading machines with automatic load programmers are now available. These devices should eliminate the problems discussed above.

<u>Transducers and recording equipment</u>. The electrical measuring equipment (transducers) and associated recording equipment operated quite satisfactorily. But there is still room for improvement in the method of making measurements. The deformation transducers undoubtedly provided accurate readings of the piston movement with respect to the

remainder of the triaxial cell, but they may not be an accurate indicator of specimen strain. It is generally accepted that stress concentrations occur at the ends of triaxial specimens which result in inherent errors in strain measurements (5)(34). However, optical tracking devices which can be attached directly to the specimen are now available. Readings from these devices, measuring the strain in the central one-third of the specimen, should eliminate much of the error contributed by end effect stresses. This method would also eliminate the measurement of deformation within the triaxial cells which is reflected as specimen deformation.

Triaxial cells. The triaxial cells, other than the aforementioned deformations which occurred under loading, performed adequately. But the heads of the cells can be modified so that simple independent control of the vertical and lateral pressures can be obtained. As mentioned in Chapter II, this method of repeated loading i.e., simultaneous changes in the lateral and vertical pressures, may be a better approximation of the actual mode of pressure distribution in the pavement structure.

Volume change devices. The inability of these devices to quickly and accurately respond to volume changes was caused by low specimen permeability and perhaps by stress concentrations at the specimen ends. Whatever the reasons, they seriously hampered quantitative determination of the relationship between specimen behavior and volume change. Perhaps external measurements of volume change would be helpful, even though they would provide only the changes in wet unit weights. Wolfskill (45) made both types of measurements and reported significant deviations between externally and internally measured volumes, even at low loading rates (0.05 in./min.).

Test Procedures

Experiment design. Probably the one factor which hampered the analysis of the results more than any other was the use of the variable, X, in the experiment design. Basically, X might be best described as a factor of safety based on the static shear strength of the soil. As discussed in Chapter IV it was desired to stress the specimens so as to obtain a wide range in the lives of the repetitive specimens. It was believed that the orthogonal factorial design using X would provide a ready means of determining the desired stresses as well as analyzing the test results. However, this did not prove to be the case because of the poor relationship of static shear strength to repetitive loading behavior. To eliminate the problem of comparing several aggregates all subjected to different stresses, it seems much more desirable to apply the same set of stresses to all materials and evaluate their relative behavior under identical stresses. The author suggests an experiment design with stresses as shown in Table VI.1. It is believed that these are representative of the in-situ stresses in the upper layers, and they are based somewhat on the behavior of several materials which have been subjected to repetitive triaxial testing.

Preparation of specimens. The method of forming specimens for the repetitive triaxial tests was carefully studied prior to the repetitive loading program, and the method described in Chapter III initially seemed satisfactory. However, as the testing progressed it was evident

TABLE VI[illegible]

Suggested [illegible]epetitive Stresses for Future Research Materials

$_1$, psi	σ_1, psi		
5	30	45	60
20	60	90	120
35	90	135	180

Note: At least four random replicate specimens should also be [illegible]sted.

that some specimens which had virtually identical unit weights performed quite differently under the same repetitive stresses. Obviously some other factor (or factors) controlled their behavior. It is believed that manual manipulation of the specimens, particularly the hand finishing operation after compaction, was responsible for much of this deviation. A method of compaction must be developed before replication error can be reduced. This method should not only be capable of achieving replicate unit weights between specimens, but it should also achieve uniformity of unit weight throughout the entire specimen. To broaden the scope of future testing, the compaction equipment should also be capable of reproducing field unit weights and moisture contents, something which the present impact method is capable of doing only within a limited range of values.

The research material. The parent materials used in this project satisfied the desired requirements, but they also had certain properties which created difficulties in specimen preparation and in the analysis of the results. For example, the rounded material contained a large portion of fines which slaked in water. Thus the initial separation, which it was necessary to accomplish by dry sieving due to the large quantity of material being handled, left the researcher at the mercy of material variability. It was difficult to recombine the sizes to produce the desired gradations, and as a result it was necessary to be content with the gradations which occurred. To then exactly reproduce these gradations with the angular and the soft materials was impossible.

While the angular material was constitutively similar to the rounded in most respects, the fine particles, which significantly influence the behavior of a granular material, were not the same. This is evident in the plasticity characteristics of the rounded and angular materials. Thus, behavioral differences between the rounded and angular materials cannot be attributed solely to angularity.

One other factor which is considered to be extremely important, particularly insofar as replication error is concerned, is the use of 1-1/2-inch maximum size materials in the 6-inch diameter triaxial cell. The gradation should be changed in future tests so the maximum size is roughly 3/4-to-1-inch. Again, changing the gradation in the upper size ranges will have little effect on the relationship of laboratory to field behavior, since it is primarily the finer constituents that are responsible for the variations in the behavior of granular materials.

Restatement of Objectives

On page 9, the particular objectives of this research were stated. To facilitate the ensuing discussion of test results, the objectives are restated below:

a. Obtain the relationship between the repeated loads and accumulated deformation for each aggregate.

b. Compare the behavior of each aggregate under repetitive loading to the other aggregates, to its triaxial classification and, if possible, to its performance in actual roadways.

c. Explain the behavior of the aggregates based on existing soil mechanics knowledge and thus extend the information gained to include other aggregates.

In the following discussions, each objective listed above and the results of the investigation on this objective will be discussed separately.

Relationship Between Repeated Loads and Accumulated Deformation for Aggregates

Figures 5.5 through 5.11 present this information expressed quantitatively in terms of the repeated vertical stress, the confining pressure, the number of repetitions and the total strain. Subject to the limitations of experimental errors discussed previously in this chapter and in Chapter V, this appears to be a sound useable relationship for these materials.

As discussed in Chapter II, Seed et al. (32), and Khuri and Buchanan (26) provided limited quantitative information of a similar nature, but to the author's knowledge this represents the first time such extensive information of this type has been developed.

Little can be discussed about these relationships. It is obvious that these curves by themselves are of limited use at present. Actual use of the relationships shown in Figures 5.5 through 5.11 would require knowledge of the stresses which the materials would be subjected to in the roadway.

The inability to develop a sound physical mathematical model to express these results was disappointing although it does not mean that such a model does not exist. To the contrary, additional study may well provide the model which is sought. But a truly physical model would require that all of the basic properties of the materials be taken into account. This would include plasticity, particle shape and hardness, and the other basic properties discussed in Chapter II. It would also require inclusion of the environmental conditions to which the aggregates were subjected.

Comparison of Behavior and Ranking of the Aggregates Based on Total Strain Characteristics

To satisfy one of the principal objectives of this research, it was desired to rate the aggregates based on total strain behavior under repetitive loading and to correlate this behavior to their Texas triaxial classifications.

For comparing relative behavior under repetitive loading, Analysis A, (graphical method) was used. Analysis B was helpful from the standpoint that the standard deviations indicated whether apparent differences were really significant. A convenient way of comparing the aggregates was to overlay transparencies of Figure 5.5 through 5.11. It was also helpful to plot the data in the form shown in Figure 6.1. Only one of these plots is contained herein, but similar plots were used for all aggregates. Even with these aids, ranking the aggregates was a difficult process because of the many interactions that occurred between the variables. For example, an aggregate would rank best up to a certain amount of repetitions and then fall below another aggregate. Also, it might rank best at 20 and 3 psi, but not at 11.5 psi confining pressure, and the same might occur at various values of strain.

The behavior of the material was examined three ways. First, each material was studied to determine which gradation was best for that material. Then, similar gradations for the three materials were

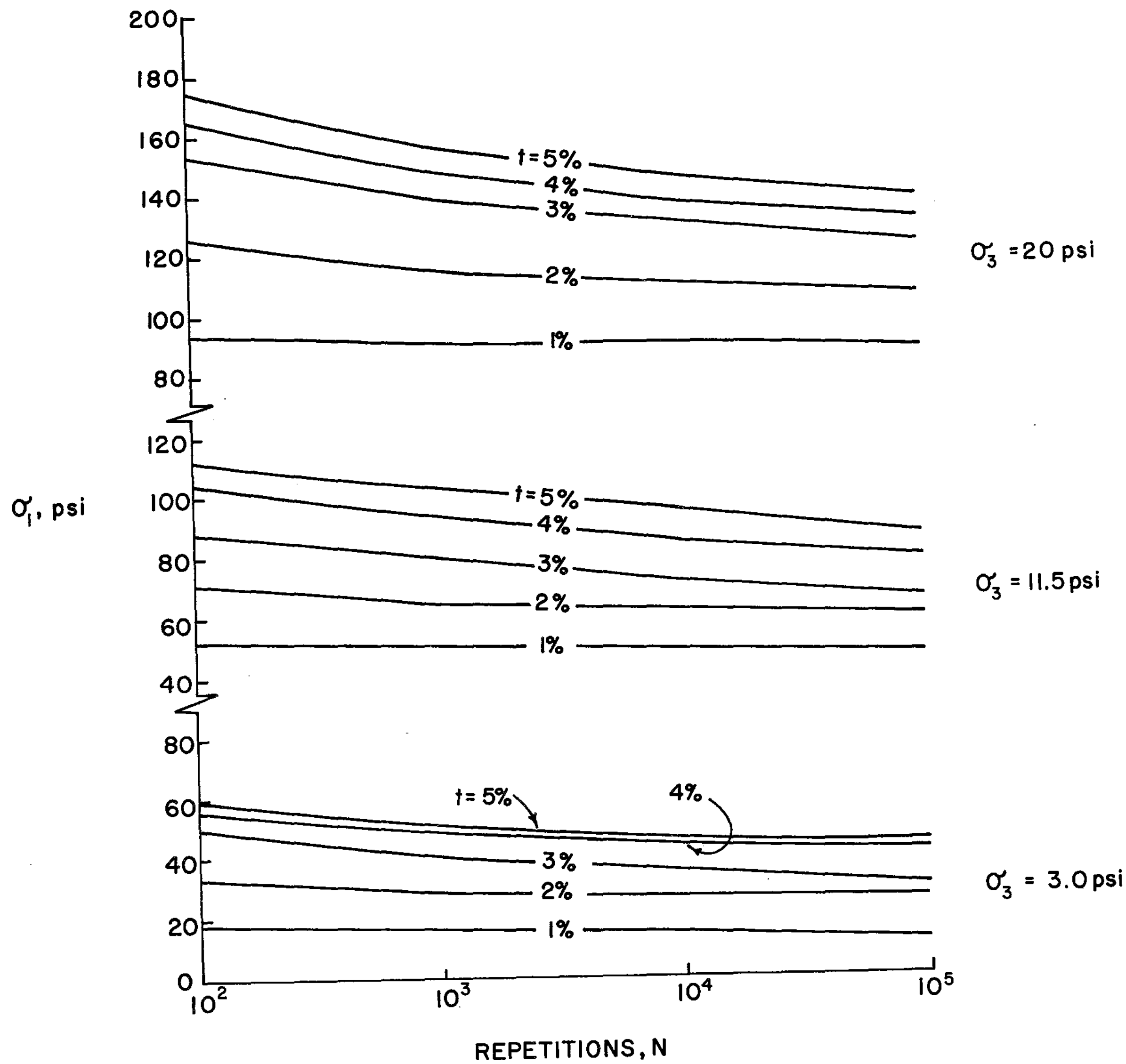

FIGURE 6.1 Rounded medium aggregate. Relationship between applied stresses, repetitions and total strain (data obtained from Figure 5.6).

compared to determine the relative influence of particle shape and hardness on the behavior of the materials. Finally, an overall comparison was made of all aggregates. The latter was particularly difficult to accomplish because of the variations in behavior mentioned in the previous paragraph.

While examining these comparisons, the reader should recall that the soft material degraded during compaction to the extent that it was finer than the fine gradations of the other two materials even though it is referred to as being coarse.

Influence of gradation.

a. Rounded material. The medium gradation of this material generally performed best. The coarse gradation appeared somewhat superior to the fine, but the difference was within the limits of experimental error. As an example of the influence of the different variables, the medium gradation was superior to the coarse at 20 and 11.5 psi confining pressures, but at 3 psi it was better only at the higher strains; it was superior to the fine gradation at 20 and 3 psi, but there was little difference between the two at 11.5 psi.

b. Angular material. Comparing the gradations of this material was difficult because of the changes that occurred with the number of repetitions. For example, up to and including 1,000 repetitions, the coarse gradation was always superior, but by 10,000 repetitions the medium and fine gradations performed better. One coarse gradation specimen lasted more than 100,000 repetitions, which did not provide sufficient data for comparison at all confining pressures. This does not mean that the coarse gradation cannot withstand 100,000 or more repetitions; it would if the applied repetitive stresses were low enough, but the material would be inferior to the medium and coarse gradations at this number of repetitions.

The distinction between the medium and fine gradations of the angular material was slight and differences were usually within the range of experimental error. In general, the medium gradation appeared to be slightly better than the fine, particularly at the high numbers of repetitions.

Influence of material shape and hardness.

a. Coarse gradations. The angular material generally rated best followed by the soft, and then the rounded materials. A notable exception was at 100,000 repetitions where the curves for the angular material were incomplete. It is assumed that the angular material would rank last at this number of repetitions with the rounded being best followed by the soft material.

The initial high shear resistance of the angular and soft materials was manifested in their ability to resist deformations under high stresses, but as the number of repetitions increased, this capability was lost.

b. Medium and fine gradations. The angular material was better than the corresponding gradation of rounded material, and the same was true for the fine gradations of the two materials.

Overall comparison of aggregates. To rank all seven aggregates, it was necessary to average their overall performances leaving the number of repetitions as the only variable. For example, if a particular material consistently ranked best at two out of the three confining pressures considered and at three out of the five values of strain, then it was given the best rank for the number of repetitions being considered. It was necessary to exercise some personal judgment in cases of extrapolation or when results were so close that experimental error could easily have affected the rankings. Table VI.2 gives the rank of each aggregate according to repetitive triaxial results and also by the Texas triaxial classification.

