DEPARTMENT OF SOILS, GEOLOGY AND FOUNDATIONS

A Strength Criterion for Repeated Loads

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Two criteria for the strength of fine-grained soils subject to repeated loads were postulated and studied. For the test conditions and soils employed, one of the proposed criteria was verified. Failure occurs when repeated stresses exceed critical values determined from slopes of deformation-number of repetitions curves. New facts concerning the stress level vs number of stress repetitions and the compaction conditions vs elastic rebound relationships suggest the need for a critical review of subgrade compaction specifications for highway and airfield pavements.

• THE STRENGTH and deformation characteristics of naturally deposited, as well as mechanically compacted, fine-grained soils under the action of gradually applied loads have been studied intensively by engineers and scientists for more than three decades. Considerable progress has been made in isolating and evaluating the factors that affect these characteristics.

During the last decade, there has been increased interest and quantity of work on the effects of repeated stresses. This interest has been caused by mounting evidence that the repeated application of stresses caused by wheel loads moving over highway and airfield pavements often affect the strength and deformation characteristics of the underlying soils in a detrimental manner, and that this effect may not be predicted satisfactorily by conventional strength tests.

Earlier investigations have been helpful in establishing marked difference in the behavior of various soils under the action of repeated loads and in partially evaluating the effects of certain factors such as degree of compaction, density, molding water content, and frequency and sequence of load applications. In view of its growing importance in soil engineering, the need for a strength criterion for soils subjected to repeated loads was recognized, and this study was undertaken in an attempt to develop such a criterion for compacted, fine-grained soils.

PREVIOUS INVESTIGATIONS

For most ferrous metals it is known that a plot of stress vs number of stress applications required to produce failure results in the familiar *s-n* diagram (Fig. 1). At a sufficiently low level of stress, a very large or infinite number of stress repetitions can be applied without causing failure of the metal. When the material is subjected to a series of completely reversed stresses, this critical level of stress is called the endurance limit, S_e. Determination of the endurance limit for ferrous metals usually requires that the sample withstand at least 10 million cycles of stress application.

For nonferrous metals, the endurance limit either does not exist or is not so easily defined as in the case of ferrous metals. For example, 500,000,000 stress cycles may fail to define the endurance limit for some nonferrous metals.

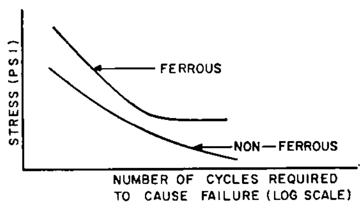


Figure 1. Typical *s-n* curves for ferrous and nonferrous metals.

The endurance limit for both ferrous and nonferrous metals has been found to be influenced by such factors as the temperature of test, speed of testing, size, shape and condition of the surface of the test specimen, medium in which the test was conducted, cold working, overstress, understress, and intrinsic stress. Further complications are added when the applied cyclic stresses are not completely reversed or where combinations of different types of stress are employed.

Repeated load or fatigue studies of such materials as portland cement concrete, asphalt paving materials, stone, and wood have been conducted and have produced a considerable amount of interesting and valuable information concerning their behavior under the action of repetitive loads. It has not been established whether these materials exhibit an endurance limit, nor have the studies led to a fundamental understanding of fatigue.

In 1943, Kersten (*1*) reported the results of both static and repeated load laboratory plate bearing tests on two statically compacted soils. Each load and unload cycle for the repeated load test required 5 min with one-half of this period devoted to each portion of the cycle. Usually no more than 300 repetitions were applied to each soil sample. Kersten's data showed that, for a given level of contact pressure, greater total penetrations were obtained under the larger bearing plates than under the smaller plates in both the static and repeated load test. For a given level of contact pressure, the initial penetration for the repeated load test was essentially the same as the corresponding penetration obtained for the static test. However, subsequent cycles of repeated loads produced increasingly larger penetrations which exceeded those obtained for the static test at all corresponding levels of applied contact pressure. For a given set of sample and test conditions, the penetration increased with larger contact pressures and with increasing number of cycles of repeated load. By selecting an arbitrary value for permissible plate penetration, Kersten was able to plot *s-n* curves (bearing value in pounds per square inch vs the number of repetitions) to show the effect of moisture content and base thickness on the shape and positioning of the *s-n* curves. These *s-n* curves resembled those for nonferrous metals, as shown in Figure 1.

Tschebotarioff and McAlpin (*2*) conducted studies of the effects of vibratory and slow repetitive loads with both controlled strain and controlled stress plunger-type loading devices. Sands, clays, and sand-clay mixtures were subjected to from 10,000 to 40,000 repetitions of vibratory and slow repetitional loads. For frequencies below the resonance range of well-graded, clean sand carrying a surcharge on the surface around the plunger, they reported that for "all densities and under all conditions of saturation, the effect on plunger penetrations of a vibratory force was several times greater than that of an equivalent static force." They found that "sand of uniform grading is particularly susceptible to the action of vibratory or slow repetitional forces. In the case of fine sand of uniform size, the continued application for 10 min of a vibratory force was found to produce deformations up to 140 times greater than those produced by a static force of equivalent magnitude" and that "the deformations produced in a soil by vibratory or by slowly repeated forces do not depend on the frequency of vibration or load repetition so long as the vibratory force itself is not magnified by resonance occurring somewhere within the vibrating system."

Buchanan and Khuri (*3*) conducted a series of repeated load triaxial tests on a lean clay soil as part of a study that dealt with soil support for rigid pavements. For one group of samples, the deviator stress was repeated while the confining pressure remained constant. For another group, both the deviator stress and confining pressure were applied and removed. Usually, no more than 40 or 50 repetitions of stress were applied to a given specimen. Their results indicated that, in the case of the lean clay, any repeated loading that exceeded the "elastic limit" caused total deformations of the soil whose "plastic" portion increased with the number of repetitions, whereas, the "elastic" portion remained fairly constant. Both the "elastic" and maximum "plastic" deformations increased with an increase in the magnitude of the applied deviator stress. However, for levels of repeated deviator stress below the soil's "elastic limit," no plastic deformations developed.

Following these earlier studies, extensive laboratory investigations were conducted by Seed and co-workers (*4* through *9*) in which compacted soil specimens were subjected to repeated applications of stress in triaxial shear. A summary of the important results of this research that are pertinent to the present study follows.

For the soils studied, and for frequencies in the range of 1 to 20 applications per min, specimen deformation depended on the number of stress applications but was independent of the frequency of applications provided the degree of saturation was not high or the soil did not possess appreciable thixotropic characteristics. For a given set of sample conditions, a given level of repeated axial load ultimately produced greater deformation than a gradually applied or static load of equal magnitude. Soil specimens may fail suddenly after having withstood a number of repeated deviator stresses of the same magnitude. The strength and stiffness of clay-like soil was increased by first applying several thousand cycles of either a repeated deviator stress of small magnitude or a repeated confining pressure. This was especially true for lower degrees of saturation even though there was no appreciable sample deformation or thixotropic effects during the load applications. For higher degrees of saturation, strength did not increase appreciably, although increased stiffness was apparent. The stiffening effect was gradually reduced as the repeated deviator stress was increased, and it was destroyed by large sample deformations. This strengthening and stiffening effect was reported to be absent or minor in sands.

After progressively increasing the repeated deviator stress on a series of identical specimens and comparing the resulting deformations with similar samples having no previous stress history, it was found that there is no simple means of evaluating the cumulative effect of a series of applications of deviator stress of different magnitudes from data concerning their individual effect. Moreover, two different soils with initial sample conditions that produced nearly identical stress-strain curves in static triaxial tests developed different total and elastic deformation in the repeated load triaxial test. It was thus apparent that for some soils, at least, the deformation characteristics, as determined by static triaxial test, were not indicative of their behavior under repeated load conditions.

Thixotropic effects were found to be an important factor in both repeated and static load tests on compacted clays having high degrees of saturation.

The complexities of the repeated stress-deformation relationships in compacted fine-grained soils suggested that rationalization of the phenomena involved might be facilitated by establishing limiting stress (rather than limiting deformation) criteria for failure under the action of repeated loads.

GENERAL HYPOTHESES

Initial efforts to obtain experimental information that would lead to a strength criterion for failure were influenced by the work of Seed, Chan, and Monismith (*4*). They had reported that a soil specimen might withstand a considerable number of repeated applications of a given deviator stress without any apparent sign of excessive deformation and then fail rather suddenly after a small number of additional applications of the same deviator stress. This indicated that an endurance strength for soil might exist that could be determined by conducting triaxial tests on a series of identical specimens subjected to increasingly higher levels of repeated deviator stress. The anticipated results from such a series of tests are shown in Figure 2. A plot of the magnitude of the applied deviator stress σ_r vs the number of stress repetitions at failure would then give the *s-n* curve as shown in Figure 3.

The belief that such a relationship might exist was strengthened by results of studies on bituminous materials conducted by Wood and Goetz (*10, 11*) and Goetz, McLaughlin, and Wood (*12*). They found that for levels of repeated axial stress equal to one-fourth of the static compressive strength, the relationship between permanent sample deformation and number of repetitions, on a logarithmic scale, was linear. For levels of repeated deviator stress equal to one-half the static compressive strength, the relationship was linear for the first few cycles but subsequently became nonlinear with the deformation increasing rapidly on the application of only a few additional stress repetitions.