At 100 repetitions the ranking of the materials was virtually identical to their ranking determined by Texas triaxial tests. But as the number of repetitions increased, the deviation between repetitive triaxial and Texas triaxial rankings gradually increased. In particular, the rankings of the coarse gradation of the angular and soft materials dropped significantly as the number of repetitions increased.

While Table VI.2 provides useful qualitative information, it is

TABLE VI.2

Ranking of Materials Based on Method Indicated

Ranking Method / Repetitions / Rank	Total Strain from Repetitive Triaxial Tests				Texas Triaxial Tests	
	100	1000	10,000	100,000	----	Class
1st	Angular, coarse	Angular, coarse	Angular, medium	Angular, medium	Angular, coarse	1
2nd	Angular, medium	Angular, fine	Angular, fine	Angular, fine	Angular, medium	1
3rd	Angular, fine	Angular, medium	Angular, coarse	Rounded, medium	Angular, fine	1
4th	Soft, coarse	Rounded, medium	Rounded, medium	Rounded, fine	Soft, coarse	2.1
5th	Rounded, medium	Soft, coarse	Soft, coarse	Rounded, coarse	Rounded, coarse	2.7
6th	Rounded, coarse	Rounded, coarse	Rounded, coarse	Soft, coarse	Rounded, medium	2.8
7th	Rounded, fine	Rounded, fine	Rounded, fine	Angular, coarse	Rounded fine	3.0

not a quantitative measure of performance. However, from the information given in Figures 5.5 through 5.11, it is possible to construct Mohr's circles and envelopes at various values of strains and repetitions. For sake of brevity, an example plot of this type is shown in Figure 6.2 for only one material. From the resulting envelopes (which should not be construed as failure envelopes) a triaxial classification or ranking can be obtained which is quantitative.

Table VI. 3 contains this information for five percent total strain at various repetitions. The rankings should not necessarily correspond to those presented in Table VI.2 for two reasons: they are based on only one value of strain, whereas the rankings in Table VI.2 are an overall evaluation at all strains. If a similar table were presented for strains of one percent the rankings would not correspond to those shown in either table. In addition, each material was classified at the point where its envelope was at the lowest position on the Texas triaxial classification chart (see page 58 for additional explanation on this matter); thus, an aggregate might perform well at a low confining pressure, but still receive a low classification if it did not perform well at higher confining pressures.

Based on Table VI.3, the three gradations of the angular material performed as follows: the medium gradation was best, the fine gradation was next, and the coarse gradation was the poorest performer. For the rounded material the medium gradation was best, and the coarse was next, followed by the fine gradation.

As the number of repetitions increased, the steady decrease in

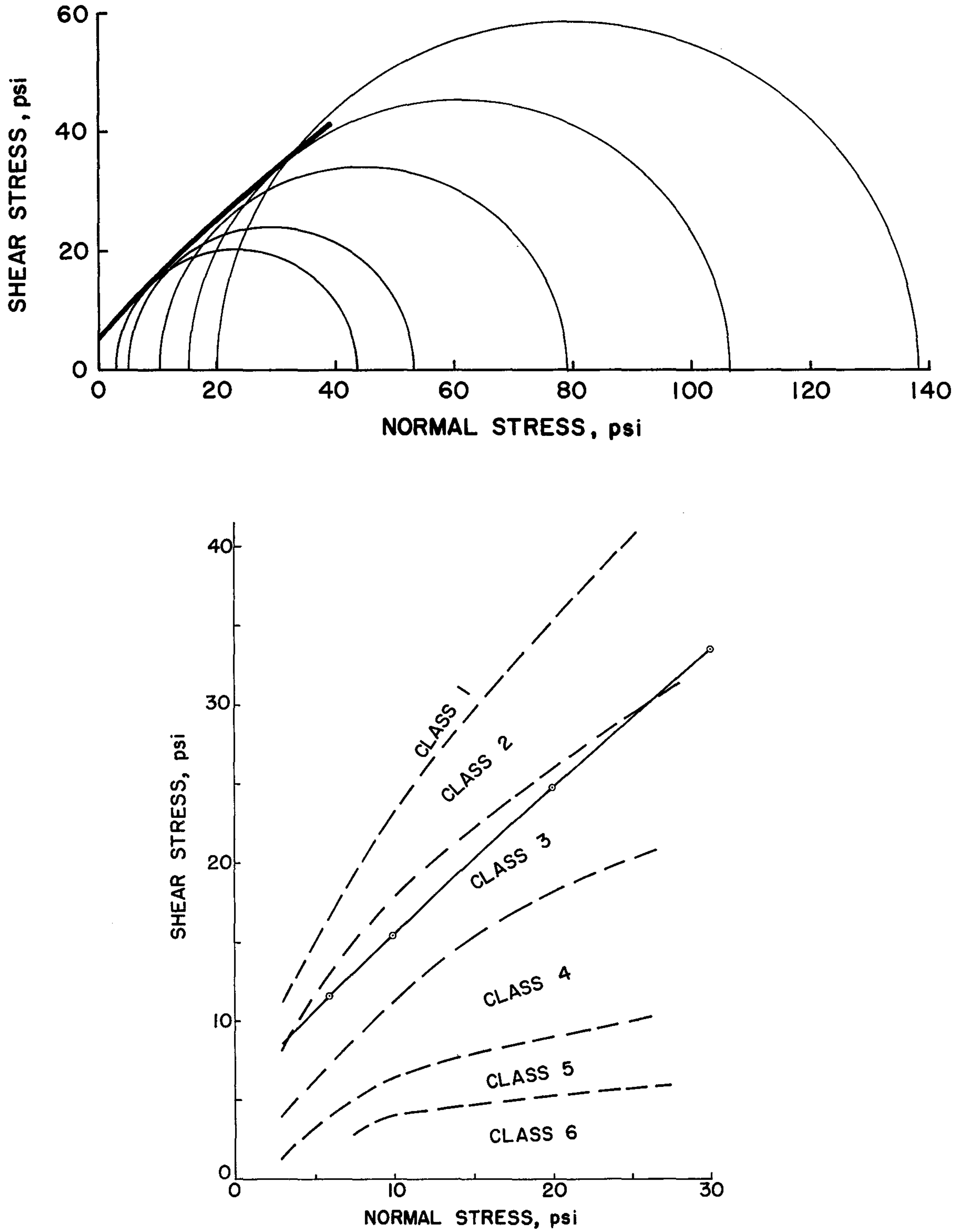

FIGURE 6.2 Rounded medium aggregate. Mohr's envelope and triaxial classification for stresses required to obtain 5 percent total strain at 100,000 repetitions.

TABLE VI.3

Ranking of Materials Based on
Texas Triaxial Method of Classification

Ranking Method / Repetitions / Rank	Repetitive Triaxial Tests at Five Percent Total Strain								Tex. Triaxial Test	
	100		1000		10,000		100,000			
	----	Class	----	Class	----	Class	----	Class	----	Class
1st	Angular medium	2.1	Angular medium	2.6	Angular medium	2.8	Angular medium	3.0	Angular coarse	1
2nd	Angular fine	2.2	Angular fine	2.7	Angular fine	3.0	Rounded medium	3.3	Angular medium	1
3rd	Angular coarse	2.2	Angular coarse	2.8	Rounded medium	3.3	Angular fine	3.4	Angular fine	1
4th	Soft coarse	2.6	Rounded medium	3.1	Soft coarse	3.4	Soft coarse	3.5	Soft coarse	2.1
5th	Rounded medium	2.8	Rounded coarse	3.3	Angular coarse	3.5	Rounded coarse	3.7	Rounded coarse	2.7
6th	Rounded coarse	3.0	Soft coarse	3.4	Rounded coarse	3.5	Rounded fine	3.8	Rounded medium	2.8
7th	Rounded fine	3.0	Rounded fine	3.4	Rounded fine	3.5	Angular coarse	---	Rounded fine	3.0

classification of the angular coarse aggregate and the steady increase of the rounded medium aggregate were quite noticeable in this method of ranking. As opposed to the results in Table VI.2, the soft aggregate generally retained its position in the rankings. Another unusual fact was the small range of triaxial classification -- from Class 3.0 to 3.8 at 100,000 repetitions -- which occurred, whereas the original Texas triaxial classifications ranged from Class 1 to 3.0.

In summary, it was obvious that rating the aggregates according to total strain is a difficult process. The different aggregates have certain ranges of stress, strains, and repetitions where they perform best. A definite ranking can be made only by knowing the actual stresses which would be applied to the aggregates, and by establishing a limiting strain.

One interesting point of behavior was the rapidity with which the angular materials reached low values of strain, say one to two percent. Some of the possible reasons for this will be discussed later.

Comparison of the Behavior of the Aggregates Based on Rebound Strains, and Changes in Unit Weights and Degrees of Saturation

As with total strains, rebound strains shown in Figure 5.18 through 5.20 were examined to determine which gradation was best for each material, then similar gradations were compared to determine the influence of particle shape and hardness on the behavior of the materials.

Influence of gradation on rebound strain.

a. Rounded material. Generally, the lowest rebound strains were exhibited by the coarse gradation. The fine appeared to have some advantage over the medium gradation.

b. Angular material. The fine gradation had the lowest rebound strains; however, it also had the smallest applied stresses, particularly at 20 psi confining pressure. Rebound strains in the medium and coarse gradations varied considerably with the confining pressure and number of repetitions, so they would have to be ranked about equal from an overall standpoint.

Influence of particle shape and hardness on rebound strain.

a. Coarse gradations. The rounded material had the lowest overall rebound strains. It was difficult to make a valid comparison between the angular and soft materials because the repetitive stresses on the soft material were much lower. In general, the soft material might be slightly superior at larger numbers of repetitions except at 3 psi confining pressure where the angular material was definitely best.

b. Medium gradations. The rounded material generally had lower rebound strains than the angular, particularly at larger numbers of repetitions. The angular material was superior only at 3 psi confining pressure.

c. Fine gradations. Again the rounded material was generally superior to the angular except at 3 psi confining pressure.

Overall, it appears that the rounded coarse gradation performed best, and the rounded material in all cases appeared to be superior to the other materials. This is, of course, somewhat in contradiction to the behavior of the materials based on total strain.

Changes in unit weights and degrees of saturation. While examining this information the reader should be aware that the values at 100 repetitions, and in some cases at 1,000 repetitions, may be somewhat in error due to lag in the volume change readings. A review of the plots for all specimens revealed a very dynamic situation during the initial loading phases; many specimens underwent an immediate increase in unit weights, but by 100 repetitions the unit weight was less than the initial unit weight. However, the trends which are reported herein are believed to be realistic.

The changes in unit weight were obviously influenced by the confining pressures. At confining pressures of 3 psi, the changes were small and in some instances a decrease in unit weight (dilatancy) was observed. Dilatancy was prevalent in the rounded and soft materials.

At confining pressures of 11.5 and 20 psi all aggregates densified under repeated loading. The greatest densification, and by far the most rapid rate of densification, was exhibited by the angular materials. In particular, the coarse gradation exhibited an immediate increase (after 100 repetitions) of about 3 pcf and by 100,000 repetitions the 20 psi specimen had increased its unit weight slightly more than 7 pcf. The unit weight of the soft coarse material also increased roughly 4 pcf, even though it was subjected to somewhat smaller stresses than the angular material.

The rounded materials exhibited rather small, gradual increases in unit weight. The medium gradation -- which had superior characteristics insofar as total strain was concerned -- densified more than the other rounded materials.

Changes in degrees of saturation generally followed the same pattern as the unit weight changes. When the unit weight decreased -- as in some 3 psi specimens -- the degree of saturation also decreased. This is understandable since it was impossible for the specimens to obtain water from the volume change device when the volume of the voids increased during dilatancy. Again, the angular coarse aggregate exhibited the highest increase in saturation as well as the fastest rate of increase. The rounded medium aggregate experienced almost as much change but it was gradual throughout the specimen's life. Overall, the angular materials exhibited the greatest increase in saturation.

Comparison of Field and Laboratory Behavior of the Research Aggregates

Although one of the primary research objectives was to compare the laboratory repetitive loading behavior with the field behavior of the research materials, it can hardly be accomplished with any degree of accuracy. The angular material, which was manufactured from the rounded, has never been used in practice. The soft material was purposely selected so it was softer than that actually used in the roadway. Research gradations of the rounded materials do not correspond with those used in the roadway, nor has this material been used under traffic conditions as severe as those which the soft material was subjected to. However, it is generally known that the rounded material performs well under the conditions for which it is used and that a somewhat better grade of soft material was unsatisfactory in actual use. If these two materials are considered, a qualitative comparison of behavior can be made. According to Table VI.2 all gradations of the rounded material performed better than the soft according to total strain characteristics. The difference is not quite as pronounced in Table VI.3 where the gradation of the rounded material appears to be of greater importance; only the medium gradation was superior to the soft material.

In general the rebound strains of all gradations of the rounded material were superior to those exhibited by the soft material. They became increasingly superior as the number of repetitions increased. Although the differences in rebound strain between all materials are rather small, they are important insofar as their influence on the life of the roadway surfacing is concerned.

Taking all factors into account, it would appear that under laboratory repetitive stressing the rounded material is somewhat superior to the soft. This also appears to be true in practice, but extensive field testing is the only proper way to evaluate the relative performance of the two materials.

In this section, it is pertinent to discuss how close the repetitive triaxial test procedure came to reproducing field conditions.

That is, were the other environmental characteristics similar enough to those which exist in the field to extrapolate the repetitive triaxial results to field conditions. Insofar as moisture content and drainage are concerned this is a difficult question to answer.

Many highway engineers feel that roadway materials should be tested at saturation, which represents the worst possible condition which could exist during the pavement's life. This has given rise to such procedures as the four-day soaked CBR test. Conversely, the Texas Highway Department subjects test specimens to partial drying followed by capillary soaking as explained in Chapter III. This procedure brings granular materials close to, or somewhat less than, their molding moisture content. In addition, this technique may produce some time-dependent changes which affect the strength of the materials.

Figure 6.3 is a comparison of the laboratory optimum moisture content versus the in-situ moisture content for over forty various base course materials sampled from highways throughout the state of Texas. These materials were selected without respect to climatic conditions, drainage conditions, or the time of year when they were sampled. The results show that the materials existed at an average moisture content about two percent less than the optimum.

The specimens in this research were molded at their optimum moisture contents -- which corresponded to a degree of saturation of roughly 80 to 85 percent -- and then allowed to drain during repetitive loading if they could. The final moisture contents of

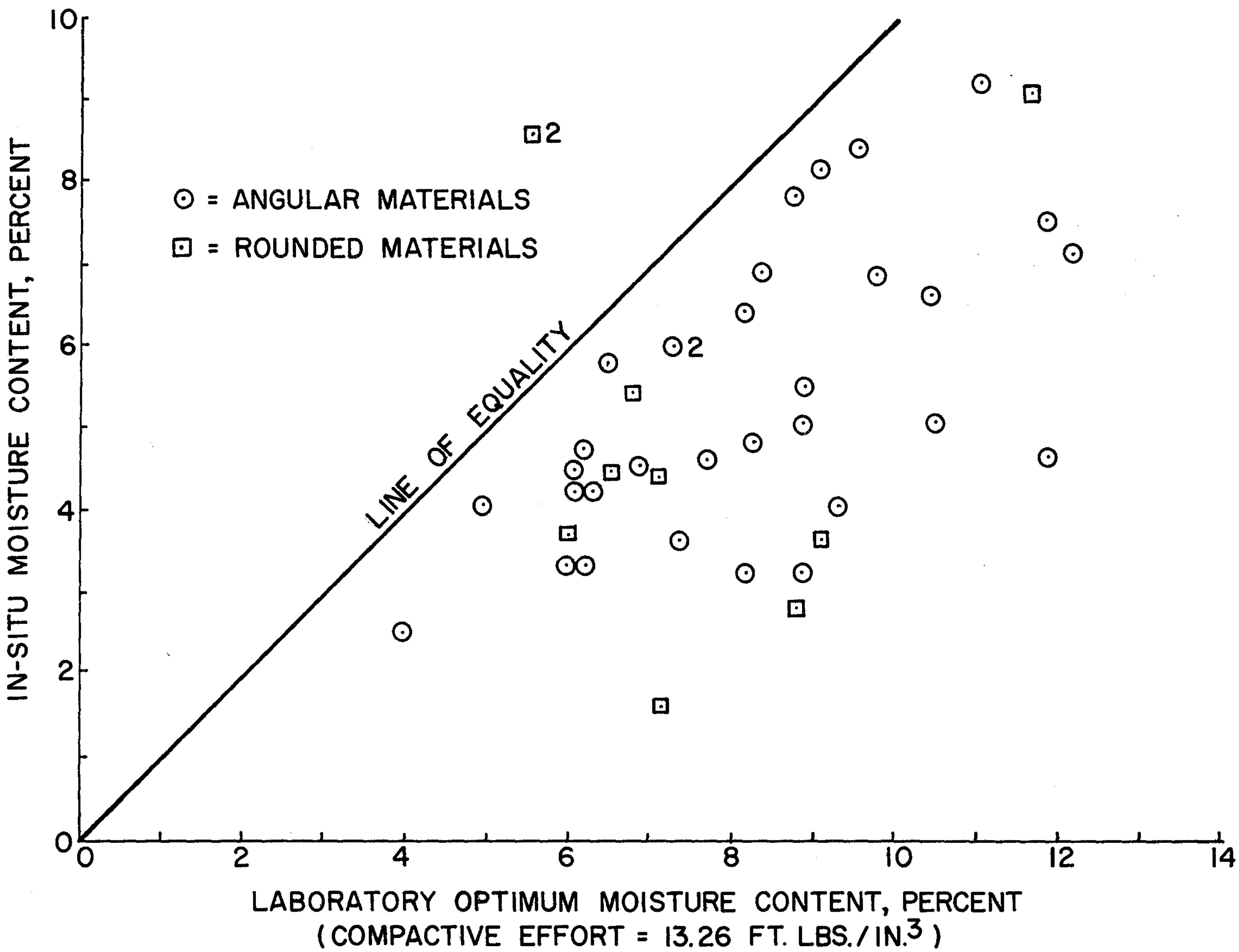

FIGURE 6.3 Comparison of in-situ to laboratory optimum moisture contents for various base courses sampled from Texas highways during 1963-64.

the long-lived repetitively loaded specimens in this research would fall well within the zone of field moisture contents shown in Figure 6.3. They got to this condition by drainage during repetitive loading, so it is highly probable that the moisture conditions for many of the repetitively loaded specimens were representative of in-situ moistures found in Texas.

The influence of drainage on the life of the aggregates was aptly shown in Figures 5.21-5.22. Obviously, if field moisture contents of base course materials are maintained at a high level due to poor drainage characteristics, the drained repetitive load test used in this research will not be applicable to field conditions.

The time-dependent changes which occur in the roadway with base course materials are another environmental factor which must be considered. It has been stated that base course materials -- particularly those consisting primarily of carbonates -- will re-cement over a period of time and in so doing, gain considerable strength. The repetitive triaxial specimens were loaded 24 hours after being molded. This time is slight compared to actual field conditions, however, present day roadway construction allows little time from the completion of base course construction to the application of traffic on the roadway. Certainly it is greater than 24 hours, but it is highly doubtful that it is sufficient time for the complex chemical reactions required to form cementing compounds such as calcium carbonate. Thus, it is not believed that the lack of a curing period prior to repetitive loading severely restricts transferring the results to roadway conditions.

Explanation of the Behavior of the Aggregates Based on Existing Soil Mechanics Knowledge

From the discussions in the previous sections, it is obvious that the Texas triaxial classification did not correlate well with the behavior of the aggregates under long-term laboratory repetitive loading. This is not a peculiar shortcoming of the Texas triaxial classification method; it probably exists with all static shear strength determinations, and it is due to the contrast in the two processes. It was pointed out in Chapter II that previous investigations had also shown that the static shear strength was not related to the ultimate performance of soils under repetitive loading. To properly determine the shear properties of materials, the environment must be characteristic of that which the materials will be subjected to. In the development of the Texas triaxial tests, an attempt was made to reproduce closely the actual roadway environmental conditions, but the influence of stress repetitions was not considered in the test. The research reported herein merely represents an attempt to include another environmental variable -- repetitions of load.

Undoubtedly several factors were responsible for the repetitive loading behavior of the various materials. The volume changes (or changes in unit weight) very definitely show differences in the mode of strain which occurred in the various materials. In fact, volume changes, degrees of saturation and their interactions cannot be generated similarly in the static process of shear testing. They have such a profound influence on the behavior of the materials that they deserve additional attention.

Casagrande (10) is generally credited with the first important work on volume change characteristics of cohesionless or granular materials. In general, when these materials are subjected to a shearing stress they expand (or dilate) if they are initially dense, or compact if initially loose. Each material has a particular void ratio -- termed its critical void ratio -- at which volume change will not occur under the action of the shearing stresses. Thus, loose and dense are relative terms with respect to the critical void ratio.

The critical void ratio for any material is not a constant but it varies with the confining pressure: it increases as the confining pressure decreases. Stated differently, at constant unit weight any of the aggregates used in this research may act as if they were in the dense state at low confining pressures and in the loose state at higher confining pressures. The strain occurring under this action would be referred to as shearing strain.

In addition, there is another volumetric change termed herein as the compressive strain -- which occurs independently of shear strain for certain circumstances. An example of this would be the volume decrease which occurs in one-dimensional compression as in a standard soil consolidation test.

It seems very likely that in repetitive triaxial tests, where a load less than the maximum shear strength is applied many times, volume decrease could occur due to compressive strain (or densification). Thus, either shear strain <u>or</u> compressive strain could result from repetitive loading. In fact, under certain loading conditions <u>both</u>

might occur simultaneously to cause failure of the specimen -- which in this research was arbitrarily defined as a total strain of five percent.

With this information, the behavior of the various materials when subjected to repetitive stresses can be described. For example, consider the respective changes in unit weight of the angular and rounded materials shown in Figure 5.18. Both were subjected to almost identical stresses at all three confining pressures. At a confining pressure of 3 psi, the rounded material experienced a decrease in unit weight and degree of saturation until failure occurred. Thus, even though many repetitions were required, the specimen exhibited primarily shear strain. At 11.5 and 20 psi confining pressures the specimens increased very little in unit weight -- roughly 1-2 pcf -- while attaining five percent strain.

On the other hand, at all confining pressures the angular material densified, or the primary strain was compressive. The angular material also experienced an immediate large increase in unit weight. It ultimately achieved a unit weight increase of 4-7 pcf while experiencing the same strain as the rounded material.

Under the action of repetitive loads at low confining pressures, the rounded material sheared and exhibited dilatancy. As the confining pressures increased the material underwent both shearing and compressive strain. The latter behavior was readily observed on the water manometer of the volume change device. During load application, suction occurred in the manometer line denoting dilatancy or shear strain. But after the load had been released, the manometer indicated that a slight increase in unit weight had occurred.

Under identical circumstances, the angular material experienced compressive strain. Observations of the water manometer indicated that dilatancy was not occurring during the application of repetitive loads.

Thus, the volume change characteristics of the rounded material indicate that it was apparently in a denser state with respect to its critical void ratio than the angular material.

The soft material responded in a manner somewhat similar to the angular material. In addition, however, the soft material also underwent some strain due to particle degradation.

These observations confirmed the thoughts of the author concerning the compaction characteristics of the materials as stated in Chapter III. With respect to the maximum unit weight which can be obtained with both materials, the angular material was in a relatively looser state than the rounded, although both were compacted with the same compactive effort. The data also explain how the various materials were able to reach a total strain of five percent with such large variations in volume change. The angular material, for example, experienced its reduction in height by compaction strain. The rounded material underwent similar deformation through a combination of compaction and shear strain; the confining pressure determined which was predominant.

Interesting additional information on the characteristics of the angular material is available from two separate sources. In

Chapter III it was shown that compaction at 125 blows per layer increased the unit weight approximately 3 pcf above that obtained at 50 blows per layer. Experience has shown that additional compaction produced little, if any, additional unit weight. Instead, it produced particle degradation. Recall from Chapter V (see Figure 5.23) that the 125 blow per layer compactive effort significantly increased the life of the material. As shown in Figure 6.4 the volume change of the high compactive effort specimen was reduced, and the initial rapid increase in unit weight, was eliminated. The important factor is that regardless of the compactive effort which was used, the specimens were able to densify, yet it was virtually impossible to compact this material to higher unit weights in the laboratory using impact type of compaction.

Al-Layla (2) used a gyratory compactor to develop the compaction characteristics of the angular material reported herein. He used a gradation which was slightly coarser than the coarse gradation used in this research. It was found that with gyratory compaction, the unit weight of the aggregate could be increased about 10 pcf above the modified AASHO compactive effort (32.55 ft. lbs. per/cu. in.) or roughly 108 percent of modified AASHO. Thus, it is quite possible to obtain greater degrees of densification by repetitive triaxial stressing or by gyratory compaction (or by traffic) than can be accomplished by impact compaction in the laboratory.

It is believed that densification under repetitive loading is one of the primary reasons why the behavior of the materials under repetitive loading could not be well correlated to their Texas triaxial

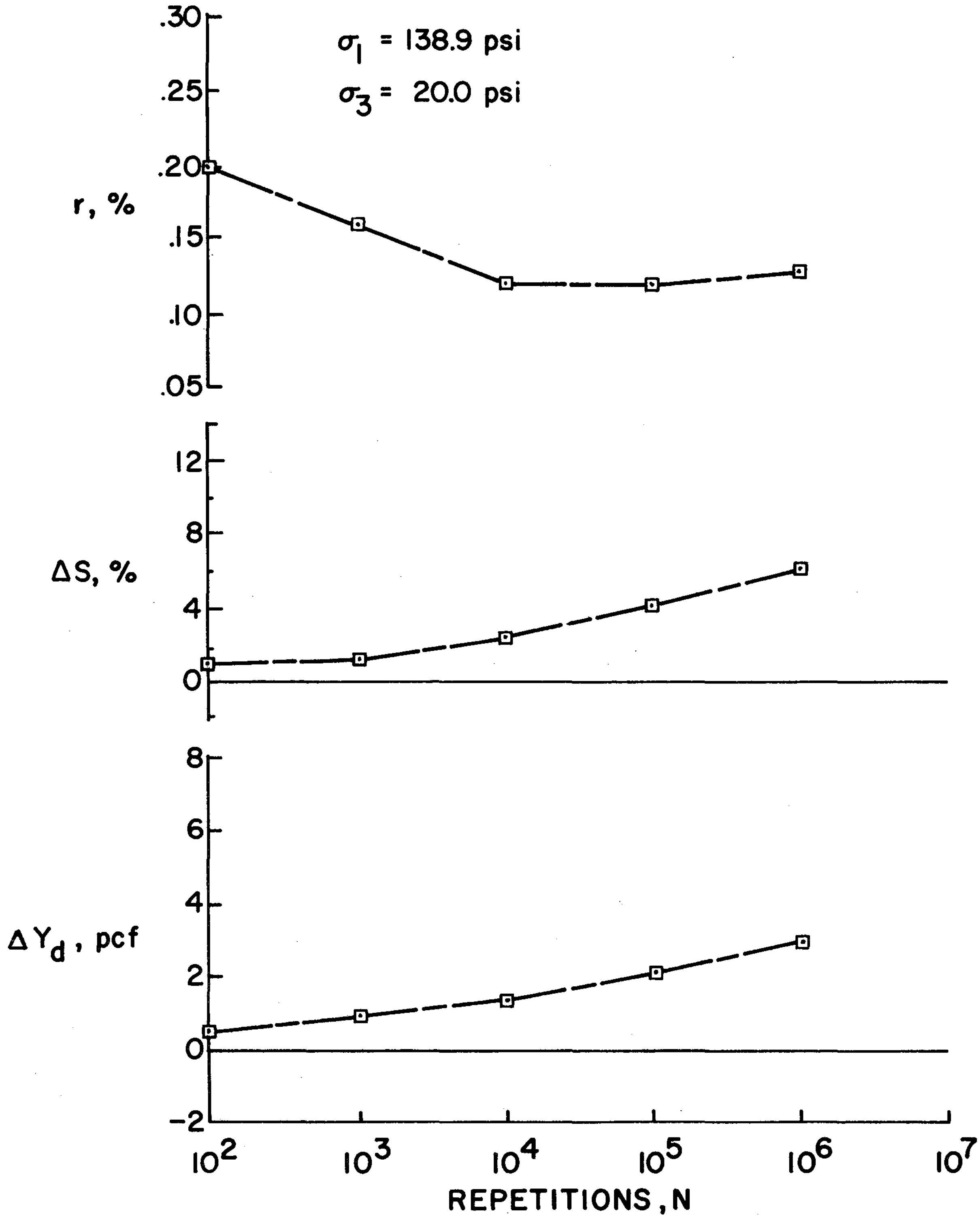

FIGURE 6.4 Rebound strain, change in dry unit weights and degrees of saturation for angular medium aggregate compacted at 125 blows per layers.

classification. The Texas triaxial test can only measure the static shear strengths of a material at any given compactive effort; it cannot estimate the amount of densification that might occur under repetitive loading. Some improvement in the Texas triaxial test method might be obtained by testing the materials at a unit weight more realistic of that expected in the roadway.