At the same time, it was postulated that a stress level might exist below which no sudden increase in deformation would occur regardless of the number of stress repetitions, and that this stress level might exist even if no continuous *s-n* curve developed (Fig. 4). As the intensity of σ_r increases, it was postulated that a condition would be reached for which the deformation curve would, after the first few cycles, rise at a constant slope; *i.e.*, $d\delta/dN$ = a constant. For levels of σ_r less than this critical condition (σ_{r_4} in Fig. 4), the curves would eventually level off and $d\delta/dN$ would approach zero. For levels of σ_r in excess of the critical value, the curves

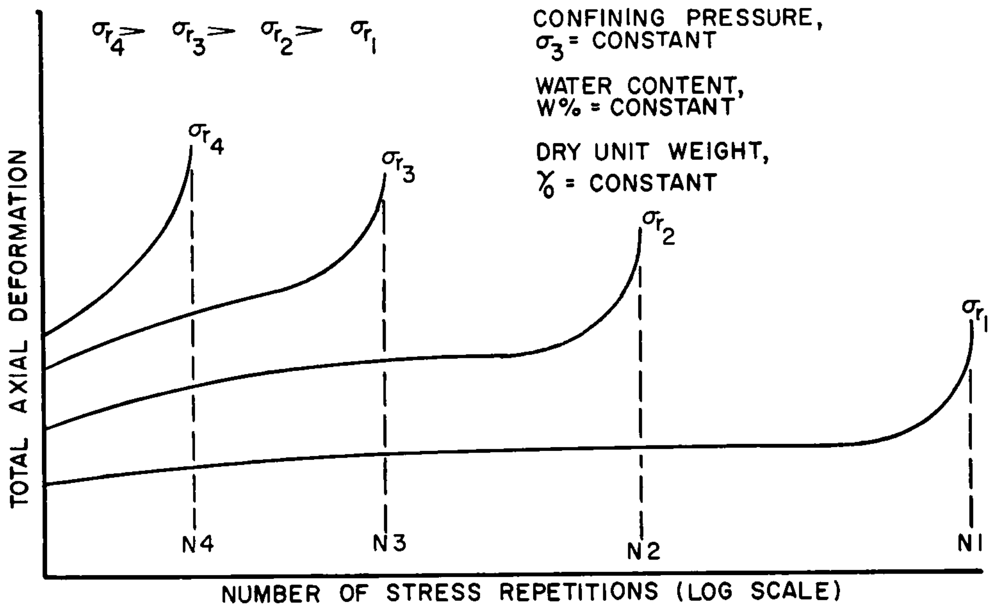


Figure 2. Hypothetical deformation vs logarithm of number of stress repetitions.

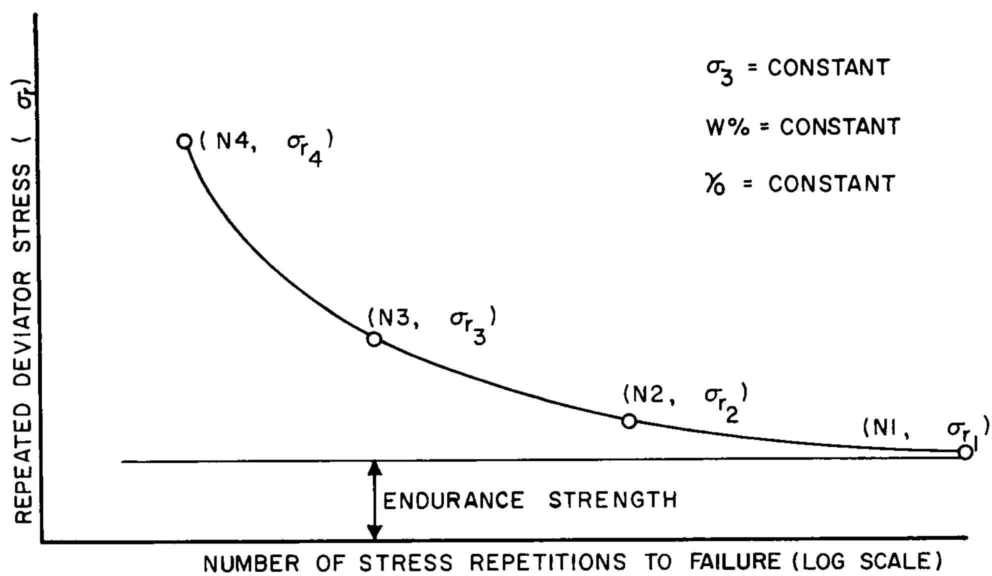


Figure 3. Hypothetical *s-n* curve for soil.

would eventually become concave upward and failure would occur either along a shear surface or by bulging.

Indirect evidence existed which indicated that the relationship shown in Figure 4 might develop for soils. Finnie and Heller (*13*) show creep strain vs time curves for two metals

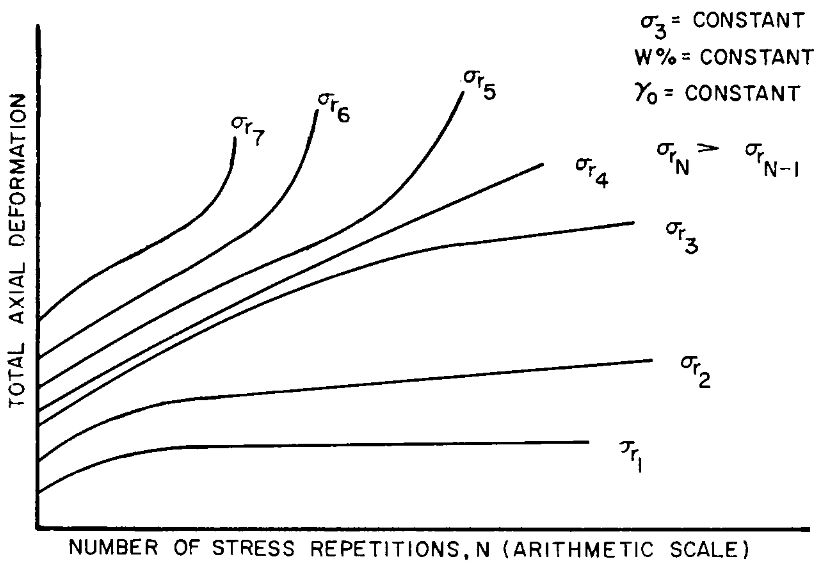


Figure 4. Hypothetical deformation vs number of repetitions.

at various levels of applied stress. These typical creep strain vs time curves, in which fracture occurs, resemble those shown in Figure 4 for which σ_r exceeds the critical stress level σ_{r4}. Moreover, Casagrande and Wilson (*14*) reported the results of creep tests on fine-grained soils in which the creep strain vs time curves for failure conditions also resembled the curves in Figure 4 at stress levels above the critical value. They stated that "failure was invariably preceded by a reversal of slope of the time-deformation curve, followed by continuous deformation at an increasing rate." Recently, after the present study was virtually completed, Vialov and Skibitsky (*15*) presented for a frozen sandy soil creep strain vs time data that have marked similarities to the Figure 4 data.

Although it was realized that creep tests may not produce the same effects as repeated load tests, the similarities were sufficient to justify further investigation. An experimental program was undertaken to check the validity of both postulated criteria.

SOILS STUDIED

Three soils were chosen for this study: a micaceous silt (Soil A) obtained in Charlottesville, Va.; a limestone residual clay (Soil B) sampled near the junction of US 11 and US 340 near Greenville, Va.; and a sand-clay (Soil C) obtained from a highway cut on Va. 639 about 6½ mi east of Ladysmith, Va.

These soils were chosen for the following reason: (a) they were typical of soils that are widespread over three or more physiographic provinces of North America and as a result are frequently encountered in engineering work; (b) the variation in physical characteristics between these three soils was sufficient to reflect a fairly broad spectrum of engineering behavior; and (c) the ease of sample preparation, due to their comparative homogeneity, and

TABLE 1
INDEX PROPERTIES AND MINERALOGICAL DATA

Soil Characteristic	Micaceous Silt (Soil A)		Limestone Residual (Soil B)		Sand-Clay (Soil C)	
Sp. gravity of solids	2.76		2.76		2.65	
Atterberg limits:						
Liquid limit	34		65		16	
Plastic limit	31		32		15	
Plasticity index	3		33		1	
Shrinkage limit	27		19		13	
Grain-size distribution:						
D_{60} (mm)	75×10^{-3}		5×10^{-3}		260×10^{-3}	
D_{10} (mm)	8×10^{-3}		—		4×10^{-3}	
D_{60}/D_{10}	9.4		—		65	
Clay Fraction	7.5		49.0		6.0	
Mineral. comp.[1]:						
Kaolinite	5.0		34		4.0	
Illite-vermiculite	1.5		10		1.0	
Hemalite	—		5		—	
Goethite	0.5		—		0.5	
Halloysite	Minor		Minor		Minor	
Calif. Bearing Ratio (soaked)	<1		5		117	
Class. unified	ML		CH-MH		SC	
Impact compaction	Stand. Proctor	Modified AASHO	Stand. Proctor	Modified AASHO	Stand. Proctor	Modified AASHO
Opt. moist. cont. (%)	16.7	12.3	29.0	22.3	8.4	6.2
Max. dry unit wt. (pcf)	105.0	118.7	89.0	104.5	127.7	140.2

[1] Approximate percent of soil fraction.

the absence of appreciable quantities of coarse particles.

Piedmont Micaceous Silt (Soil A)

Soil A is a residual soil formed from a tan-colored quartz mica schist which is a part of the Lynchburg Formation. Its pedological classification is Elioak silt-loam. It was obtained from pier excavations at depths ranging from 6 to 10 ft below the ground surface in the C-horizon. As the name implies, it contains a high percentage of fine mica particles, many of which were barely visible to the unaided eye. Soil A is a fine sandy-silt of very low plasticity. Its dry strength is low, and compacted specimens have a tendency to swell under low confining pressures.

Index properties and mineralogical data for this soil are given in Table 1. Figure 5 shows a grain-size distribution curve, and Figure 6 shows impact and static compaction curves.

Valley of Virginia Limestone Residual (Soil B)

Samples of Soil B were obtained from a depth of 1 to 2 ft below the surface and adjacent to a borrow pit that had been used in the construction of a nearby highway fill. When moist, its color is a rich red-brown and when dry, a medium red. Soil B is a highly plastic clay with a considerable fraction of silt-size particles. It has a high dry strength. In the undisturbed state, it appears to have a fragmented structure. When remolded, it is highly impervious.

The limestone rock from which it derived is a part of the Beekmantown Formation, and the soil as obtained from the pit contained a few small chert nodules which were removed by sieving. Soil B is typical of many

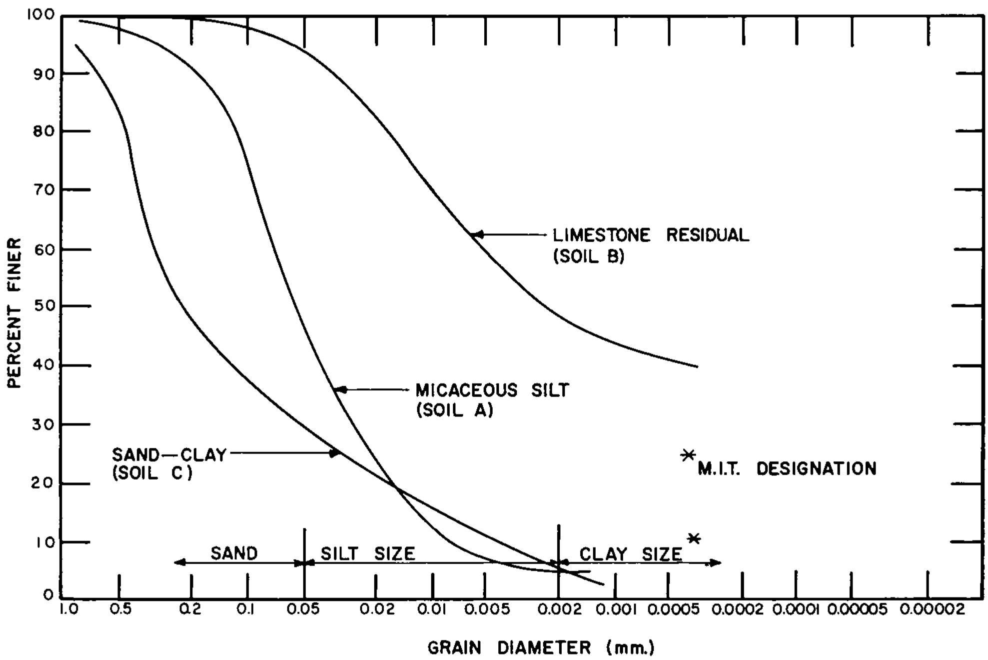


Figure 5. Grain-size distribution curves.

limestone residual soils found in the United States east of the Mississippi River. It is classified pedologically as Frederick clay-loam from the B-horizon, primarily.