According to the present Texas standards the angular materials were tested at about the unit weights which would be required of these materials in the field (D_A). The rounded material was tested at the same compactive effort but at unit weights which were roughly 2 pcf greater than the actual unit weight which would be required of this material in the field.

<u>If the rounded material had been tested at the required field unit weights, more than likely it would have been very inferior to the angular material insofar as repetitive load behavior was concerned.</u>

Rebound and total strain characteristics of the materials can be further explained with basic principles of soil mechanics. For example, both the rebound and total strain must have been influenced by the pressures of the air and water in the interstices of the specimens. As the specimens were repetitively loaded, their low permeability restricted the escape of considerable quantities of water. This caused the degree of saturation and the pore pressures to increase as the unit weight increased. Eventually the compressibility of the material within the voids was decreased by reduction of the air and gas. As the pore pressures increased, the effective strength decreased. At some point in this cycle the deleterious effect of decrease in effective strength overwhelmed any benefits which were derived from increase in the unit weight of the material. This action is believed to have caused the upward trends of both the rebound and total strain which often occurred in later stages in the life of many of the specimens. (This phenomenon can be observed in Figure 5.1 and Figures 5.18 through 5.20). It is also believed to have been responsible for the inability to fit a suitable mathematical model to the data. Such a model would have to include the effect of pore pressures, and their value was unknown.

CHAPTER VII

SUMMARY AND CONCLUSIONS

General

One of the most pressing needs in highway research is reproduction of field environmental conditions in laboratory specimens. The influence of rapid, repetitive loads on highway materials -- particularly flexible bases -- is an important environmental factor which has not been properly evaluated in the laboratory. As a result, if a flexible pavement design method accounts for load repetitions at all, it does so without regard to proper laboratory evaluation. Yet sufficient evidence has been presented in the past, as well as in the research reported herein, to show that static strengths and stress-strain characteristics are not necessarily valid indicators of the performance of roadway materials under repetitive loads.

Research Program

In this research, the repetitive stress-deformation characteristics were evaluated for three materials typifying flexible bases used by the Texas Highway Department. They were:

A. "Rounded" material. A caliche gravel.

B. "Angular" material. A material produced by crushing large particles of the caliche gravel.

C. "Soft" material. A soft crushed limestone.

Properties of the materials are presented in Table III.1. Each material -- except the soft -- was tested at coarse, medium and fine gradations within the allowable gradation range specified by the Texas Highway Department for Type A, Grade 1 flexible base course materials; the soft material was tested only in the coarse gradation.

This arrangement produced a total of seven aggregates, i.e., rounded coarse, angular coarse, rounded medium, etc. Specimens of each aggregate were subjected to a range of repetitive vertical stresses and confining pressures as shown in Table IV.1. Failure under repetitive loading was arbitrarily defined as five percent total strain.

Test Equipment

Considering that the repetitive loading apparatus was designed and constructed locally at a low initial cost and at a time when suitable equipment was not available for testing large diameter specimens of granular materials, the apparatus has functioned adequately. It does have some shortcomings which must be eliminated. In particular, it must be capable of consistently applying accurate and reproducible load patterns and especially it must be capable of doing this during the initial load phases regardless of the deformation characteristics of the specimen. Commercial repetitive loading machines are now available which can do this.

The measuring equipment and the triaxial cells worked satisfactorily. The major improvement which needs to be made is in the method of measuring specimen strain. With the present arrangement, it is impossible to eliminate the non-uniform end stresses, and it is difficult to separate strain occurring in the triaxial cell from the measured strain to obtain the specimen strain. Probably the easiest way to eliminate this difficulty is to measure strain directly on the specimen at points beyond the influence of concentrated end stresses. Optical tracking devices are available which appear to be capable of performing this function.

Finally, the volume change devices are not adequate substitutes for measurement of the pore air and pore water pressures. At best, they give qualitative information from which unit weight and degree of saturation can be estimated. This is certainly helpful in explaining some aspects of material behavior, but to actually understand how the material reacts to load, the effective stresses must be known. Measurement of pore pressures during dynamic load applications will certainly be a difficult problem to solve for partly saturated specimens, but it must be accomplished to determine the effective stresses.

Test Procedures

With two exceptions the test procedures -- many of which evolved during this research -- were satisfactory. Perhaps the greatest difficulty resulted from the experiment designs in which the applied repetitive loads were based on the static shear strengths of the materials. As already mentioned in this chapter, there does not seem to be a definite relationship between static strength characteristics and repetitive loading characteristics. The wide range of stresses applied to the various specimens not only proved to be a major obstacle in analyzing the results, but also made the testing more difficult because of the many load patterns which had to be set on the loading apparatus. It would be more desirable to apply the same set of stresses to all materials using the stresses suggested in Table VI.1 for flexible base materials.

The other major difficulty involved the method of fabricating the triaxial specimens. The impact method (the Texas Highway Department compaction method modified for 6-inch diameter by 12-inch high specimens) was not suitable for producing replicate specimens. It is felt that the main reason for this was non-uniformity of unit weights which resulted primarily from the hand-finishing operation needed to level the compacted surface of the specimens. The operation must somehow be eliminated, possibly with some type of gyratory compaction apparatus. Also, limiting the maximum size of aggregates in the specimen to 3/4-1 inch should reduce specimen variability.

However, the scatter of results obtained in metal fatigue tests on relatively homogeneous materials indicated that perfect replication of results cannot be expected in less homogeneous soils or granular materials, regardless of the pains taken in forming the test specimens. Some type of statistical analysis and several replicate specimens will always be necessary in repetitive triaxial testing.

Test Results

The results of this investigation were analyzed with respect to the total strain, rebound strain and degradation which occurred in the aggregates. The remaining data (primarily changes in unit weights and degrees of saturation) were used as aids in describing the behavior of the various aggregates.

Total strain. The total strain occurring in the specimens were analyzed two ways -- graphically and statistically. The end result of the graphical analysis was a set of curves (Figures 5.5 -5.11) relating the principal stresses and number of repetitions to the total strain. These curves are suitable for design use: if the applied principal stresses in the roadway are known, total strain can be obtained for any desired number of repetitions.

Similar curves (Figures 5.12 - 5.17) were obtained from the statistical analysis, but they are not considered suitable for design purposes. This was primarily due to the statistical model which was used. Because a suitable physical model could not be found, a second degree response surface which bore no physical relationship to material behavior was fitted to the data using normal multiple regression techniques. However, the statistical analysis did provide a measure of the experimental testing error. The standard deviations (see Table V.1) ranged from 0.38 to 1.0 percent total strain for the seven aggregates.

Using the graphical analysis, the relative behavior of the materials was compared two ways:

A. An "average" comparison which gave equal weight to behavior at all confining pressures and strains shown in Table VI.2.

B. A comparison based on stresses required to produce five percent total strain at 100, 1,000, 10,000 and 100,000 repetitions expressed in terms of Mohr's envelopes which in turn were superimposed on the Texas classification charts to provide a triaxial classification for this level of strain and number of repetitions shown in Table VI.3.

The two methods produced somewhat different results, but one method does not appear to have any particular advantage over the other except the latter can be expressed quantitatively. Both methods showed that Texas triaxial classifications were valid indicators of repetitive load performance for small numbers of repetitions, say 100 or so. But as the number of applied load repetitions increased, this relationship did not hold. For example, the angular coarse aggregate, which was the best performing material at 100 load repetitions and also had the highest Texas triaxial classification, ranked as the poorest performer after 100,000 repetitions. On the other hand, the rounded medium aggregate, which ranked close to the bottom in static triaxial classifications, was one of the best performers after 100,000 repetitions.

Repetitive loading, however, did not completely reshuffle the order of performance, although it did have a tendency to close the performance gap which existed under static triaxial tests. For example, the initial Texas triaxial classifications ranged from Class 1 to 3.0, whereas according to Method B (explained above) the classifications ranged from Class 3.0 to 3.8 at five percent strain and after 100,000 load repetitions.

In general, the angular materials, which were best according to static triaxial tests, suffered the greatest loss in performance with the number of repetitions, followed closely by the soft material. The angular coarse and the soft coarse aggregates, in that order, suffered the greatest loss in performance in the research.

The medium gradations of the materials performed better than the coarse or fine.

Rebound strain. The rounded materials in all cases appeared to have lower rebound strains than the other materials. The rounded coarse gradation performed best of all.

Degradation. The amount of degradation that occurred under repetitive triaxial loading was insignificant except for the soft material. It degraded in the sizes smaller than the No. 10 sieve, with the maximum degradation being approximately five percent on the Nos. 80 and 200 sieves.

Explanation of repetitive loading behavior of aggregates. Based on the analysis of total strain, rebound strain, and the changes in unit weights and degrees of saturation, it is felt that the behavior of the materials can be explained as follows:

> The high shear strengths of the angular and soft materials enabled them to withstand high repetitive stresses better than the weaker rounded materials, although the rounded materials often performed better (or required more repetitions) at the low values of total strain. The compaction characteristics of the angular and soft materials were such that they were tested at lower relative densities than the rounded materials even though identical compactive efforts were used on all aggregates. Thus, repetitive loading produced considerable densification which was manifested in a rapid initial increase in total strain and in high rebound strains. The primary mode of strain in the angular and soft materials was

compressive (or densification), not shear, and, in fact, repetitive loading produced greater densification in the angular material than could be produced by impact compaction in the laboratory. For the soft material some strain resulted from particle degradation.

As long as the repetitive loads were not too high, the rounded materials performed well in spite of having lower shear strengths. Because of their higher relative densities with respect to the angular and soft materials, they a) required more repetitions to achieve a particular total strain, b) more or less underwent slow, gradual increases in unit weight, and c) exhibited smaller rebound strains. Apparently there was some stress -- although poorly defined -- at which the better compaction characteristics of the rounded material outweighed the higher shear strengths of the angular and soft materials.

The changes in unit weights with the number of repetitions (Figures 5.18 through 5.20)tend to confirm the above explanations.

For all aggregates, as the number of repetitions increased, the degree of saturation (and the pore pressures) increased since the excess pore air and water could not dissipate as rapidly as densification occurred. This caused a reduction in effective strengths which overshadowed any strength increases which might have resulted from increases in unit weights. For long life specimens this resulted in an increased rate of total strain and higher rebound strains as the number of repetitions became larger.

Overall comparison of materials. Based on total strain and rebound strain, the rounded material would appear to be at least equivalent to the angular material, and both materials were superior to the soft. The comparison is predicated on the assumption that excessive resilience (or rebound strain) is just as detrimental to roadway performance as excessive total strain; it must be qualified by explaining that it is based on performance at vertical stresses which are equal to or less than the expected roadway stresses, say 120-140 psi. This is opposed to generally accepted concepts of the behavior of rounded versus angular materials. In this respect, the report by Haynes (21) which compared the results of repetitive triaxial tests on the AASHO Road Test gravel and crushed stone might be considered. He stated:

> At low degrees of saturation, the gravel material was found to be superior (insofar as stability-rebound characteristics are concerned) to the crushed stone at all levels of relative density. ...The stability of the gravel was less affected by a change in relative density than that of the crushed stone. At higher degrees of saturation the variation in stability between the two materials seemed to disappear and they performed in a similar manner.
>
> ... Field moisture determination showed that the gravel existed at a higher degree of saturation than the crushed stone base during the time traffic was on the road. The variation in degree of saturation in turn no doubt accounts for the major portion of the difference in field performance of the two materials.
>
> ... The primary reason for the improved performance in the pavement using crushed stone as a base as contrasted to the performance of the pavement having a gravel base was not due to the inherent strength characteristics of the base materials but was due rather to the external effects of moisture on these strength properties.
>
> The data further suggest that if the gravel studied existed in a road at a degree of saturation equal to that of the crushed stone, its performance would be at least equal and, at low degrees of saturation, superior to that of the crushed stone.

It might be pointed out that Haynes eliminated about 20 percent of the + 3/4 inch crushed stone, but then compacted it in the laboratory to the in-situ unit weight. As a result the crushed stone was compacted to a much higher relative density than actually achieved in the field, and also at a compactive effort which considerably exceeded that used to fabricate the laboratory specimens of the gravel. This placed the crushed stone at a distinct advantage over the gravel, yet, as stated above, the gravel performed well when compared to the crushed stone.

Applicability of results to field conditions. Elaborate experience records are not available for the materials used in this research. However, Haynes had the benefit of precisely known field conditions, i.e., the extent to which the AASHO Road Test crushed stone outperformed the gravel in the field. Yet, his test results indicated near equality of the two materials.

This suggest that some environmental effect may be lacking in repetitive triaxial tests. In Haynes's case it may have been the influence of freezing and thawing in the roadway which influenced the degree of saturation to an extent not reproducible in the laboratory. This is not believed to be such an important factor in this research. Based on the comparison of in-situ moisture contents as shown in Figure 6.3, it is felt that the repetitive tests are fairly representative of moisture conditions which may be encountered on Texas highways. However, the relative densities should be investigated further before applicability of the results to field conditions can be determined precisely.

It is felt that the two most important facts gained from this research were:

A. Results of repetitive triaxial tests can be expressed quantitatively.

B. Unit weight (or relative density) may have a greater influence on repetitive loading behavior than either particle shape or particle hardness.

CHAPTER VIII

RECOMMENDATIONS

While interesting quantitative and qualitative information was obtained from this program, the research must still be defined as exploratory in nature. Based on the information gained from the research program, the following recommendations are appropriate:

A. Repetitive triaxial tests must be continued on flexible base course materials. It is necessary to test a wide variety of typical materials.

B. Until more is learned about repetitive triaxial performance and its relationship to field performance, it is suggested that testing be entirely confined to aggregates actually used in roadway construction; that they be tested at moisture contents, unit weights and gradations typical of field placement conditions; and that field performance of the aggregates be closely observed for later correlation with the repetitive triaxial tests.

C. The repetitive triaxial apparatus should be redesigned to include programmable servo-operated mechanisms to insure accurate and repeatable load applications or alternatively, a commercial apparatus capable of performing this function should be obtained. It should be capable of varying vertical and lateral pressures simultaneously.

D. Measuring devices should be improved. In particular, a method of measuring localized strain -- both axial and lateral --

should be devised, and methods should be developed for measuring pore pressures in partly saturated granular materials.

E. A better method of fabricating test specimens should be developed. This is an absolute necessity for production of replicate specimens, and also for producing specimens at in-situ unit weights and moisture contents as suggested in B. above. Present impact methods can accomplish the latter only within a relatively narrow range.

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APPENDIX A

MEMBRANE PERMEABILITY AND DIFFUSION CHARACTERISTICS

As mentioned in Chapter IV, it was necessary to know the amount of air diffusion and water leakage through the butyl membrane for calculating correct degrees of saturation and unit weights at any time during the life of a specimen. Poulos (30) has presented an excellent review of previous research of this subject and the results of his own research. He shows that leakage may occur in the membrane bindings and in various valves and fittings used on the triaxial apparatus, as well as through the membrane.

In this particular research it was desired to determine the leakage for the entire system since this was the quantity reflected on the volume change devices. Owing to the large membrane area, there was reason to believe that most of the leakage or diffusion occurred through the membrane itself and not through the fittings, bindings or valves.

Test procedures. The triaxial apparatus was assembled in the manner described on pages 92 and 93, except a 6-inch diameter by 12-inch high coarse porous stone was substituted for the actual specimen. Also, a thermometer was placed inside the triaxial cell to measure the temperature of the confining medium.

After sealing the porous stone in the membrane, the volume change device was attached, confining fluid (water) was added, and a confining pressure of 20 psi was applied. Readings on the volume change device were made regularly after cessation of the initial volume changes resulting from the application of the lateral pressure. Simultaneous temperature readings in the confining fluid and laboratory barometric pressures were also recorded.

To measure any moisture which might have been transmitted through the membrane in either liquid or vapor form but which was too minute to be measured in the volume change device, the porous stone was weighed prior to the test and then afterwards.

Test results. Figure A.1 contains the test results. The air volume values have been corrected to standard atmospheric pressure and for a temperature of 25°C; this temperature value is within ± 1.5°C of the laboratory temperatures observed over the time of repetitive triaxial testing. Even with these corrections, the scatter of the results was quite pronounced. However, an average air diffusion value of 3.7 cm^3/day for a confining pressure of 20 psi appeared acceptable.

No changes of water level were observed in the water trap during the test periods, and weight change in the porous stone was insignificant, indicating that the membranes had insignificant water permeability.

If there are no convection currents in the chamber water so a partial pressure gradient of air can develop in the water, Poulos suggests that the volume of air diffused into or out of a specimen can be approximated by:

$$J_a = \frac{8}{\sqrt{\pi}} \frac{K_s}{\sqrt{D}} \frac{\Delta p_p}{D_s} \sqrt{t} \; V \qquad \text{Equation (A.1)}$$

where: J_a = volume of air flow at standard atmospheric pressure, cm^3

K_s = permeability constant of membrane, cm^4/gm sec

Δp_p = initial partial pressure difference in air across membrane, gms/cm^2

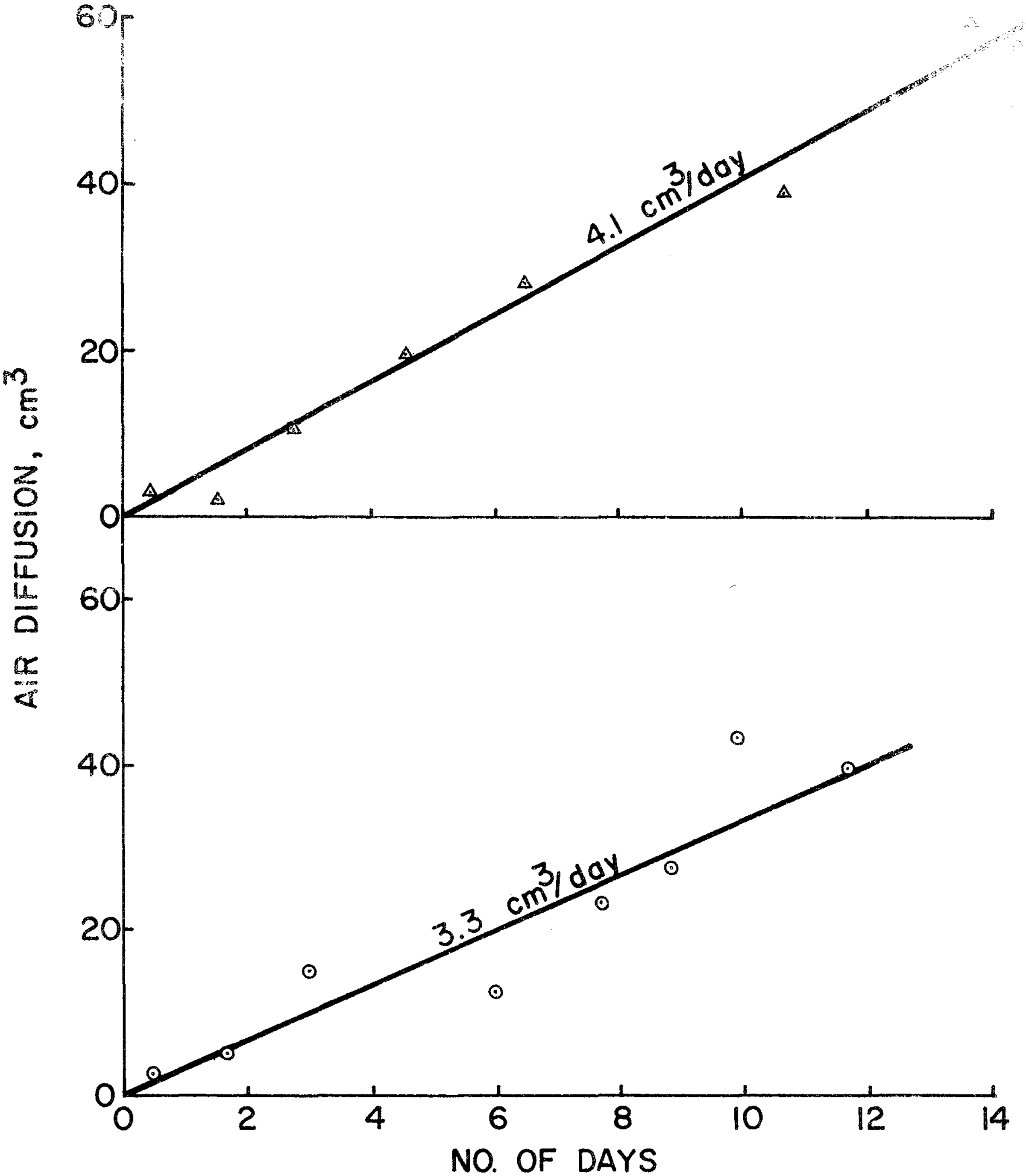

FIGURE A.1 Rate of air diffusion through butyl rubber membranes.

t = time, sec

V = total volume of soil specimen, cm^3

D = coefficient of diffusion of air in confining fluid cm^2/sec

D_s = diameter of specimen, cm

Equation (A.1) can be solved for K_s using the experimentally determined value of J_a. With Δp_p as 1408 gms/cm^2, J_a as the measured volume of air flow($3.7\ cm^3$ in one day) and D as $2 \times 10^{-5}\ cm^2/sec$, a value of $0.24 \times 10^{-11}\ cm^4/gm\ sec$ was obtained for K_s.

If the convection currents were significant enough to allow partial pressure gradients to develop only in the membranes and not in the water, Poulos suggests that the rate of air flow be computed using:

$$q_s = (K_s\ A\ \Delta p_p)/L \qquad \text{Equation (A.2)}$$

where: Δp_p = partial pressure difference of air across the membrane, gm/cm^2

A = area of flow, cm^2

L = thickness of membrane, cm

K_s = as before

q_s = rate of flow measured at standard atmospheric pressure, cm^3/sec

Solving for K_s produces a value of $0.17 \times 10^{-11}\ cm^4/gm\ sec$.

In reality, an average of the values from Equations (A.1) and(A.2) probably gives as suitable a value of K_s as can be obtained.

Under actual test conditions, the continual flexing of the specimen would prevent partial pressure gradients from developing in the water and Equation (A.2) should be used to calculate the rate of air flow. Entering the average value of K_s into the equation produces the following:

$$q_s = 4.5 \text{ cm}^3/\text{day at a confining pressure of 20 psi}$$

$$q_s = 2.6 \text{ cm}^3/\text{day at a confining pressure of 11.5 psi}$$

$$q_s = 0.7 \text{ cm}^3/\text{day at a confining pressure of 3 psi}$$

The above calculations are based on the assumption that the specimen pore air pressures were atmospheric, which was obviously not true. In fact, it is difficult to even estimate their magnitude. Initially, the air pressures were probably high, but thereafter they must have decreased although they certainly remained greater than atmospheric. Thus, the air diffusion during the initial stages of loading would be less than calculated, but it would probably be too small to affect void ratio and degree of saturation calculations for short term tests. On specimens which were repetitively loaded for several days, the calculated values of diffusion are probably somewhat high.

In summary, it appears almost impossible to calculate highly accurate degrees of saturation and unit weights owing to the difficulty of obtaining accurate air diffusion values under actual test conditions, but the methods and calculations described above do give a value whichis believed to be sufficiently accurate for the desired purposes.

APPENDIX B

TESTS OF EQUIPMENT

Determination of Piston Friction

To evaluate piston friction, an 8-inch high specimen was mounted in the triaxial cell in the same manner in which 12-inch high repetitive triaxial specimens were normally installed. A load transducer was placed on top of this specimen and centered beneath the loading piston which travelled through the head of the triaxial cell. The cell was then placed in the repetitive loading apparatus and the specimen was subjected to repetitive loads. The recorder was run at 20 inches per second -- the speed normally used to check the load magnitude. Simultaneous tracings were obtained from the outputs of the interior transducer and the exterior transducer. The latter was attached, as usual, to the hydraulic loading cylinder and bore directly on the loading piston of the triaxial cell. The difference in load readings between the two transducers should represent the friction loss resulting from movement of the piston through the head of the triaxial cell.

Trials were made on two separate specimens. The head on the first one tilted badly after one trial had been made. On the second specimen, five trials were made at different numbers of repetitions. The results are presented graphically in Figure B.1 which shows the difference between exterior and interior load readings for all trials. Negative numbers indicate the interior transducer gave higher loads than the exterior one. Each point represents a time range of 10 milliseconds.