Index properties and mineralogical data for this soil are given in Table 1. The grain-size distribution curve is shown in Figure 5 and impact and static compaction curves in Figure 7.

Coastal Plain Sand-Clay (Soil C)

Soil C was sampled from a depth of about 4 to 6 ft below the ground surface from what appears to be the Brandywine Terrace. In the natural state it was hard and offered considerable resistance to excavation with a hand shovel.

Soil C is a tan-colored silty sand with sufficient clay binder to impart to it moderate dry strength. The clay binder provided just enough cohesion in the moist, compacted samples to permit handling, trimming, and testing. Nevertheless, the portion of the soil that passed the No. 40 sieve is essentially nonplastic.

A small quantity of coarse sand and fine gravel present in the sample was removed by sieving.

Compacted specimens of this soil were not susceptible to appreciable swelling.

Surface soils similar to Soil C are scattered widely throughout much of the Atlantic Coastal Plain Province. They have been encountered and utilized extensively in highway and other engineering work. Pedologically it is thought to be the Sassafras soil.

Index properties and mineralogical data for Soil C are given in Table 1. A grain-size distribution curve is shown in Figure 5 and impact and static compaction curves in Figure 8.

APPARATUS AND PROCEDURE

The experimental devices and techniques employed for the preparation

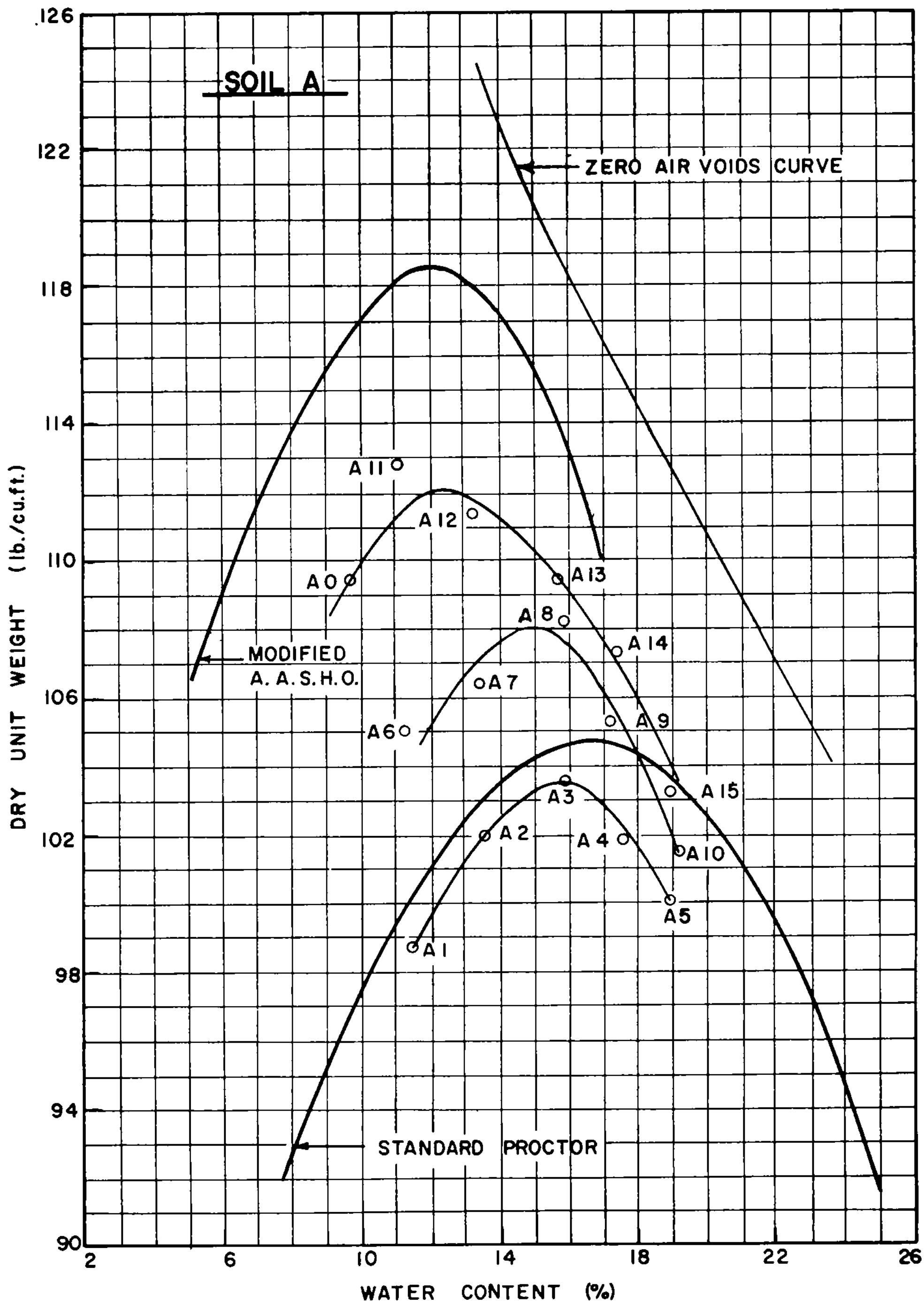


Figure 6. Compaction curves, Soil A.

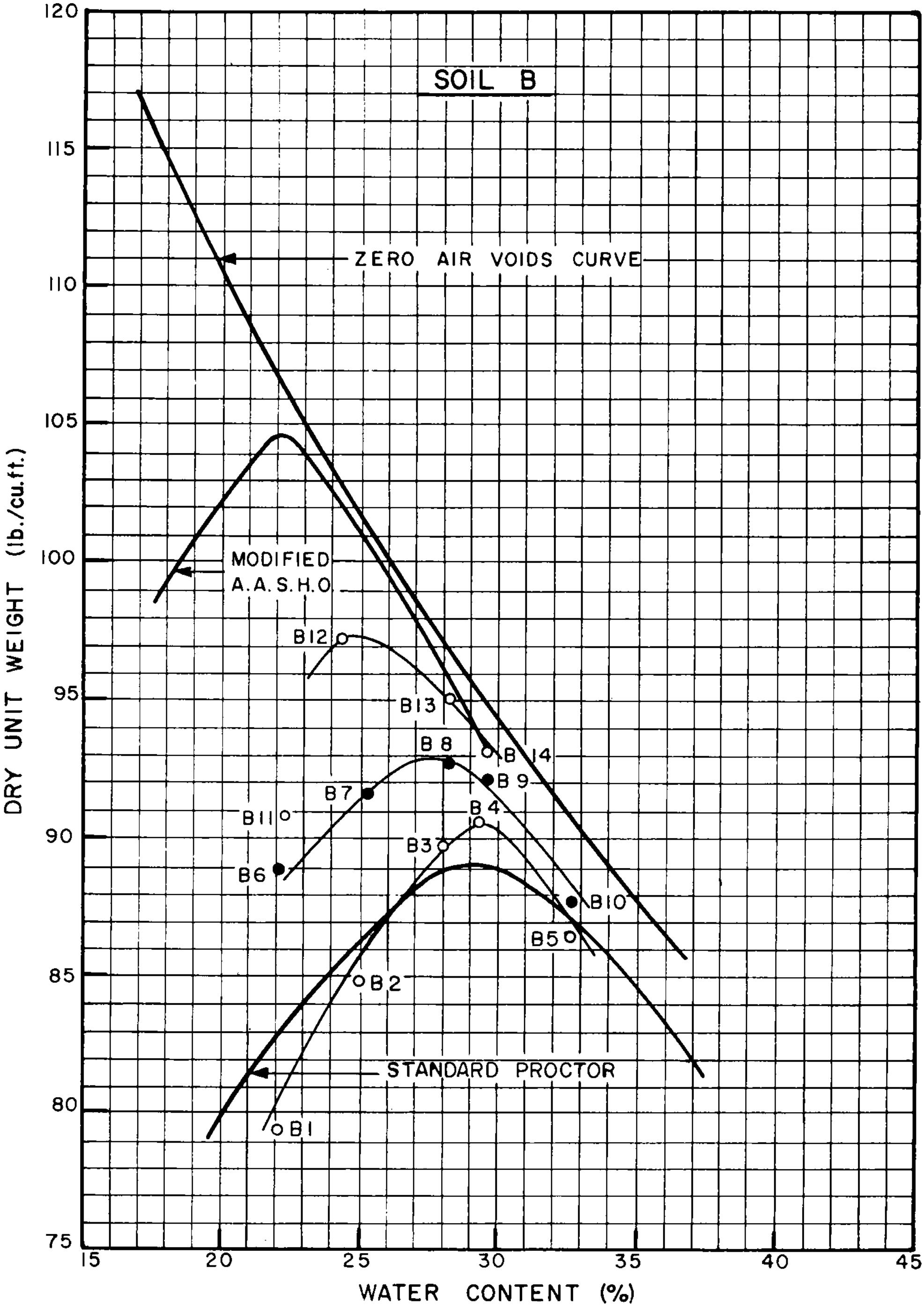


Figure 7. Compaction curves, Soil B.