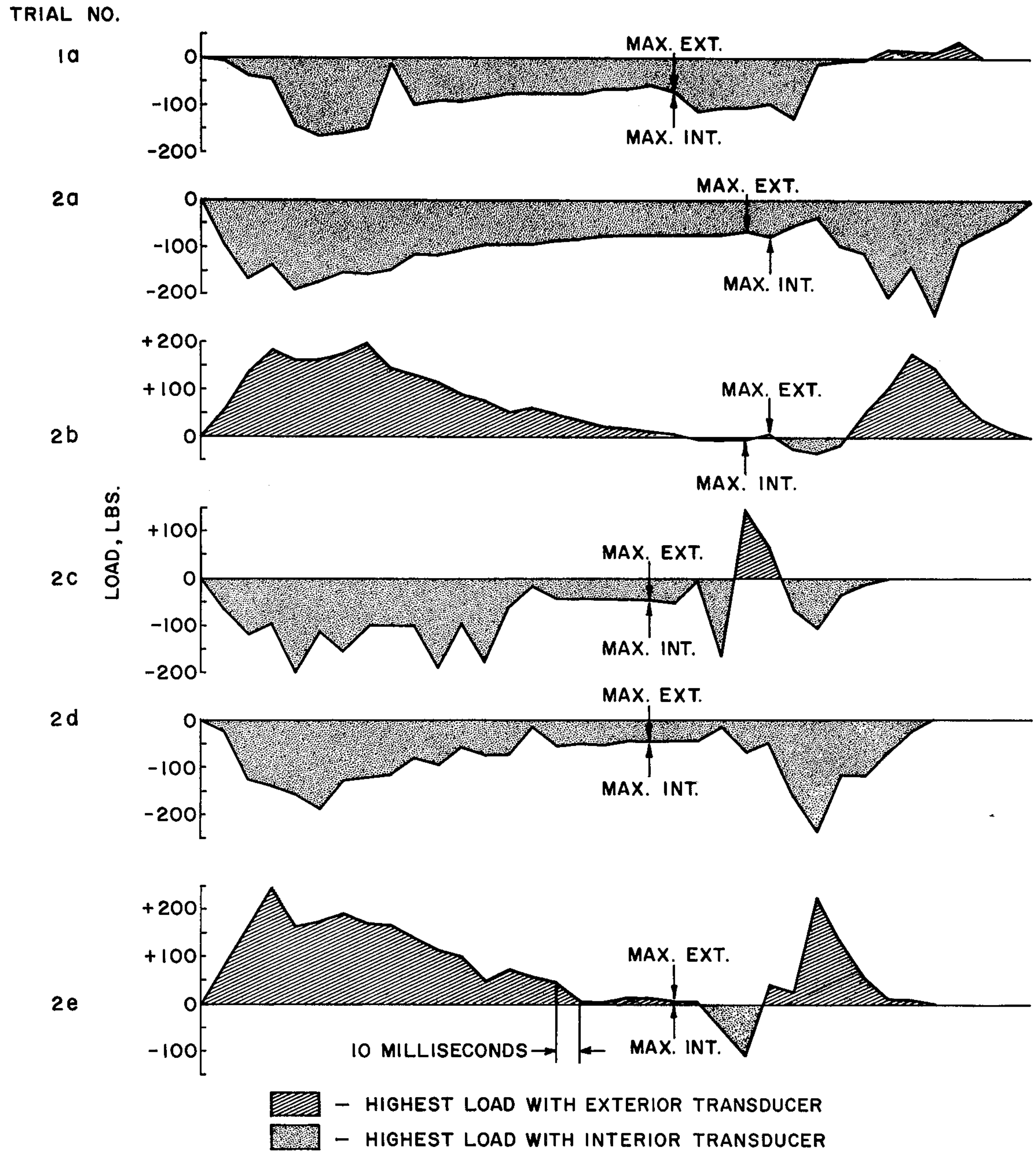

FIGURE B.1 Differences between exterior and interior transducers used for determining piston friction.

Generally, there was considerable difference in the readings of the two load transducers from the beginning of loading until they reached maximum load, indicating that they were out of phase with each other in the early stages of loading. However, both transducers reached their maximum load at essentially the same time. In one instance (Trial 2a) the exterior transducer reached its maximum 10 milliseconds earlier than the interior one, and in another case (Trial 2b), the interior was 10 milliseconds earlier than the exterior. In the remaining four trials, both transducers reached maximum load at the same instant.

Out of the six separate trials, the interior transducer produced the highest overall readings four times (Trials 1a, 2a, 2c, and 2d), and while these differences may seem rather large, at the maximum readings the interior transducer was only 44-72 lbs. higher than the exterior. In Trials 2c and 2e the exterior transducer gave higher overall readings, but at the maximum readings the difference was only 6 lbs.

The results are not conclusive insofar as what phenomena occurred. In general, the differences appeared to be fairly random and unimportant so far as the maximum load is concerned; however, there may be some evidence of the fact that rather than friction occurring in the piston, just the opposite is happening. The only way which this could occur would be inertial forces gained from the movement of the piston.

Response of Load and Deformation Recording System

While the load and deformation transducers were designed for rapid and accurate response times, it was necessary to check the entire recording system to assure that the required characteristics were being obtained.

The deformation transducer system was checked by mounting transducers on the triaxial cell pistons in the usual manner for making measurements. Stops were placed so the pistons could only move a known distance which was accurately measured by a dial gauge of 0.0001-inches accuracy. The oscillographic recorder was then operated at a speed of 20 inches per second and the piston was given a sharp tap to produce instantaneous movement of the transducer.

Figure B.2 is a tracing of a typical recording from several trials which were made. The recording shows an almost immediate rise time as indicated by the vertical line. In addition, the recording readings correspond to the measured dial gauge readings within 0.0002-inches, which is well within the limits of reading accuracy on the oscillograph recordings.

To check the load transducers, they were placed in a special apparatus whereby a weight could be dropped from a known distance on the transducers. As the weight was released, oscillographic recordings were made at the recording speed which was normally used to check the force being applied to the specimen. A typical recording is shown in Figure B.3.

Again it was noted that the recordings were vertical indicating an immediate response time, and that there was no overshoot. While it would have been desirable to calculate the force which was applied to

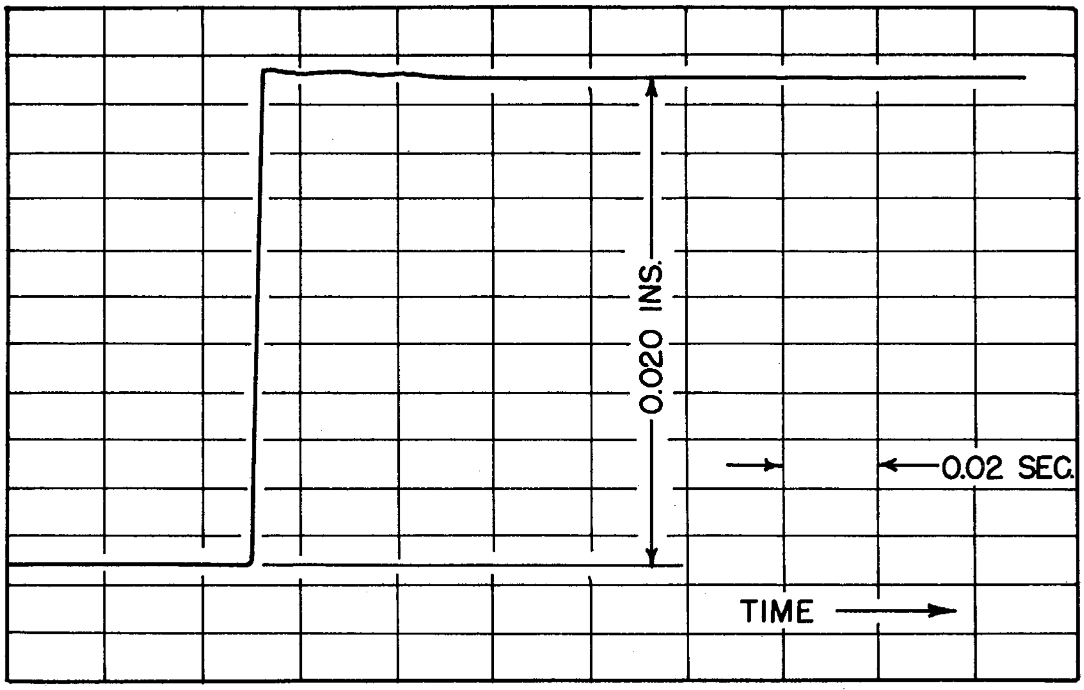

FIGURE B.2 Typical recording from deformation transducer subjected to instantaneous deformation.

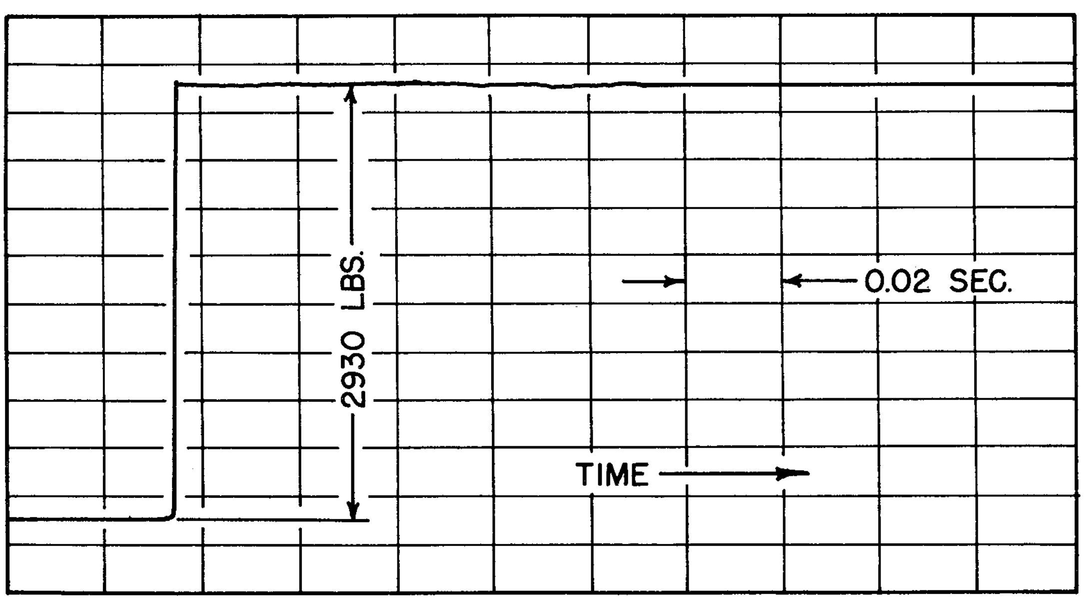

FIGURE B.3 Typical recording from load transducer subjected to impact load.

the transducer by knowing the mass and acceleration of the weight, this was not considered feasible due to the friction losses in the falling weight system.

Triaxial Cell Deformation

To determine the deformation characteristics of the triaxial cells, the load-deformation relationship of a concrete cylinder -- capped smoothly on both ends -- was first established. The concrete cylinder was then placed in a triaxial cell and the load-deformation relationship of the entire system was determined with the measuring devices normally used during repetitive triaxial tests. Triaxial cell deformation was obtained by subtracting the concrete cylinder's deflection from the deflection of the entire system. This produced the curve shown in Figure B.4, which is expressed in terms of strain rather than deformation. It is an average curve for the several cells.

Deformation of the cells determined in this manner may not be entirely correct, since the stiffness of the concrete cylinder is much greater than that of an actual triaxial specimen. This may create cell deflection incomparable to that occurring if an actual specimen were being loaded. To date, it has not been possible to locate a material which has a) deformation characteristics approximating the research material, and b) a reproducible load-deformation relationship.

The results indicate that a large amount of deformation took place initially under a rather small load, after which the load-deformation characteristics of the cells were relatively linear.

Apparently the initial high deformation was caused by seating of the various components in the triaxial cell. Since several porous stones were broken during testing, it is suspected that much of the seating involved deformation of the porous stones to conform to the surfaces of the metal pedestal and cap of the triaxial apparatus. In fact, check tests with various porous stones mounted in the same triaxial cell indicated somewhat variable load-deformation characteristics. Attempts were made to mill one surface of the porous stones flat, but this was expensive and not entirely satisfactory.

The deformation of the triaxial cell is of significant magnitude to influence specimen rebound strain, but is reproducible enough that its effect can be taken into account by subtracting it from the results.

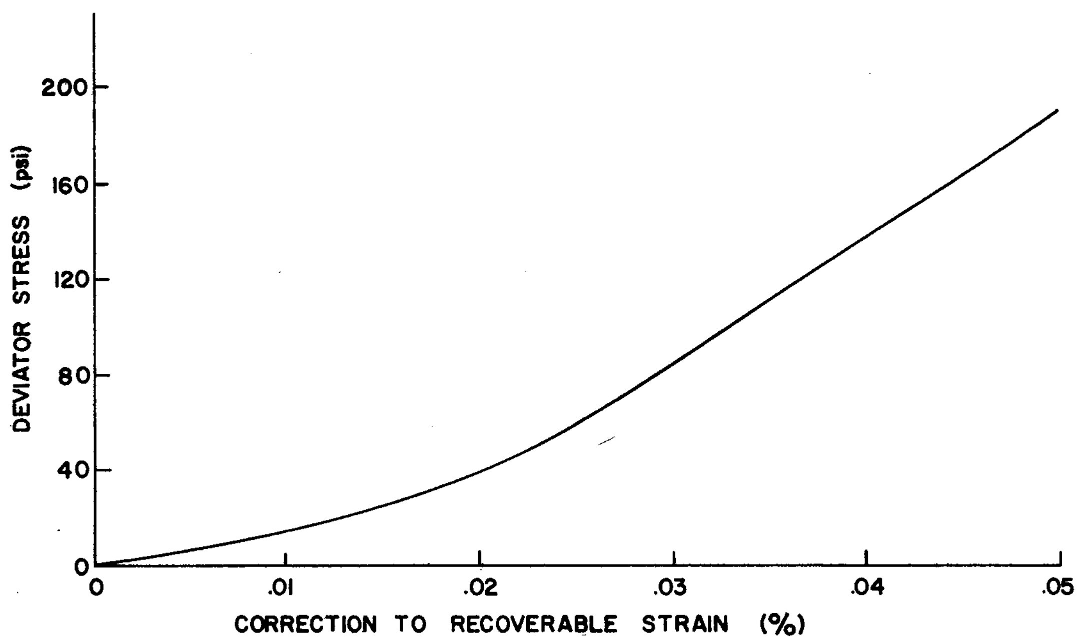

FIGURE B.4 Deformation characteristics of triaxial cells.

APPENDIX C

UNIT WEIGHTS AND GRADATIONS FOR RESEARCH SPECIMENS

The following graphs present a record of dry unit weights and gradations for the majority of specimens used in the research program. They include:

A. Gradation specimens which were not compacted.

B. Compaction specimens which were compacted at optimum moisture content, but were not stressed.

C. Texas triaxial specimens.

D. Repetitive triaxial specimens.

For various reasons, not all of the specimens presented in the graphs were considered in the analysis of test results. Specimen numbers which are underlined represent those specimens used in the analyses.