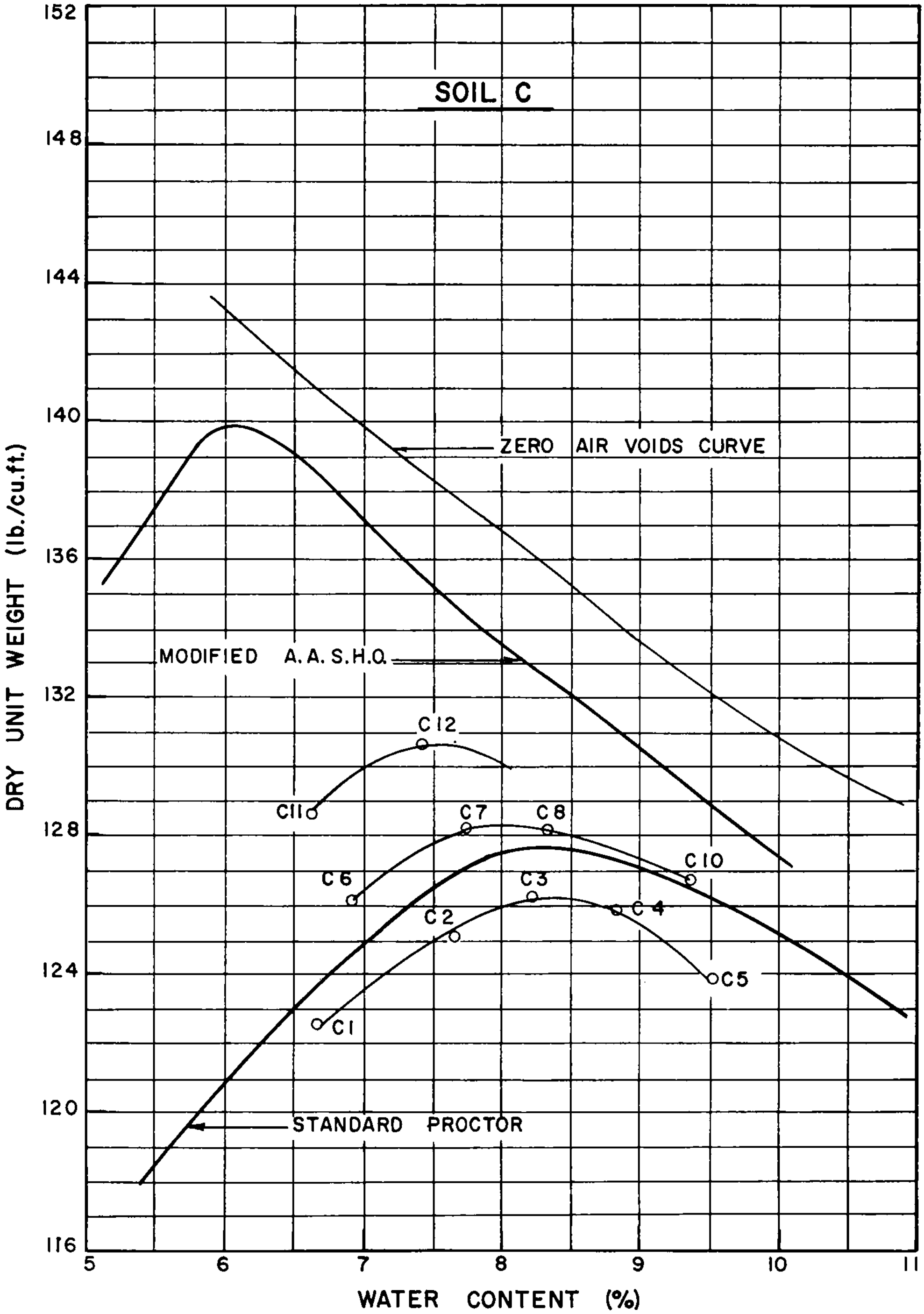


Figure 8. Compaction curves, Soil C.

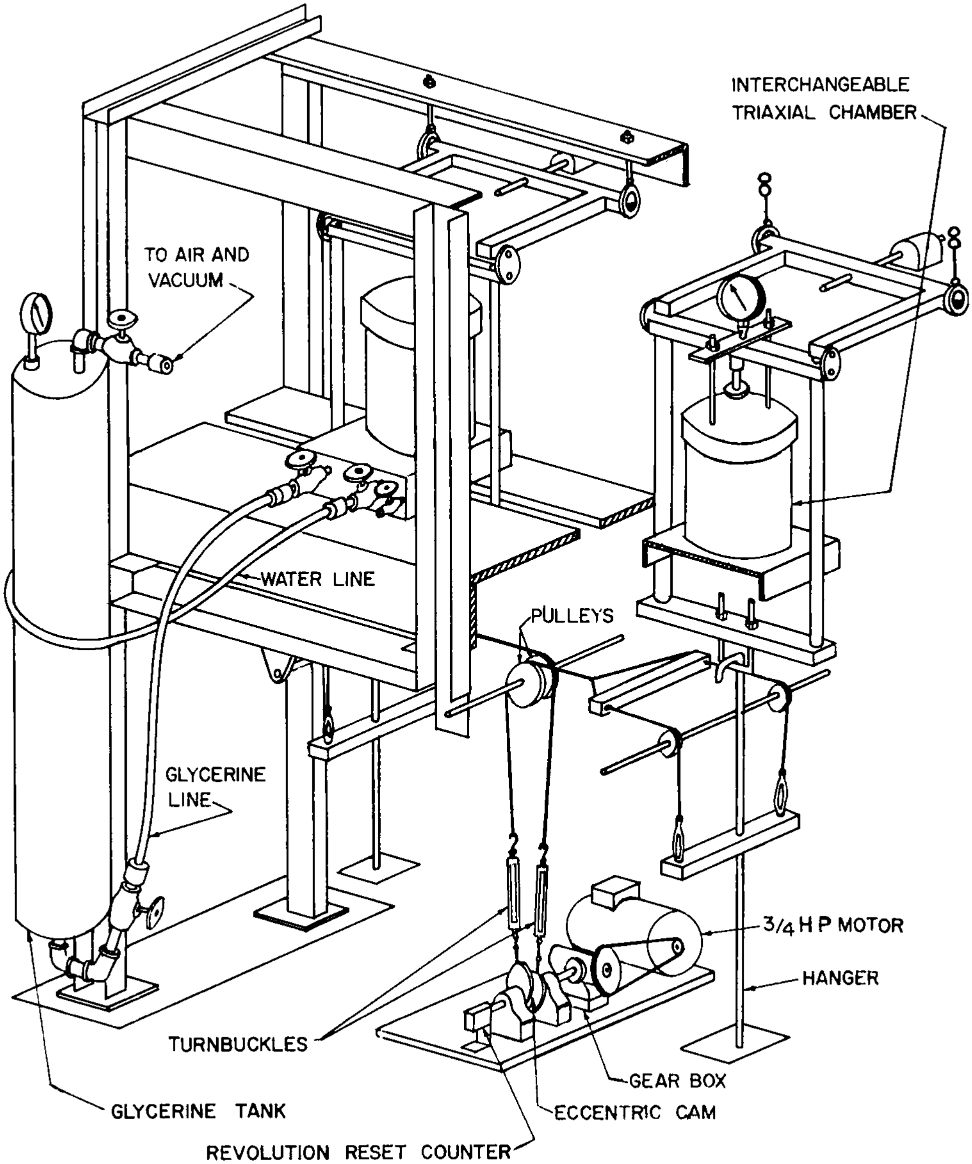


Figure 9. Repeated load triaxial device.

of raw soil, static compaction of soil cakes, cutting of duplicate specimens from soil cakes, preservation of samples, preparation of test specimens, and triaxial testing with gradually applied loads were similar in most respects to those previously employed and described by Leonards (*16*).

All triaxial test specimens were 2.80 in. long and 1.40 in. in diameter and were cut from cylindrical cakes approximately 10 in. in diameter and

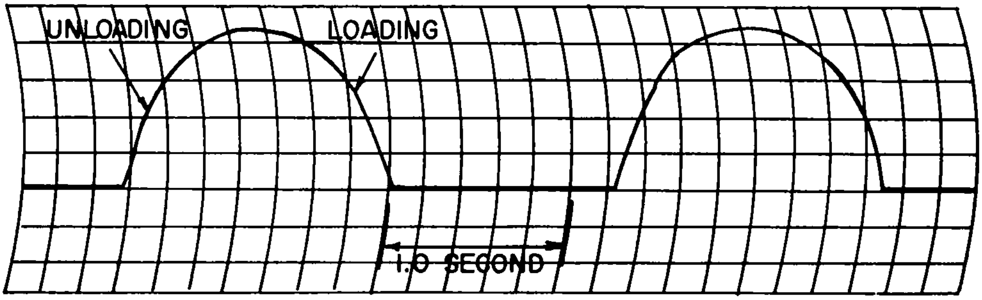


Figure 10. Loading cycle waveform.

$3\frac{1}{2}$ in. high that had been prepared by a static compaction process. Usually, 20 identical specimens were obtained from each of these soil cakes. From 12 to 15 of these soil cakes were prepared for each soil so that three levels of compaction and five levels of moisture content for each level of compaction were obtained for each soil. The degree of saturation of the as-compacted samples varied from 42 to 80 percent for Soil A, from 52 to 96 percent for Soil B, and from 50 to 80 percent for Soil C. The coordinates of points (such as A-2 and B-7) on the compaction curves (Figs. 6, 7, and 8) represent the dry unit weight and molding water content of each soil cake. The samples from each soil cake were stored for at least one month before being tested.

Repeated Load Triaxial Testing

The repeated load triaxial devices employed consisted of two modified double-bay triaxial testing units. The units contained necessary piping and connections for circulating water through the test specimens. Glycerin was used initially in the triaxial cell to which a confining pressure was applied from an air reservoir.

A schematic diagram of the mechanism for applying and removing the axial stress, including the cable hanger system, motor, speed-reducing pulley and gear system, pillow blocks, and cams employed in this device, is shown in Figure 9. The mechanism was designed as a compromise between convenience of operation and low initial cost. Minor repairs, such as replacing worn-out cable, were occasionally required; otherwise, the equipment was rugged and served satisfactorily. The fact that the two cams were mounted 180 deg out of phase relieved the system of much of its work, for one cam was lifting the weights while the other was lowering them, and the torques applied to the main shaft essentially canceled one another. Loads of as much as 160 kg were applied to individual specimens by this mechanism.

The frequency of load application was held essentially constant between 20 to 22 cycles per min. The form of the load-unload cycle employed was obtained with the aid of a Brush recorder (Fig. 10). The waveform can be varied by employing different shaped cams. The duration of the loading cycle can be varied by adjusting the large turnbuckle attached to the cam housing.

Some difficulty with the control of load duration was experienced in the early stages of the study. This problem was more serious for those tests in which large initial deformations developed. It resulted from the loop on the loading hanger moving downward with the deformation of the specimen while the hook on the loading rod moved up and down with respect to a fixed position. The load duration after the first cycle was, therefore, reduced until an adjustment of the large turnbuckle could be made, thereby lowering the hook on the loading rod. The problem was largely overcome by estimating the initial deformation and lowering the loop of the loading rod a corresponding amount before the start of the test. The gap between hanger and loading rod loops was set with "go" gages.

Test Procedures

The triaxial chamber containing a test specimen was filled with glycerin and the confining pressure applied. All drainage valves on the triaxial equipment were closed.

The weight comprising the axial load was then added to the hanger system, first making certain that the loading hooks were disengaged. The gear system was turned by hand until one set of the weights was raised to its highest point. Further lifting of the weights was accomplished by adjusting the large turnbuckle. Once this pair of loading hooks cleared each other, they were engaged and the weights were again lowered into position for starting the test. The gap between the two hooks was then given a final adjustment to account for the initial sample deformation. To this point no axial load had been applied to the specimen. The loading hooks for the companion specimen remained disengaged during these operations.

With the deformation dial for the first specimen and the counter set at zero, the motor was started and simultaneous readings of load application and axial deformation (to the nearest 0.001 in.), were recorded at 1, 2, 5, 10, and 20 cycles. The motor was stopped at 30 cycles and the loading hooks for the second and companion sample were positioned. The initial gap between these two hooks was set and the deformation dial gage for this specimen was set at zero. During this short interval while the motor was stopped, the axial load was applied continuously to the first specimen; however, the axial deformations observed were small and had little effect on the final results.

The motor was again started and readings of the number of load repetitions and corresponding axial deformations were recorded for both specimens. Readings were taken at predetermined intervals until failure of the specimen occurred or at least 40,000 stress cycles had been applied. Most samples were subjected to from 60,000 to 80,000 stress cycles and a few samples were subjected to over 400,000 repetitions. With few exceptions, a complete set of rebound readings was obtained for each sample. Following the completion of each test the sample was removed from the triaxial chamber for moisture content determinations.

Of the 20 identical specimens available from each soil cake, 9 as-compacted samples were subjected to repeated loads (3 samples at each of 3 levels of confining pressure). Six samples were saturated with water and then subjected to repeated loads (3 samples at each of 2 levels of confining pressure). The 5 remaining samples were subjected to conventional triaxial tests on both as-compacted and saturated samples.

For repeated load tests on soaked specimens, water was circulated upward through the specimen under a pressure equal to about one-fourth of the applied confining pressure until sample deformations had essentially ceased. Shortly before the start of repeated load applications the excess water pressure was removed and all drainage valves were closed. In all other respects the tests were similar to the as-compacted specimen tests.

A series of special tests was conducted to check the effects of thixotropic action on each of the three soils studied. Twenty identical specimens were prepared for each soil. Immediately after these samples had been prepared, 10 were subjected to a series of conventional and repeated load triaxial tests in which both as-compacted and saturated samples were employed. The 10 remaining samples in their protective wax and aluminum foil covering were stored in the humid room for two weeks and then subjected to the same series of tests.

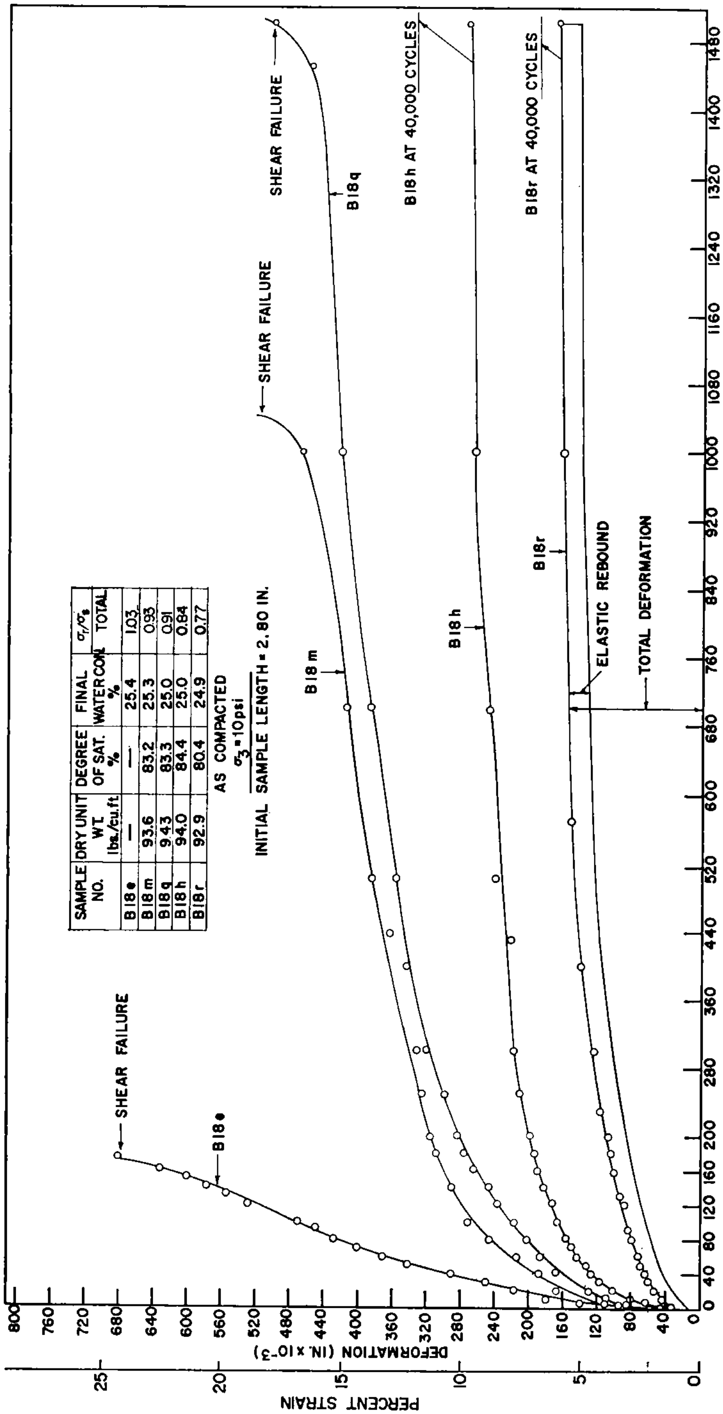
RESULTS

The test data obtained are presented in a series of eight comprehensive tables, reported elsewhere (*17*). Only a summary of the most pertinent results are included in this report. Figure 11 shows a set of deformation vs number of load repetition curves obtained for as-compacted specimens of Soil B, the limestone residual soil. Five levels of repeated deviator stress at one confining pressure are represented. The curves show that a large portion of the total deformation at any given stress level occurred during the first few hundred cycles. For samples Bl8r and Bl8h, which were subjected to lower levels of repeated deviator stress, σ_r/σ_s (ratio of the repeatedly applied deviator stress to the static deviator stress causing shear failure) equal to 0.77 and 0.84, respectively, the curves leveled out; *i.e.*, the slope $d\delta/dN$ approached zero; after several thousand cycles, the curves retained their shape even though more than 40,000 cycles were applied. For samples Bl8m and Bl8q, which were subjected to high levels of repeated deviator stress, the slopes of the curves were greater than for specimens Bl8h and Bl8r. Furthermore, after 950 and 1,500 repetitions, respectively, both Bl8m and Bl8q failed suddenly during the application of only a few additional load repetitions. Sample Bl8e was subjected to a still higher level of repeated deviator stress, $\sigma_r/\sigma_s = 1.03$. This sample developed a slip plane after 120 stress repetitions. The elastic rebound for sample Bl8r is also shown. At this low stress level, the rebound reaches an equilibrium value after a small number of stress repetitions has been applied. The ratio, σ_{rc}/σ_s, representing the postulated failure criterion is between 0.84 and 0.91.

A typical set of deformation vs number of load applications curves for the sand-clay, Soil C, is shown in Figure 12. The curve for specimen C9^{2}x closely approximates the failure condition which was postulated earlier. The slope was essentially constant after the first few cycles. For levels of deviator stress less than the critical value, the curves leveled out; and for levels in excess of the critical value, a shear failure developed.

Although slight variations between specimens and slight differences in load duration made the exact determination of the critical ratio of σ_r/σ_s difficult, it was usually possible to bracket this condition closely and consistently for a given set of initial sample conditions with no more than three test specimens. By interpolation, a reliable measure of the critical ratio was obtained for both as-compacted and soaked specimens of Soils B and C at various levels of dry unit weight, water content, and confining pressure.

Curves representing the variation of the critical ratio of σ_r/σ_s with water content at two levels of compactive effort and confining pressure are shown for Soil B in Figure 13. Both soaked and as-compacted samples are represented by these curves. Average final water contents were used to plot the as-compacted curves. For corresponding initial conditions these same water contents were employed to plot the curves for soaked specimens. Values of the average final water content of soaked specimens are shown in parentheses beside each



SAMPLE NO.	DRY UNIT WT. lbs./cu.ft	DEGREE OF SAT. %	FINAL WATER CON. %	σ_r/σ_s TOTAL
B18e	—	—	25.4	1.03
B18m	93.6	83.2	25.3	0.93
B18q	94.3	83.3	25.0	0.91
B18h	94.0	84.4	25.0	0.84
B18r	92.9	80.4	24.9	0.77

Figure 11. Deformation vs number of load applications, Soil B.