The solid horizontal lines represent the values which were desired based on information which is presented in Chapter III. The graphs are intended to show the deviations which occurred from the desired values.

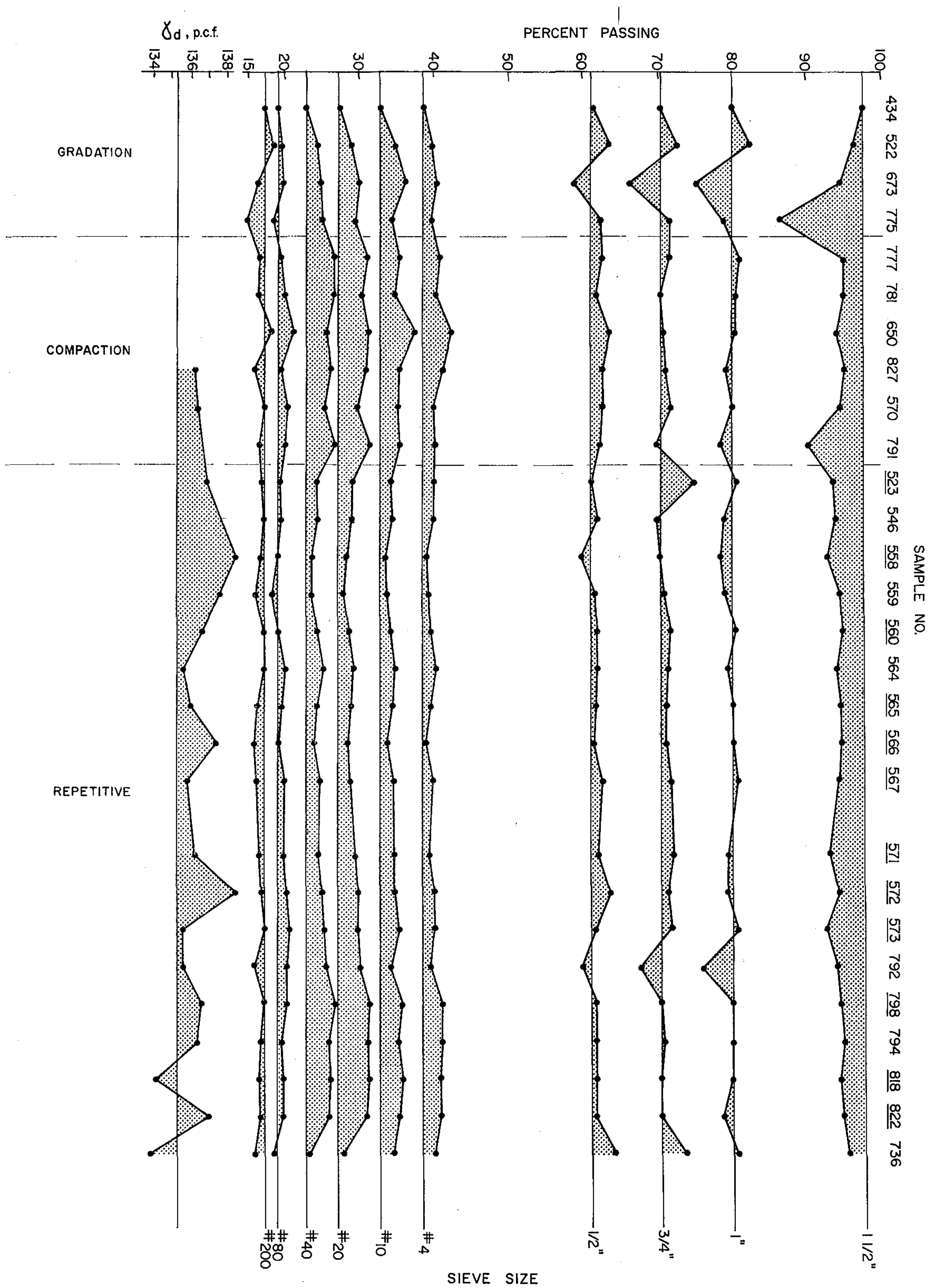

FIGURE C. 1 Rounded coarse aggregate. Gradations and compacted dry unit weights for specimens used in research program.

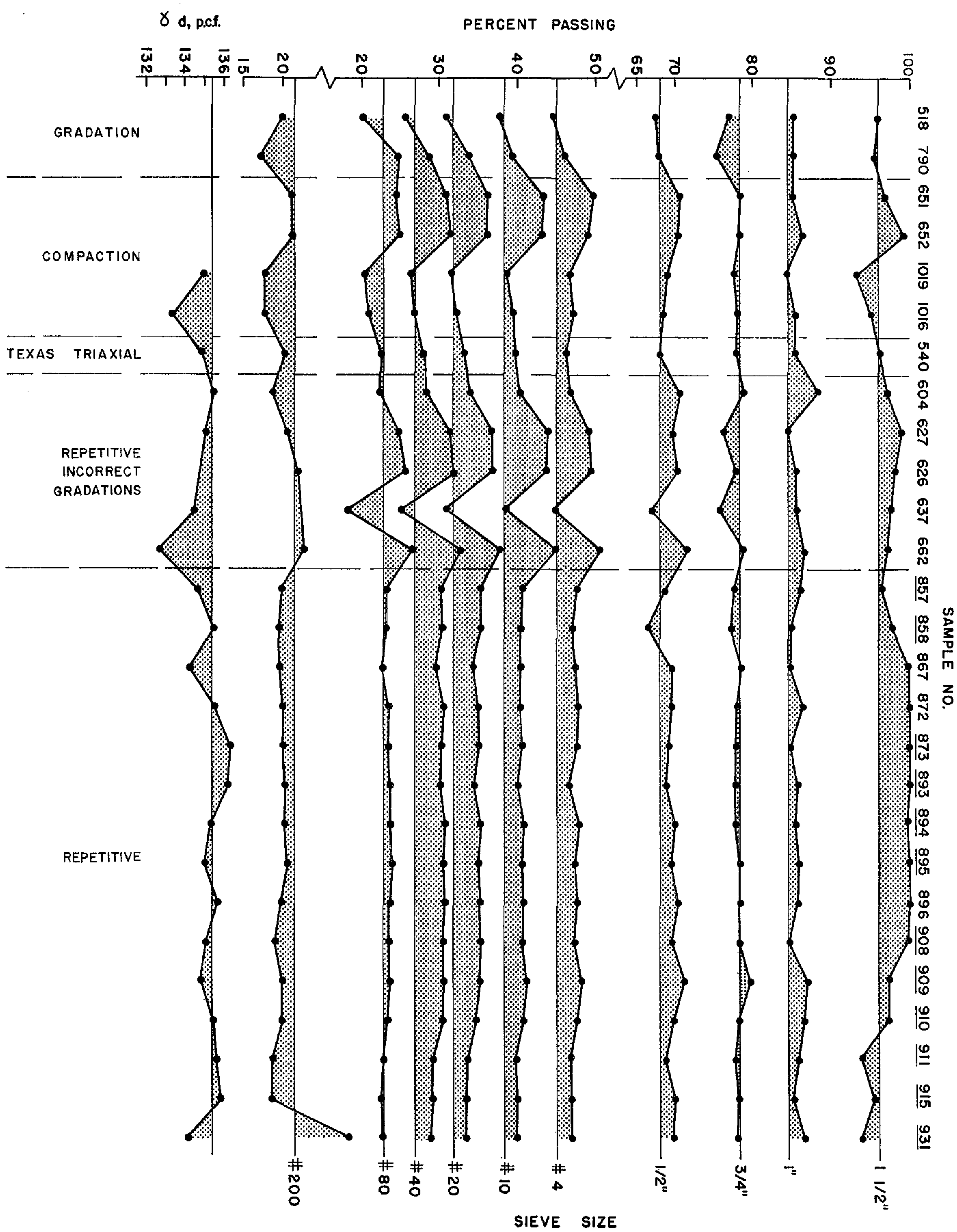

FIGURE C.2 **Rounded medium aggregate. Gradations and compacted dry** unit weights for specimens used in research program.

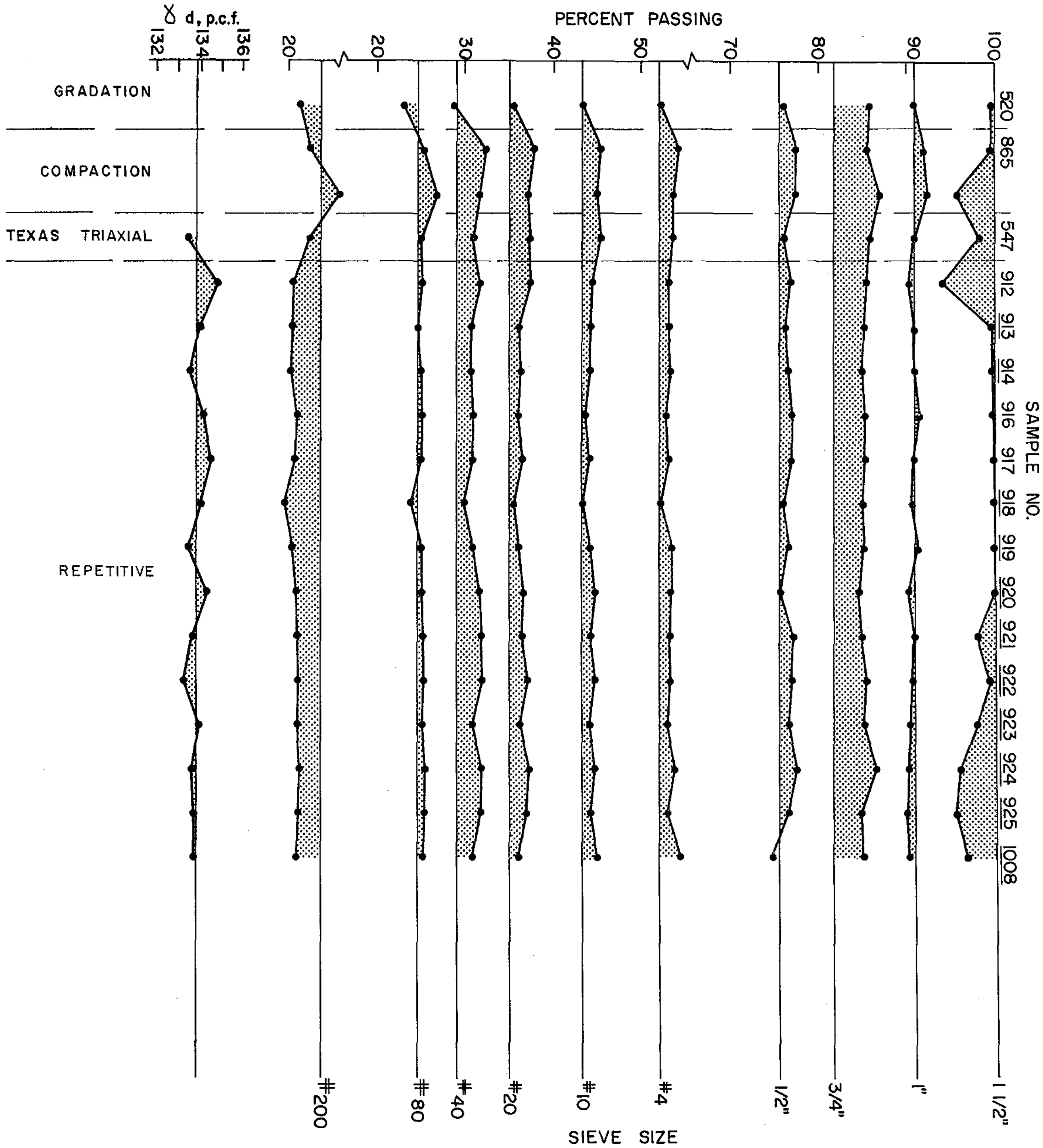

FIGURE C.3 Rounded fine aggregate. Gradations and compacted dry unit weights for specimens used in research program.