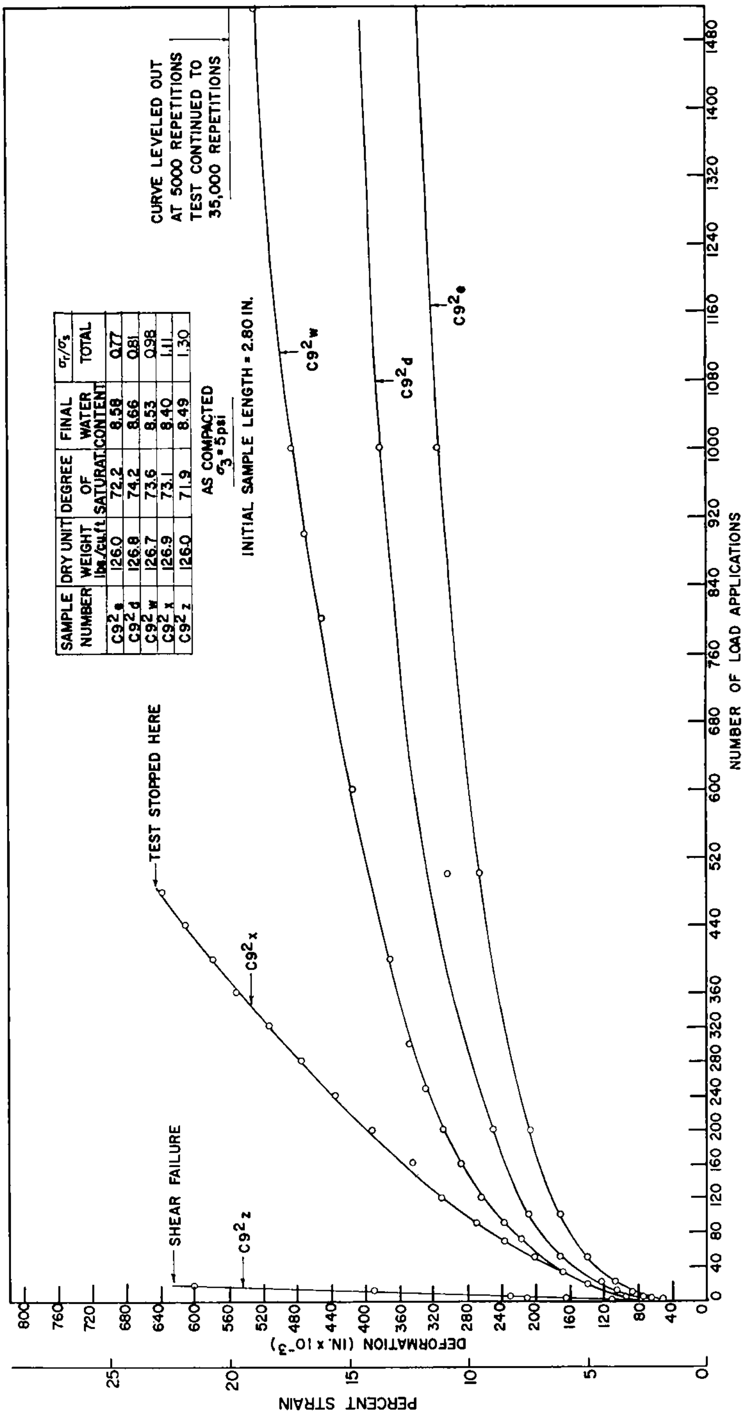


Figure 12. Deformation vs number of load applications, Soil C.

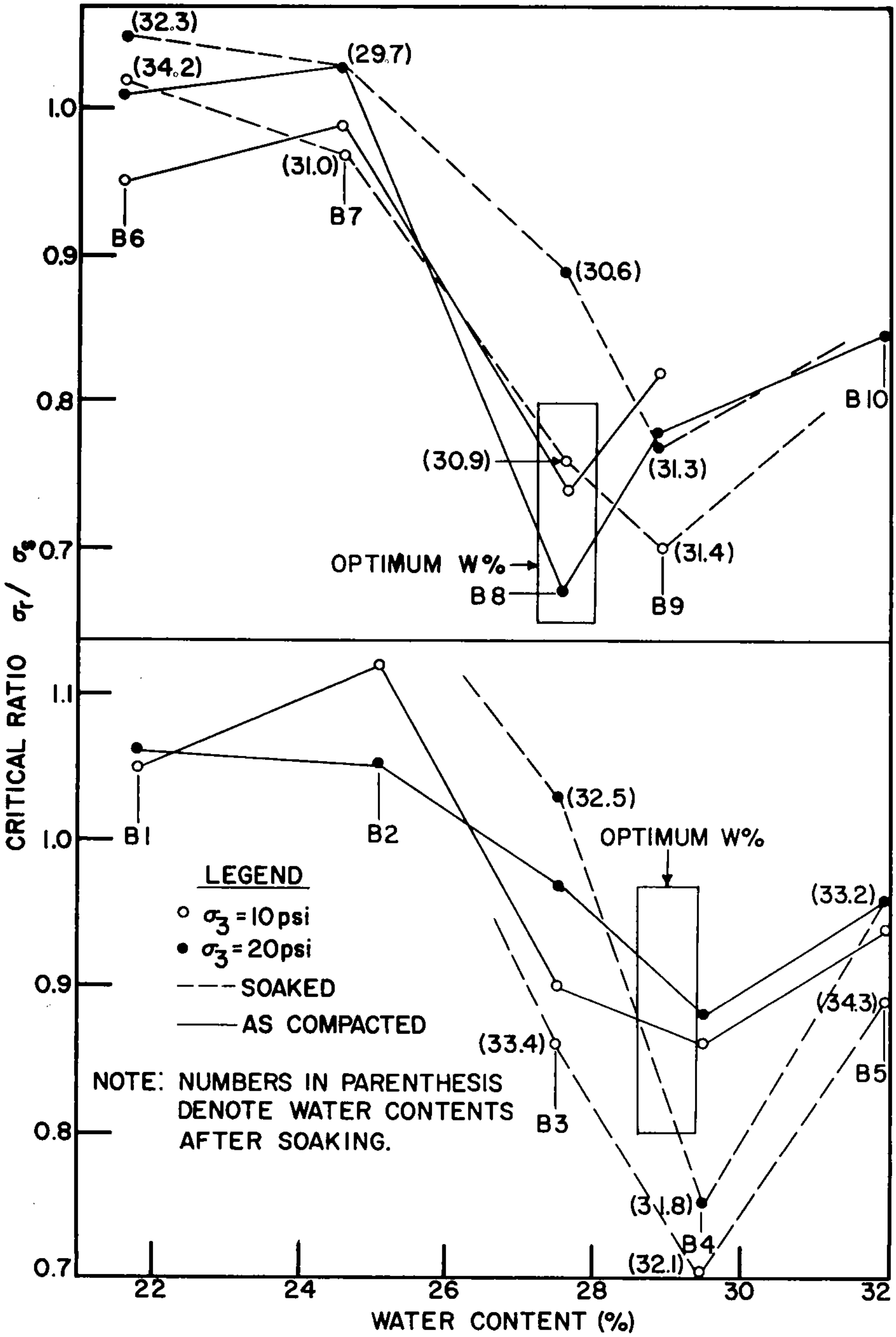


Figure 13. Critical ratio, σ_r/σ_s vs water content, Soil B.

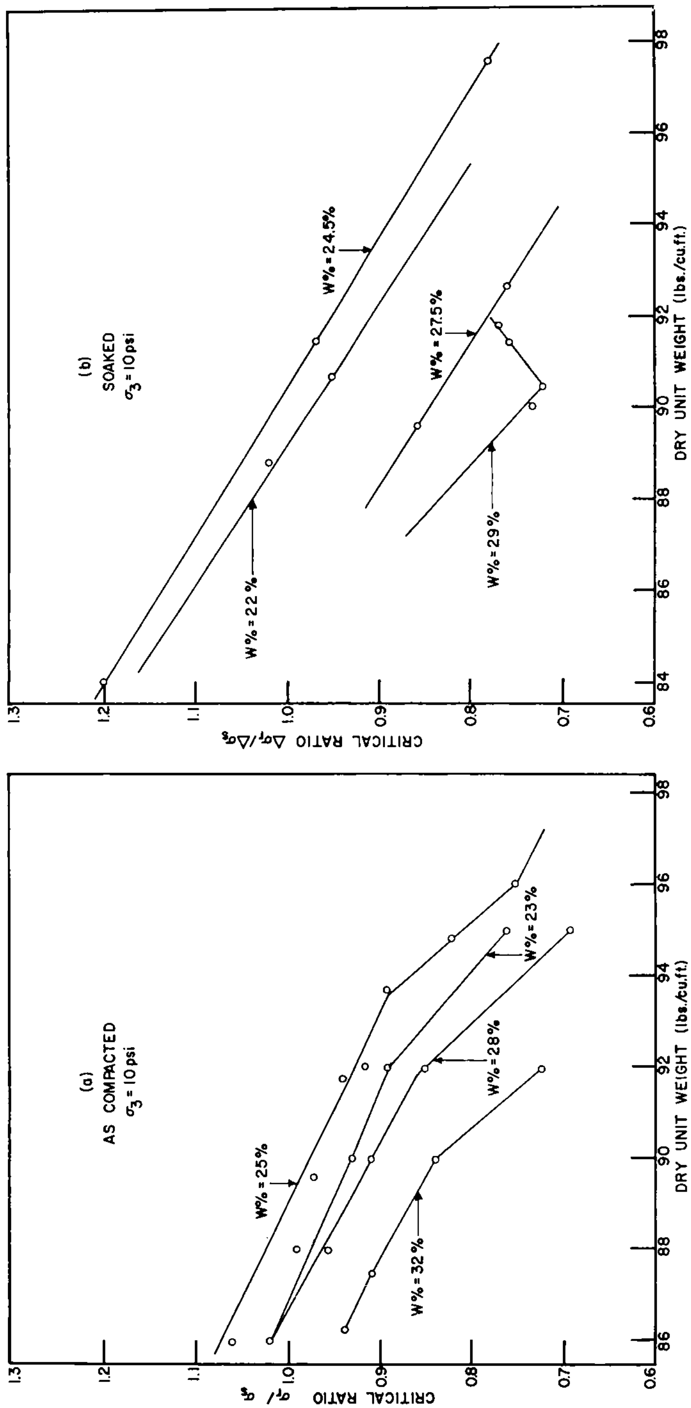


Figure 14. Critical ratio, σ_r/σ_s vs dry unit weight at constant water content, Soil B.

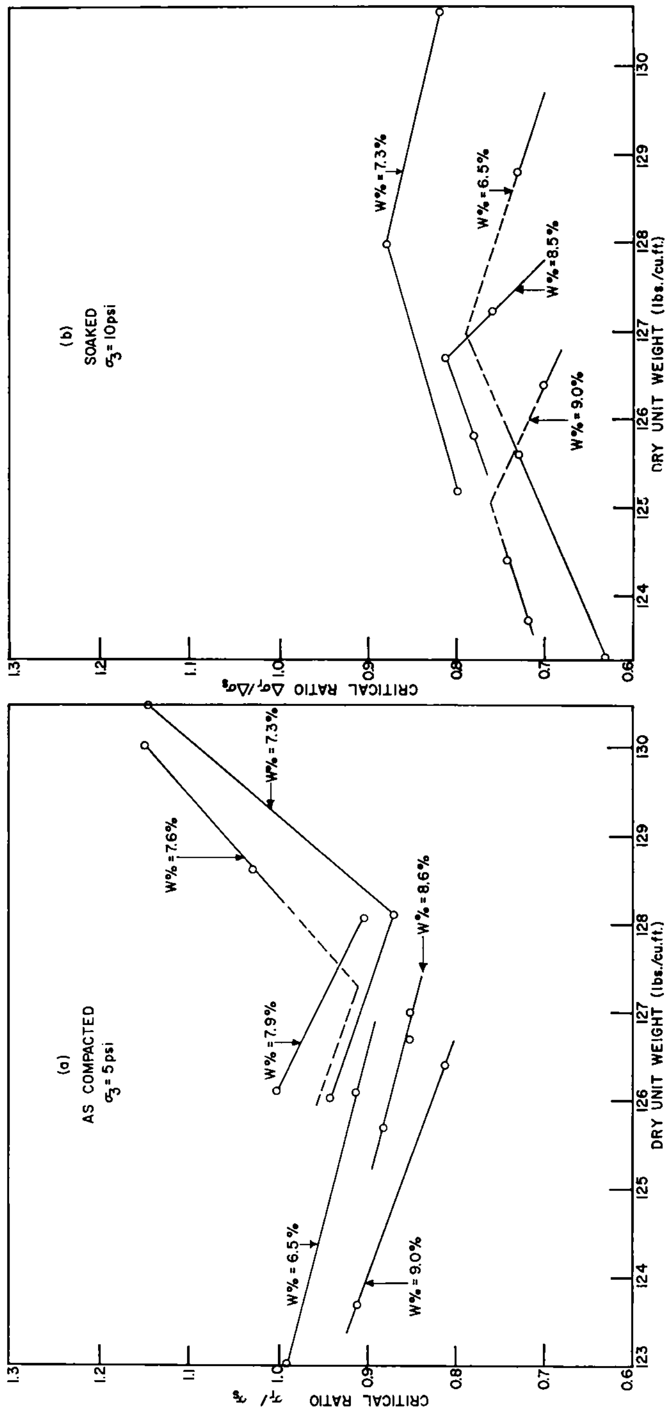


Figure 15. Critical ratio, σ_r/σ_s vs dry unit weight at constant water content, Soil C.

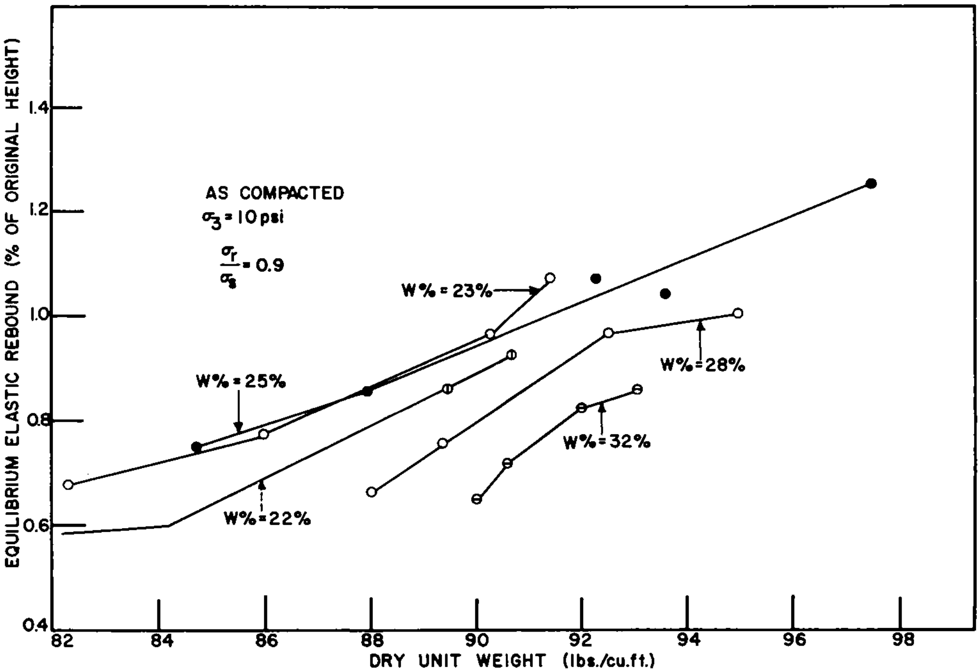


Figure 16. Elastic rebound vs dry unit weight at constant water content, Soil B.

plotted point. It is significant that the critical ratio of σ_r/σ_s drops considerably in the water content range near the optimum for the compactive effort used.

Curves illustrating the relationship between the critical value of σ_r/σ_s and dry unit weight at constant water content for both soaked and as-compacted specimens and one level of confining pressure are shown for Soils B and C in Figures 14 and 15. At any given water content, the critical ratio of σ_r/σ_s drops significantly as the compactive effort is increased in the case of Soil B. No such pattern is observed for Soil C.

An essentially complete set of rebound readings was obtained for all samples subjected to repeated stress applications. For the three soils tested and for any given set of initial conditions, the elastic rebound eventually reached a constant or equilibrium value provided the stress level was less than the strength under repeated loading. An interesting relationship between the equilibrium elastic rebound and dry unit weight (at constant moisture content) developed in the case of the limestone residual (Soil B). This is shown in Figure 16 for a single level of confining pressure and indicates that for Soil B the equilibrium elastic rebound increased with increasing dry weight at constant water content. Moreover, for a given compactive effort, the equilibrium elastic rebound reached its maximum value, for Soil B, at or near optimum moisture content. This is shown in Figure 17 for three levels of compactive effort and two levels of confining pressure. For Soil A, the micaceous silt, the equilibrium elastic rebound was not strongly influenced by either

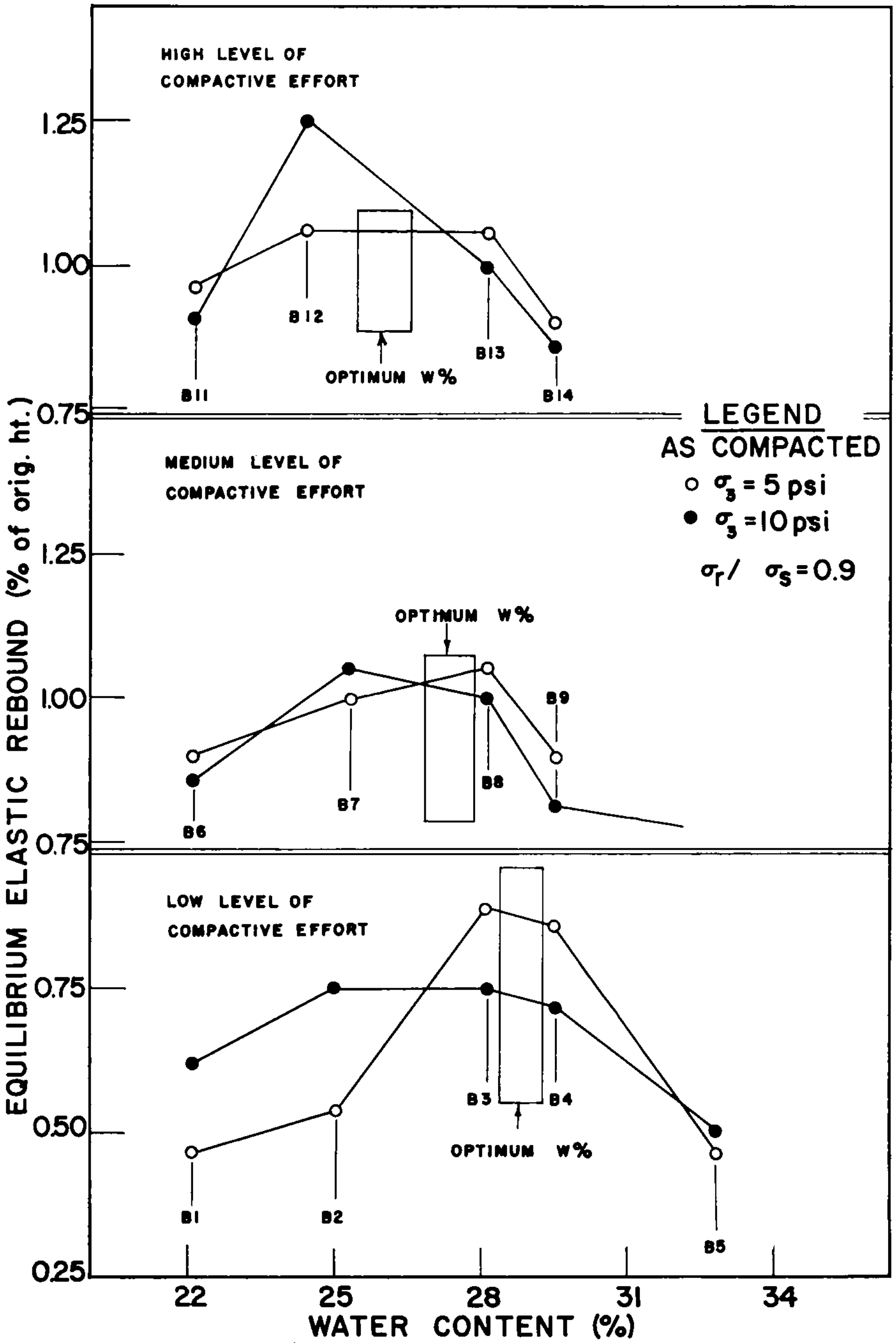


Figure 17. Equilibrium elastic deformation vs molding water content, Soil B.

dry unit weight, degree of compaction, or water content. It was, however, influenced by both confining pressure σ_3 and the ratio of repeated deviator stress to conventional or static deviator stress σ_r/σ_s as shown in Figure 18.

Special tests were conducted to determine the effects of thixotropy on the three soils employed. For the soils studied, for the degrees of saturation obtained by the compaction process employed, and for the period of sample aging used, no measurable thixotropic effects were observed.

During the course of the experimental phase of this study it became apparent that some samples were losing moisture during either the period of sample preparation or that of testing. A study conducted to determine the cause of this drying showed little, if any, migration of moisture between the ends and center of the specimen. However, a radial migration of moisture was noted; *i.e.*, the outer portions of the sample were drier than the interior. Only those specimens that remained in the triaxial chamber more than 24 hr (30,000 load repetitions) gave evidence of this radial drying. It was concluded that the glycerin, which has an affinity for water and was employed as the liquid confining medium, was the cause of this difficulty even though double membranes separated by a silicone grease were used. Distilled water was subsequently employed as the confining medium for repeated load tests; for the times involved in this testing program, no appreciable further changes in water content were noted.

The data obtained using glycerine as the confining medium showed that in all but a few instances the critical value of σ_r/σ_s was determined on samples that were not subject to the effects of this drying. That is, the pattern of specimen deformation or failure was sufficiently established in most instances well before 30,000 cycles had been applied.