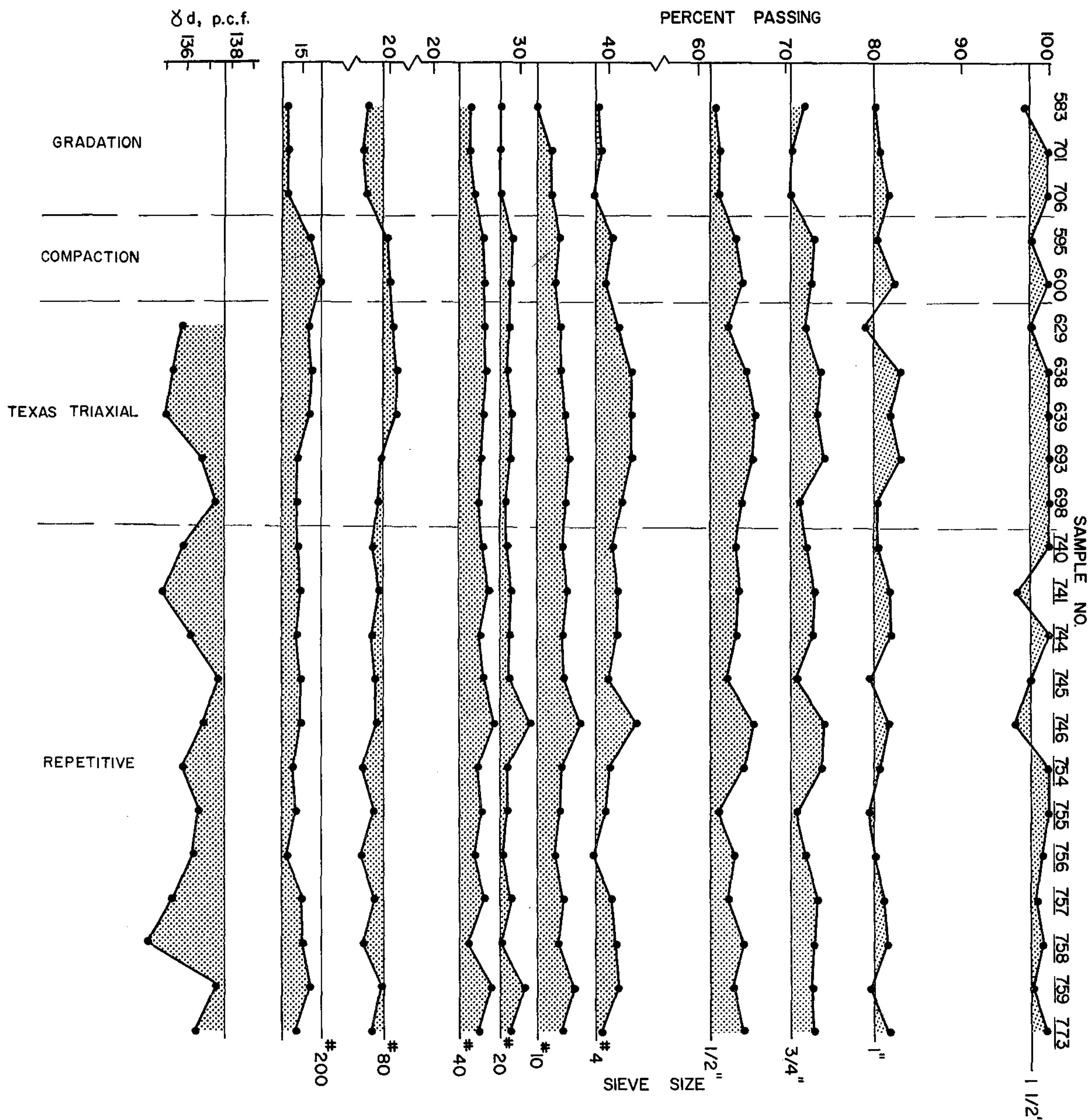

FIGURE C.4 Angular coarse aggregate. Gradations and compacted dry unit weights for specimens used in research program.

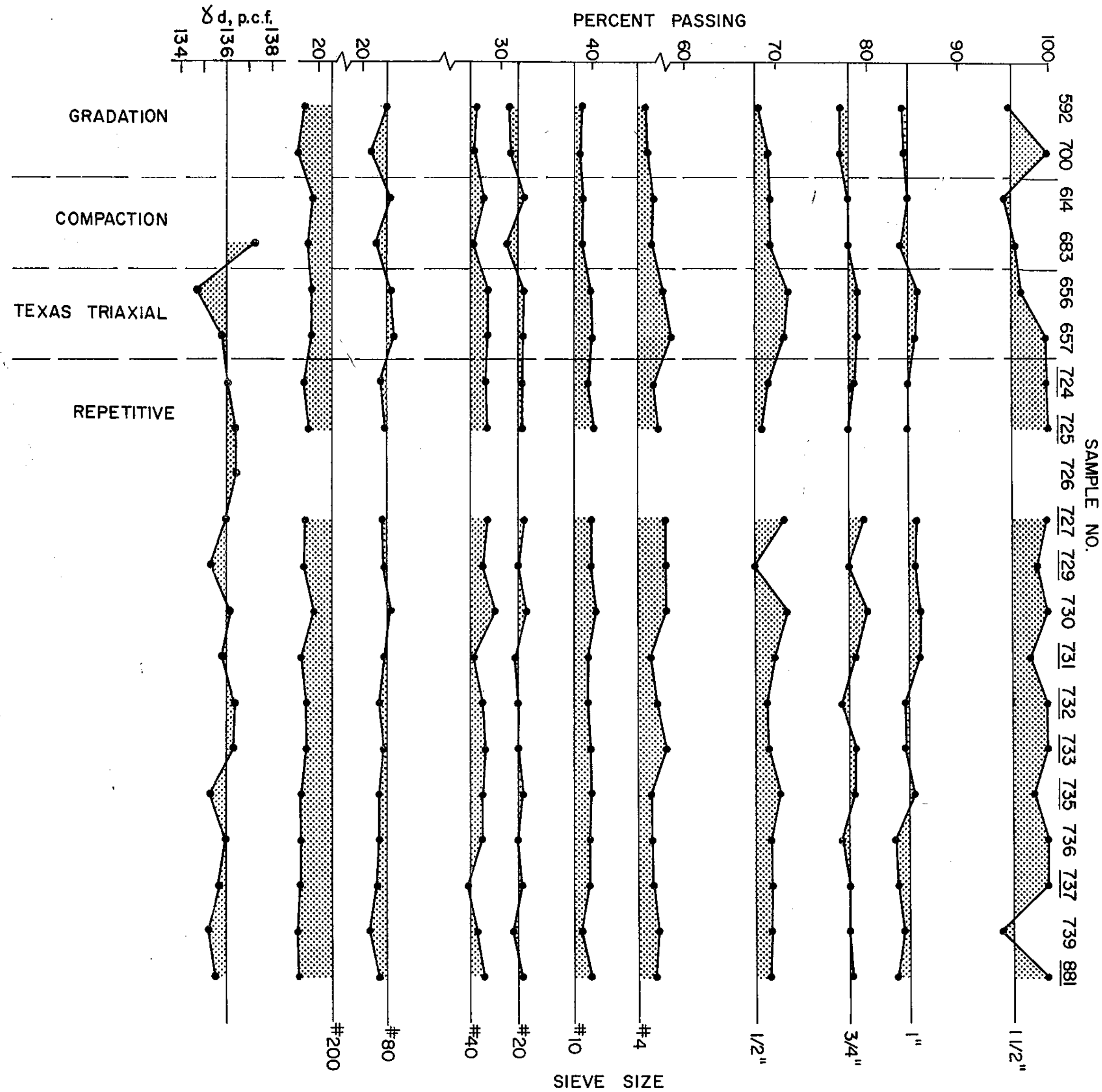

FIGURE C.5 Angular medium aggregate. Gradations and compacted dry unit weights for specimens used in research program.

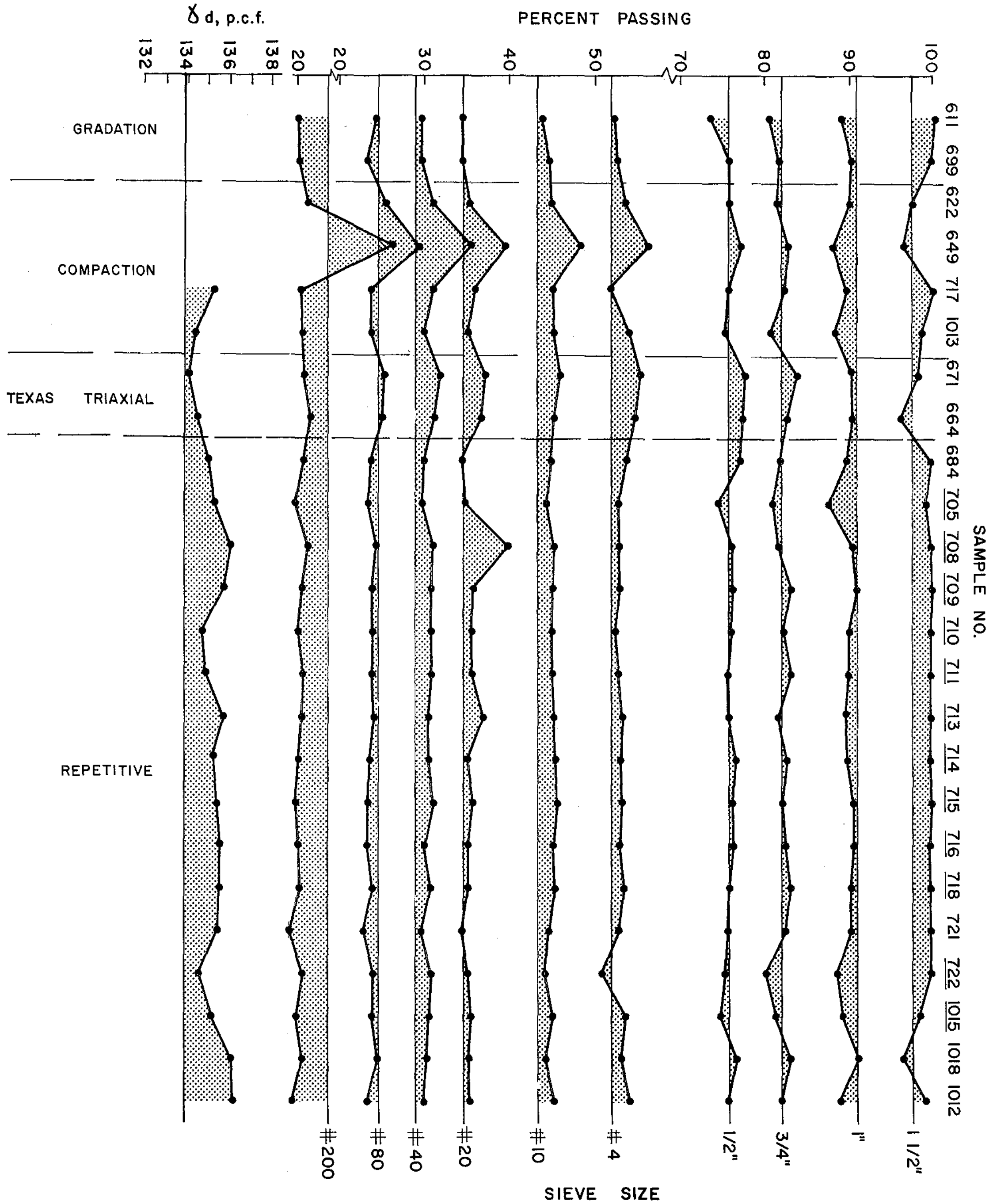

FIGURE C.6 Angular fine aggregate. Gradations and compacted dry unit weights for specimens used in research program.

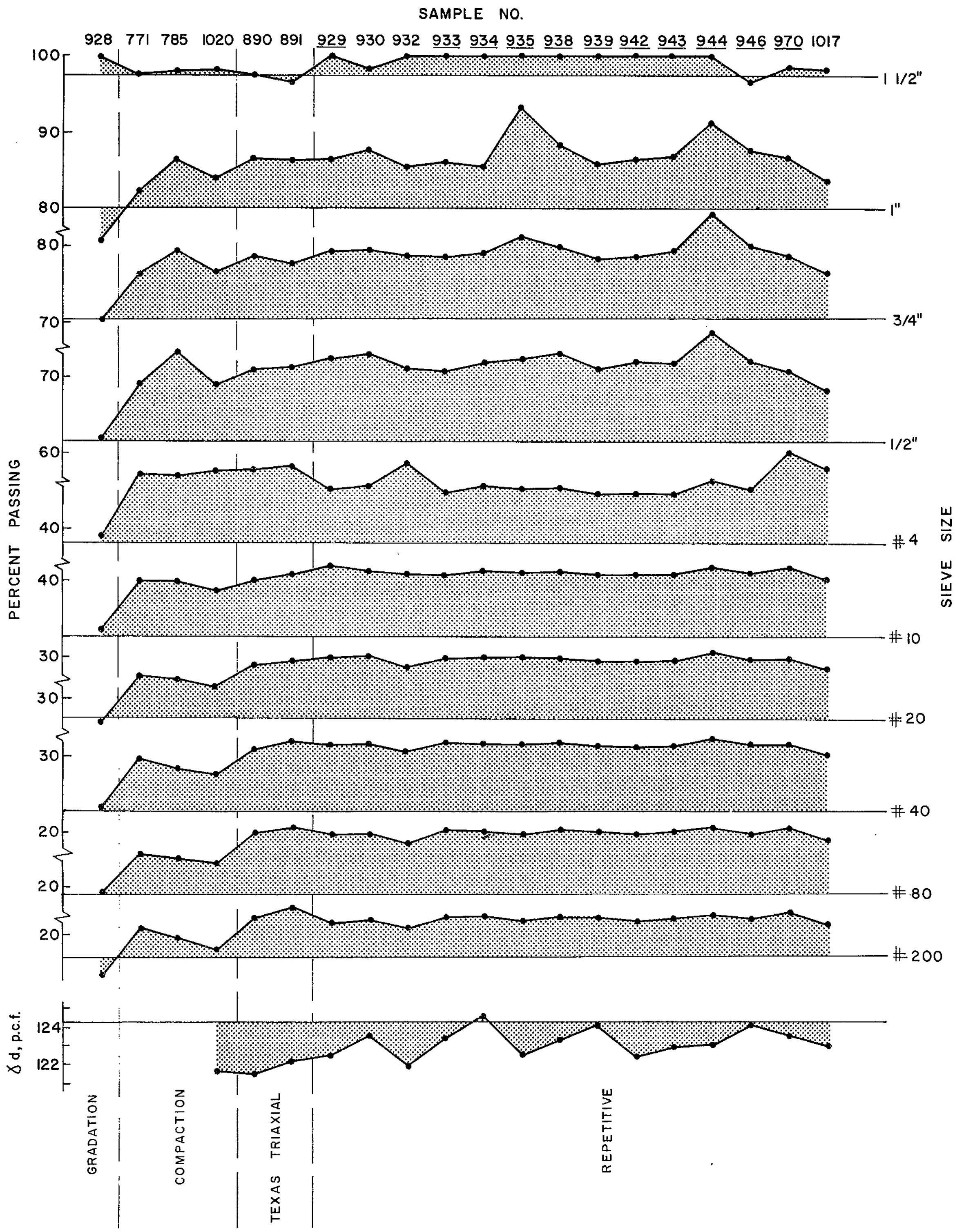

FIGURE C.7 Soft coarse aggregate. Gradations and compacted dry unit weights for specimens used in research program.