ANALYSIS OF RESULTS

The performance of a highway pavement cannot be assessed solely from the behavior of a single component. Failure may occur from a lack of stability in the wearing course, from deflections in the base course due to traffic compaction, from cumulative shear deformations in the subgrade, and from temporary rebound in both the subgrade and base courses. Furthermore, the pattern of deflections produced, which is influenced by the intensity, shape, and position of the contact stresses and by the speed at which the vehicle traverses the pavement, is of critical importance in determining whether the pavement can withstand the associated deflections and rebound conditions. Nevertheless, from the standpoint of subgrade behavior, the implications of the results previously discussed are clear. These are shown in Figure 19.

At a low stress level (Curve A, Fig. 19), the deflection pattern will be too small to cause distress in the pavement. Furthermore, after a few thousand load repetitions, the additional cumulative deflections will be small and the subgrade will behave essentially as an elastic system. Thus, mathematical models that replace the subgrade by an elastic system could predict its behavior with satisfactory precision. In many cases, the required thickness of a flexible pavement to achieve this condition for heavy truck loading is considerably greater than those obtained from the design procedures used in this country. In Europe, the practice of constructing semirigid base courses (9 to 12 in. of soil cement, bituminous

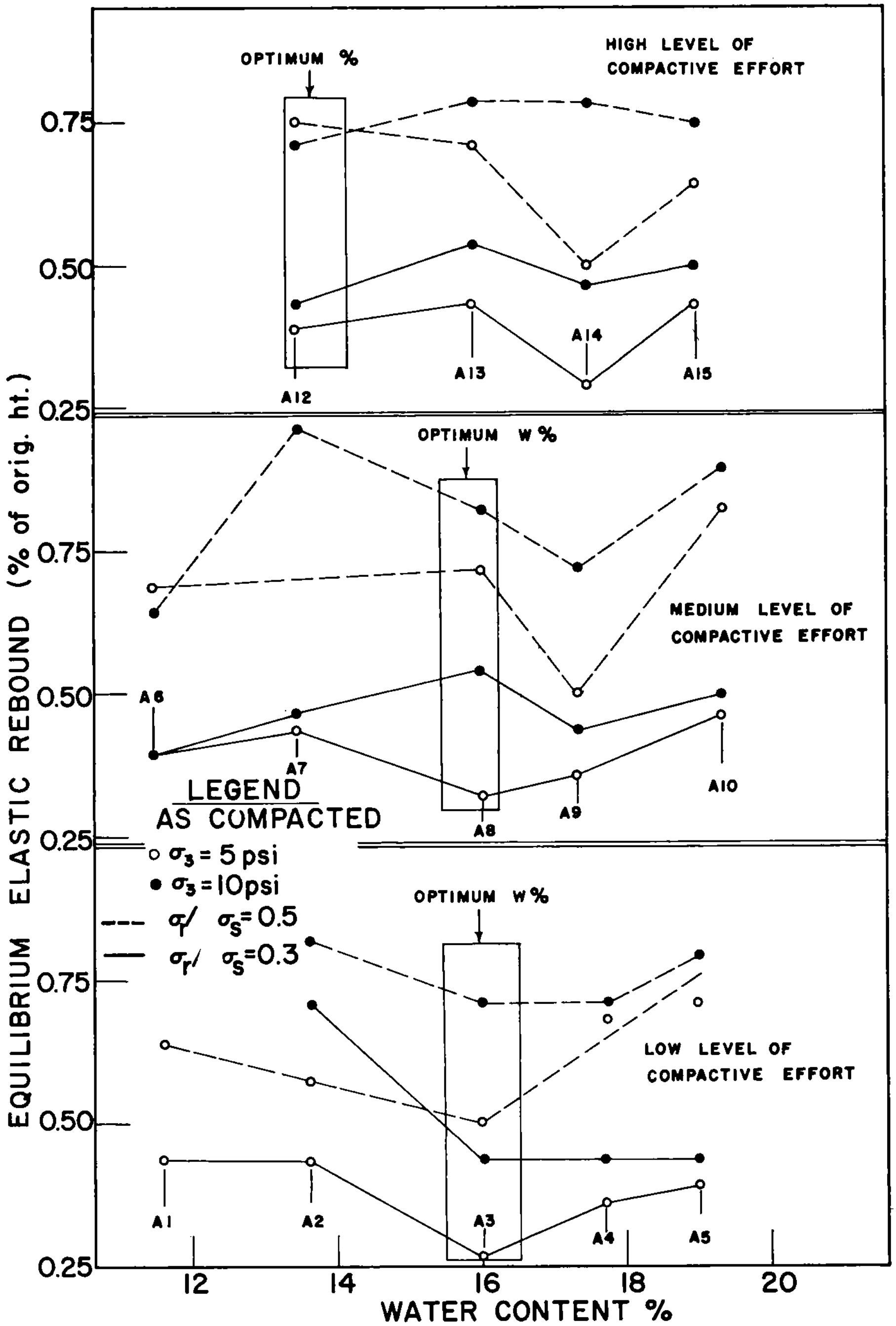


Figure 18. Equilibrium elastic deformation vs molding water content, Soil A.

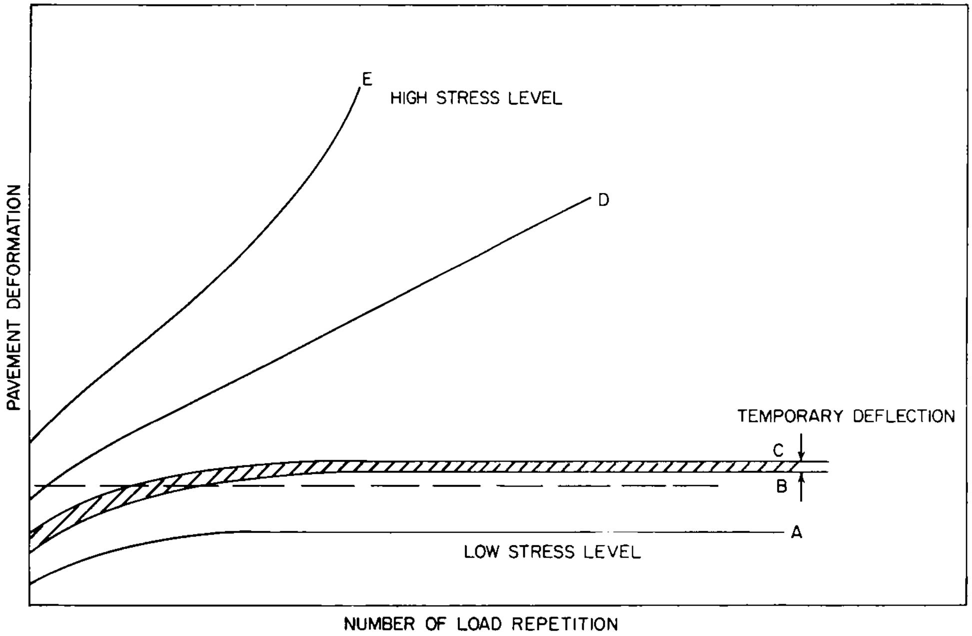


Figure 19. Schematic representation of the effect of repeated loads on the deformation pattern of flexible pavements.

bound macadam, etc.) for flexible pavements on main roads is gaining increasing favor (*18*). The semirigid base reduces the stress in the subgrade to a level shown by Curve A without necessitating the use of excessively thick pavements.

If the stress level is increased (Curve C, Fig. 19), the cumulative deflection pattern may exceed that required to cause distress in the pavement (represented by the horizontal dashed line). Nevertheless, modest repairs will be effective, as the additional cumulative deflections will be tolerable provided the temporary deflection (Curve C minus Curve B) is not large enough to cause a fatigue failure in the wearing course. Thus, if the moisture-density conditions of the subgrade (and of the base course as well, in some instances) are adversely selected to result in high temporary deflections, a pavement that might otherwise have proved adequate will experience fatigue failure. In many cases—particularly for secondary roads—it may be more economical to design the pavement with the intent of carrying out minor repairs rather than make it thick enough initially to avoid completely any signs of distress.

If the stress level is high enough to result in the condition represented by by Curve D (failure criterion), even extensive maintenance will not result in a satisfactory pavement as additional load repetitions develop a continuously increasing deflection pattern at a rate that is practically unabated. The use of elastic theory to assess this condition is entirely unrealistic; a suitable viscoelastic model may be helpful. Furthermore, the stress level in the subgrade is still below its strength as measured by any procedure applying a continuously increasing load to failure, such as the widely used CBR and triaxial tests.

If the stress level is further increased (Curve E), the pavement will rapidly deteriorate and complete replacement will be necessary soon after construction. The stress in the subgrade will be approximated by the failure condition in a triaxial or CBR test; however, a suitable factor of safety is usually applied, based on empirical correlations. As pointed out earlier, because the cumulative deflection pattern is not necessarily reflected by the static stress-strain curve, design charts based on such empirical correlations are inherently incapable of assessing the effect of repeated load and rebound conditions. Thus, they can do no better than be correct some of the time, be too conservative at other times, and occasionally result in failures.

CONCLUSIONS

For the soils studied and the test conditions employed, the following criterion of failure for compacted fine-grained soils acted on by repeated loads of constant magnitude has been established.

A critical level of repeated deviator stress, σ_{rc}, exists at which the slope of the deformation vs number of repetitions curve is constant after the first few load applications. For levels of deviator stress in excess of this critical value, the deformation curves eventually turn concave upward, their slopes continue to increase until failure occurs either by sliding along a shear plane or by excessive bulging. For levels of deviator stress less than the critical value, the deformation curves eventually approach a horizontal asymptote. It is proposed that σ_{rc} be termed the "strength" of compacted clay subject to repeated loading.

The ratio of σ_{rc} to the deviator stress at failure in a conventional triaxial test, σ_{rc}/σ_s, may be taken as a measure of the strength reduction due to the effects of repeated loads. For highly plastic clays, there are strong indications that this ratio is a minimum at or near optimum moisture content at any given level of compactive effort, and that it decreases significantly with increasing compactive effort. For such clays the equilibrium elastic rebound is also greatly increased as the compactive effort is increased. These facts warrant reexamination of current compaction specifications for highway and airfield pavements.

Design charts for flexible pavements are usually based on strength tests in which a continuously increasing load is applied to failure—modified by empirical correlations. Such charts are inherently incapable of assessing the effect of repeated load and rebound conditions on the performance of flexible pavements unless the stress level in the subgrade is sufficiently small to result in essentially "elastic" conditions.

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